ARCH 631

APPLIED ARCHITECTURAL STRUCTURES

COURSE NOTE SET
Fall 2012

by

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TABLE OF CONTENTS

ARCH 631. Applied Architectural Structures

Instructor: Prof. Anne B. Nichols
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Office Hours: 9-10 am and 2-3 pm MW
11-12 pm TR
(*and by appointment M-R)*

Prerequisites: ARCH 331 & ARCH 431 (or equivalent hours and content)

Course Description: Structural analysis of building structural systems: components, frames, shapes; selection and economics of structural systems; survey of current structural design codes; supervision practices in structural construction.

Goals: ARCH 631 encompasses structural analysis of building structural systems: components, frames, and shapes. Also covered is the selection and economics of structural systems; survey of current structural design codes; supervision practices in structural construction. Case studies and writing exercises will be utilized. The course follows the content areas of the ARE 4.0 section of Structural Systems section: **General Structures** is the application of general structural principles to building design and construction considering the code requirements, implication of alternate systems, materials and construction details along with site and environmental characteristics. **Wind Forces, Seismic Forces & Lateral Forces** is the application of lateral force principles to the design and construction of buildings to resist lateral, wind and seismic forces considering the code requirements, implication of alternative systems, materials and constructions details along with site and environmental characteristics.

Objective: To synthesize knowledge of components, systems and framing with environmental loads (particularly hazard) and design codes and standards.

Text: Structures, 6th ed., Daniel L. Schodek and Martin Bechthold, (2007) Pearson – Prentice Hall, ISBN 0-13-178939-2

Recommended Texts:
A Structures Primer, Kaufman, (2010) Prentice Hall, ISBN 978-0-13-230256-3
Understanding Structures, Moore, (1999) McGraw-Hill, ISBN 9780070432536
The Structural Basis of Architecture, Sandaker, et.al, (2011) Routledge, ISBN 978-0415415477

References: AIA Publications
Adoptable codes (ICBO, SBCCI, BOCA, CABO)
International Building Code, International Residential Code
Structural Design Codes (ACI, PCI, AISC, MSJC, etc.)
Material and Professional Standard Documents (ANSI, ASCE, ASTM, ASHRAE)

Timetable: 9:35-10:50 am Lecture T,R (section 600)

Grading:

Assignments	20%
Mid-term Exams	40%
Team Project	20%
Final Exam	20%

Letter Grades (Approximate):

90-100	A
80-89	B
70-79	C
60-69	D
0-59	F

Policy:

1) **Attendance:** Necessary. Required.* And subject to University Policy. See Part I Section 7 in Texas A&M University Student Rules: http://student-rules.tamu.edu/ Absences related to illness or injury must be documented according to http://shs.tamu.edu/attendance.htm *including* the Explanatory Statement for Absence from class for 3 days or less. Doctor visits not related to immediate illness or injury are not excused absences.

2) **Lecture:** The lecture slides should be viewed prior to class. Class–will also require problem solving with the lecture examples, assignments, and case studies. The lecture slide handouts are available on the class web page (see #3) and Vista (see #9). *Use of electronic devices during lecture is prohibited.*

3) **Notes:** The notes and related handouts are available on the class web page at http://faculty.arch.tamu.edu/anichols/631frame.html, or on Vista (see #8). A bound set can be purchased from the Notes-n-Quotes at 701 W. University, directly across from the Mitchell Physics Building in the Northgate Neighborhood.

4) **Assignments:** Due as stated on the assignment statements. Only *one* assignment without University excuse may be turned in for credit no later than one week after the due date **and** before final exams begin. All other assignments and projects will receive *no credit* if late without a recognized excuse or after final exams have begun.

5) **Team Project:** A term project to be completed in teams is due the last week of class. Presentations of the projects will be made during class periods.

6) **Mid-term Exams:** Mid-term exams will be given in lecture at any time during the period. Make-up exams without an excuse will not be given.

7) **Teaching Assistant:** Kara Wetzel .... (kewetz@neo.tamu.edu)

8) **Structures Help Desk:** Ryan Buys .... (syubnayr@neo.tamu.edu)
ARCA129 845-6580 Posted Hours (link)

9) **Vista:** Vista is a web course tool for posting, reading messages and replying as well as recording scores and is accessed with your neo account. This will be used to post questions and responses by class members and the instructor, for posting scores and for e-mail. It can be accessed at http://elearning.tamu.edu/

10) **Final Exam:** The final exam will be comprehensive and is officially scheduled for ***12:30-2:30 PM Friday, December 7***.

11) **Other Resources:** The Student Learning Center provides tutoring in math and physics. See their schedule at http://slc.tamu.edu/tutoring.shtml The Student Counseling Center has programs for study and learning (PASS), and tutoring services. See the resources at http://scs.tamu.edu/

12) **Aggie Honor Code:** "An Aggie does not lie, cheat, or steal or tolerate those who do."
The University policy will be strictly enforced. See Part I Section 20 in Texas A&M University Student Rules: http://student-rules.tamu.edu/ Plagiarism (deliberate misrepresentation of someone else's work as your own) will be treated strictly according to University policy as outlined by the Office of the Aggie Honor System: http:/www.tamu.edu/aggiehonor/

13) **The American with Disabilities Act** (ADA) is a federal anti-discrimination statute that provides comprehensive civil rights protection for persons with disabilities. Among other things, this legislation requires that all students with disabilities be guaranteed a learning environment that provides for reasonable accommodation of their disabilities. If you believe you have a disability requiring accommodation, please contact the Department for

* Except for death in the family, medical or deans' excuse, and natural disasters.

Student Life, Services for Students with Disabilities, in Cain Hall or call 845-1637. Also contact Prof. Nichols at the beginning of the semester.

14) **Grievances:** For grievances other than those listed in Part III in Texas A&M University Student Rules: http://student-rules.tamu.edu/ the *instructor* must be the first point of contact.

Learning Objectives:

1) The student will be able to read a text or article about structural technology, identify the key concepts and related equations, and properly apply the concepts and equations to appropriate structural problems (**relevance)**. The student will also be able to define the answers to key questions in the reading material. The student will be able to evaluate their own skills, or lack thereof, with respect to reading and comprehension of structural concepts, **clarity** of written communication, reasonable determination of **precision** in numerical data, and **accuracy** of computations.
2) The student will be able to read a problem statement, interpret the structural wording in order to identify the concepts and select equations necessary to solve the problem presented (**significance)**. The student will be able to identify common steps in solving structural problems regardless of the differences in the structural configuration and loads, and apply these steps in a clear and structured fashion (**logic)**. The student will be able to draw representational structural models and diagrams, and express information provided by the figures in equation form. The student will compare the computational results in a design problem to the requirements and properly decide if the requirements have been met. The student will take the corrective action to meet the requirements.
3) The student will create structural models with a computer application based on the concepts of the behavior and loading of the structural member or assemblage. The student will be able to interpret the modeling results and relate the results to the solution obtained by manual calculations.
4) The student will be able to articulate the physical phenomena, behavior and design criteria which influence structural space and form. (**depth)** The student will be able to identify the structural purpose, label, behavior, advantages and disadvantages, and interaction of various types of structural members and assemblies. (**breadth)** The student will be able to identify the configuration, label, behavior, advantages and disadvantages, and interaction of various types of structural members and assemblies with respect to materials (e.g. reinforced concrete beams or frames). The student will draw upon existing organizational and communication skills to clearly present concepts and personal interpretation of structural knowledge in writing assignments and examinations (**clarity, precision, accuracy, relevance, depth, breadth, logic, significance)**.
5) The student will interact and participate in group settings to facilitate peer-learning and teaching. In addition, the student will be able to evaluate the comprehension of concepts, clarity of communication of these concepts or calculations, and the precision and accuracy of the data used in the computations in the work of their peers. The student will participate in the classification and identification of structural components and assemblages and purposes with a case study chosen by a group in order to show synthesis of structural knowledge including modeling and analysis.

Tentative Schedule *(subject to change at any time throughout the semester)*

Lecture	Text Topic	Articles/Problems
1.	Structures: An Overview Introduction to Structural Analysis and Design	**Read*:** Ch. 1 **Solve:** Assignment 1 (*start*)
2.	Review of Statics and Mechanics	**Read:** Ch. 2; note sets 2.1 & 2.2 **Reference:** *Appendices 1-5*
3.	Overview of Building Codes	**Read:** Ch. 3; note sets 3.1 & 3.2 **Reference:** *note sets 3.3, 3.4 & 3.5*
4.	Overview of Design Philosophies and Beams	**Read:** § 6.1-6.4.1 & § 8.1-8.3 **Reference:** *Appendices 6-9; note set 4.2* **Due:** Assignment 1 over material from lectures 1-2
5.	Trusses & Columns	**Read:** Ch. 4 & § 7.1-7.4.2 **Reference:** *note set 5.1*
6.	Funicular Structures: Cables & Arches	**Read:** Ch. 5 **Due:** Assignment 2 over material from lectures 3-4
7.	Rigid Frames: Analysis & Design	**Read:** Ch. 9; note set 7.1 **Reference:** *note set 7.2* **Due:** CPR 1 Text over material from lecture 4
8.	Plates and Grids	**Read:** Ch. 10 & § 8.4; note set 8.1 **Due:** Assignment 3 over material from lectures 5-6 & CPR 1 Reviews
9.		**Mid-term Exam**
10.	Reinforced Concrete Construction	**Read:** § 15.3, 6.4.4-6.4.7, 7.4.5 & 8.4.6, Appendix 12; note set 10.1
11.	CASE STUDY – Reinforced Concrete	**Read:** note set 11
12.	CASE STUDY – Reinforced Concrete	**Read:** note set 11 **Due:** Assignment 4 over material from lecture 7
13.	Membrane, Net, and Shell Structures	**Read:** Ch. 11 & 12; note set 13.1
14.	Structural Planning & Design Issues	**Read:** Ch. 13; note set 14 **Due:** Assignment 5 over material from lectures 7-8

*Note: Material in the Class Note Set not specifically mentioned above are provided as references or aids.

Lecture	Text Topic	Articles/Problems
15.	Design for Lateral Loads Wind and Flood	**Read:** § 14.1; note set 15.1 **Re-read:** § 1.3.1, 1.3.2, 3.3.3 **Due:** CPR 2 Text over material from lecture 10
16.	Design for Lateral Loads Seismic	**Read:** § 14.2; note sets 16.1, 16.2 & 16.3 **Re-read:** § 3.3.4 **Due:** Assignment 6 over material from lectures 10-12 & CPR 2 Reviews
17.	Structural Connections: Wood and Steel	**Read:** § 16.1-16.3; note set 17.1
18.		**Mid-term Exam**
19.	Wood Construction	**Read:** § 15.2, 6.4.2, & 7.4.3; note set 19.1
20.	CASE STUDY - Wood	**Read:** note set 20 **Due:** Assignment 7 over material from lectures 13-15
21.	Steel Construction	**Read:** § 15.4, 6.4.3 & 7.4.4; note set 21.1 **Due:** CPR 3 Text over material from lectures 15 and 17
22.	CASE STUDY – Steel	**Read:** note set 22 **Due:** Assignment 8 over material from lectures 15-17 & CPR 3 reviews
23.	Masonry Construction	**Read:** note set 23.1
24.	Foundations and Retaining Walls	**Read:** §15.5; note sets 24.1 & 24.2 **Due:** Assignment 9 over material from lectures 19-22
25.		**Mid-term Exam**
	Thanksgiving Break	
26.	Project Presentations	
27.	Project Presentations	
28.	Construction & Inspection Review	**Reference:** *note set 28.1* **Due:** Assignment 10 over material from lectures 23-24 & Project Report
	Final Exam Period	**Exam**

*Note: Materials in the Class Note Set not specifically mentioned above are provided as references or aids.

	Sun	Mon	Tue	Wed	Thu	Fri	Sat
AUGUST	19	20	21	22	23	24 last day to register	25
	26 freshman convocation	27 classes begin	28 **Lect 1**	29	30 **Lect 2**	31 last day to add	1
SEPTEMBER	2	3	4 **Lect 3**	5	6 **Lect 4** #1 due academic convocation	7	8
	9	10	11 **Lect 5**	12	13 **Lect 6** #2 due	14	15
	16	17	18 **Lect 7** CPR 1 text due	19	20 **Lect 8** #3 & CPR 1 rev. due	21	22
	23	24	25 **Lect 9** Exam 1	26	27 **Lect 10**	28	29
	30	1	2 **Lect 11**	3	4 **Lect 12** #4 due	5	6
OCTOBER	7	8	9 **Lect 13**	10	11 **Lect 14** #5 due	12	13
	14	15 (mid-term grades due)	16 **Lect 15** CPR 2 text due	17	18 **Lect 16** #6 & CPR 2 rev. due	19	20
	21	22 college classes canceled for Symposium	23 **Lect 17**	24	25 **Lect 18** Exam 2	26	27
	28	29	30 **Lect 19**	31	1 **Lect 20** #7 due	2 last day to Q-drop	3
NOVEMBER	4	5	6 **Lect 21** CPR 3 text due	7	8 **Lect 22** #8 & CPR 3 rev. due	9	10
	11	12	13 **Lect 23**	14	15 **Lect 24** #9 due pre-registration begins	16	17
	18 Bonfire Remembrance day	19	20 **Lect 25** Exam 3	21	22 Thanksgiving Holiday	23	24
	25	26	27 **Lect 26** presentations	28	29 **Lect 27** presentations	30	1
DECEMBER	2	3 (dead day) Friday classes	4 **Lect 28** #10, project & evals due (dead day) Thursday classes	5 Reading	6 Days	7 Final exams 12:30-2:30am 631 FINAL	8
	9	10	11	12	13	14 Commencement (and Saturday)	15
	16	17 Grades due	18	19	20	21	22
	23	24	25	26 Winter Holiday	27	28	29

ARCH 631. Student Understandings

1) I understand that there are intellectual standards in this course and that I am responsible for monitoring my own learning.__________
2) I understand that the class will focus on application and synthesis, not on lecture.__________
3) I understand that I am responsible for preparing for lecture with the assigned reading by internalizing key concepts, recognizing key questions, and evaluating what makes sense and what doesn't make sense to me.__________
4) I understand that I will be held regularly responsible for assessing my own work using criteria and standards discussed in class.__________
5) I understand that if at any time in the semester I feel unsure about my "grade", I may request an assessment from the instructor.__________
6) I understand that there are <u>13 practice assignments</u>, one due every week during the bulk of the semester.__________
7) I understand that there are <u>group projects</u> and I will be responsible to take an active part in advancing the work of the group.__________
8) I understand that I will occasionally be required to assess the work of my classmates in an objective manor using the same criteria and standards used to assess my own work.__________
9) I understand that there are <u>3 graded exams</u> distributed throughout the semester.__________
10) I understand that there is a final exam in the course.__________
11) I understand that I must do a <u>Learning Evaluation</u> in which I make my "case" for receiving a particular grade using criteria provided in class and citing evidence from my work across the semester.__________
12) I understand that the work of the course requires <u>consistent classroom attendance</u> and active participation.__________
13) I understand that I will regularly be required to demonstrate that I have prepared for lecture.__________
14) I understand that the class will not be graded on a curve. I understand that it is theoretically possible for the whole class to get an A or an F.__________
15) I understand the basis of the final grade as outlined in the syllabus.__________
16) I understand that I will uphold academic integrity and abide by the Aggie Honor Code.__________

NAME (sign and print) ________________________________

____

DATE ______________

Academic Integrity

Academic integrity is defined differently from culture to culture. In some cultures, sharing homework assignments, providing friends with answers to test questions, and allowing someone to copy one's papers is considered "helping". In the US, each of these actions is considered a violation of academic integrity, and violators are punished severely. At Texas A & M University, the least severe punishment is a suspension for a stated period of time (possibly a semester or longer), which could cause a student to have to reapply for continuation of their academic program. Following the suspension, an academic department could refuse to readmit the individual. Furthermore, it usually means a loss of legal non-immigrant status as student. Once this legal status is lost, an international student is no longer eligible for employment in the US and must apply for reinstatement to legal status by the Immigration and Naturalization Services. But, applying does not guarantee that reinstatement to legal status will occur.

While most international students adapt very well to the US academic system, it is important to understand the US expectations for students in relation to academic integrity and to avoid problems that could be caused by lack of cultural understanding about these issues. Indeed, many international students are simply unaware of the US expectations and university rules about this important issue and would never violate them on purpose. While this lack of understanding can have serious consequences, there are a variety of resources to assist you in learning what the rules and expectations are.

For example, the most common problem of international students in relation to academic integrity concerns giving appropriate credit to others when using someone else's ideas in a written paper, such as a dissertation or theses. This is called plagiarism. The majority of this handout focuses on this problem and how to avoid it. However, your best source of information about appropriate writing styles and formats for research papers, theses, or dissertations is the faculty member teaching the class or advising you in your research project. You should also ask your professor if it is appropriate to work with another student on homework or other class projects.

Students who are accused of scholastic dishonesty have rights within the university regulations that govern the process. For more information about the process, the punishments, and your rights and responsibilities as a student, refer to the Academic Integrity brochure published by the Department of Student Life or to the Texas A & M University Student Rules and Regulations, available in your academic department, at International Student Services, and at http://student-rules.tamu.edu.

<u>Plagiarism</u> – Plagiarism occurs when someone else's ideas are described or words are used in a written document, but that individual is not given appropriate or correct credit for those ideas or words. The effect that is created is that the writer of the document appears to be taking credit for ideas or words that are not his or her own. In essence, it becomes a theft of ideas. This is a very serious offense in a university, which is an institution designed to explore ideas and create new knowledge.

<u>Fabrication of Results/Data</u> – It is never appropriate to create false data to include in a written paper or to leave data out of a study to make the results appear more significant. The academic community condemns this above all other forms of scholastic dishonesty.

<u>Reference Citation</u> – There are appropriate ways to give credit for someone else's ideas in a written paper that you are writing. This is called making a reference citation. Each academic discipline is different in its expectations for correctly paraphrasing and documenting sources. For example, some disciplines use footnotes; some use endnotes. Some use names in the body of the text; some use numbers. Each uses its own stylistic format from a scientific journal or professional association. Therefore, it is vital to ask your academic advisor for help in learning what is appropriate for your academic major or a specific class. There are some basic examples listed on this page to point out some of these differences.

<u>Literature Review</u> – This part of a thesis or dissertation is a review of the current state of the discipline about a chosen research topic. It is never the writer's own ideas, and it is never copied from a previously written dissertation or thesis of a research group member. It is a compilation of the ideas of others, so each idea that comes from another source must be quoted or paraphrased and appropriately given credit through a reference citation.

How to Avoid Plagiarism When Writing a Paper

1. **<u>Quotation Marks</u>** -- Never copy the ideas of someone else word for word without using quotation marks. In most disciplines it is not acceptable to use too many quotations in a written paper. In general, the reason to use quotations is that the original writer has used such impressive wording that it must be read as it was originally written or it will lose its impact or importance. In most cases, such quotations are not longer than a few sentences. Always check with your academic advisor if you have questions about what is appropriate for your field.

2. **<u>Use of Tables, Charts, and Figures</u>** – Sometimes, you will want to use a chart or some type of diagram in your paper that was originally created by someone else. Generally, this is all right to do if you provide credit to the original author. However, here are circumstances where it is not appropriate. For example, if a paper is being published, it may be necessary to obtain permission in advance from the author before the table or chart can be used in your published paper. If you are not sure about whether you should use a table or figure in your papers, ask your professor.

3. **Paraphrasing** – This occurs when you read something from an author that you want to include, but you summarize the original author's ideas and write them in your own words. Even using paraphrasing, it is still necessary to provide a reference citation naming the original author, to let the reader know that this idea is not your original idea. If you use too many of the original author's wording or phrasing in your paraphrase, you may still be guilty of plagiarism. If you are not sure that your paraphrasing is acceptable, ask the professor for whom you are writing the paper. Below are some examples of this.

Paraphrasing Sample 1 :

Original Text
"The association of lipids with proteins not only solubilizes lipids but also aids in their transport into cells. Triacylglycerols are transported to tissues either in chylomicrons or in VLDL."[1]

Note the use of quotation marks and the reference number "1" at the end of the quotation to indicate the reference citation.

Reference Citation:

1. Mathews, C. K., & van Holde, K. E. (1990) *Biochemistry*, pp. 576, Benjamin/Cummings Press, Menlo Park, CA.

This reference citation would appear, numbered, in a list at the end of the paper. Using the reference citation provides proper credit to the original author. Remember that citation formats are discipline specific. Ask your professor what is appropriate for your papers in class or your research group.

Bad Paraphrase of Original Text:
Lipids associate with proteins not only to solubilize themselves in blood but also to help their transport into cells. Triacylglycerols are carried to tissues by chylomicrons or VLDL particles.[1]

This paraphrase is worded too much like the original quotation. Even though the phrasing is not exactly the same as the original, this pharphrasing could still be considered plagiarism.

Better Paraphrase of Original Text:

Plasma lipoproteins are composed of lipid and protein domains and are responsible for delivering the water-insoluble lipids to cells. For example, chylomicrons or VLDL particles are involved in the delivery of triacylglycerols.[1]

Now, the idea of the original author is more obviously stated in the words of the writer. Notice that both paraphrases still use the reference number "1" at the end of the idea to indicate that the original idea came from a source other than the writer of this paper.

<u>Paraphrasing Sample 2 :</u>

<u>Original Text</u>
"Skills are dimensions of the ability to behave effectively in situations of action. *Skill* is a hybrid term that refers both to a property of concrete behavior and to a property of theories of action" (Argyris & Schon, 1974, p. 12).

Note the use of quotation marks and inclusion of names of the authors, the date of the original publication, and the page number where the quotation appeared in the original text. Some disciplines use this type of reference citation format instead of numbers as used in sample 1.

<u>Reference Citation:</u>
Argyris, C. and Schon, D.A. (1974). <u>Theory in practice: Increasing professional effectiveness</u>. San Francisco, California: Jossey-Bass Publishers.

Note that this reference citation is different than that in Sample 1. In this discipline, the references are also listed at the end of the paper, but they are listed alphabetically by the author's last name. To make the last name easy to see, the first line of the citation sometimes hangs over the following lines as this example shows. Again, always ask your professor what is most appropriate for your papers in class or your research group.

<u>Bad Paraphrase of Original Text:</u>
In active situations, skills are aspects of one's ability to act effectively. Referring to both to definite action as well as a theory of action, *skill* is a hybrid term (Argyris & Schon, 1974).

This paraphrase is worded too much like the original quotation. Even though the phrasing is not exactly the same as the original, this pharphrasing could still be considered plagiarism.

<u>Better Paraphrase of Original Text:</u>
The term "skill" can refer both to physical actions that allow one to perform effectively in a specific situation as well as to mental concepts that comprise one's theories of action (Argyris & Schon, 1974).

Now, the idea of the original author is more obviously stated in the words of the writer. Notice that both paraphrases still use reference citations to indicate that the original idea came from a source other than the writer of this paper.

List of Symbol Definitions

a	long dimension for a section subjected to torsion (in, mm); acceleration (ft/sec^2, m/sec^2); acceleration due to gravity, 32.17 ft/sec^2, 9.81 m/sec^2 (*also see g*) unit area (in^2, ft^2, mm^2, m^2); distance used in beam formulas (ft, m); depth of the effective compression block in a concrete beam (in, mm); equivalent square column size in spread footing design (in, ft, mm, m)
$\mathcal{a}$	area bounded by the centerline of a thin walled section subjected to torsion (in^2, mm^2)
A	area, often cross-sectional (in^2, ft^2, mm^2, m^2)
A_b	nominal cross section bolt area (in^2, ft^2, mm^2, m^2)
A_e	net effective area, equal to the total area ignoring any holes and modified by the lag factor, U, (in^2, ft^2, mm^2, m^2) *(see A_{net})*
A_g	gross area, equal to the total area ignoring any holes (in^2, ft^2, mm^2, m^2)
A_{gv}	gross area in shear, equal to the total area ignoring any holes (in^2, ft^2, mm^2, m^2)
A_{net}	net effective area, equal to the gross area subtracting any holes (in^2, ft^2, mm^2, m^2) *(see A_e)*
A_{nt}	net area in shear of a bolted connection subject to shear rupture (in^2, ft^2, mm^2, m^2)
A_{nv}	net area in tension of a bolted connection subject to shear rupture (in^2, ft^2, mm^2, m^2); net shear area for a masonry member (in^2, ft^2, mm^2, m^2)
A_p	bearing area (in^2, ft^2, mm^2, m^2)
A_{throat}	area across the throat of a weld (in^2, ft^2, mm^2, m^2)
A_s	area of steel reinforcement in concrete beam design (in^2, ft^2, mm^2, m^2)
$A_{s'}$	area of compression steel reinforcement in concrete beam design (in^2, ft^2, mm^2, m^2)
A_v	area of concrete shear stirrup reinforcement (in^2, ft^2, mm^2, m^2); seismic coefficient for acceleration
A_{web}	web area in a steel beam equal to the depth x web thickness (in^2, ft^2, mm^2, m^2)
A_1	area of column in spread footing design ((in^2, ft^2, mm^2, m^2)
A_2	projected bearing area of column load in spread footing design ((in^2, ft^2, mm^2, m^2)
ASD	Allowable Stress Design
b	width, often cross-sectional (in, ft, mm, m); narrow dimension for a section subjected to torsion (in, mm); number of truss members (*also see n)*; rectangular column dimension in concrete footing design (in, mm, m); distance used in beam formulas (ft, m)
b_E	effective width of the flange of a concrete T beam cross section (in, mm)
b_f	width of the flange of a steel or concrete T beam cross section (in, mm)
b_o	perimeter length for two-way shear in concrete footing design (in, ft, mm, m)
b_w	width of the stem of a concrete T beam cross section (in, mm)

B spread footing dimension in concrete design (ft, m);
dimension of a steel base plate (in, mm, m)

B_s width within the longer dimension of a rectangular spread footing that reinforcement must be concentrated within for concrete design (ft, m)

B_1 factor for determining M_u for combined bending and compression

c distance from the neutral axis to the top or bottom edge of a beam (in, mm, m);
rectangular column dimension in concrete footing design (in, mm, m)

c_1 coefficient for shear stress for a rectangular bar in torsion

c_2 coefficient for shear twist for a rectangular bar in torsion

CL, ℄ center line

C compression label;
compression force (lb, kips, N, kN);
dimension of a steel base plate for concrete footing design (in, mm, m);
seismic design coefficient dependent on the building period of vibration;
constant for moment calculation of plates with respect to boundary conditions;
coefficient for eccentrically loaded bolt groups

C_a constant for moment calculation of plates with respect to boundary conditions

C_b modification factor for LRFD steel beam design;
constant for moment calculation of plates with respect to boundary conditions

C_d pressure coefficient for wind force calculation

C_D load duration factor for wood design

C_F size factor for wood design

C_{fu} flat use factor for wood design

C_H shear stress factor for wood design

C_i incising factor for wood design

C_L beam stability factor for wood design

C_m modification factor for combined stress in steel design

C_M wet service factor for wood design

C_p column stability factor for wood design

C_r repetitive member factor for wood design

C_s seismic design coefficient based on soil, response and acceleration

C_v web shear coefficient for steel design

C_V glulam volume factor for wood design

C_t temperature factor for wood design;
seismic coefficient based on structural system and number of stories to determine building period

d diameter of a circle (in, mm, m);
depth, often cross-sectional (in, mm, m);
perpendicular distance from a force to a point in a moment calculation (in, mm, m) ;
effective depth from the top of a reinforced concrete beam to the centroid of the steel (in, mm);
effective depth from the top of a reinforced masonry member to the centroid of the steel (in, mm);
critical cross section dimension of a rectangular timber column cross section related to the profile (axis) for buckling (in, mm, m);
symbol in calculus to represent a very small change (like the greek letters for d, *see* δ & Δ)

d' effective depth from the top of a reinforced concrete beam to the centroid of the compression steel (in, mm)

d_b depth of a steel wide flange section (in, mm);
bar diameter of concrete reinforcement (in, mm)

d_f depth of a steel column flange (wide flange section) (in, mm)

d_x difference in the x direction between an area centroid and the centroid of the composite shape (in, mm)

d_y difference in the y direction between an area centroid and the centroid of the composite shape (in, mm)

D diameter of a circle (in, mm, m);
dead load for LRFD design

DL dead load

e dimensional change to determine strain (in, mm) *(see s or* ε*)*;
eccentric distance of application of a force (P) from the centroid of a cross section (in, mm)

E modulus of elasticity (psi; ksi, kPa, MPa, GPa);
earthquake load for LRFD design

E_c modulus of elasticity of concrete (psi; ksi, kPa, MPa, GPa)

E_s modulus of elasticity of steel (psi; ksi, kPa, MPa, GPa)

f symbol for stress (psi, ksi, kPa, MPa);
symbol for function with respect to some variable, ie. f(t)

f_a calculated axial stress (psi, ksi, kPa, MPa)

f_b calculated bending stress (psi, ksi, kPa, MPa)

f_c calculated compressive stress (psi, ksi, kPa, MPa)

f'_c concrete design compressive stress (psi, ksi, kPa, MPa)

f_{cr} calculated column stress based on the critical column load P_{cr} (psi, ksi, kPa, MPa)

f_m calculated compressive stress in masonry (psi, ksi, kPa, MPa)

f'_m masonry design compressive stress (psi, ksi, kPa, MPa)

f_n natural frequency of a suspended cable (sec^{-1}, Hz)

f_p calculated bearing stress (psi, ksi, kPa, MPa)

f_r calculated radial stress for a glulam timber (psi, ksi, kPa, MPa)

f_s calculated steel stress for reinforced masonry (psi, ksi, kPa, MPa)

f_t calculated tensile stress (psi, ksi, kPa, MPa)

f_v calculated shearing stress (psi, ksi, kPa, MPa)

f_x combined stress in the direction of the major axis of a column (psi, ksi, kPa, MPa)

f_y yield stress (psi, ksi, kPa, MPa)

F force (lb, kip, N, kN);
capacity of a nail in shear (lb, kip, N, kN);
hydraulic fluid load for LRFD design

F_a allowable axial stress (psi, ksi, kPa, MPa)

F_b allowable bending stress (psi, ksi, kPa, MPa)

F_b' allowable bending stress for combined stress for wood design (psi, ksi, kPa, MPa)

F_c allowable compressive stress (psi, ksi, kPa, MPa)
critical unfactored compressive stress for LRFD steel design

F_{cr} flexural buckling (column) stress in ASD and LRFD (psi, ksi, kPa, MPa)

$F_{c\perp}$ allowable compressive stress perpendicular to the wood grain (psi, ksi, kPa, MPa)

$F_{connector}$ resistance capacity of a connector (lb, kips, N, kN)

F_{cE} intermediate compressive stress for ASD wood column design dependant on material (psi, ksi, kPa, MPa)

F_{cr} critical column stress due to buckling (psi, ksi, kPa, MPa)

F_c' allowable compressive stress for ASD wood column design (psi, ksi, kPa, MPa)

$F^*{}_c$ intermediate compressive stress for ASD wood column design dependant on load duration (psi, ksi, kPa, MPa)

F_e elastic critical buckling stress is steel design

F_{EXX} yield strength of weld material (psi, ksi, kPa, MPa)

$F_{horizontal\text{-}resist}$ resultant frictional force resisting sliding in a footing or retaining wall (lb, kip, N, kN)

F_n nominal stress (psi, ksi, kPa, MPa)

F_{nv} nominal shear stress (psi, ksi, kPa, MPa)

F_{nt} nominal tensile stress (psi, ksi, kPa, MPa)

F_p allowable bearing stress parallel to the wood grain (psi, ksi, kPa, MPa)

F_r allowable radial stress for a curved glulam (psi, ksi, kPa, MPa)

$F_{sliding}$ resultant force causing sliding in a footing or retaining wall (lb, kip, N, kN)

F_t allowable tensile stress (psi, ksi, kPa, MPa)

F_v allowable shear stress (psi, ksi, kPa, MPa);
allowable shear stress in a welded connection (psi, ksi, kPa, MPa)

F_{vm} allowable shear stress in the reinforced masonry (psi, ksi, kPa, MPa)

F_{vs} allowable shear stress in the reinforcement for masonry (psi, ksi, kPa, MPa)

F_x force component in the x coordinate direction (lb, kip, N, kN)

F_y force component in the y coordinate direction (lb, kip, N, kN);
yield stress (psi, ksi, kPa, MPa)

F_{yw} yield stress in the web of a steel wide flange section (psi, ksi, kPa, MPa)

F_u ultimate stress a material can sustain prior to failure (psi, ksi, kPa, MPa)

F.S. factor of safety *(also see SF)*

g acceleration due to gravity, 32.17 ft/sec^2, 9.81 m/sec^2 *(also see a)*
gage spacing of staggered bolt holes (in, mm)

G shear modulus (psi; ksi, kPa, MPa, GPa);
gigaPascals (10^9 Pa or 1 kN/mm^2);
relative stiffness of columns to beams in a rigid connection *(see Ψ)*

h depth, often cross-sectional (in, ft, mm, m);
sag of a cable structure (ft, m);
height (in, ft, mm, m);
effective height of a wall or column, *(see ℓ_e)*

h_c height of the web in a wide flange section (in, ft, mm, m) *(also see t_w)*

h_f depth of a flange in a T section (in, ft, mm, m);
height of a concrete spread footing (in, ft, mm, m)

h_n building height for determination of period for seismic design

H hydraulic soil load for LRFD design;
height of retaining wall (ft, m)

H_A horizontal load from active soil or water pressure (lb, k, N, kN)

I moment of inertia (in^4, mm^4, m^4);
seismic importance factor based on building occupancy

$\bar{I}$ moment of inertia about the centroid (in^4, mm^4, m^4)

$\bar{I}_T$ moment of inertia about the centroid of a composite shape (in^4, mm^4, m^4) *(also see $\hat{I}$)*

$\hat{I}$ moment of inertia about the centroid of a composite shape (in^4, mm^4, m^4) *(also see I_c)*

I_c moment of inertia about the centroid of a composite shape (in^4, mm^4, m^4)

I_{min} minimum moment of inertia of I_x and I_y (in^4, mm^4, m^4)

I_{net} moment of inertia of plate area excluding bolt holes (in^3, mm^3, m^3)

I_o moment of inertia about the centroid (in^4, mm^4, m^4)

$I_{transformed}$ moment of inertia of a multi-material section transformed to one material (in^4, mm^4, m^4)

I_x moment of inertia with respect to an x-axis (in^4, mm^4, m^4)

I_y moment of inertia with respect to a y-axis (in^4, mm^4, m^4)

j number of connections in a truss (*also see n)*;
multiplier by effective depth of concrete or masonry section for moment arm, jd (*see d)*

J, J_o polar moment of inertia (in^4, mm^4, m^4)

k	kips (1000 lb); shape factor for plastic design of steel beams, $M_p/M_{y;}$ effective length factor for columns (*also K);* distance from outer face of flange to the web toe of fillet of a wide flange section (in, mm); spring constant (lb/in, N/mm); multiplier by effective depth of masonry section for neutral axis, kd
kg	kilograms
kN	kiloNewtons (10^3 N)
kPa	kiloPascals (10^3 Pa)
K	effective length factor with respect to column end conditions (*also k)*; masonry mortar strength designation
K_A	empirically derived coefficient based on soil properties
K_{cE}	material factor for wood column design
ℓ	length (in, ft, mm, m); cable span (ft, m)
ℓ_d	development length of concrete reinforcement (in, ft, mm, m)
ℓ_{dc}	development length of compression reinforcement in concrete footing design (in, ft, mm, m)
l_{dh}	development length for hooks (in, ft, mm, m)
ℓ_e	effective length that can buckle for wood column design (in, ft, mm, m)
ℓ_n	effective clear span for concrete one-way slab design (ft, m)
lb	pound force
L	length (in, ft, mm, m); live load for LRFD design; spread footing dimension in concrete design (ft, m)
L_b	unbraced length of a steel beam in LRFD design (ft, m)
L_c	clear distance between the edge of a bolt hole and the edge of the next hole or edge of the connected steel plate in the direction of the load (in, mm)
L_d	development length of reinforcement in concrete (ft, m)
L_e	effective length that can buckle for column design (ft, m)
L_m	projected length for bending in concrete footing design (ft, m)
L_p	maximum unbraced length of a steel beam in LRFD design for full plastic flexural strength (in, ft, mm, m)
L_r	roof live load in LRFD design; maximum unbraced length of a steel beam in LRFD design for inelastic lateral-torsional buckling (in, ft, mm, m)
L'	length of the one-way shear area in concrete footing design (ft, m)
LL	live load
LRFD	Load and Resistance Factor Design

m mass (lb-mass, g, kg);
meters;
moment per unit width (lb-ft/ft, kN-m/m);
edge dimension in a steel base plate (in, mm)

mm millimeters

M moment of a force or couple (lb-ft, kip-ft, N-m, kN-m);
bending moment (lb-ft, kip-ft, N-m, kN-m);
masonry mortar strength designation

M_a required bending moment in steel ASD beam design (unified) (lb-ft, kip-ft, N-m, kN-m)

M_A moment value at quarter point of unbraced beam length for LRFD beam design (lb-ft, kip-ft, N-m, kN-m)

M_B moment value at half point of unbraced beam length for LRFD beam design (lb-ft, kip-ft, N-m, kN-m);
nominal moment capacity of a reinforced concrete beam at the balanced steel ratio (ρ_b) for limiting strains in both concrete and steel (lb-ft, kip-ft, N-m, kN-m)

M_C moment value at three quarter point of unbraced beam length for LRFD beam design (lb-ft, kip-ft, N-m, kN-m)

M_m moment capacity of a reinforced masonry beam (lb-ft, kip-ft, N-m, kN-m)

M_n nominal moment capacity of a reinforced concrete beam based on steel yielding and concrete design strength (lb-ft, kip-ft, N-m, kN-m)

$M_{overturning}$ resulting moment from all forces on a footing or retaining wall causing overturning (lb-ft, kip-ft, N-m, kN-m)

M_p internal bending moment when all fibers in a cross section reach the yield stress (lb-ft, kip-ft, N-m, kN-m) *(also see M_{ult})*

M_{resist} resulting moment from all forces on a footing or retaining wall resisting overturning (lb-ft, kip-ft, N-m, kN-m)

M_u factored moment calculated in concrete design from load factors (lb-ft, kip-ft, N-m, kN-m)

M_{ult} internal bending moment when all fibers in a cross section reach the yield stress (lb-ft, kip-ft, N-m, kN-m) *(also see M_p)*

M_y internal bending moment when the extreme fibers in a cross section reach the yield stress (lb-ft, kip-ft, N-m, kN-m)

M_1 smaller end moment used to calculate C_m for combined stresses in a beam-column (lb-ft, kip-ft, N-m, kN-m)

M_2 larger end moment used to calculate C_m for combined stresses in a beam-column (lb-ft, kip-ft, N-m, kN-m)

MPa megaPascals (10^6 Pa or 1 N/mm^2)

n number of truss joints or members, nails or bolts;
modulus of elasticity transformation coefficient for steel to concrete;
edge dimension in a steel base plate (in, mm)

n.a. neutral axis (axis connecting beam cross-section centroids)

n' equivalent edge dimension in a steel base plate for design (in, mm)

N Newtons (kg-m/sec^2);
bearing-type connection with bolt threads included in shear plane;
normal load (lb, kip, N, kN);
bearing length on a wide flange steel section (in, mm);
dimension of a steel base plate (in, mm, m);
masonry mortar strength designation

N_ϕ meridional in-plane internal force per unit length in a shell (lb/ft, N/m, kN/m)

N_θ hoop in-plane internal force per unit length in a shell (lb/ft, N/m, kN/m)

o.c. on-center

O point of origin;
masonry mortar strength designation

p pitch of nail spacing (in, mm) (*also see s)*;
pressure (lb/in^2, lb/ft^2, kip/in^2, kip/ft^2, Pa, MPa):
unit weight of soil for determining active lateral pressure (lb/ft^3, kN/m^3)

p_A active soil pressure (lb/ft^3, kN/m^3)

p_r internal pressure (lb/in^2, lb/ft^2, kip/in^2, kip/ft^2, Pa, MPa)

P force, concentrated (point) load (lb, kip, N, kN)

P_a required axial force in ASD steel design (unified) (lb, kip, N, kN)

P_c available axial strength for steel unified design (lb, kip, N, kN)

P_{cr} critical (failure) load in column calculations (lb, kip, N, kN)

P_{e1} Euler buckling strength in steel unified design (lb, kip, N, kN)

P_n maximum column load capacity in LRFD steel and concrete design (lb, kip, N, kN);
nominal axial load for a tensile member or connection in LRFD steel (lb, kip, N, kN)

P_o maximum axial force with no concurrent bending moment in a reinforced concrete column (lb, kip, N, kN)

P_r required axial force in steel unified design (lb, kip, N, kN)

P_u factored column load calculated from load factors in LRFD steel and concrete design (lb, kip, N, kN);
factored axial load for a tensile member or connection in LRFD steel (lb, kip, N, kN)

Pa Pascals (N/m^2)

q shear flow (lb/in, kips/ft, N/m, kN/m));
soil bearing pressure (lb/ft^2, kips/ft^2, N/m^2, Pa, MPa)

$q_{allowed}$ allowable soil bearing pressure (lb/ft^2, kips/ft^2, N/m^2, Pa, MPa)

q_h static wind velocity pressure for wind force calculation (lb/ft^2, kips/ft^2, N/m, Pa, MPa)

q_{net} net allowed soil bearing pressure (lb/ft^2, kips/ft^2, N/m, Pa, MPa)

q_u factored soil bearing pressure in concrete design from load factors (lb/ft^2, kips/ft^2, N/m, Pa, MPa)

Q first moment area used in shearing stress calculations (in^3, mm^3, m^3)

$Q_{connected}$ first moment area used in shear calculations for built-up beams (in^3, mm^3, m^3)

Q_x first moment area about an x axis (using y distances) (in^3, mm^3, m^3)

Q_y first moment area about an y axis (using x distances) (in^3, mm^3, m^3)

r radius of a circle or arc (in, mm, m);
radius of gyration (in, mm, m)

r_o polar radius of gyration (in, mm, m)

r_x radius of gyration with respect to an x-axis (in, mm, m)

r_y radius of gyration with respect to a y-axis (in, mm, m)

R force, reaction or resultant (lb, kip, N, kN);
radius of curvature of a beam or radius of a shell (ft, m);
rainwater or ice load for LRFD design;
seismic response modification based on structural type;
calculated reduction in live load limited to 60% (in percent);
generic load quantity (force, shear, moment, etc.) for LRFD design

R_a required strength (ASD-unified) *(also see V_a, M_a)*

R_n concrete beam design ratio = M_u/bd^2 (lb/in^2, MPa)
nominal value for LRFD design to be multiplied by ϕ *(also see P_n, M_n)*
nominal value for ASD design to be divided by the safety factor Ω

R_u design value for LRFD design based on load factors *(also see P_u, M_u)*

R_w seismic response modification based on structural type

R_x reaction or resultant component in the x coordinate direction (lb, kip, N, kN)

R_y reaction or resultant component in the y coordinate direction (lb, kip, N, kN)

s strain (=change in length divided by length) (no units);
displacement with respect to time (ft, m);
length of a segment of a thin walled section (in, mm);
pitch of nail spacing (in, mm) *(also see p);*
longitudinal center-to-center spacing of any two consecutive holes (in, mm);
spacing of stirrups in reinforced concrete beams (in, mm)

s.w. self-weight

S section modulus (in^3, mm^3, m^3);
snow load for LRFD design;
allowable strength of a weld for a given size (lb/in, kips/in, N/mm, kN/m)
seismic soil profile;
masonry mortar strength designation

S_{net} section modulus of plate area excluding bolt holes (in^3, mm^3, m^3)

$S_{required}$ section modulus required to not exceed allowable bending stress (in^3, mm^3, m^3)

S_x section modulus with respect to the x-centroidal axis (in^3, mm^3, m^3)

S_y section modulus with respect to the y-centroidal axis (in^3, mm^3, m^3)

SC slip critical bolted connection

SF safety factor *(also see F.S.)*

S4S surface-four-sided

t thickness (in, mm, m);
time (sec, hrs)

t_f thickness of the flange of a steel beam cross section (in, mm, m)

t_w thickness of the web of a steel beam cross section (in, mm, m)

T tension label;
tensile force (lb, kip, N, kN);
torque (lb-ft, kip-ft, N-m, kN-m);
throat size of a weld (in, mm);
effect of thermal load for LRFD design;
seismic building period (sec);
depth in web of wide flange section from fillet to fillet (in, mm)

U shear lag factor for steel tension member design *(see A_e and A_{net})*

U_{bs} reduction coefficient for block shear rupture

v velocity (ft/sec, m/sec, mi/h);
shear force per unit length (lb/ft, k/ft, N/m, kN/m) *(see q)*

V shearing force (lb, kip, N, kN);
seismic base shear force (lb, kip, N, kN)

V_a required shear in steel ASD design (unified) (lb, kip, N, kN)

V_c shear force capacity in concrete (lb, kip, N, kN)

V_n nominal shear force capacity for concrete design (lb, kip, N, kN)

V_s shear force capacity in steel (lb, kip, N, kN)

V_u factored shear calculated in concrete design from load factors (lb, kip, N, kN)

V_{u1} factored one-way shear calculated in concrete footing design from load factors (lb, kip, N, kN)

V_{u2} factored two-way shear calculated in concrete footing design from load factors (lb, kip, N, kN)

w load per unit length on a beam (lb/ft, kip/ft, N/m, kN/m);
load per unit area on a surface (lb/ft^2, kip/ft^2, N/m^2, kN/m^2) *(see w')*;
width dimension (in, ft, mm, m)

w_c weight of reinforced concrete per unit volume (lb/ft^3, N/m^3)

w_u factored load per unit length on a beam from load factors (lb/ft, kip/ft, N/m, kN/m);
factored load per unit area on a surface from load factors (lb/ft^2, kip/ft^2, N/m^2, kN/m^2)

w' load per unit area on a surface (lb/ft^2, kip/ft^2, N/m^2, kN/m^2) *(see w)*;

W weight (lb, kip, N, kN);
total load from a uniform distribution (lb, kip, N, kN);
wind load for LRFD design;
seismic building weight (lb, kip, N, kN);
wide flange shape designation (i.e. W 21 x 68)

x a distance in the x direction (in, ft, mm, m)

$\bar{x}$ the distance in the x direction from a reference axis to the centroid of a shape (in, mm)

$\hat{x}$ the distance in the x direction from a reference axis to the centroid of a composite shape (in, mm)

X bearing-type connection with bolt threads excluded from shear plane;
design constant for steel base plate design based on concrete bearing capacity

y	a distance in the y direction (in, ft, mm, m); distance from the neutral axis to the y-level of a beam cross section (in, mm)
$\bar{y}$	the distance in the y direction from a reference axis to the centroid of a shape (in, mm)
$\bar{y}_T$	the distance in the y direction from a reference axis to the centroid of a composite shape (in, mm) *(also see* $\hat{y}$ *)*
$\hat{y}$	the distance in the y direction from a reference axis to the centroid of a composite shape (in, mm) *(also see* $\bar{y}_T$ *)*
z	the distance from a unit area to a reference axis (in, ft, mm, m) *(also see* d_x *and* d_y*)*
Z	plastic section modulus of a steel beam (in^3, mm^3); seismic geographic factor based on zone
′	symbol for feet
″	symbol for inches
#	symbol for pounds
=	symbol for equal to
≈	symbol for approximately equal to
∝	symbol for proportional to
≤	symbol for less than or equal to
$\int$	symbol for integration
α	coefficient of thermal expansion (/°C, /°F); angle, in a math equation (degrees, radians)
β	angle, in a math equation (degrees, radians)
β_c	ratio of long side to short side of the column in concrete footing design
β_1	coefficient for determining stress block height, *a*, based on concrete strength, f'_c
δ	elongation (in, mm) *(also see e)*
δ_P	elongation due to axial load (in, mm)
δ_s	shear deformation (in, mm)
δ_T	elongation due to change in temperature (in, mm)
Δ	beam deflection (in, mm); story drift (in, mm); an increment
Δ_{LL}	beam deflection due to live load (in, mm)
Δ_{max}	maximum calculated beam deflection (in, mm)
Δ_{TL}	beam deflection due to total load (in, mm)
ΔT	change in temperature (°C, °F)
ε	strain (no units)
ε_t	thermal strain (no units)

ϕ	diameter symbol; angle of twist (degrees, radians); resistance factor in LRFD steel design and reinforced concrete design; angle defining the shell cutoff (degrees, radians)
κ	limit of timber slenderness for intermediate length columns (no units)
λ	design constant for steel base plate design
μ	Poisson's ratio (*also see* ν*);* coefficient of static friction
ν	Poisson's ratio (*also see* μ*)*
γ	specific gravity of a material (lb/in^3, lb/ft^3, N/m^3,kN/m^3); angle, in a math equation (degrees, radians); shearing strain (no units); load factor in LRFD design
γ_D	dead load factor in LRFD steel design
γ_L	live load factor in LRFD steel design
θ	angle, in a trig equation, ex. $\sin\theta$ (degrees, radians); slope of the deflection of a beam at a point (degrees, radians)
π	pi (180°)
ρ	radial distance (in, mm); radius of curvature in beam deflection relationships (ft, m); reinforcement ratio in concrete beam design = A_s/bd (or possibly A_s/bt, A_s/bh) (no units)
ρ_b	balanced reinforcement ratio in concrete beam design
ρ_g	reinforcement ratio in concrete column design = A_{st}/A_g
ρ_{max}	maximum reinforcement ratio allowed in concrete beam design for ductile behavior
σ	engineering symbol for normal stress (axial or bending)
τ	engineering symbol for shearing stress
υ_c	shearing stress capacity in concrete design (psi; ksi, kPa, MPa);
ω	load per unit length on a beam (lb/ft, kip/ft, N/m, kN/m) (*see w*); load per unit area (lb/ft^2, kips/ft^2, N/m^2, Pa, MPa)
ω'	load per unit volume (lb/ft, kip/ft, N/m, kN/m) (*see* γ)
Σ	summation symbol
Ω	safety factor for ASD of steel (unified)
Ψ	relative stiffness of columns to beams in a rigid connection *(see G)*

Structural Glossary

Allowable strength: *Nominal strength* divided by the *safety factor.*

Allowable stress: *Allowable strength* divided by the appropriate section property, such as section modulus or cross section area.

Applicable building code: Building code und which the structure is designed.

ASD (Allowable Strength Design): Method of proportioning *structural components* such that the *allowable strength* equals or exceeds the *required strength* of the component under the action of the *ASD load combinations.*

ASD load combination: Load combination in the *applicable building code* intended for allowable strength design *(allowable stress design).*

ASTM standards: The American Society of Testing and Materials specifies standards for performance and testing of construction materials.

Axial force: A force that is acting along the longitudinal axis of a structural member.

Base shear: A lateral (wind or seismic) force acting at the base of a structure.

Beam: Structural member that has the primary function of resisting bending moments.

Beam-column: Structural member that resists both axial force and bending moment.

Bearing (local compressive yielding): *Limit state* of *local* compressive *yielding* due to the action of a member bearing against another member or surface.

Bending moment: A force rotating about a point; causes bending in beams, etc.

Block shear rupture: In a connection, *limit state* of tension fracture along one path and shear yielding or shear fracture along another path.

Bracing: Diagonal members that are used to stiffen a structure, by utilizing the inherent in-plane stiffness of a triangular framework.

Braced frame: An essentially vertical truss system that provides resistance to lateral forces and provides stability for the *structural system*.

Buckling: *Limit state* of sudden change in the geometry of a structure or any of its elements under a critical loading condition.

Buckling strength: Nominal strength for *buckling* or *instability limit states.*

Built-up member, cross-section, section, shape: Member, cross-section, section or shape fabricated from elements that are nailed, welded, glued or bolted together.

Camber: Curvature fabricated into a beam or truss so as to compensate for deflection induced by loads.

Cantilevers: Structural elements or systems that are supported only at one end.

Cement: The generic name for cementitious (binder) materials used in concrete, which is a commonly used building material.

Center of gravity: The location of resultant gravity forces on an object or objects.

Centroid: The center of mass of a shape or object.

Chord member: Primary member that extends, usually horizontally, through a truss *connection.*

Cold-rolled steel structural member: Shape manufactured by roll forming cold-or hot- rolled coils or sheets without manifest addition of heat such as would be required for hot forming.

Collector: An element that transfers load from a diaphragm to a resisting element.

Column: Structural member that has the primary function of resisting axial force.

Component (of vector): One of several vectors combined to a resultant vector.

Composite: Condition in which steel and concrete elements and members work as a unit in the distribution of internal forces.

Composite materials: Those consisting of a combination of two of more distinct materials, together yielding superior characteristics to those of the individual constituents.

Compression: A force that tends to shorten or crush a member or material.

Concentrated force: A force acting on a single point.

Concentrated load: An external concentrated force (also known as a point load).

Concrete: Material composed mainly of cement, crushed rock or gravel, sand and water.

Concrete crushing: Limit state of compressive failure in concrete having reached the ultimate strain.

Connection: A connection joins members to transfer forces or moments from one to the other.

Cope: Cutout made in a structural member to remove a flange and conform to the shape of an intersecting member.

Couple: A couple is a system of two equal forces of opposite direction offset by a distance. A couple causes a moment whose magnitude equals the magnitude of the force times the offset distance.

Cover plate: Plate welded or bolted to the flange of a member to increase cross-sectional area, section modulus or moment of inertia.

Creep: Plastic deformation that proceeds with time.

Damping: Reduces vibration amplitude, like amplitude seismic vibration.

Dead load: The weight of a structure or anything permanently attached to it.

Deflection: Deflection is the vertical moment under gravity load of beams for example, while lateral movement under wind of seismic load is called drift.

Deformation: A change of the shape of an object or material.

Design load: Applied *load* determined in accordance with either *LRFD load combinations* or *ASD load combinations*, whichever is applicable.

Design strength: Resistance factor multiplied by the *nominal strength*, øR*n*.

Design stress range: Magnitude of change in stress due to the repeated application and removal of service live *loads*. For locations subject to stress reversal it is the algebraic difference of the peak stresses.

Design stress: Design strength divided by the appropriate section property, such as section modulus or cross section area.

Determinate structure: A structure with the number of reactions equal to the number of static equations (3).

Diagonal Bracing: Inclined structural member carrying primarily axial force in a *braced fame*.

Diaphragm plate: Plate possessing in-plane shear stiffness and strength, used to transfer forces to the supporting elements.

Diaphragm: Roof, floor or other membrane or bracing system that transfers in-plane forces to the lateral force resisting system.

Displacement: May be a translation, a rotation, or a combination of both.

Distributed load: An external force which acts over a length or an area.

Double curvature: Deformed shape of a beam with one or more inflection points within the span.

Double-concentrated forces: Two equal and opposite forces that form a couple on the same side of the loaded member.

Drift: Lateral deflection of structure due to lateral wind or seismic load.

Ductibility: The capacity of a material to deform without breaking; it is measured as the ratio of total strain at failure, divided by the strain at the elastic limit.

Durability: Ability of a material, element or structure to perform its intended function for its required life without the need for replacement or significant repair, but subject to normal maintenance.

Dynamic equilibrium: Equilibrium of a moving object without change of motion.

Dynamic load: Cyclic load, such as gusty wind or seismic loads.

Effective length factor, K: Ratio between the *effective length* and the unbraced length of the member.

Effective length: Length of an otherwise identical *column* with the same strength when analyzed with pinned end conditions.

Effective net area: Net area modified to account for the effect of shear lag.

Effective section modulus: Section modulus reduced to account for buckling of slender compression elements.

Effective width: Reduced width of a plate or slab with an assumed uniform stress distribution which produces the same effect on the behavior of a structural member as the actual plate or slab with its nonuniform stress distribution.

Elastic: A material or structure is elastic if it returns to its original geometry upon unloading.

Elastic/plastic: Materials that have both an elastic zone and a plastic zone (i.e. steel).

Elastic limit: The point of a stress/strain graph beyond which deformation of a material is plastic, i.e. remains permanently deformed.

Elastic modulus: The linear slope value relating material stress to strain.

End-bearing pile: A pile supported on firm soil or rock.

Energy: The work to move a body a distance; energy is the product of forces times distance.

Epicenter: The point on the Earth's surface above the hypocenter where an earthquake originates.

Equilibrium: An object is in equilibrium if the resultant of all forces acting on it has zero magnitude.

External force: A force acting on an object; external forces are also called applied forces.

Factored load: Product of a *load factor* and the *nominal load*.

Fatigue: Limit state of crack initiation and growth resulting from repeated application of live *loads*, usually by reversing the loading direction.

Fillet weld: Weld of generally triangular cross section made between intersecting surfaces of elements.

Fitted bearing stiffener: Stiffener used at a support or concentrated *load* that fits tightly against one or both flanges of a *beam* so as to transmit load through bearing.

Fixed connection: A connection that resists axial and shear forces and bending moments.

Flexure: Bending deformation (of increasing curvature).

Flexural buckling: Buckling mode in which a compression member deflects laterally without twist or change in cross-sectional shape.

Flexural-torsional buckling: Buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

Force: Resultant of distribution of stress over a prescribed area, or an action that tends to change the shape of an object, move an object, or change the motion of an object.

Foundations: There are two basic types: ‘shallow,’ which includes pad footing, strip footings and rafts and ‘deep’ i.e. piles. The choice is a function of the strength and stiffness of the underlying strata and the load to be carried, the aim being to limit differential settlement on the structure and more importantly the finishes.

Fully restrained moment connection: Connection capable of transferring moment with negligible rotation between connected members.

Funicular: The shape of a chain or string suspended form two points under any load.

Gravity: An attractive force between objects; each object accelerates at the attractive force divided by its mass.

Groove weld: Weld in a groove between connection elements.

Gusset plate: Plate element connecting truss members of a strut or brace to a *beam* or *column*.

Hertz: Cycles per second.

Horizontal diaphragm: A floor or roof deck to resist lateral load.

Horizontal shear: Force at the interface between steel and concrete surfaces in a *composite beam*.

Indeterminate structure: A structure with more unknown reactions than static equations (3).

Inelastic: Inelastic (plastic) strain implies permanent deformation.

Inertia: Tendency of objects at rest to remain at rest and objects in motion to remain in motion.

In-plane instability: *Limit state* of a *beam-column* bent about its major axis while *lateral buckling* or *lateral-torsional buckling* is prevented by *lateral bracing*.

Instability: Limit state reached in the loading of a structural component, frame or structure in which a slight disturbance in the *loads* or geometry produces large displacements.

Internal force: The force within an object that resists external forces, also called resisting force.

Joint: Area where two or more ends, surfaces, or edges are attached. Categorized by type of *fastener* or weld used and method of force transfer.

Joist: A repetitive light beam.

K-connection: Connection in which forces in *branch members* or connecting elements transverse to the *main member* are primarily equilibrated by forces in other branch members or connecting elements on the same side of the main member.

Kern: The core of a post or other compression member which limits eccentric stresses being tensile.

Lacing: Plate, angle or other steel shape, in a lattice configuration, that connects two steel shapes together.

Lap joint: Joint between two overlapping connection elements in parallel planes.

Lateral bracing: Diagonal bracing, shear walls or equivalent means for providing in-plane lateral stability.

Lateral load resisting system: Structural system designed to resist lateral loads and provide stability for the structure as a whole.

Lateral load: Load, such as that produced by wind or earthquake effects, acting in a lateral direction.

Lateral-torsional buckling: Buckling mode of a flexural member involving deflection normal to the plane of bending occurring simultaneously with twist about the shear center of the cross-section.

Length effects: Consideration of the reduction in strength of a member based on its *unbraced length*.

Limit state: Condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (*serviceability limit state*) or to have reached its ultimate load-carrying capacity (*strength limit state*).

Linear: A structural or material behavior is linear if its deformation is directly proportional to the loading.

Line of action: The line of action defines the location and incline of a vector.

Linear elastic: A force-displacement relationship which is both linear and elastic.

Live load: Any load not permanently attached to the structure.

Load: Force or other action that results from the weight of building materials, occupants and their possessions, environmental effects, differential movement, or restrained dimensional changes.

Load effect: Forces, stresses and deformations produced in a *structural component* by the applied *loads*.

Load factor: Factor that accounts for deviations of the *nominal load* from the actual *load*, for uncertainties in the analysis that transforms the load into a *load effect* and for the probability that more than one extreme load will occur simultaneously.

Local bending: Limit state of large deformation of a flange under a concentrated tensile force.

Local buckling: Limit state of buckling of a compression element within a cross section.

Local crippling: Limit state of local failure of web plate in the immediate vicinity of a concentrated *load* or reaction.

Local yielding: Yielding that occurs in a local area of an element.

LRFD (Load and Resistance Factor Design): Method of proportioning *structural components* such that the *design strength* equals or exceeds the *required strength* of the component under the action of the *LRFD load combinations*.

LRFD load combination: Load combination in the *applicable building code* intended for strength design *(load and resistance factor design)*.

Main member: Chord member or column to which *branch members* or other connecting elements are attached.

Mass: Mass is the property of an object to resist acceleration.

Magnitude: a scalar value of physical units, such as force or displacement.

Modulus of elasticity: The proportional constant relating stress/strain of material in the linear elastic range: calculated as stress divided by strain. The modulus of elasticity is the slope of the stress-strain graph, usually denoted as E, also as Young's Modulus Y, or E-Modulus.

Moment: A force causing rotation without translation; defined as force times lever arm.

Moment of inertia: Moment of inertia is the capacity of an object to resist bending or buckling, defined as the sum of all parts of the object times the square of their distance from the centroid.

Moment connection: Connection that transmits bending moment between connected members.

Moment frame: Framing system that provides resistance to lateral loads and provides stability to the *structural system*, primarily by shear and flexure of the framing members and their connections.

Net area: Gross area reduced to account for removed material.

Nominal dimension: Designated or theoretical dimension, as in the tables of section properties.

Nominal load: Magnitude of the *load* specified by the *applicable building code*.

Nominal strength: Strength of a structure or component (without the *resistance factor* or *safety factor* applied) to resist *load effects*, as determined in accordance with this *Specification*.

Normal stress: Stress acting parallel to the axis of an object in compression and tension; normal stress is usually simply called stress.

Out-of-plane buckling: Limit state of a beam-column bent about its major axis while lateral buckling or *lateral-torsional buckling* is not prevented by lateral bracing.

Overlap connection: Connection in which intersecting *branch members* overlap.

Overturn: Topping, or tipping over under lateral load.

Permanent load: Load in which variations over time are rare or of small magnitude. All other *loads* are *variable loads*.

Pin connection: A pin connection transfers axial and shear forces but no bending moment.

Pin support: A pin support resists axial and shear forces but no bending moment.

Pitch: Longitudinal center-to-center spacing of fasteners. Center-to-center spacing bolt threads along axis of bolt.

Plastic: Material may be elastic or plastic. Plastic deformation of a structure or material under load remains after the load is removed.

Plastic analysis: Structural analysis based on the assumption of rigid-plastic behavior, in other words, that equilibrium is satisfied throughout the structure and the stress is at or below the yield stress.

Plastic hinge: Yielded zone that forms in a structural member when the *plastic moment* is attained. The member is assumed to rotate further as if hinged, except that such rotation is restrained by the *plastic moment*.

Plastic moment: Theoretical resisting moment developed within a fully yielded cross section.

Plastic stress distribution method: Method for determining the stresses in a composite member assuming that the steel section and the concrete in the cross section are fully plastic.

Plate girder: Built-up *beam*.

Plug weld: Weld made in a circular hole in one element of a joint fusing that element to another element.

Post-buckling strength: Load or force that can be carried by an element, member, or frame after initial buckling has occurred.

Pressure: Similar to stress, the force intensity at a point, except that pressure is acting on the surface of an object rather than within the object.

Prying action: Amplification of the tension force in a bolt caused by leverage between the point of applied load, the bolt and the reaction of the connected elements.

Punching load: Component of *branch member* force perpendicular to a *chord*.

P-δ effect: Effect of *loads* acting on the deflected shape of a member between joints or nodes.

P-Δ effect: Effect of *loads* acting on the displaced location of joints or nodes in a structure. In tiered building structures, this is the effect of *loads* acting on the laterally displaced location of floors and roofs.

Radius of gyration: A mathematical property, determining the stability of a cross section, defined as: $r = \sqrt{I/A}$, where I = moment of inertia and A = cross section area.

Reaction: The response of a structure to resist applied load.

Required strength: Forces, stresses and deformations acting on the *structural component*, determined by either *structural analysis*, for the *LRFD* or *ASD load combinations*, as appropriate, or as specified by the *Specification* or Standard.

Resilience: The property of structures to absorb energy without permanent deformation of fracture.

Resistance factor ϕ*:* Factor that accounts for unavoidable deviations of the *nominal strength* from the actual strength and for the manner and consequences of failure.

Resultant: The resultant of a system of forces is a single force or moment whose magnitude, direction, and location make it statically equivalent to the system of forces.

Retaining wall: Wall used to hold back soil or other materials.

Roller support: In two dimensions, a roller support restrains one translation degree of freedom.

Rupture strength: In a *connection*, strength limited by tension or shear rupture.

Safety factor: Factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual *load* from the *nominal load*, uncertainties in the analysis that transforms the load into a *load effect*, and for the manner and consequence of failure.

Scalar: A mathematical entity with a numeric value but no direction (in contrast to a vector).

Section modulus: The property of a cross section defined by its shape and orientation; section modulus is denoted S, and S = I/c, where I = moment of inertia about the centroid and c is the distance from the centroid to the edge of the section,

Service load combination: Load combination under which serviceability limit states are evaluated.

Service load: Load under which *serviceability limit states* are evaluated.

Serviceability limit state: Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability or the comfort of its occupants or function of machinery, under normal usage.

Shear: A sliding force, pushing and pulling in opposite directions.

Shear buckling: Buckling mode in which a plate element, such as the web of a beam, deforms under pure shear applied in the plane of the plate.

Shear connector: Headed stud, cannel, plate or other shape welded to a steel member and embedded in concrete of a *composite member* to transmit shear forces at the interface between the two materials.

Shear connector strength: Limit state of reaching the strength of a *shear connector*, as governed by the connector bearing against the concrete in the slab or by the *tensile strength* of the connector.

Shear modulus: The ratio of shear stress divided by the shear strain in a linear elastic material.

Shear rupture: Limit state of *rupture (fracture) due to shear*.

Shear strain: Strain measuring the intensity of racking in a material. Shear strain is measured as the change in angle of a small square part of a material.

Shear stress: Stress acting parallel to an imaginary plane cut through an object.

Shear wall: Wall that provides resistance to lateral loads in the plane of the wall and provides stability for the structural system.

Shear yielding: Yielding that occurs due to shear.

Shear yielding (punching): In a connection, *limit state* based on out-of-plane shear strength of the *chord* wall to which *branch members* are attached.

Slip: In a bolted connection, *limit state* of relative motion of connected parts prior to the attainment of the *available strength* of the connection.

Slip-critical connection: Bolted *connection* designed to resist movement by friction on the faying surface of the connection under the clamping forces of the bolts.

Slot weld: Weld made in an elongated hole fusing an element to another element.

Splice: Connection between two structural elements joined at their ends to forma single, longer element.

Stability: Condition reached in the loading of a structural component, frame or structure in which a slight disturbance in the *loads* or geometry does not produce large displacements.

Static equilibrium: Equilibrium of an object at rest.

Stiffness: The capacity of a material to resist deformation.

Stiffened element: Flat compression element with adjoining out-of-plane elements along both edges parallel to the direction of loading.

Stiffener: Structural element, usually an angle or plate, attached to a *member* to distribute *load*, transfer shear or prevent buckling.

Stiffness: Resistance to deformation of a member or structure, measured by the ratio of the applied force (or moment) to the corresponding displacement (or rotation).

Strain: Change of length along an axis, calculated as $\varepsilon = \Delta L/L$, where L is the original length and ΔL is the change of length.

Strength: The capacity of a material to resist breaking.

Strength design: A design method based on factored load and ultimate strength for concrete design.

Strength limit state: Limiting condition affecting the safety of the structure, in which the ultimate load-carrying capacity is reached.

Stress: Force per unit area caused by axial force, moment, shear or torsion.

Stress concentration: Localized stress considerably higher than average (even in uniformly loaded cross sections of uniform thickness) due to abrupt changes in geometry or localized loading.

Stress resultant: A system of forces which is statically equivalent to a stress distribution over an area.

Stress: The internal reaction to an applied force, measured in force per unit area.

Structure: Composition of elements that define form and resist applied loads.

Structural Aluminum: Elements manufactured of aluminum for structural purposes, generally 50% larger than comparable steel elements due to the lower *modulus of elasticity.*

Structural Steel: Elements manufactured of steel with properties designated by *ASTM standards*, including A36, A992 & A572.

Strong axis: Major principal centroidal axis of a cross section.

Structural analysis: Determination of *load effects* on members and *connections* based on principles of structural mechanics.

Structural component: Member, connector, connecting element or assemblage.

Structural system: An assemblage of load-carrying components that are joined together to provide interaction or interdependence.

T-connection: Connection in which the *branch member* or connecting element is perpendicular to the *main member* and in which forces transverse to the main member are primarily equilibrated by shear in the main member.

Tensile rupture: Limit state of rupture (fracture) due to tension.

Tensile strength (of material): Maximum tensile stress that a material is capable of sustaining as defined by ASTM.

Tensile strength (of member): Maximum tension force that a member is capable of sustaining.

Tensile yielding: Yielding *that occurs due to tension*.

Tension: A force that tends to elongate or enlarge an object.

Tension and shear rupture: In a bolt, *limit state* of rupture (fracture) due to simultaneous tension and shear *force*.

Tie plate: Plate element used to join parallel components of a *built-up column*, girder or strut rigidly connected to the parallel components and designed to transmit shear between them.

Torsion: A twisting moment.

Torsional bracing: Bracing resisting twist of a *beam* or *column*.

Torsional buckling: Buckling mode in which a compression member twists about its shear center axis.

Torsional yielding: Yielding that occurs due to torsion.

Translation: Motion of an object along a straight line path without rotation.

Transverse reinforcement: Steel reinforcement in the form of closed ties or welded wire fabric providing confinement for the concrete surrounding the steel shape core in an *encased concrete composite column*.

Transverse stiffener: Web *stiffener* oriented perpendicular to the flanges, attached to the web.

Truss: A linear support system consisting of triangular panels usually with pin joints.

Ultimate strength: The utmost strength reached by a material before breaking.

Unbraced length: Distance between braced points of a member, measured between the centers of gravity of the bracing members.

Uneven load distribution: In a *connection*, condition in which the load is not distributed through the cross section of connected elements in a manner that can be readily determined.

Unframed end: The end of a member not restrained against rotation by stiffeners of connection elements.

Unstiffened elements: Flat compression element with an adjoining out-of-plane element along one edge parallel to the direction of loading.

Uplift: Upward force, usually wind uplift.

Variable load: Load not classified as *permanent load*.

Vector: A mathematical entity having a magnitude, line of action, and a direction in space.

Vertical bracing system: System of *shear walls, braced frames* or both, extending through one or more floors of a building.

Vertical diaphragm: A wall to resist lateral load.

Vibration: The cyclic motion of an object.

Wall: A vertical element to resist load and define space; shear walls also resist lateral loads.

Weak axis: Minor principal centroidal axis of a cross section.

Web buckling: Limit state of lateral instability of a web.

Web compression buckling: Limit state of out-of-plane compression buckling of the web due to a concentrated compression force.

Web sideway buckling: Limit state of lateral buckling of the tension flange opposite the location of a concentrated compression force.

Weld metal: Portion of a fusion weld that has been completely melted during welding. Weld metal has elements of filler metal and base metal melted in the weld thermal cycle.

Working stress: The same as allowable stress.

Yield moment: In a member subjected to bending, the moment at which the extreme outer fiber first attains the *yield stress*.

Yield point: First stress in a material at which an increase in strain occurs without an increase in stress as defined by ASTM.

Yield strength: Stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain as defined by ASTM.

Yield strain: The strain of a material which occurs at the level of yield stress.

Yield stress: Generic term to denote either *yield point* or *yield strength*, as appropriate for the material.

Yielding: Limit state of inelastic deformation that occurs after the *yield stress* is reached.

Yielding (plastic moment): Yielding throughout the cross section of a member as the bending moment reaches the *plastic moment*.

Yielding (yield moment): Yielding at the extreme fiber on the cross section of a member when the bending moment reached the *yield moment*.

References:

AISC, *Specifications for Structural Steel Buildings*, 13th ed. (2005)
Jacqueline Glass, *Encyclopaedia of Architectural Technology,* Wiley, Cornwall (2002)

Statics Primer

Notation:

a = name for acceleration
A = area (net = with holes, bearing = in contact, etc...)
(C) = shorthand for *compression*
d = perpendicular distance to a force from a point
d_x = difference in the x direction between an area centroid ($\bar{x}$) and the centroid of the composite shape ($\hat{x}$)
d_y = difference in the y direction between an area centroid ($\bar{y}$) and the centroid of the composite shape ($\hat{y}$)
F = name for force vectors, as is *A, B, C, T and P*
F_x = force component in the x direction
F_y = force component in the y direction
g = acceleration due to gravity
h = name for height
$\bar{I}$ = moment of inertia about the centroid
I_x = moment of inertia with respect to an x-axis
I_y = moment of inertia with respect to a y-axis
L = beam span length
m = name for mass
M = moment due to a force or internal bending moment
N = name for normal force to a surface
p = pressure
Q_x = first moment area about an x axis (using y distances)
Q_y = first moment area about an y axis (using x distances)
R = name for resultant vectors
R_x = resultant component in the x direction
R_y = resultant component in the y direction
tail = start of a vector (without arrowhead)
tip = direction end of a vector (with arrowhead)
(T) = shorthand for *tension*
V = internal shear force
w = name for distributed load
$w_{s(elf)}$ $w_{(t)}$ = name for distributed load from self weight of member
W = name for force due to weight
x = x axis direction or algebra variable
$\bar{x}$ = the distance in the x direction from a reference axis to the centroid of a shape
y = y axis direction or algebra variable
$\bar{y}$ = the distance in the y direction from a reference axis to the centroid of a shape
α = angle, in math
β = angle, in math
γ = angle, in math
μ = coefficient of static friction
θ = angle, in a trig equation, ex. $\sin\theta$, that is measured between the x axis and *tail* of a vector
Σ = summation symbol

Newton's Laws of Motion

Newton's laws govern the behavior of physical bodies, whether at rest or moving:

- **First Law.** *A particle originally at rest, or moving in a straight line with constant velocity, will remain in this state provided the particle is not subjected to an unbalanced force.*
- **Second Law.** *A particle of mass* **m** *acted upon by an unbalanced force experiences an acceleration that has the same direction as the force and a magnitude that is directly proportional to the force. This is expressed mathematically as:* $\overline{F} = m\overline{a}$,

where **F** *and* **a** *are vector (directional) quantities, and* **m** *is a scalar quantity.*

- **Third Law.** *The mutual forces of action and reaction between two particles are equal, opposite, and collinear.*

Units

Units are necessary to define quantities. Standards exist to relate quantities in a convention system, such as the International System of Units (SI) or the U.S. Customary system.

Units	Mass	Length	Time	Force
SI	kg	m	s	$N = \frac{kg \cdot m}{s^2}$
Absolute English	lb	ft	s	$Poundal = \frac{lb \cdot ft}{s^2}$
Technical English	$slug = \frac{lb_f \cdot s^2}{ft}$	ft	s	lb_{force}
Engineering English	lb	ft	s	lb_{force}

$$lb_{force} = lb_{(mass)} \times 32.17\, {}^{ft}\!/_{s^2}$$

gravitational constant	$g_c = 32.17\, {}^{ft}\!/_{s^2}$	(English)	$F=mg$
	$g_c = 9.81\, {}^{m}\!/_{s^2}$	(SI)	
conversions (pg. vii)	$1\,in = 25.4\,mm$ $1\,lb = 4.448\,N$		

Conversions

Conversion of a quantity from a category within a unit system to a more useful category or to another unit system is very common. Tables of conversion can be found in most physics, statics and design texts.

Numerical Accuracy

Depends on 1) accuracy of data you are given

2) accuracy of the calculations performed

The solution CANNOT be more accurate than the less accurate of #1 and #2 above!

DEFINITIONS: *precision* the number of significant digits
accuracy the possible error

Relative error measures the degree of accuracy: $\frac{relative\ \ error}{measurement} \times 100 = degree\ \ of\ \ accuracy\ \ (\%)$

For engineering problems, accuracy *rarely* is less than 0.2%.

Math for Structures

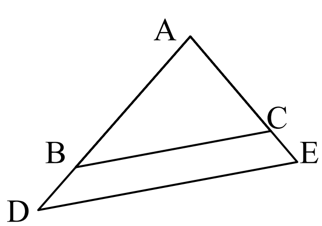
1. Parallel lines never intersect.
2. Two lines are *perpendicular* (or *normal*) when they intersect at a right angle = 90°.
3. *Intersecting* (or *concurrent)* lines cross or meet at a point.
4. If two lines cross, the opposite angles are identical:
5. If a line crosses two parallel lines, the intersection angles with the same orientation are identical:
6. If the sides of two angles are parallel and intersect in the same fashion, the angles are identical.
7. If the sides of two angles are parallel, but intersect in the opposite fashion, the angles are *supplementary*: $\alpha+\beta = 180°$.
8. If the sides of two angles are perpendicular and intersect in the same fashion, the angles are identical.
9. If the sides of two angles are perpendicular, but intersect in the opposite fashion, the angles are *supplementary*: $\alpha+\beta = 180°$.
10. If the side of two angles bisects a right angle, the angles are *complimentary*: $\alpha+\gamma = 90°$.
11. If a right angle bisects a straight line, the remaining angles are *complimentary*: $\alpha+\gamma = 90°$.
12. The sum of the interior angles of a triangle = 180°.
13. For a right triangle, that has one angle of 90°, the sum of the other angles = 90°.
14. For a right triangle, the sum of the squares of the sides equals the square of the

$$AB^2 + AC^2 = CB^2$$

hypotenuse:

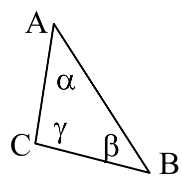
15. Similar triangles have identical angles in the same orientation. Their sides are related by:

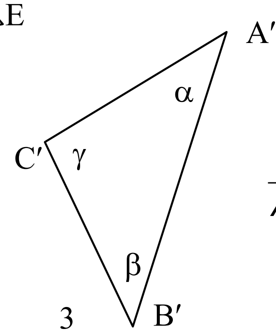
Case 1:



$$\frac{AB}{AD} = \frac{AC}{AE} = \frac{BC}{DE}$$

Case 2:





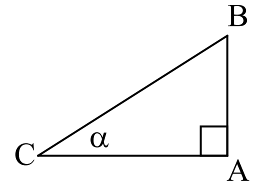
$$\frac{AB}{A'B'} = \frac{AC}{A'C'} = \frac{BC}{B'C'}$$

16. For right triangles:

$$sin = \frac{opposite\ side}{hypotenuse} = sin\,\alpha = \frac{AB}{CB}$$

$$cos = \frac{adjacent\ side}{hypotenuse} = cos\,\alpha = \frac{AC}{CB}$$

$$tan = \frac{opposite\ side}{adjacent\ side} = tan\,\alpha = \frac{AB}{AC}$$

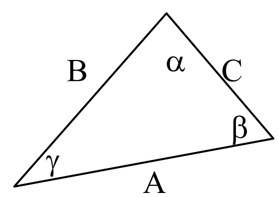


(SOHCAHTOA)

17. If an angle is greater than 180° and less than 360°, *sin* will be less than 0.
If an angle is greater than 90° and less than 270°, *cos* will be less than 0.
If an angle is greater than 90° and less than 180°, *tan* will be less than 0.
If an angle is greater than 270° and less than 360°, *tan* will be less than 0.

18. LAW of SINES (any triangle)

$$\frac{\sin\alpha}{A} = \frac{\sin\beta}{B} = \frac{\sin\gamma}{C}$$



19. LAW of COSINES (any triangle)

$$A^2 = B^2 + C^2 - 2BC\cos\alpha$$

20. Surfaces or areas have dimensions of width and length and units of length *squared* (ex. in^2 or inches x inches).

21. Solids or volumes have dimension of width, length and height or thickness and units of length *cubed* (ex. m^3 or m x m x m)

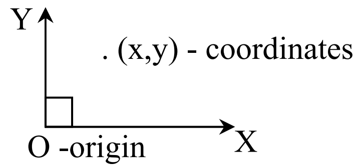
22. Force is defined as mass times acceleration. So a weight due to a mass is accelerated upon by gravity: F = m·g $g = 9.81\,{}^{m}\!/\!_{sec^2} = 32.17\,{}^{ft}\!/\!_{sec^2}$

23. Weight can be determined by volume if the unit weight or *density* is known: W = V·γ where ·V is in units of $length^3$ and γ is in units of force/unit volume

24. Algebra: If a·b= c·d then it can be rewritten:

$a \cdot b + k = c \cdot d + k$	add a constant
$c \cdot d = a \cdot b$	switch sides
$a = \frac{c \cdot d}{b}$	divide both sides by *b*
$\frac{a}{c} = \frac{d}{b}$	divide both sides by *b·c*

25. Cartesian Coordinate System



26. Solving equations with one unknown:

1st order polynomial: $2x-1=0\cdots$ $2x=1\cdots$ $x=\frac{1}{2}$

$ax+b=0\cdots$ $x=\dfrac{-b}{a}$

2nd order polynomial

$ax^2+bx+c=0\cdots$ $x=\dfrac{-b\pm\sqrt{b^2-4ac}}{2a}$ two answers (radical *cannot* be negative)

$x^2-1=0\cdots$
$(a=1, b=0, c=-1)$ $x=\dfrac{-0\pm\sqrt{0^2-4(-1)}}{2\cdot 1}\cdots$ $x=\pm 1$

27. Solving 2 linear equations simultaneously:
One equation consisting only of variables can be rearranged and then substituted into the second equation:

ex:	$5x-3y=0$	add 3y to both sides to rearrange	$5x=3y$
	$4x-y=2$	divide both sides by 5	$x=\frac{3}{5}y$
		substitute x into the other equation	$4(\frac{3}{5}y)-y=2$
		add like terms	$\frac{7}{5}y=2$
		simplify	$\boxed{y=\frac{10}{7}=1.43}$

Equations can be added and factored to eliminate one variable:

ex:	$2x+3y=8$		$2x+3y=8$
	$4x-y=2$	multiply both sides by 3	$\underline{12x-3y=6}$
		and add	$14x+0=14$
		simplify	$\boxed{x=1}$
		put x=1 in an equation for y	$2\cdot 1+3y=8$
		simplify	$3y=6$
			$\boxed{y=2}$

28. Derivatives of polynomials

$$y = constant \qquad \frac{dy}{dx} = 0$$

$$y = x \qquad \frac{dy}{dx} = 1$$

$$y = ax \qquad \frac{dy}{dx} = a$$

$$y = x^2 \qquad \frac{dy}{dx} = 2x$$

$$y = x^3 \qquad \frac{dy}{dx} = 3x^2$$

29. The minimum and maximum of a function can be found by setting the derivative = 0 and solving for the unknown variable.

30. Calculators (and software) process equations by an "order of operations", which typically means they process functions like exponentials and square roots before simpler functions such as + or -. BE SURE to specify with parenthesis what order you want, or you'll get the wrong answers. It is also important to have degrees set in your calculator for trig functions.

For instance, Excel uses – for sign (like -1) first, then will process exponents and square roots, times and divide, followed by plus and minus. If you type 4x10^2 and really mean (4x10)^2 you will get an answer of 400 instead of 1600.

<u>General Procedure for Analysis</u>

1. Inputs
Outputs
"Critical Path" $\Longrightarrow$ <u>GIVEN:</u>
<u>FIND:</u>
<u>SOLUTION</u> } *on graph paper*

2. Draw simple diagram of body/bodies & forces acting on it/them.

3. Choose a reference system for the forces.

4. Identify key geometry and constraints.

5. Write the basic equations for force components.

6. Count the equations & unknowns.

7. SOLVE

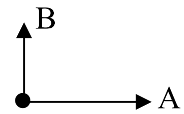
8. "Feel" the validity of the answer. (Use common sense. Check units…)

Example: Two forces, A & B, act on a particle. What is the resultant?

1. GIVEN: Two forces on a particle and a diagram with size and orientation

   FIND: The "resultant" of the two forces

   SOLUTION:



2. Draw what you know (the diagram, any other numbers in the problem statement that could be put on the drawing….)

3. Choose a reference system. What would be the easiest? Cartesian, radian?

4. Key geometry: the location of the particle as the origin of all the forces
   Key constraints: the particle is "free" in space

5. Write equations:
$$\textit{size of } A^2 + \textit{size of } B^2 = \textit{size of resultant}$$
$$\sin\alpha = \frac{\textit{size of } B}{\textit{size of } A + B}$$

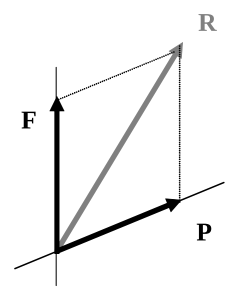
6. Count: Unknowns: 2, magnitude and direction $\leq$ Equations: 2 $\therefore$ can solve

7. Solve: graphically or with equations

8. "Feel": Is the result bigger than A and bigger than B? Is it in the right direction? (like A & B)

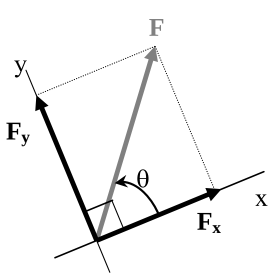
Forces

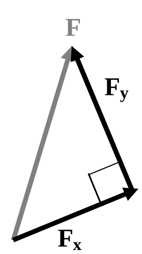
Forces are vectors, which means they have a direction, size and point or line of application. External forces can be moved along the line of action by the *law of transmissibility*. Internal forces are within a material or at a connection between elements.



Force systems can be classified as *concurrent, collinear, coplanar, coplanar-parallel,* or *space.*

Because they are vector quantities, they cannot be simply added. They must be *graphically* added or analytically added by resolving forces into *components* using trigonometry and summed.





θ is: *between x & F*

$F_x = F \cdot \cos\theta$

$F_y = F \cdot \sin\theta$

$F = \sqrt{F_x^2 + F_y^2}$

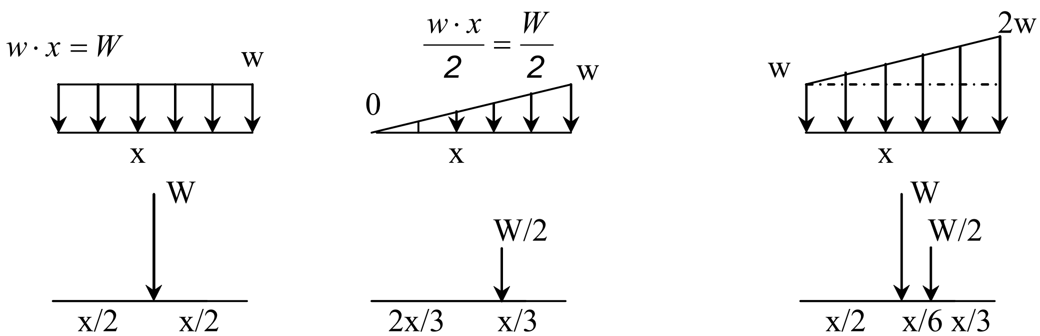
$\tan\theta = \dfrac{F_y}{F_x}$

$$R_x = \sum F_x\ , \quad R_y = \sum F_y \quad \text{and} \quad R = \sqrt{R_x^2 + R_y^2} \qquad \tan\theta = \frac{R_y}{R_x}$$

Types of Forces

Forces can be classified as *concentrated* at a point or *distributed* over a length or area. *Uniformly distributed* loads are quite common and have units of lb/ft or N/m. The total load is commonly wanted from the distribution, and can be determined based on an "area" calculation with the load value as the "height".

Equivalent force systems are the reorganization of the loads in a system so there is a equivalent force put at the same location that would cause the same translation **and** rotation (see Moments).

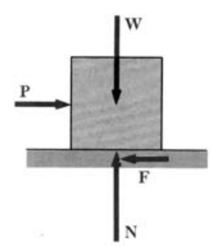


To determine a distributed load due to hydrostatic pressure, the height of the water, *h,* is multiplied by the material density, γ (62.4 lb/ft^3): $p = h\gamma$.

To determine a weight of a beam member per length, the cross section area, *A*, is multiplied by the material density, γ (ex. concrete = 150 lb/ft^3): $w_{s.w.} = A\gamma$. *(Care must be taken with units.)*

Friction

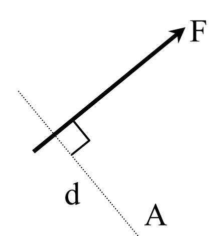
Friction is a resulting force from the contact of two materials and a normal force. It can be *static* or *kinematic.* Static friction is defined as the product of the normal force, *N*, with the coefficient of static friction, μ, which is a constant dependant upon the materials in contact: $F = \mu N$



Moments

Moments are the tendency of forces to cause rotation and are *vector* quantities with rotational direction. Most physics texts define positive rotation as *counter clockwise.* With the sign convention, moments can be added.

Moments are defined as the product of the force magnitude, *F*, with the perpendicular distance from the point of interest to the line of action of the force, $d_\perp$: $M = F \cdot d_\perp$



Moment *couples* can be identified with forces of equal size in opposite direction that are *parallel.* The equations is $M = F \cdot d_\perp$ where F is the size of *one* of the forces.

Support Conditions

Reaction forces and moments occur at supports for structural elements. The force component directions and moments are determined by the motion that is resisted, for example no rotation will mean a reaction moment. Supports are commonly modeled as these general types, with the drawing symbols of triangles, circles and ground:

Structural Analysis, 4th ed., R.C. Hibbeler

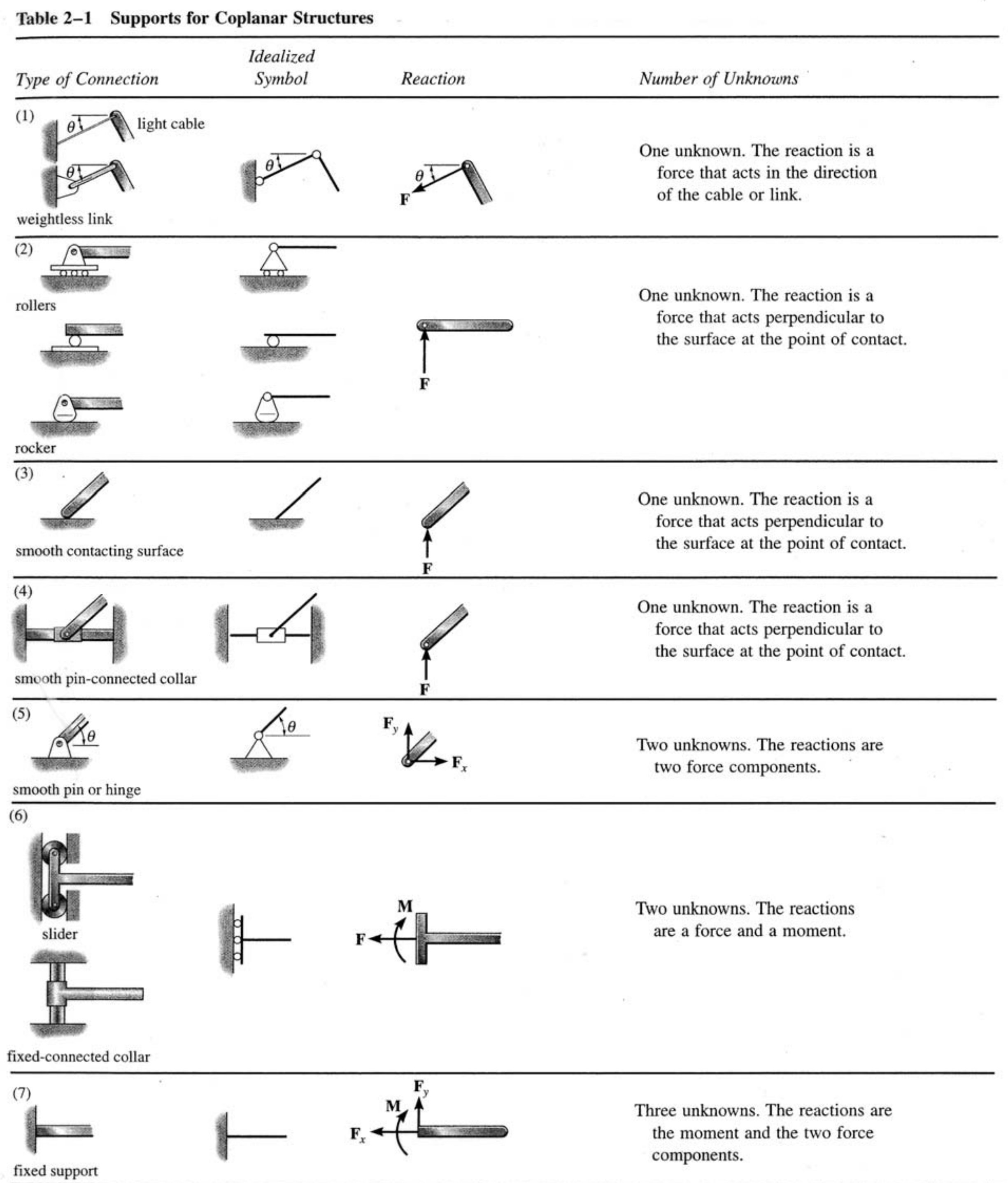


Table 2–1 Supports for Coplanar Structures

Type of Connection	Idealized Symbol	Reaction	Number of Unknowns
(1) light cable; weightless link	θ	θ, F	One unknown. The reaction is a force that acts in the direction of the cable or link.
(2) rollers; rocker		F	One unknown. The reaction is a force that acts perpendicular to the surface at the point of contact.
(3) smooth contacting surface		F	One unknown. The reaction is a force that acts perpendicular to the surface at the point of contact.
(4) smooth pin-connected collar		F	One unknown. The reaction is a force that acts perpendicular to the surface at the point of contact.
(5) smooth pin or hinge	θ	F_y, F_x	Two unknowns. The reactions are two force components.
(6) slider; fixed-connected collar		M, F	Two unknowns. The reactions are a force and a moment.
(7) fixed support		F_y, M, F_x	Three unknowns. The reactions are the moment and the two force components.

Equilibrium

Equilibrium is the state when all the external forces acting on a rigid body form a system of forces equivalent to zero. There will be no rotation or translation. The forces are referred to as balanced.

$$R_x = \sum F_x = 0 \qquad R_y = \sum F_y = 0 \qquad \textbf{AND} \qquad \sum M = 0$$

Equilibrium for a point already satisfies the sum of moments equal to zero because a force acting through a point will have zero moment from a zero perpendicular distance. This is a very useful concept to apply when summing moments for a rigid body. If the point summed about has unknown forces acting through it, that force variable will not appear in the equilibrium equation as an unknown quantity, allowing for much easier algebra.

Free Body Diagrams

1. Determine the free body of interest. (What body is in equilibrium?)

2. Detach the body from the ground and all other bodies (*"free" it).*

3. Indicate all external forces which include:

   - action on the free body by the **supports & connections**

   - action on the free body by other bodies

   - the weigh effect (=force) of the free body itself (force due to gravity)

4. All forces should be clearly marked with magnitudes and direction. The sense of forces should be those acting *on the body* not by the body.

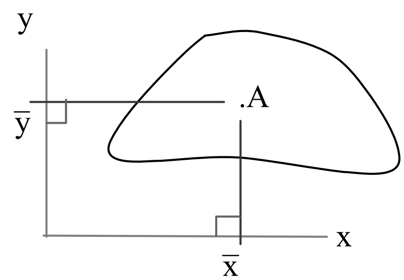
5. Dimensions/angles should be included for moment computations and force computations.

6. Indicate the unknown angles, distances, forces or moments, such as those reactions or constraining forces where the body is supported or connected.

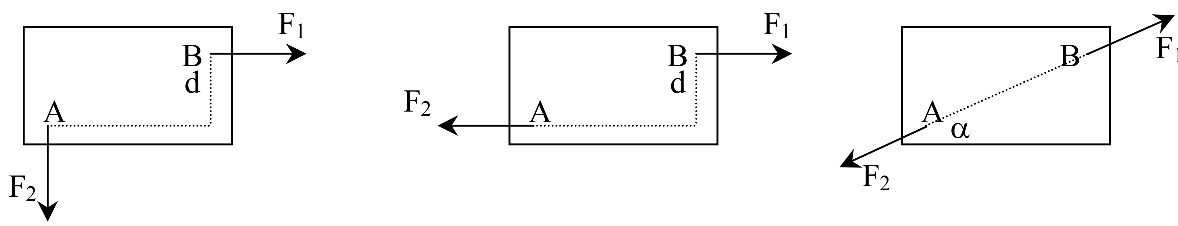
- The line of action of any unknown should be indicated on the FBD. The sense of direction is determined by the type of support. (Cables are in tension, etc…) *If the sense isn't obvious, assume a sense.* When the reaction value comes out positive, the assumption was correct. When the reaction value comes out negative, the direction is *opposite* the assumed direction. *DON'T CHANGE THE ARROWS ON YOUR FBD OR SIGNS IN YOUR EQUATIONS.*

With the 3 equations of equilibrium, there can be no more than 3 unknowns for statics. If there are, and the structure is stable, it means that it is *statically indeterminate* and other methods must be used to solve the unknowns. When it is not stable, it is *improperly constrained* and may still look like it has 3 unknowns. It will prove to be unsolvable.

Conditions for Equilibrium of a Rigid Body

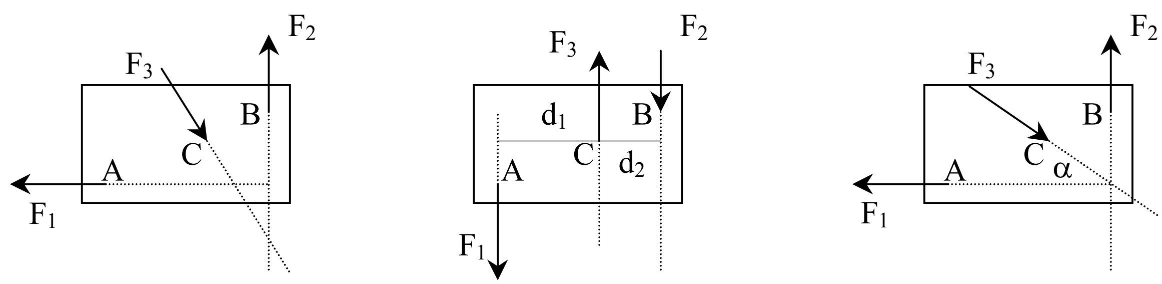


1. **Two-force body**: Equilibrium of a body subjected to two forces on two points requires that those forces be **equal** and **opposite** and act in the same line of action.



(A) (B) (C)

2. **Three-force body**: Equilibrium of a body subjected to three forces on three points requires that the line of action of the forces be concurrent (intersect) or parallel AND that the resultant equal zero.



(A) -no (B) (C)

Geometric Properties

Area is an important quantity to be calculated in order to know material quantities and to find geometric properties for beam and column cross sections. Charts are available for common mathematical relationships.

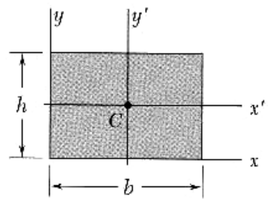
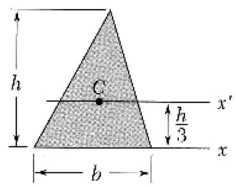
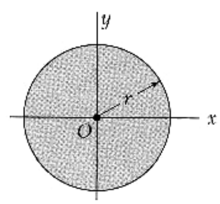
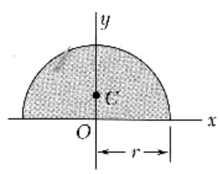
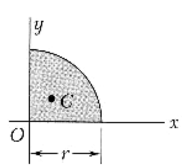
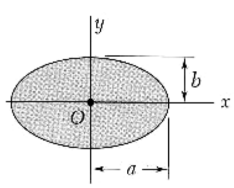
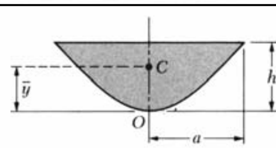
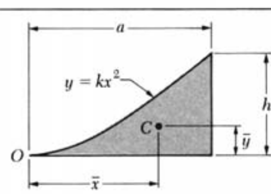
Centroid For a uniform material, the geometric center of the area is the *centroid* or center of gravity. It can be determined with calculus.

$$\bar{x} = \frac{\Sigma(x\Delta A)}{A} \qquad \bar{y} = \frac{\Sigma(y\Delta A)}{A}$$

First Moment Area The product of an area with respect to a distance about an axis is called the first moment area, *Q*. The quantity is useful for shear stress calculations and to determine the moment of inertia.

$$Q_x = \int y dA = \bar{y}A \qquad Q_y = \int x dA = \bar{x}A$$

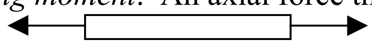
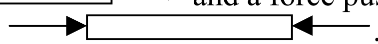
Geometric Properties of Areas

Rectangle		$\bar{I}_{x'} = \frac{1}{12}bh^3$ $\bar{I}_{y'} = \frac{1}{12}b^3h$ $I_x = \frac{1}{3}bh^3$ $I_y = \frac{1}{3}b^3h$ $J_C = \frac{1}{12}bh(b^2 + h^2)$	Area = bh $\bar{x}$ = b/2 $\bar{y}$ = h/2
Triangle		$\bar{I}_{x'} = \frac{1}{36}bh^3$ $I_x = \frac{1}{12}bh^3$ $\bar{I}_{y'} = \frac{1}{36}b^3h$	Area = $bh/2$ $\bar{x} = b/3$ $\bar{y} = h/3$
Circle		$\bar{I}_x = \bar{I}_y = \frac{1}{4}\pi r^4$ $J_O = \frac{1}{2}\pi r^4$	Area = $\pi r^2 = \pi d^2/4$ $\bar{x}$ = 0 $\bar{y}$ = 0
Semicircle		$\bar{I}_x = 0.1098r^4$ $\bar{I}_y = \pi r^4/8$	Area = $\pi r^2/2 = \pi d^2/8$ $\bar{x}$ = 0 $\bar{y} = 4r/3\pi$
Quarter circle		$\bar{I}_x = 0.0549r^4$ $\bar{I}_y = 0.0549r^4$	Area = $\pi r^2/4 = \pi d^2/16$ $\bar{x} = 4r/3\pi$ $\bar{y} = 4r/3\pi$
Ellipse		$\bar{I}_x = \frac{1}{4}\pi ab^3$ $\bar{I}_y = \frac{1}{4}\pi a^3b$ $J_O = \frac{1}{4}\pi ab(a^2 + b^2)$	Area = πab $\bar{x}$ = 0 $\bar{y}$ = 0
Semiparabolic area Parabolic area		$\bar{I}_x = 16ah^3/175$ $\bar{I}_y = 4a^3h/15$	Area = $4ah/3$ $\bar{x}$ = 0 $\bar{y} = 3h/5$
Parabolic spandrel		$\bar{I}_x = 37ah^3/2100$ $\bar{I}_y = a^3h/80$	Area = $ah/3$ $\bar{x} = 3a/4$ $\bar{y} = 3h/10$

Moment of Inertia The moment of inertia is the second area moment of an area, and is found using calculus. For a composite shape, the moment of inertia can be found using the *parallel axis theorem*: $I_x = \bar{I}_x + Ad_y^{\,2}$ $I_y = \bar{I}_y + Ad_x^{\,2}$

The theorem states that the sum of the centroid of each composite shape about an axis (subscript axes) can be added but must be added to the second moment area of the shape by the distance between parallel axes (opposite axes direction).

Internal Forces

If a body is in equilibrium, it holds that any *section* of that body is in equilibrium. *Two force-bodies* will have internal forces that are *in line* with the body (end points), while *three-force bodies* will see an internal force that will not be axial, in addition to an internal moment called a *bending moment*. An axial force that is pulling the body from both ends is referred to as a *tensile* force,  and a force pushing on the body at both ends is referred to as a *compressive* force .

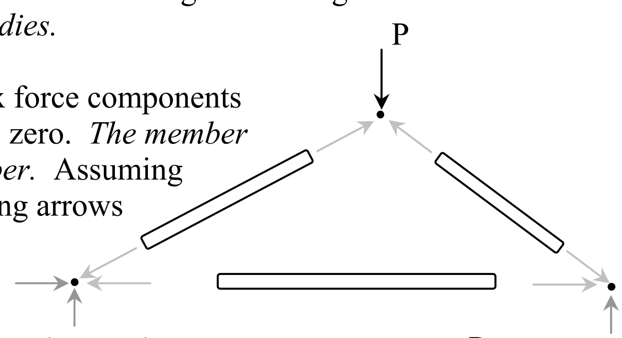
Cable Analysis

Cables can only see tensile forces. If cables are straight, they are two-force bodies and the geometry of the cable determines the direction of the force.

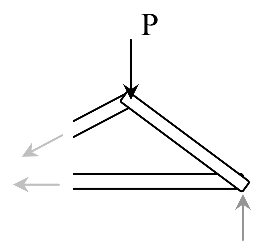
If cables drape (*are funcular)* by having distributed or gravity loads, the internal vertical force component changes, while the internal horizontal force component does not.

Truss Analysis

Truss members are assembled such that the pins connecting them are the only location of forces (internal and external). This loading assumption relies on there being no bending in the members, and all truss members are then *two-force bodies.*



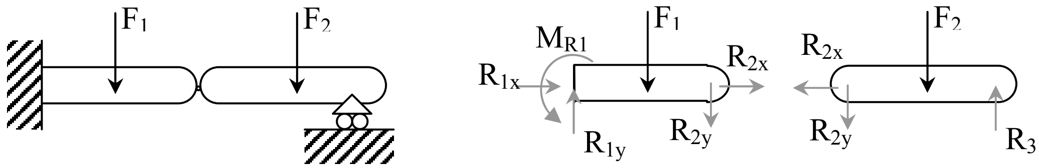
Equilibrium of the joints will only need to satisfy the x force components summing to 0 and the y force components summing to zero. *The member forces will have direction in the geometry of the member.* Assuming the unknown forces in tension is represented by drawing arrows "away" from the joint. When compression forces are known, they must be drawn "in" to the point.



Equilibrium of the section will only be possible if the section cut is through three or less members exposing three or less unknown forces. This method relies on the sum of moment equilibrium equation. The member forces are in the direction of the members, and the line of action of those forces runs through the member location in order to find the perpendicular distance. It is helpful to find points of intersection of unknown forces to sum moments.

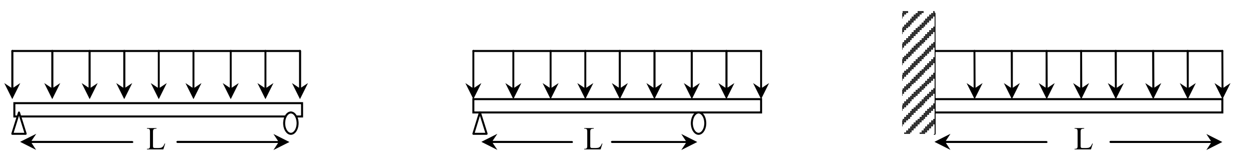
Pinned Frame. Arch and Compound Beam Analysis

Connecting or "internal" pins, mean a frame is made up of multiple bodies, just like a truss. But unlike a truss, the member will not all be *two-force bodies*, so there may be three equations of equilibrium required for each member in an assembly, in addition to the three equations of equilibrium for the entire structure. The force reactions on one side of the pin are equal and opposite those to the other side, so there are only two unknown component forces per pin.



Beam Analysis

Statically determinate beams have a limited number of support arrangements for a limit of three unknown reactions. The cantilever condition has a *reaction moment.*

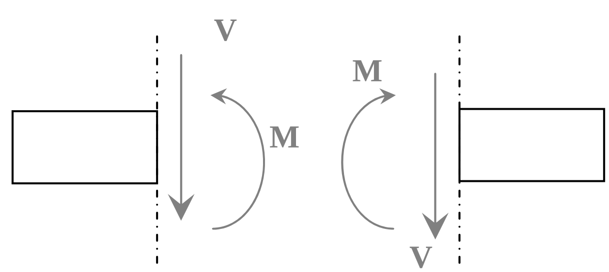


simply supported (most common)   overhang   cantilever

The internal forces and moment are particularly important for design. The axial force *(*commonly equal zero) is labeled *P,* while the transverse force is called *shear*, *V*, and the internal moment is called *bending moment*, *M*.

The sign convention for *positive shear* is a downward force on a left section cut (or upward force on a right section cut).

The sign convention for *positive bending moment* corresponds to a downward deflection (most common or positive curvature.) That is a *counter clockwise* moment on a right section cut and a *clockwise* moment of a left section cut.

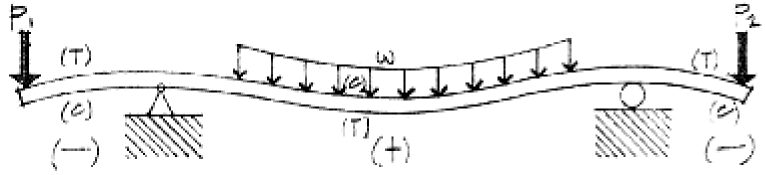


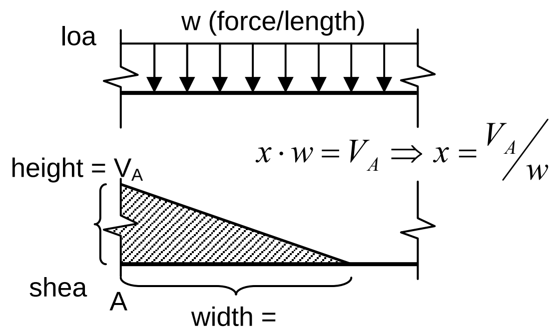
Shear and Bending Moment Diagrams

Diagrams of the internal shear at every location along the beam and of the internal bending moment are extremely useful to locate maximum quantities to design the beams for. There are two primary methods to construct them. The *equilibrium method* relies on section cuts over distances and writes expressions based on the variable of distance. These functions are plotted as lines or curves. The *semi-graphical method* relies on the calculus relationship between the "load" curve (or *load diagram)*, shear curve, and bending moment curve. If the area under a curve is known, the result in the next plot is a *change* by the amount of the area.

The location of the **maximum bending moment** corresponds to the location of **zero shear.**

On the deflected shape of a beam, the point where the shape changes from smile up to frown is called the ***inflection*** *point.* The bending moment value at this point is **zero.**





Semigraphical Method Proceedure:

1. Find all support forces.

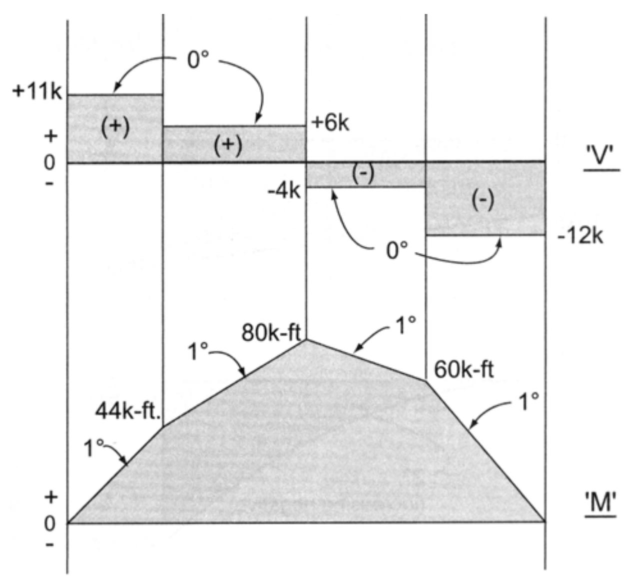
V diagram:

2. At free ends and at simply supported ends, the shear will have a zero value.
3. At the left support, the shear will equal the reaction force.
4. The shear will not change in x until there is another load, where the shear is reduced if the load is negative. If there is a distributed load, the change in shear is the area under the loading.
5. At the right support, the reaction is treated just like the loads of step 4.
6. At the free end, the shear should go to zero.

M diagram:

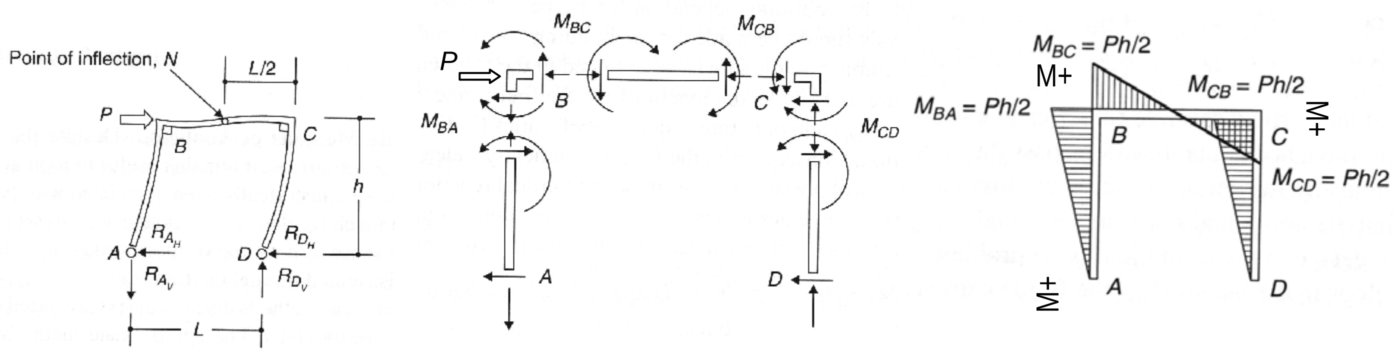
7. At free ends and at simply supported ends, the moment will have a zero value.
8. At the left support, the moment will equal the reaction moment (if there is one).
9. The moment will not change in x until there is another load or applied moment, where the moment is reduced if the applied moment is negative. If there is a value for shear on the V diagram, the change in moment is the area under the shear diagram.

   For a triangle in the shear diagram, the width will equal the height ÷ w!
10. At the right support, the moment reaction is treated just like the moments of step 9.
11. At the free end, the moment should go to zero.



Indeterminate Structures

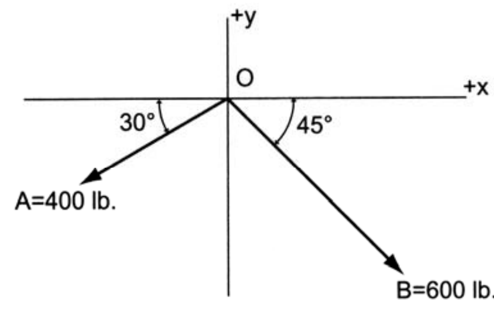
Structures with more unknowns than equations of equilibrium are *statically indeterminate*. The number of excess equations is the degree to which they are indeterminate. Other methods must be used to generate the additional equation. These structures will usually have *three-force* bodies, and possibly *rigid connections* which mean internal axial, shear and bending moment at the members and at the joints. Bending moment and shear diagrams can be constructed.



Example 1 Determine the resultant vector analytically with the component method.

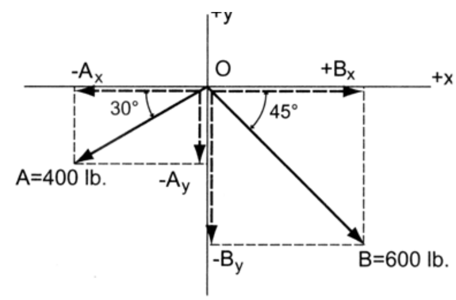
Example Problem 2.9 (Figure 2.29)

This is the same problem as Example Problem 2.2, which was solved earlier using the graphical methods.



(a)

$$-A_x = -A\cos 30° = -(400\text{ lb.})(0.866) = -346.4\text{ lb.}$$
$$-A_y = -A\sin 30° = -(400\text{ lb.})(0.50) = -200\text{ lb.}$$
$$+B_x = +B\cos 45° = +(600\text{ lb.})(0.707) = +424.2\text{ lb.}$$
$$-B_y = -B\sin 45° = -(600\text{ lb.})(0.707) = -424.2\text{ lb.}$$

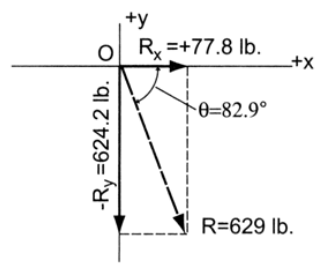


$$R_x = \sum F_x = -A_x + B_x$$
$$= -346.4\text{ lb.} + 424.2\text{ lb.} = +77.8\text{ lb.}$$

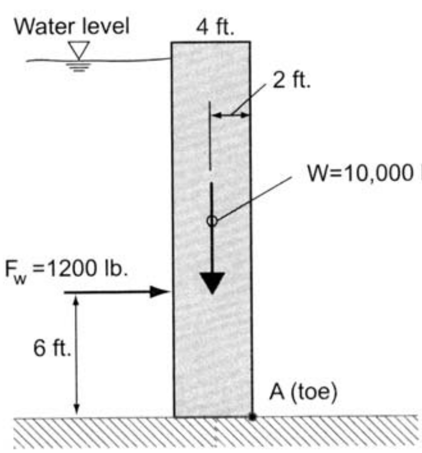
$$R_y = \sum F_y = -A_y - B_y$$
$$= -200\text{ lb.} - 424.2\text{ lb.} = -624.2\text{ lb.}$$

$$R = \sqrt{(R_x)^2 + (R_y)^2} = \sqrt{(+77.8)^2 + (-624.2)^2} = 629\text{ lb.}$$

$$\tan\theta = \left(\frac{R_y}{R_x}\right) \qquad \theta = \tan^{-1}\left(\frac{624.2}{77.8}\right) = 82.9°$$



Example 2



Example Problem 2.13 (Figure 2.35)

A 1-foot-wide slice of a 4-foot-thick concrete gravity dam weighs 10,000 pounds and the equivalent force due to water pressure behind the dam is equal to 1200 pounds. The stability of the dam against overturning is evaluated about the "toe" at *A*.

Determine the resultant moment at *A* due to the two forces shown. Is the dam stable?

$$M_A = -(F_w) \times (6\text{ ft.}) + (W) \times (2\text{ ft.})$$
$$M_A = -(1200\text{ lb.})(6\text{ ft.}) + (10{,}000\text{ lb.})(2\text{ ft.})$$
$$= +12{,}800\text{ lb.-ft.}$$

Yes, because the ground will stop the rotation.

Example 3

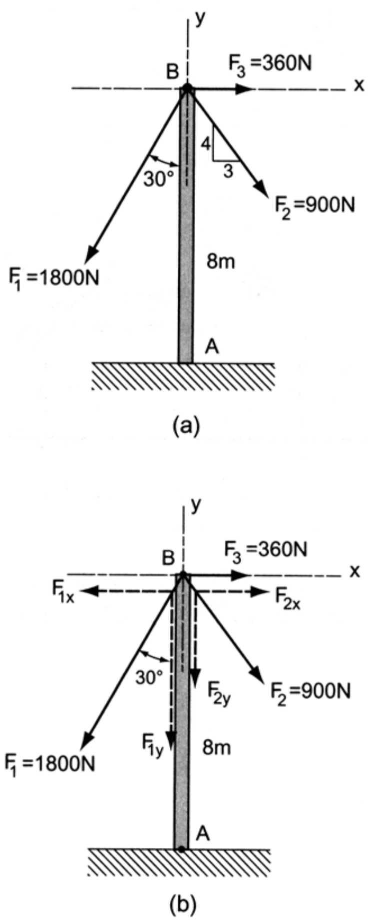


Figure 2.40 (a) Three forces on a vertical pole. (b) Forces resolved into x *and* y *components.*

Example Problem 2.17

A 8-meter vertical pole is used to support three cable forces as shown in Figure 2.40a. Determine the moment at the base of the pole at ***A***.

Solution (Figure 2.40b):

Resolve forces $\boldsymbol{F}_1$ and $\boldsymbol{F}_2$ into their respective x and y components.

$$F_{1x} = F_1 \sin 30° = (1800\text{ N})(0.5) = 900\text{ N}$$

$$F_{1y} = F_1 \cos 30° = (1800\text{ N})(0.866) = 1560\text{ N}$$

$$F_{2x} = \frac{3}{5}F_2 = \frac{3}{5}(900\text{ N}) = 540\text{ N}$$

$$F_{2y} = \frac{4}{5}F_2 = \frac{4}{5}(900\text{ N}) = 720\text{ N}$$

The moment at the base of the pole at ***A*** is the algebraic sum of the moments due to force $\boldsymbol{F}_3$ and the component forces of $\boldsymbol{F}_1$ and $\boldsymbol{F}_2$.

$$M_A = +(F_{1x})(8\text{ m}) - (F_{2x})(8\text{ m}) - (F_3)(8\text{ m})$$

$$M_A = +(900\text{ N})(8\text{ m}) - (540\text{ N})(8\text{ m}) - (360\text{ N})(8\text{ m})$$

$$M_A = +(7200\text{ N-m}) - (4320\text{ N-m}) - (2880\text{ N-m}) = 0$$

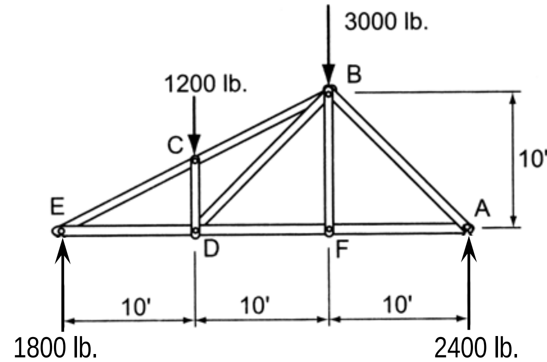
A zero resultant moment at ***A*** means that there is no tendency for the pole to rotate about the base for this particular combination of forces. Also, note that the vertical components of forces $\boldsymbol{F}_1$ and $\boldsymbol{F}_2$ did not appear in the moment equation because neither had a moment arm.

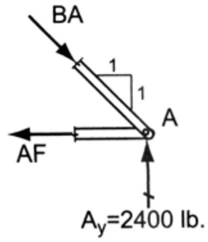
Forces that intersect the reference point have no moment arms and will cause no tendency for rotation about the point.

Example 4

Example Problem 4.1 (Method of Joints)

An asymmetrical roof truss, shown in Figure 4.4, supports two vertical roof loads. ~~Determine the support reactions at each end, then~~ Using the method of joints, solve for all member forces. ~~Summarize the results of all member forces on a FBD (this diagram is referred to as a *force summation diagram*).~~

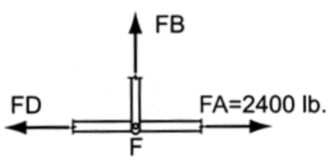




$$\sum F_y = -\left(\frac{BA}{\sqrt{2}}\right) + 2400 \text{ lb.} = 0$$

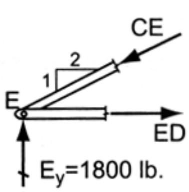
$$BA = +2400\sqrt{2} \text{ lb.} = +3390 \text{ lb.}$$

$$\sum F_x = \left(+\frac{BA}{\sqrt{2}}\right) - AF = 0; \qquad AF = \left(+\frac{2400\sqrt{2}\text{ lb.}}{\sqrt{2}}\right) = +2400 \text{ lb. (tension)}$$



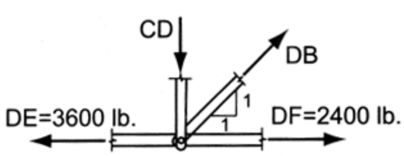
$$\sum F_x = -FD + 2400 \text{ lb.} = 0; \qquad FD = +2400 \text{ lb. (tension)}$$

$$\sum F_y = 0; \qquad \therefore FB = 0$$ special case!



$$\sum F_y = \left(\frac{-CE}{\sqrt{5}}\right) + 1800 \text{ lb.} = 0; \qquad CE = +(1800 \text{ lb.})(\sqrt{5}) = +4025 \text{ lb. (compression)}$$

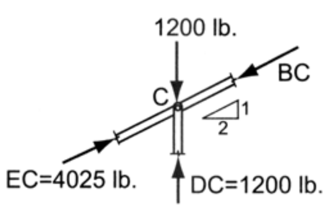
$$\sum F_x = \left(\frac{-2CE}{\sqrt{5}}\right) + ED = 0; \qquad ED = +\left(\frac{2 \times 4025 \text{ lb.}}{\sqrt{5}}\right) = +3600 \text{ lb. (tension)}$$



$$\sum F_x = (-3600 \text{ lb.}) + (2400 \text{ lb.}) + \left(\frac{DB}{\sqrt{2}}\right) = 0$$

$$DB = (1200\sqrt{2} \text{ lb.}) = +1696 \text{ lb. (tension)}$$

$$\sum F_y = +DB_y - CD = 0 \qquad CD = \frac{DB}{\sqrt{2}} = \frac{1200\sqrt{2}\text{ lb.}}{\sqrt{2}} = 1200 \text{ lb. (compression)}$$



$$\sum F = +EC_x - BC_x = 0$$

$$BC_x = \frac{2BC}{\sqrt{5}} \text{ and } EC_x = \frac{(2 \times 4025 \text{ lb.})}{\sqrt{5}} = 3600 \text{ lb.}$$

$$\sum F_y = \left(+\frac{4025 \text{ lb.}}{\sqrt{5}}\right) - 1200 \text{ lb.} + 1200 \text{ lb.} - \left(\frac{4025 \text{ lb.}}{\sqrt{5}}\right) = 0$$

$$0 = 0; \qquad \text{checks}$$

Example 5

Figure 3.15 shows a radial three-hinged arch, so named because the shape of the two-member structure is an arc of a circle with a 42-ft radius that is pinned at its two external supports with a third pin connecting the two members at the crown of the arch. Such frames are commonly used to form circular dome and barrel arch buildings and, as in this case, arch bridges.

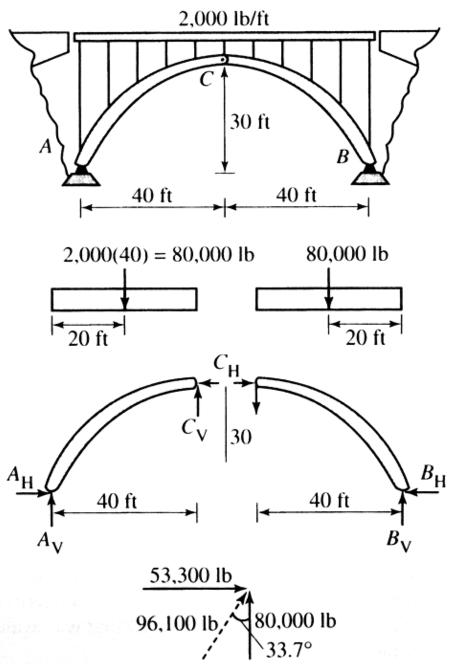


FIGURE 3.15

This bridge structure consists of four arches spaced 18 ft apart, with each supporting a roadway deck having a uniform dead (including allowance for the arch self-weight) plus averaged live load of 2,000 plf. As shown, this horizontal load is delivered to the arch through vertical columns spaced 8 ft apart, each delivering the same vertical load to the supporting arch. In this instance, or whenever four or more uniformly spaced equal concentrated loads act on a structural element, it is reasonable to assume the element is uniformly loaded.

We want to know the external reaction components at supports A and B. Since there are four support reactions–two per hinge–we cannot simply determine them by application of the three equilibrium equations to the entire 80-ft structure. By taking it apart at pin C, however, we see that we have a total of six unknowns (two per pin) and three equations of equilibrium for each of the two separated members–six equations and six unknowns. Note that the two components of the force in hinge C must be assumed to be equal and opposite on the left and right members.

By summing moments at A and B, respectively, we get the following two equations with the two unknown components of force in pin C:

$$80{,}000(20) - C_H(30) - C_V(40) = 0$$

$$-80{,}000(20) + C_H(30) - C_V(40) = 0$$

From these, $C_H = 53{,}300$ lb and $C_V = 0$. Summing vertical forces on each arch element shows us that $A_V = B_V = 80{,}000$ lb, and summation of horizontal forces on both members indicates that the outward kick of the arch members, called the horizontal thrust, is

$$A_H = B_H = C_H = 53{,}300 \text{ lb}$$

Thus, the force with which the foundation reacts to support the arch bridge is given as

$$F = \sqrt{(80{,}000^2 + 53{,}300^2)} = 96{,}100 \text{ lb}$$

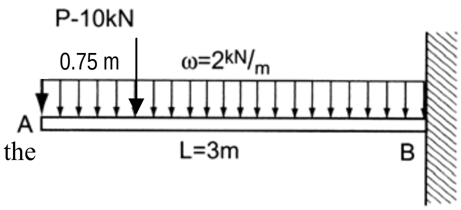
This force makes an angle with a vertical axis of

$$z = \text{arc tan}\left(\frac{53{,}300}{80{,}000}\right) = 33.7°$$

Actually, we could have made quick work of determining the arch reaction components by applying the simple arch equations discussed in the last chapter. Since it is uniformly loaded, the vertical component of the reaction at A would be $V = wL/2 = 2000(80)/2 = 80{,}000$ lb. The horizontal component would be $H = wL^2/8s = 2000(80^2)/(8 \times 30) = 53{,}300$ lb.

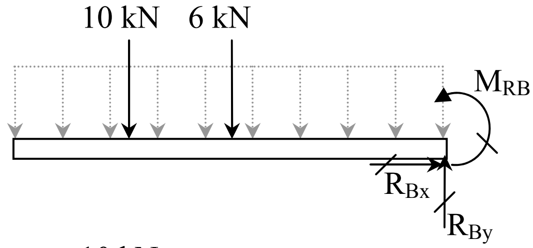
Example 6

Example Problem 8.5 (Semi-Graphical Method)



A cantilever beam supports a uniform load of $\omega = 2^{kN}/_{m}$ over its entire span, plus a concentrated load of 10 kN at 0.75 m from the free end. Construct the V and M diagrams (Figure 8.29).

SOLUTION:
Determine the reactions:

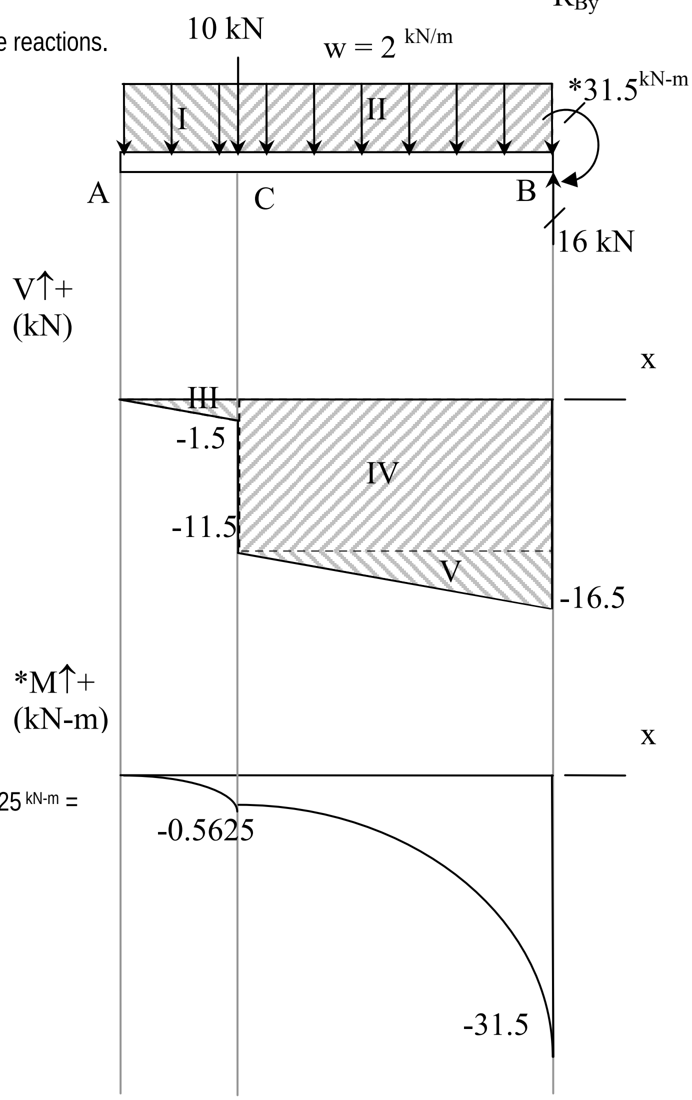


$\sum F_x = R_{Bx} = 0$ R_{Bx} = 0 kN

$\sum F_y = -10kN - (2^{kN}/_{m})(3m) + R_{By} = 0$ R_{By} = 16 kN

$\sum M_B = (10kN)(2.25m) + (6kN)(1.5m) + M_{RB} = 0$ M_{RB} = -31.5$^{kN\text{-}m}$

Draw the load diagram *with the distributed load* as given with the reactions.



Shear Diagram:

Label the load areas and calculate:

Area I = (-2 $^{kN/m}$)(0.75 m) = -1.5 kN
Area II = (-2 $^{kN/m}$)(2.25 m) = -4.5 kN

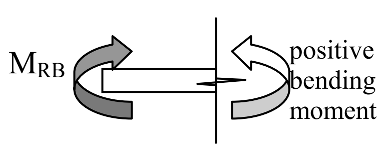
V_A = 0
V_C = V_A + Area I = 0 -1.5 kN = -1.5 kN *and*
V_C = V_C + force at C = -1.5 kN -10 kN= -11.5 kN
V_B = V_C + Area II = -11.5 kN -4.5 kN = -16 kN *and*
V_B = V_B + force at B = -16 kN +16 kN= 0 kN

Bending Moment Diagram:

Label the load areas and calculate:

Area III = (-1.5 kN)(0.75 m)/2 = -0.5625 $^{kN\text{-}m}$
Area IV = (-11.5 kN)(2.25m) = -25.875 $^{kN\text{-}m}$
Area V = (-16 – 11.5 kN)(2.25m)/2 = -5.0625 $^{kN\text{-}m}$

M_A = 0
M_C = M_A + Area III = 0 - 0.5625 $^{kN\text{-}m}$ = - 0.5625 $^{kN\text{-}m}$
M_B = M_C + Area IV + Area V = - 0.5625 $^{kN\text{-}m}$ - 25.875 $^{kN\text{-}m}$ - 5.0625 $^{kN\text{-}m}$ =
= -31.5 $^{kN\text{-}m}$ *and*
M_B = M_B + moment at B = -31.5 $^{kN\text{-}m}$ + 31.5 $^{kN\text{-}m}$ = 0 $^{kN\text{-}m}$



Mechanics of Materials Primer

Notation:

A = area (net = with holes, bearing = in contact, etc...)
b = total width of material at a horizontal section
d = diameter of a hole
D = symbol for diameter
E = modulus of elasticity or Young's modulus
f = symbol for stress
$f_{allowable}$ = allowable stress
$f_{critical}$ = critical buckling stress in column calculations from $P_{critical}$
f_v = shear stress
f_p = bearing stress (see P)
$F_{allowed}$ = allowable stress (used by codes)
$F_{connector}$ = shear force capacity per connector
I = moment of inertia with respect to neutral axis bending
J = polar moment of inertia
K = effective length factor for columns
L = length
L_e = effective length that can buckle for column design, as is ℓ_e, $L_{effective}$
M = internal bending moment, as is M'
n = number of connectors across a joint
p = pitch of connector spacing
P = name for axial force vector, as is P'
P_{crit} = critical buckling load in column calculations, as is $P_{critical}$, P_{cr}
Q = first moment area about a neutral axis
$Q_{connected}$ = first moment area about a neutral axis for the connected part
r = radius of gyration or radius of a hole
S = section modulus
t = thickness of a hole or member
T = name for axial moment or torque
V = internal shear force
y = vertical distance
α = coefficient of thermal expansion for a material
δ = elongation or length change
δ_T = elongation due or length change due to temperature
ε = strain
ε_T = thermal strain (no units)
ϕ = angle of twist
γ = shear strain
π = pi (3.1415 radians or 180°)
θ = angle of principle stress
= slope of the beam deflection curve
ρ = name for radial distance
σ = engineering symbol for normal stress
τ = engineering symbol for shearing stress
Δ = displacement due to bending
ΔT = change in temperature
$\int$ = symbol for integration

Mechanics of Materials is a basic engineering science that deals with the relation between externally applied load and its effect on deformable bodies. The main purpose of Mechanics of Materials is to answer the question of which requirements have to be met to assure STRENGTH, RIGIDITY, AND STABILITY of engineering structures.

Normal Stress

Stress that acts along an *axis* of a member; can be internal or external; can be compressive or tensile.

$$f = \sigma = \frac{P}{A_{net}}$$ Strength condition: $$f = \frac{P}{A_{net}} < f_{allowable}\ or\ F_{allowed}$$

Shear Stress (non beam)

Stress that acts perpendicular to an *axis or length* of a member, or **parallel** to the cross section is called shear stress.

Shear stress cannot be assumed to be uniform, so we refer to *average shearing stress.*

$$f_v = \tau = \frac{P}{A_{net}}$$ Strength condition: $$f_v = \frac{P}{A_{net}} < \tau_{allowable}\ or\ F_{allowed}$$

Bearing Stress

A compressive normal stress acting *between two bodies.*

$$f_p = \frac{P}{A_{bearing}}$$

Torsional Stress

A shear stress caused by torsion (moment around the axis).

$$f_v = \frac{T\rho}{J}$$

Bolt Shear Stress

Single shear - forces cause only one shear "drop" across the bolt. $$f = \frac{P}{1A_{bolt}}$$

Double shear - forces cause two shear changes across the bolt. $$f = \frac{P}{2A_{bolt}}$$

Bearing of a bolt on a bolt hole – The bearing surface can be represented by *projecting* the cross section of the bolt hole on a plane (into a rectangle).

$$f_p = \frac{P}{A} = \frac{P}{td}$$

Bending Stress

A normal stress caused by bending; can be compressive or tensile. The stress at the neutral surface or *neutral axis*, which is the plane at the *centroid* of the cross section is zero.

$$f_b = \frac{My}{I} = \frac{M}{S}$$

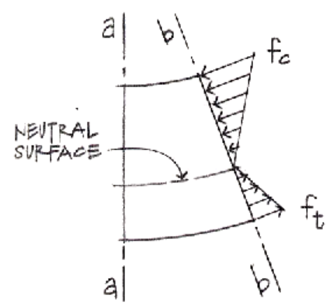
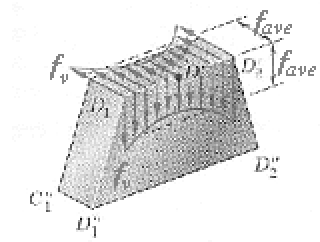


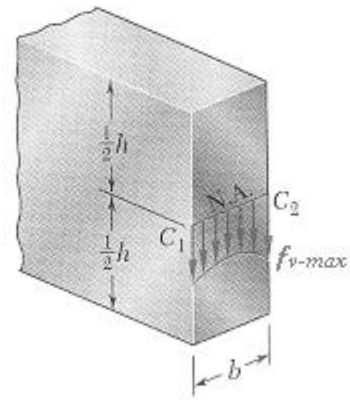
Figure 8.8 Bending stresses on section b-b.

Beam Shear Stress

$f_{v-ave} = 0$ on the beam's surface. Even if Q is a maximum at y = 0, we don't know that the thickness is a *minimum* there.

$$f_v = \frac{V}{\Delta A} = \frac{V}{b \cdot \Delta x} \qquad \boxed{f_{v-ave} = \frac{VQ}{Ib}}$$





Rectangular Sections

f_{v-max} *occurs at the neutral axis:* $$f_v = \frac{VQ}{Ib} = \frac{3V}{2A}$$

Webs of Beams

In steel W or S sections the thickness varies from the flange to the web. We neglect the shear stress in the flanges and consider the shear stress in the web to be constant:

$$f_{v-max} = \frac{3V}{2A} \approx \frac{V}{A_{web}}$$

Connectors in Bending

Typical connections needing to resist shear are plates with nails or rivets or bolts in composite sections or splices. The pitch (spacing) can be determined by the capacity in shear of the connector(s) to the shear flow over the spacing interval, p.

$$\frac{V_{longitudinal}}{p} = \frac{VQ}{I} \qquad nF_{connector} \geq \frac{VQ_{connected\,area}}{I} \cdot p$$

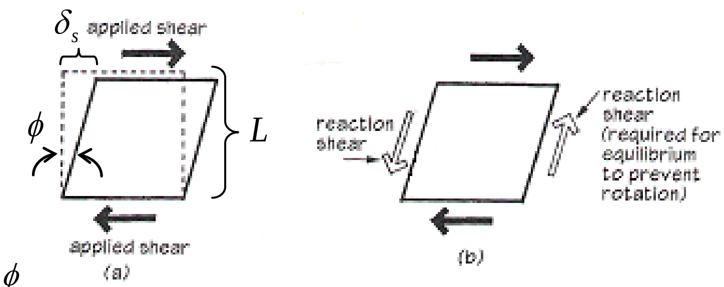
where

p = pitch length
n = number of connectors connecting the connected area to the rest of the cross section
F = force capacity in one connector
$Q_{connected\ area} = A_{connected\ area} \times y_{connected\ area}$
$y_{connected\ area}$ = distance from the centroid of the connected area to the neutral axis

Normal Strain

In an axially loaded member, normal strain, ε is the change in the length, δ with respect to the original length, L. $$\varepsilon = \frac{\delta}{L}$$

Shearing Strain



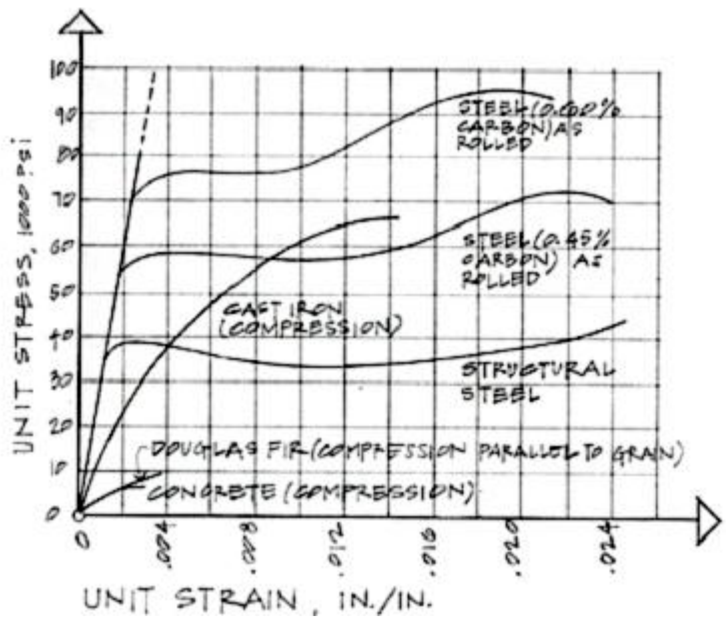
In a member loaded with shear forces, shear strain, γ is the change in the sheared side, δ_s with respect to the original height, L. For small angles: $\tan\phi \cong \phi$.

$$\gamma = \frac{\delta_s}{L} = \tan\phi \cong \phi$$

In a member subjected to twisting, the shearing strain is a measure of the angle of twist with respect to the length and distance from the center, ρ:

$$\gamma = \frac{\rho\phi}{L}$$

Stress vs. Strain



Behavior of materials can be measured by recording deformation with respect to the size of the load. For members with constant cross section area, we can plot stress vs. strain.

BRITTLE MATERIALS - ceramics, glass, stone, cast iron; show abrupt fracture at small strains.

DUCTILE MATERIALS – plastics, steel; show a yield point and large strains (considered *plastic)* and "necking" (give warning of failure)

SEMI-BRITTLE MATERIALS – concrete; show no real yield point, small strains, but have some "strain-hardening".

Linear-Elastic Behavior

In the straight portion of the stress-strain diagram, the materials are *elastic*, which means if they are loaded and unloaded no permanent **deformation** occurs.

True Stress & Engineering Stress

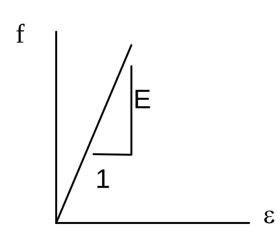
True stress takes into account that the area of the cross section changes with loading.

Engineering stress uses the original area of the cross section.

Hooke's Law – Modulus of Elasticity

In the linear-elastic range, the slope of the stress-strain diagram is *constant*, and has a value of E, called Modulus of Elasticity or Young's Modulus.

$$f = E \cdot \varepsilon$$



Isotropic Materials – have the **same** E with any direction of loading.

Anisotropic Materials – have **different** E's with the direction of loading.

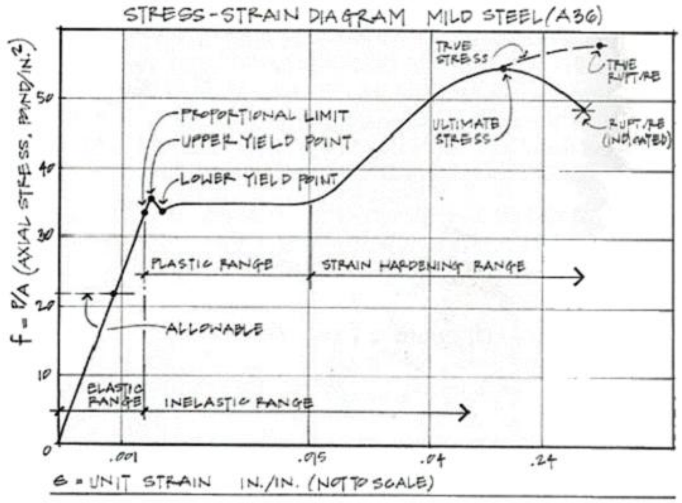
Orthotropic Materials – have **directionally based** E's

Table D-1 Elastic moduli of selected materials

Material	Modulus of elasticity E (10^6 psi)	Modulus of elasticity E (GPa)	Shear modulus G (10^6 psi)	Shear modulus G (GPa)	Poisson's ratio ν
Aluminum	10	70	3.8	26	0.33
Aluminum alloys	10–12	70–80	3.8–4.4	26–30	0.33
2014-T6	10.6	73	4	28	0.33
6061-T6	10	70	3.8	26	0.33
7075-T6	10.4	72	3.9	27	0.33
Brick (compression)	1.5–3.5	10–24			
Cast iron	12–25	80–170	4.5–10	31–69	0.2–0.3
Gray cast iron	14	97	5.6	39	0.25
Concrete (compression)	2.6–4.4	18–30			0.1–0.2
Copper	17	115	6.2	43	0.35
Copper alloys	14–18	96–120	5.2–6.8	36–47	0.33–0.35
Brass	14–16	96–110	5.2–6	36–41	0.34
80% Cu, 20% Zn	15	100	5.5	38	0.33
Naval brass	15	100	5.5	38	0.33
Bronze	14–17	96–120	5.2–6.3	36–44	0.34
Manganese bronze	15	100	5.6	39	0.35
Glass	7–12	50–80	2.9–5	20–33	0.20–0.27
Magnesium	5.8	40	2.2	15	0.34
Nickel	30	210	11.4	80	0.31
Nylon	0.3–0.4	2–3			0.4
Rubber	0.0001–0.0006	0.001–0.004	0.00004–0.0002	0.0003–0.0014	0.44–0.50
Steel	28–32	190–220	10.8–12.3	75–85	0.28–0.30
Stone (compression)					
Granite	6–10	40–70			0.2–0.3
Marble	7–14	50–100			0.2–0.3
Titanium	16	110	5.8	40	0.33
Titanium alloys	15–18	100–124	5.6–6.8	39–47	0.33
Tungsten	52	360	22	150	0.2
Wood (bending)					
Ash	1.5–1.6	10–11			
Oak	1.6–1.8	11–12			
Southern pine	1.6–2	11–14			
Wrought iron	28	190	10.9	75	0.3

Plastic Behavior & Fatigue

Permanent deformations happen outside the linear-elastic range and are called *plastic* deformations. Fatigue is damage caused by reversal of loading.

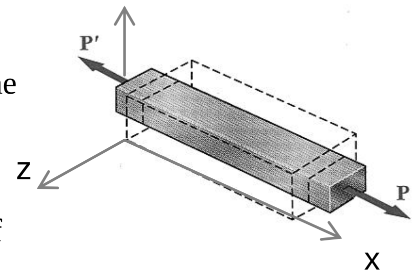


- The proportional limit (at the end of the **elastic** range) is the greatest stress valid using Hooke's law.
- The elastic limit is the maximum stress that can be applied before permanent deformation would appear upon unloading.
- The yield point (at the ***yield stress***) is where a ductile material continues to elongate without an increase of load. (May not be well defined on the stress-strain plot.)
- The ultimate strength is the largest stress a material will see before rupturing, also called the *tensile strength.*
- The rupture strength is the stress at the point of rupture or failure. It may not coincide with the ultimate strength in ductile materials. In brittle materials, it will be the same as the ultimate strength.
- The fatigue strength is the stress at failure when a member is subjected to reverse cycles of stress (up & down or compression & tension). This can happen at much lower values than the ultimate strength of a material.
- Toughness of a material is how much work (a combination of stress and strain) us used for fracture. It is the area under the stress-strain curve.

Concrete does not respond well to tension and is tested in compression. The strength at crushing is called the *compression strength.*

Materials that have time dependent elongations when loaded are said to have *creep.* Concrete and wood creep. Concrete also has the property of shrinking over time.

Poisson's Ratio



For an isometric material that is homogeneous, the properties are the same for the cross section:

$$\varepsilon_y = \varepsilon_z$$

There exists a linear relationship while in the linear-elastic range of the material between *longitudinal strain* and *lateral strain:*

$$\mu = -\frac{lateral\ strain}{axial\ strain} = -\frac{\varepsilon_y}{\varepsilon_x} = -\frac{\varepsilon_z}{\varepsilon_x} \qquad \varepsilon_y = \varepsilon_z = -\frac{\mu f_x}{E}$$

Positive strain results from an increase in length with respect to overall length.

Negative strain results from a decrease in length with respect to overall length.

μ is the *Poisson's ratio* and has a value between 0 and ½, depending on the material

Relation of Stress to Strain

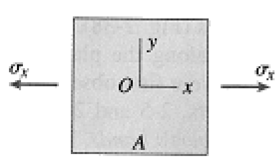
$$f = \frac{P}{A};\ \ \varepsilon = \frac{\delta}{L} \text{ and } E = \frac{f}{\varepsilon} \text{ so } E = \frac{P/A}{\delta/L} \text{ which rearranges to: } \qquad \delta = \frac{PL}{AE}$$

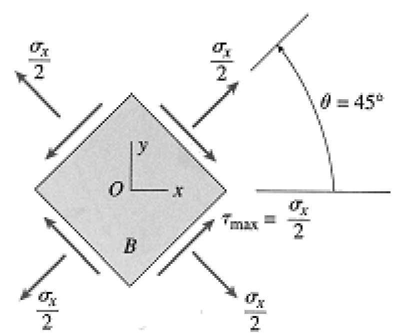
Stress Concentrations

In some sudden changes of cross section, the stress concentration changes (and is why we used *average* normal stress). Examples are sharp notches, or holes or corners.

Plane of Maximum Stress

When both normal stress and shear stress occur in a structural member, the *maximum stresses can occur at some other planes* (angle of θ).





Maximum Normal Stress happens at $\theta = 0°$ AND

Maximum Shearing Stress happens at $\theta = 45°$ with only normal stress in the *x* direction.

Thermal Strains

Physical restraints limit deformations to be the same, or sum to **zero**, or be proportional with respect to the rotation of a rigid body.

We know axial stress relates to axial strain: $\delta = \frac{PL}{AE}$ which relates δ to P

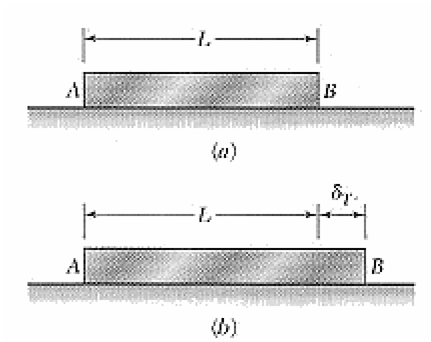
Deformations can be caused by the *material* reacting to a change in energy with temperature. In general (there are some exceptions):

- Solid materials can **contract** with a decrease in temperature.
- Solid materials can **expand** with an increase in temperature.

The change in length per unit temperature change is the *coefficient of thermal expansion,* α. It has units of $/{}^{\circ}F$ or $/{}^{\circ}C$ and the deformation is related by: $\delta_T = \alpha(\Delta T)L$

Coefficient of Thermal Expansion

Material	Coefficients (α) [in./in./°F]	Coefficients (α) [mm/mm/°C]
Wood	3.0 x 10^{-6}	5.4 x 10^{-6}
Glass	4.4 x 10^{-6}	8.0 x 10^{-6}
Concrete	5.5 x 10^{-6}	9.9 x 10^{-6}
Cast Iron	5.9 x 10^{-6}	10.6 x 10^{-6}
Steel	6.5 x 10^{-6}	11.7 x 10^{-6}
Wrought Iron	6.7 x 10^{-6}	12.0 x 10^{-6}
Copper	9.3 x 10^{-6}	16.8 x 10^{-6}
Bronze	10.1 x 10^{-6}	18.1 x 10^{-6}
Brass	10.4 x 10^{-6}	18.8 x 10^{-6}
Aluminum	12.8 x 10^{-6}	23.1 x 10^{-6}

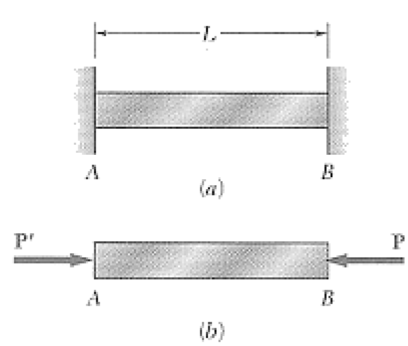


Thermal Strain: $\varepsilon_T = \alpha\Delta T$

There is **no stress** associated with the length change with free movement, BUT if there are restraints, thermal deformations or strains *can cause internal forces and stresses.*

How A Restrained Bar Feels with Thermal Strain

1. Bar pushes on supports because the material needs to expand with an increase in temperature.
2. Supports push *back.*
3. Bar is restrained, can't move and the reaction causes internal *stress*.



Superposition Method

If we want to solve a statically indeterminate problem that has extra support forces:

- We can remove a support or supports that *makes the problem look statically determinate*
- Replace it with a reaction and treat it like it is an applied force
- Impose geometry restrictions that the support imposes

Beam Deflections

$$\theta = slope = \frac{1}{EI}\int M(x)dx$$

If the bending moment changes, M(x) across a beam of constant material and cross section then the curvature will change:

The slope of the n.a. of a beam, θ, will be tangent to the radius of curvature, R:

The equation for deflection, y, along a beam is:

$$y = \Delta = \frac{1}{EI}\int \theta dx = \frac{1}{EI}\iint M(x)dx$$

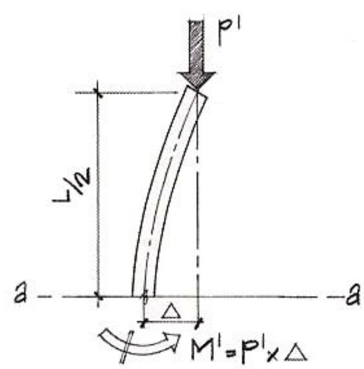
Elastic curve equations can be found in handbooks, textbooks, design manuals, etc...Computer programs can be used as well.

Elastic curve equations can be **superpositioned** ONLY if the stresses are in the elastic range.

Column Buckling

Stability is the ability of the structure to support a specified load without undergoing unacceptable (or sudden) deformations. A column loaded centrically can experience unstable equilibrium, called *buckling*, because of how tall and slender they are. This instability is sudden and not good.

Buckling can occur in sheets (like my "memory metal" cookie sheet), pressure vessels or slender (narrow) beams not braced laterally.



The critical axial load to cause buckling is related to the deflected shape we could get (or determine from bending moment of P·Δ) as a function of the end conditions.

Swiss mathematician Euler determined the relationship between the critical buckling load, the material, section and effective length (as long as the material stays in the elastic range):

$$P_{critical} = \frac{\pi^2 EI_{min}}{(L)^2} \quad \text{or} \quad P_{cr} = \frac{\pi^2 EI}{(L_e)^2} = \frac{\pi^2 EA}{\left(\frac{L_e}{r}\right)^2}$$

and the critical stress (*if less than the normal stress)* is:

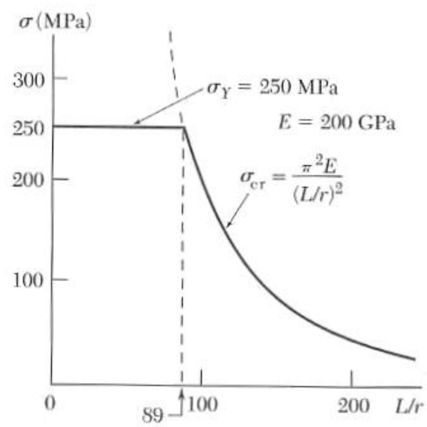
$$f_{critical} = \frac{P_{critical}}{A} = \frac{\pi^2 EAr^2}{A(L_e)^2} = \frac{\pi^2 E}{\left(\frac{L_e}{r}\right)^2}$$

where I=Ar^2 and L_e/r is called the slenderness ratio. The smallest I of the section will govern.

Radius of gyration is a relationship between I and A. It is useful for comparing columns of different shape cross section shape.

$$r_x = \sqrt{\frac{I_x}{A}} \qquad r_y = \sqrt{\frac{I_y}{A}}$$

Yield Stress and Buckling Stress

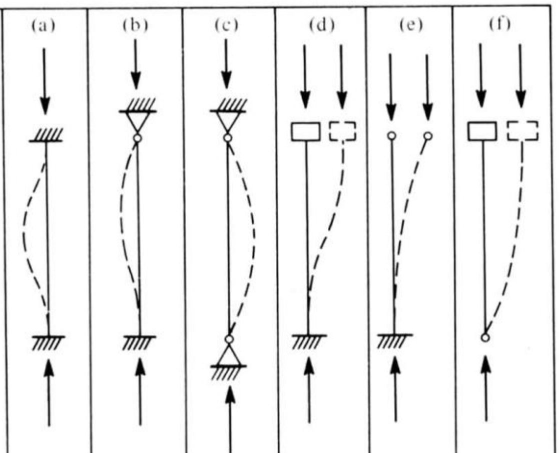


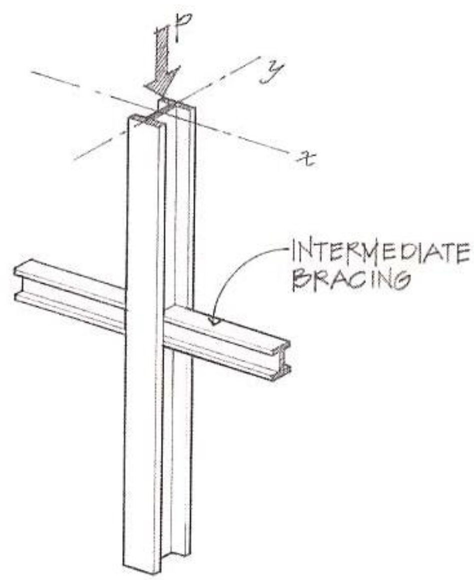
The two design criteria for columns are that they do not buckle and the strength is not exceeded. Depending on slenderness, one will control over the other.

Effective Length and Bracing

Depending on the end support conditions for a column, the effective length can be found from the deflected shape (elastic equations). If a very long column is braced intermittently along its length, the column length that will buckle can be determined. The effective length can be found by multiplying the column length by an effective length factor, K.

$$L_e = K \cdot L$$

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design values when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.10	2.0
End conditions code	Rotation fixed, Translation fixed Rotation free, Translation fixed Rotation fixed, Translation free Rotation free, Translation free					



Example 1

Example Problem 6.8 (Figures 6.18 to 6.20)

A pipe storage rack is used for storing pipe in a shop. The support rack beam is fastened to the main floor beam using steel straps $\frac{1}{2}" \times 2"$ in dimension. Round bolts are used to fasten the strap to the floor beam in single shear. (a) If the weight of the pipes impose a maximum tension load of 10,000 pounds in each strap, determine the tension stress developed in the steel strap. (b) Also, what diameter bolt is necessary to fasten the strap to the floor beam if the allowable shear stress for the bolts equals $F_v = 15{,}000\,{}^{\text{lb.}}/_{\text{in.}^2}$?

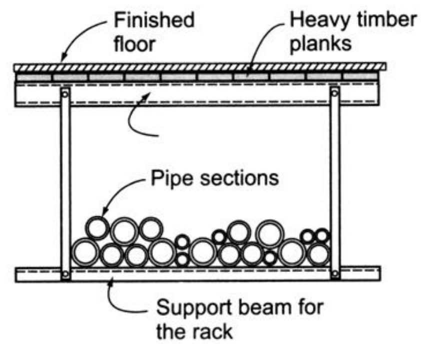


Figure 6.18 Pipe storage rack.

Solution:

a. The tensile stress developed in the steel strap (Figure 6.19) can be determined using the direct stress formula.

$$f_t = \frac{P}{A} = \frac{10{,}000\text{ lb.}}{\left(\frac{1}{2}" \times 2"\right)} = 10{,}000\,{}^{\text{lb.}}/_{\text{in.}^2}$$

In mild steel (A36), the maximum permissible tensile stress (allowable) is equal to

$$\underset{\text{(allowable)}}{F_t} = 22{,}000\text{ psi}$$

Therefore, the strap size is adequate to support the tensile load safely.

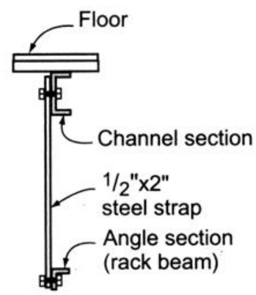


Figure 6.19 Section.

b. To determine the size bolt necessary to carry the load safely in single shear, the design form of the equation must be used.

$$f_v = \frac{P}{A'}; \qquad A = \frac{P}{F_v} = \frac{10{,}000\text{ lb.}}{15{,}000\,{}^{\text{lb.}}/_{\text{in.}^2}} = 0.67\text{ in.}^2$$

$$A = \frac{\pi D^2}{4}; \quad D^2 = \frac{4 \times A}{\pi} = \frac{4 \times 0.67\text{ in.}^2}{3.14}$$

$$= 0.854\text{ in.}^2$$

$D = 0.92$ in.; Use: 1"ϕ bolt.

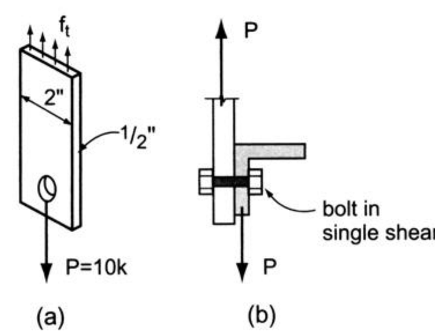
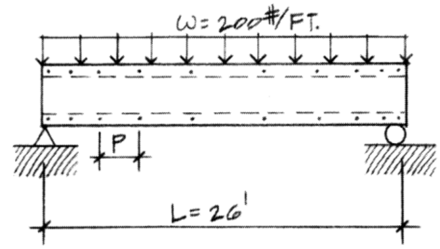
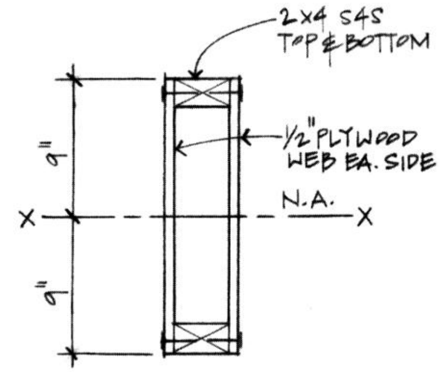


Figure 6.20 Bolt in single shear.

Example 2

8.11 A built-up plywood box beam with 2 × 4 S4S top and bottom flanges is held together by nails. Determine the pitch (spacing) of the nails if the beam supports a uniform load of 200 #/ft. along the 26-foot span. Assume the nails have a shear capacity of 80# each.

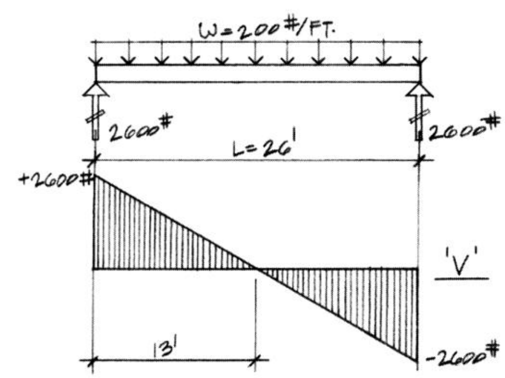




Solution:

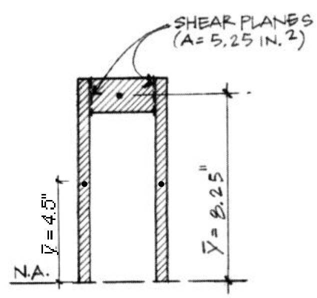
Construct the shear (V) diagram to obtain the critical shear condition and its location

Note that the condition of shear is critical at the supports, and the shear intensity decreases as you approach the center line of the beam. This would indicate that the nail spacing P varies from the support to midspan. Nails are closely spaced at the support, but increasing spacing occurs toward midspan, following the shear diagram.



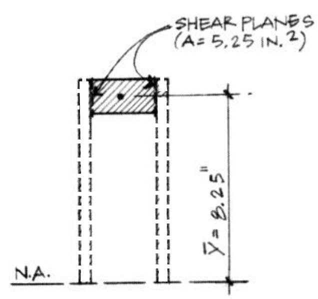
$$f_v = \frac{VQ}{Ib}$$

$$I_x = \frac{(4.5'')(18'')^3}{12} - \frac{(3.5'')(15'')^3}{12} = 1{,}202.6 \text{ in.}^4$$



$$\varepsilon = \frac{\delta}{L}$$

$$f_{v-\max} = \frac{(2{,}600\#)(83.3in.^3)}{(1{,}202.6in.^4)(\tfrac{1}{2}"+\tfrac{1}{2}")} = 180.2\,psi$$



$$Q = A\bar{y} = (5.25 \text{ in.}^2)(8.25'') = 43.3 \text{ in.}^3$$

Shear force $= f_v \times A_v$

where:

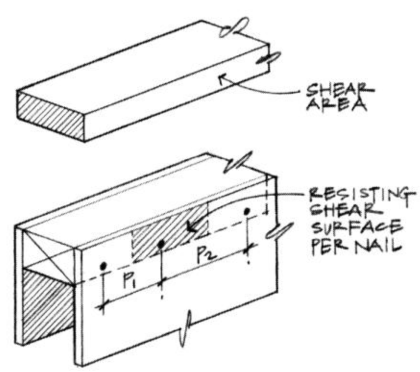
A_v = shear area

Assume:

F = Capacity of two nails (one each side) at the flange; representing two shear surfaces

$$(\text{n})F \geq f_v \times b \times p = \frac{VQ}{Ib} \times bp$$

$$\therefore\ (\text{n})F \geq p \times \frac{VQ}{I}; \qquad p \leq \frac{(\text{n})FI}{VQ}$$



At the maximum shear location (support) where V = 2,600#

$$p \leq \frac{(2 \text{ nails} \times 80 \text{ \#/nail})(1{,}202.6 \text{ in.}^4)}{(2{,}600\#)(43.3 \text{ in.}^3)} = 1.71''$$

Example 3

6.4 THERMAL EFFECTS

Most structural materials expand in volume when subjected to heat and contract when cooled. Whenever a design prevents the change in length of a member subjected to temperature variation, internal stresses develop. Sometimes these *thermal stresses* may be sufficiently high to exceed the elastic limit and cause serious damage. Free, *unrestrained members* experience no stress changes with temperature changes, but dimensional change results. For example, it is common practice to provide expansion joints between sidewalk pavements to allow movement during hot summer days. Prevention of expansion on a hot day would undoubtedly result in severe buckling of the pavement.

The dimensional change due to temperature changes is usually described in terms of the change in a linear dimension. The change in length of a structural member, ΔL, is directly proportional to both the temperature change (ΔT) and the original length of the member L. *Thermal sensitivity*, called the *coefficient of linear expansion* (α), has been determined for all engineering materials (see Table 6.3). Careful measurements have shown that the ratio of strain ε to temperature change ΔT is a constant:

$$\alpha = \frac{\text{strain}}{\text{temp. change}} = \frac{\varepsilon}{\Delta T} = \frac{\delta/L}{\Delta T}$$

Solving this equation for the deformation:

where:

$$\delta = \alpha L \Delta T$$

where:

α = coefficient of thermal expansion or contraction
L = original length of the member (inches or mm)
ΔT = change in temperature (°F or °C)
δ = total change in length (in. or mm)

Of perhaps even greater importance in engineering design are the stresses developed by restraining the free expansion and contraction of members subjected to temperature variations. To calculate these temperature stresses, it is useful to determine first the free expansion or contraction of the member involved and, second, the force and unit stress developed in forcing the member to attain its original length. The problem from this point on is exactly the same as those solved in the earlier portions of this chapter dealing with axial stresses, strains, and deformations. The amount of stress developed by restoring a bar to its original length L is:

$$f = \varepsilon E = \frac{\delta}{L}E = \frac{\alpha L \Delta T E}{L} = \alpha \Delta T E$$

$$\therefore f = \alpha \Delta T E$$

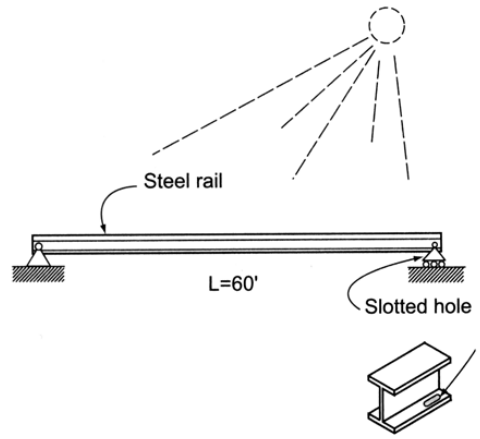


Figure 6.57 Steel rail subjected to thermal change.

Example Problem 6.21 (Figure 6.57)

A 60' length of steel rail is laid on a day when the temperature is 40°F. In order to prevent the rail from developing any internal stresses due to a thermal increase of 60°F, what is the amount of deformation that needs to be accommodated with respect to the slotted connection at the rail end(s)? $E_{st} = 29 \times 10^3$ ksi.

Solution:

Steel has a coefficient of expansion $\alpha = 6.5 \times 10^{-6}/°\text{F}$ (see Table 6.3).

Using the deformation equation due to thermal change:

$$\delta = \alpha L \Delta T = (6.5 \times 10^{-6}/°\text{F})\left(60' \times 12\,{}^{\text{in.}}/_{\text{ft.}}\right)(60°\text{F})$$
$$= 0.28''$$

This amount of deformation (0.28") for a 60'-long rail section may not seem large but if there are no provisions made to allow movement during thermal changes, large internal stress may result. If the rail section in this example has a cross-sectional area of $A = 10.5$ in.^{2}, determine the amount of internal compressive stress that can result if the rail is restrained from moving.

$$f = \alpha \Delta T E = (6.5 \times 10^{-6}/°\text{F})(60°\text{F})\left(29 \times 10^3\,{}^{\text{k}}/_{\text{in.}^2}\right)$$
$$= 11.31 \text{ ksi}$$

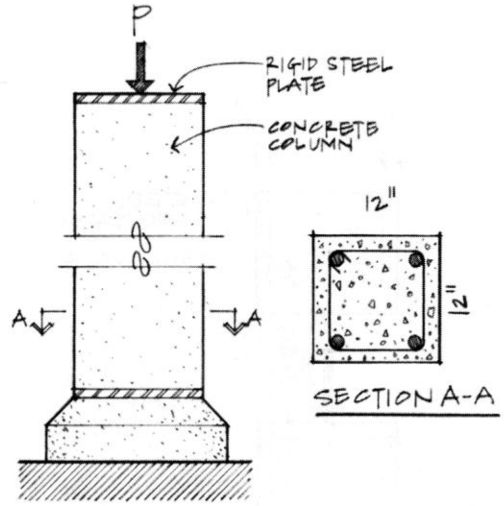
(a very large internal stress which can potentially cause the rail to buckle)

Example 4

A short concrete column measuring 12 in. square is reinforced with four #8 bars ($A_s = 4 \times 0.79$ in.^{2} = 3.14 in.^{2}) and supports an axial load of 250k. Steel bearing plates are used top and bottom to ensure equal deformations of steel and concrete. Calculate the stress developed in each material if:

$E_c = 3 \times 10^6$ psi and

$E_s = 29 \times 10^6$ psi



Solution:

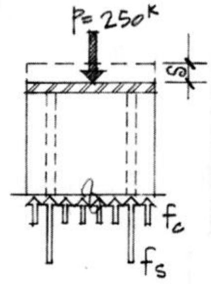
From equilibrium:

$$[\Sigma F_y = 0] - 250\text{ k} + f_s A_s + f_c A_c = 0$$

$$A_s = 3.14\text{ in.}^2$$

$$A_c = (12'' \times 12'') - 3.14\text{ in.}^2 \cong 141\text{ in.}^2$$

$$3.14 f_s + 141 f_c = 250\text{ k}$$



From the deformation relationship:

$$\delta_s = \delta_c; \;\; L_s = L_c$$

$$\therefore \frac{\delta_s}{L} = \frac{\delta_c}{L}$$

and

$$\varepsilon_s = \varepsilon_c$$

Since

$$E = \frac{f}{\varepsilon}$$

and

$$\frac{f_s}{E_s} = \frac{f_c}{E_c}$$

$$f_s = f_c \frac{E_s}{E_c} = \frac{29 \times 10^3 (f_c)}{3 \times 10^3} = 9.67 f_c$$

Substituting into the equilibrium equation:

$$3.14\,(9.76 f_c) + 141 f_c = 250$$

$$30.4 f_c + 141 f_c = 250$$

$$171.4 f_c = 250$$

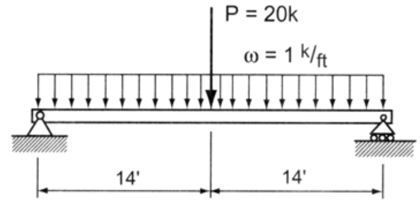
$$f_c = 1.46\text{ ksi}$$

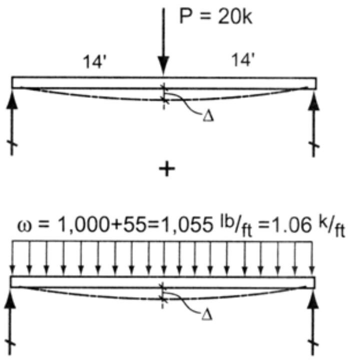
$$\therefore f_s = 9.67\,(1.46)\text{ ksi}$$

$$f_s = 14.1\text{ ksi}$$

Example 5

Determine the deflection in the steel beam if it is a W15 x 88. E = 30x10^3 ksi.





Deflection Check:

$$\Delta_{\text{actual}} = \frac{PL^3}{48EI} + \frac{5\omega L^4}{384EI}$$

$$\Delta_{\text{actual}} = \frac{(20\text{ k})(28')^3(1728)}{48(30 \times 10^3)(890)} + \frac{5(1.06\text{ k/ft.})(28')^4(1728)}{384(30 \times 10^3)(890)}$$

$$\Delta_{\text{actual}} = 0.59'' + 0.55'' = 1.14''$$

Example 6

Example Problem 10.6 (Figures 10.28 to 10.30)

A W8×40 steel column supports trusses framed into its web, which serve to fix the weak axis and light beams that attach to the flange, simulating a pin connection about the strong axis. If the base connection is assumed as a pin, determine the critical buckling load the column is capable of supporting.

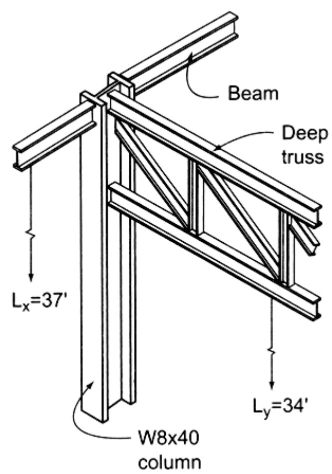


Figure 10.28 Truss/column framing.

Solution:

W8×40; ($A = 11.7$ in.^{2}, $r_x = 3.53''$, $I_x = 146$ in.^{4}, $r_y = 2.04''$, $I_y = 49.1$ in.^{4})

The first step is to determine the critical axis for buckling (i.e., which one has the larger KL/r).

Weak Axis:

$$L_e = KL = 0.7\,(34') = 23.8'$$

$$\frac{KL}{r_y} = \frac{23.8' \times 12^{\text{in.}}/_{\text{ft.}}}{2.04''} = 140$$

Strong Axis:

$$L_e = L; \quad K = 1.0; \quad KL = 37'$$

$$\frac{KL}{r_x} = \frac{\left(37' \times 12^{\text{in.}}/_{\text{ft.}}\right)}{3.53'} = 125.8$$

The weak axis for this column is critical since

$$\frac{KL}{r_y} > \frac{KL}{r_x}$$

$$P_{\text{cr.}} = \frac{\pi^2 EI_y}{(KL)^2} = \frac{(3.14)^2(29 \times 10^3\text{ ksi})(49.1\text{ in.}^4)}{\left(23.8' \times 12^{\text{in.}}/_{\text{ft.}}\right)^2}$$

$$= 172.1\text{ k}$$

$$f_{\text{critical}} = \frac{P_{\text{crit.}}}{A} = \frac{172.1\text{ k}}{11.7\text{ in.}^2} = 14.7\text{ ksi}$$

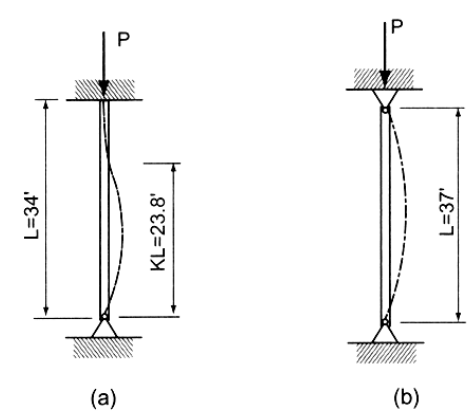


Figure 10.30 (a) Weak axis. (b) Strong axis.

Conversion Factors and Units of Measurement

Simplified Engineering for Architects and Builders, 10th ed., Ambrose & Tripeny, 2006

TABLE 3 Factors for Conversion of Units

To Convert from U.S. Units to SI Units, Multiply by:	U.S. Unit	SI Unit	To Convert from SI Units to U.S. Units, Multiply by:
25.4	in.	mm	0.03937
0.3048	ft	m	3.281
645.2	in.^{2}	mm^2	1.550×10^{-3}
16.39×10^3	in.^{3}	mm^3	61.02×10^{-6}
416.2×10^3	in.^{4}	mm^4	2.403×10^{-6}
0.09290	ft^2	m^2	10.76
0.02832	ft^3	m^3	35.31
0.4536	lb (mass)	kg	2.205
4.448	lb (force)	N	0.2248
4.448	kip (force)	kN	0.2248
1.356	ft-lb (moment)	N-m	0.7376
1.356	kip-ft (moment)	kN-m	0.7376
16.0185	lb/ft^3 (density)	kg/m^3	0.06243
14.59	lb/ft (load)	N/m	0.06853
14.59	kip/ft (load)	kN/m	0.06853
6.895	psi (stress)	kPa	0.1450
6.895	ksi (stress)	MPa	0.1450
0.04788	psf (load or pressure)	kPa	20.93
47.88	ksf (load or pressure)	kPa	0.02093
$0.566 \times (°F - 32)$	°F	°C	$(1.8 \times °C) + 32$

TABLE 2 Units of Measurement: SI System

Name of Unit	Abbreviation	Use in Building Design
Length		
Meter	m	Large dimensions, building plans, beam spans
Millimeter	mm	Small dimensions, size of member cross sections
Area		
Square meters	m^2	Large areas
Square millimeters	mm^2	Small areas, properties of member cross sections
Volume		
Cubic meters	m^3	Large volumes
Cubic millimeters	mm^3	Small volumes
Mass		
Kilogram	kg	Mass of material (equivalent to weight in U.S. units)
Kilograms per cubic meter	kg/m^3	Density (unit weight)
Force, Load		
Newton	N	Force or load on structure
Kilonewton	kN	1000 Newtons
Stress		
Pascal	Pa	Stress or pressure (1 pascal = 1 N/m^2)
Kilopascal	kPa	1000 pascals
Megapascal	MPa	1,000,000 pascals
Gigapascal	GPa	1,000,000,000 pascals
Temperature		
Degree Celsius	°C	Temperature

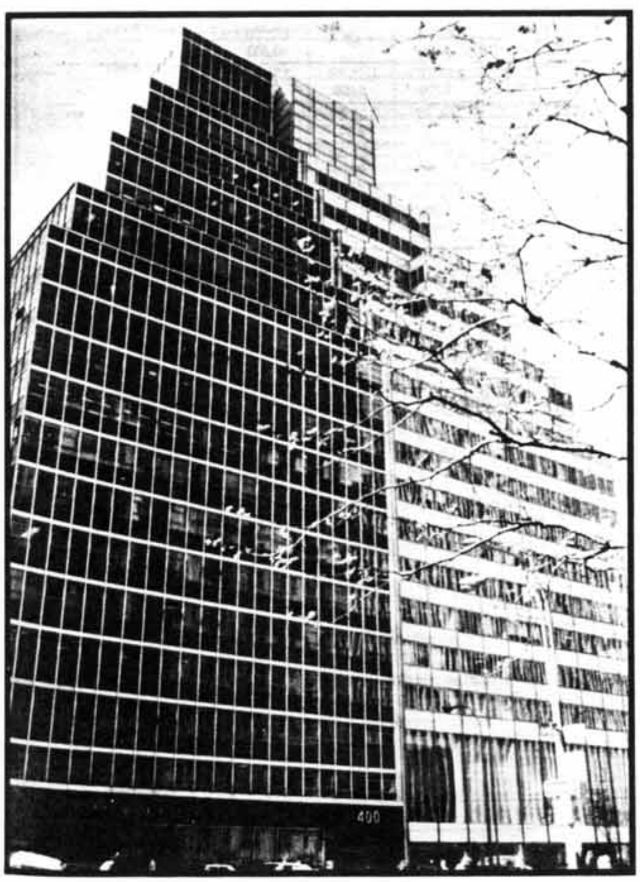
TABLE 1 Units of Measurement: U.S. System

Name of Unit	Abbreviation	Use in Building Design
Length		
Foot	ft	Large dimensions, building plans, beam spans
Inch	in.	Small dimensions, size of member cross sections
Area		
Square feet	ft^2	Large areas
Square inches	$in.^2$	Small areas, properties of cross sections
Volume		
Cubic yards	yd^3	Large volumes, of soil or concrete (commonly called simply "yards")
Cubic feet	ft^3	Quantities of materials
Cubic inches	$in.^3$	Small volumes
Force, Mass		
Pound	lb	Specific weight, force, load
Kip	kip, k	1000 pounds
Ton	ton	2000 pounds
Pounds per foot	lb/ft, plf	Linear load (as on a beam)
Kips per foot	kips/ft, klf	Linear load (as on a beam)
Pounds per square foot	lb/ft^2, psf	Distributed load on a surface, pressure
Kips per square foot	k/ft^2, ksf	Distributed load on a surface, pressure
Pounds per cubic foot	lb/ft^3	Relative density, unit weight
Moment		
Foot-pounds	ft-lb	Rotational or bending moment
Inch-pounds	in.-lb	Rotational or bending moment
Kip-feet	kip-ft	Rotational or bending moment
Kip-inches	kip-in.	Rotational or bending moment
Stress		
Pounds per square foot	lb/ft^2, psf	Soil pressure
Pounds per square inch	$lb/in.^2$, psi	Stresses in structures
Kips per square foot	$kips/ft^2$, ksf	Soil pressure
Kips per square inch	$kips/in.^2$, ksi	Stresses in structures
Temperature		
Degree Fahrenheit	°F	Temperature

<u>Design and Technology in Architecture,</u> Revised Ed., David Guise, 1991, Van Nostrand Reinhold, NY.

Chapter 7

BUILDING CODES AND ZONING ORDINANCES



In a complex society, regulation is one of the facts of life. The buildings in which people live, work, and play are subject to many controls. Local and regional government agencies have been established to protect the public and the environment from dangerous and undesirable conditions that sometimes occur when man-made structures are erected, and the result is innumerable building codes and zoning ordinances. In the United States, these rules and regulations can vary from community to community, but all are based on fundamental construction methods necessary to protect public safety and welfare.

BUILDING CODES

Building codes not only affect the selection of the materials that an architect uses to build a structure; they can influence the size and shape of the building as well. For example, depending upon how fire-resistant the selected construction materials are, the codes will permit different maximum areas per floor and different total numbers of floors for the building.

The impact of the relationship between building materials and the size of a building can be most easily demonstrated by an example. Assume that an architect is planning to design a resort. He wishes to use exposed, laminated-wood beams and other wood construction in order to create a rustic atmosphere. Chart 1, reproduced from the National Building Code, known as BOCA, lists hotels under the use-group R-1. When R-1 is intersected with construction type 4 (heavy timber), the chart shows that the maximum height permitted for a structure of this type is 4 stories or 50 feet, whichever is greater. The chart also indicates that no more than 14,400 square feet of space is permitted on each floor. If the client's needs can be accommodated within these height and area limitations, then all is fine. If not, a different type of framing system will have to be considered—one that permits either more height or more area.

Chart 2 provides specific information regarding the amount of fire protection that is required for each particular part of a building. The information differs based on the variations in construction types listed in Chart 1. The building code can also be referred to for further explanation, and indeed the chart often refers the reader to the code. For example, under type 4 construction, Chart 2 shows that bearing walls require a two-hour rating (see 1 under Structure Element).

Park Avenue, New York, New York.

CHART 1

Table 501
HEIGHT AND AREA LIMITATIONS OF BUILDINGS
Height limitations of buildings (shown in upper figure as stories and feet above grade), and area limitations of one or two story buildings facing on one street or public space not less than 30 feet wide (shown in lower figure as area in square feet per floor). See Note a.

NP — Not permitted
NL — Not limited

Use Group	Type of construction									
	Noncombustible					Noncombustible/Combustible			Combustible	
	Type 1		Type 2			Type 3		Type 4	Type 5	
	Protected Note b		Protected		Unprotected	Protected	Unprotected	Heavy timber	Protected	Unprotected
Note a	1A	1B	2A	2B	2C	3A	3B	4	5A	5B
A-1 Assembly, theaters	NL	NL	5 St. 65' 19,950	3 St. 40' 13,125	2 St. 30' 8,400	3 St. 40' 11,550	2 St. 30' 8,400	3 St. 40' 12,600	1 St. 20' 8,925	1 St. 20' 4,200
A-2 Assembly, nightclubs and similar uses	NL	NL 7,200	3 St. 40' 5,700	2 St. 30' 3,750	1 St. 20' 2,400	2 St. 30' 3,300	1 St. 20' 2,400	2 St. 30' 3,600	1 St. 20' 2,550	1 St. 20' 1,200
A-3 Assembly — Lecture halls, recreation centers, terminals, restaurants other than night clubs	NL	NL	5 St. 65' 19,950	3 St. 40' 13,125	2 St. 30' 8,400	3 St. 40' 11,550	2 St. 30' 8,400	3 St. 40' 12,600	1 St. 20' 8,925	1 St. 20' 4,200
A-4 Assembly, churches — Note d	NL	NL	5 St. 65' 34,200	3 St. 40' 22,500	2 St. 30' 14,400	3 St. 40' 19,800	2 St. 30' 14,400	3 St. 40' 21,600	1 St. 20' 15,300	1 St. 20' 7,200
B Business	NL	NL	7 St. 85' 34,200	5 St. 65' 22,500	3 St. 40' 14,400	4 St. 50' 19,800	3 St. 40' 14,400	5 St. 65' 21,600	3 St. 40' 15,300	2 St. 30' 7,200
E Educational — Note c,d	NL	NL	5 St. 65' 34,200	3 St. 40' 22,500	2 St. 30' 14,400	3 St. 40' 19,800	2 St. 30' 14,400	3 St. 40' 21,600	1 St. 20' 15,300 Note e	1 St. 20' 7,200 Note e
F-1 Factory and industrial, moderate — Note i	NL	NL	6 St. 75' 22,800	4 St. 50' 15,000	2 St. 30' 9,600	3 St. 40' 13,200	2 St. 30' 9,600	4 St. 50' 14,400	2 St. 30' 10,200	1 St. 20' 4,800
F-2 Factory and industrial, low — Note i	NL	NL	7 St. 85' 34,200	5 St. 65' 22,500	3 St. 40' 14,400	4 St. 50' 19,800	3 St. 40' 14,400	5 St. 65' 21,600	3 St. 40' 15,300	2 St. 30' 7,200
H High hazard — Note f	5 St. 65' 16,800	3 St. 40' 14,400	3 St. 40' 11,400	2 St. 30' 7,500	1 St. 20' 4,800	2 St. 30' 6,600	1 St. 20' 4,800	2 St. 30' 7,200	1 St. 20' 5,100	NP
I-1 Institutional, residential care	NL	NL	9 St. 100' 19,950	4 St. 50' 13,125	3 St. 40' 8,400	4 St. 50' 11,550	3 St. 40' 8,400	4 St. 50' 12,600	3 St. 40' 8,925	2 St. 35' 4,200
I-2 Institutional, incapacitated	NL	8 St. 90' 21,600	4 St. 50' 17,100	2 St. 30' 11,250	1 St. 20' 7,200	1 St. 20' 9,900	NP	1 St. 20' 10,800	1 St. 20' 7,650	NP
I-3 Institutional, restrained	NL	6 St. 75' 18,000	4 St. 50' 14,250	2 St. 30' 9,375	1 St. 20' 6,000	2 St. 30' 8,250	1 St. 20' 6,000	2 St. 30' 9,000	1 St. 20' 6,375	NP
M Mercantile	NL	NL	6 St. 75' 22,800	4 St. 50' 15,000	2 St. 30' 9,600	3 St. 40' 13,200	2 St. 30' 9,600	4 St. 50' 14,400	2 St. 30' 10,200	1 St. 20' 4,800
R-1 Residential, hotels	NL	NL	9 St. 100' 22,800	4 St. 50' 15,000	3 St. 40' 9,600	4 St. 50' 13,200	3 St. 40' 9,600	4 St. 50' 14,400	3 St. 40' 10,200	2 St. 35' 4,800
R-2 Residential, multiple-family	NL	NL	9 St. 100' 22,800	4 St. 50' 15,000 Note g	3 St. 40' 9,600	4 St. 50' 13,200 Note g	3 St. 40' 9,600	4 St. 50' 14,400	3 St. 40' 10,200	2 St. 35' 4,800
R-3 Residential, one- and two-family	NL	NL	4 St. 50' 22,800	4 St. 50' 15,000	3 St. 40' 9,600	4 St. 50' 13,200	3 St. 40' 9,600	4 St. 50' 14,400	3 St. 40' 10,200	2 St. 35' 4,800
S-1 Storage, moderate	NL	NL	5 St. 65' 19,950	4 St. 50' 13,125	2 St. 30' 8,400	3 St. 40' 11,550	2 St. 30' 8,400	4 St. 50' 12,600	2 St. 30' 8,925	1 St. 20' 4,200
S-2 Storage, low — Note h	NL	NL	7 St. 85' 34,200	5 St. 65' 22,500	3 St. 40' 14,400	4 St. 50' 19,800	3 St. 40' 14,400	5 St. 65' 21,600	3 St. 40' 15,300	2 St. 30' 7,200
U Utility, miscellaneous	NL	NL								

Note a. See the following sections for general exceptions to Table 501:
Section 501.4 Allowable area reduction for multistory buildings.
Section 502.2 Allowable area increase due to street frontage.
Section 502.3 Allowable area increase due to automatic sprinkler system installation.
Section 503.1 Allowable height increase due to automatic sprinkler system installation.
Section 504.0 Unlimited area one-story buildings.

Note b. Buildings of Type 1 construction permitted to be of unlimited tabular heights and areas are not subject to special requirements that allow increased heights and areas for other types of construction (see Section 501.5).

Note c. For tabular area increase in buildings of Use Group E, see Section 502.4.

Note d. For height exceptions for auditoriums in buildings of Use Groups A-4 and E, see Section 503.2.

Note e. For height exceptions for day care centers of Type 5 construction, see Section 503.3.

Note f. For exceptions to height and area limitations for buildings of Use Group H, see Article 6 governing the specific use. For other special fireresistive requirements governing specific uses, see Section 904.0.

Note g. For exceptions to height of buildings for Use Group R-2 of Types 2B and 3A construction, see Section 904.2.

Note h. For height and area exceptions for open parking structures, see Section 607.0.

Note i. For exceptions to height and area limitations for special industrial uses, see Section 501.1.1.

Note j. 1 foot = 304.8 mm; 1 square foot = 0.093 m².

CHART 2

Table 401
FIRERESISTANCE RATINGS OF STRUCTURE ELEMENTS (IN HOURS)

Structure element		Type of construction Section 401.0									
		Noncombustible					Noncombustible/Combustible			Combustible	
		Type 1 Section 402.0		Type 2 Section 403.0			Type 3 Section 404.0		Type 4 Section 405.0	Type 5 Section 406.0	
		Protected		Protected		Unprotected	Protected	Unprotected	Heavy timber Note c	Protected	Unprotected
Note a		1A	1B	2A	2B	2C	3A	3B	4	5A	5B
1 Exterior walls	Loadbearing	4	3	2	1	0	2	2	2	1	0
		Not less than the rating based on fire separation distance (see Section 905.2)									
	Nonloadbearing	Not less than the rating based on fire separation distance (see Section 905.2)									
2 Fire walls and party walls (Section 907.0)		4	3	2	2	2	2	2	2	2	2
		Not less than the rating required by Table 907.1									
3 Fire separation assemblies (Section 909.0)	Fire enclosure of of exits (Sections 817.11, 909.0 and Note b)	2	2	2	2	2	2	2	2	2	2
	Shafts (other than exits) and elevator hoistways (Sections 909.0, 915.0 and Note b)	2	2	2	2	2	2	2	2	1	1
	Mixed use separation (Section 313.0)	Fireresistance rating corresponding to the rating required by Table 313.1.2									
	Other separation assemblies (Note i)	1	1	1	1	1	1	1	1	1	1
		Note d									
4 Fire partitions (Section 910.0)	Exit access corridors (Notes f, g)	1	1	1	1	1	1	1	1	1	1
		Note d									
	Tenant spaces separations (Note f)	1	1	1	1	0	1	0	1	1	0
		Note d									
5 Dwelling unit separations (Sections 910.0, 913.0 and Notes f and j)		1	1	1	1	1	1	1	1	1	1
		Note d									
6 Smoke barriers (Section 911.0 and Note g)		1	1	1	1	1	1	1	1	1	1
7 Other nonbearing partitions		0	0	0	0	0	0	0	0	0	0
		Note d									
8 Interior bearing walls, bearing partitions, columns, girders, trusses (other than roof trusses) and framing (Section 912.0)	Supporting more than one floor	4	3	2	1	0	1	0	see Sec. 405.0	1	0
	Supporting one floor only or a roof only	3	2	1½	1	0	1	0	see Sec. 405.0	1	0
9 Structural members supporting wall (Section 912.0 and Note g)		3	2	1½	1	0	1	0	1	1	0
		Not less than fireresistance rating of wall supported									
10 Floor construction including beams (Section 913.0 and Note h)		3	2	1½	1	0	1	0	see Sec. 405.0 Note c	1	0
11 Roof construction, including beams, trusses and framing, arches and roof deck (Section 914.0 and Notes e, i)	15' or less in height to lowest member	2	1½	1	1	0	1	0	see Sec. 405.0 Note c	1	0
				Note d							
	More than 15' but less than 20' in height to lowest member	1	1	1	0	0	0	0	see Sec. 405.0	1	0
		Note d									
	20' or more in height to lowest member	0	0	0	0	0	0	0	see Sec. 405.0	0	0
		Note d									

Note a. For fireresistance rating requirements for structural members and assemblies which support other fireresistance rated members or assemblies, see Section 912.1.
Note b. For reductions in the required fireresistance rating of exit and shaft enclosures, see Sections 817.11 and 915.3.
Note c. For substitution of other structural materials for timber in Type 4 construction, see Section 1703.1.1.
Note d. Fireretardant-treated wood permitted, see Sections 904.3 and 1702.4.
Note e. For permitted uses of heavy timber in roof construction in buildings of Types 1 and 2 construction, see Section 914.4.
Note f. For reductions in required fireresistance ratings of exit access corridors, tenant separations and dwelling unit separations, see Section 810.4 and 810.4.1.
Note g. For exceptions to the required fireresistance rating of construction supporting exit access corridor walls, tenant separation walls in covered mall buildings, and smoke barriers, see Sections 911.4 and 912.2.
Note h. For buildings having habitable or occupiable stories or basements below grade, see Section 807.3.1.
Note i. Not less than the rating required by code.
Note j. For Use Group R-3, see Section 309.4.
Note k. 1 foot = 304.8 mm.

Upon further investigation, to determine the required fire rating for columns supporting more than one floor (intersect line 8 under Structure Element with Type 4 construction), Chart 2 refers the reader to the building code because the information is too detailed to be included in a chart. The text of the code under that section states that columns supporting floor loads cannot be less than 8 inches by 8 inches in cross-section or less than 6 inches by 8 inches when supporting roofs.

Building codes are concerned with innumerable items and often run to hundreds and even thousands of pages. Every section of the code obviously has some impact on a building, but not every section has a major impact of a form-giving nature. The list that follows notes code items that often affect a building's overall design. The list is selective; individual architects might include different items or omit some of the items on this list.

1. Total permitted area as a function of construction materials.
2. Total permitted height as a function of construction materials.
3. Number and location of required stairs and exits.
4. Required amount of natural and/or artificial light.
5. Required amount of natural and/or artificial ventilation.
6. Required number and types of plumbing fixtures (for washrooms).
7. Pipe spaces required for plumbing and storm-drainage systems.
8. Heating equipment.
9. Air-conditioning equipment.
10. Elevator machine rooms and shafts.
11. Electric-equipment spaces and shafts.
12. Fire-protection systems.
13. Fire-extinguishing equipment.
14. Total building size as a function of building use.

ZONING ORDINANCES

While building codes tend to tell an architect how a structure can be built, zoning regulations tell him or her where the structure can be built and how bulky it can be. They define the areas of a community in which buildings intended for certain specific uses can be constructed. For example, manufacturing is often allowed only in a particular area, which is usually some distance from residential areas. Zoning ordinances can also limit the overall bulk of buildings and the percentage of the ground they can cover. In addition, they may mandate such things as how many parking spaces must be provided; the amount of open space; yards and plaza sizes; and, in major cities, the type of vertical setbacks that are required.

Among the major items covered by most zoning ordinances are the following:

1. Building use permitted in each area of the community.
2. Lot-area regulations.
3. Yard-size regulations.
4. Building height and setback requirements.
5. Distances between buildings.
6. Parking and truck-dock requirements.
7. Ratio of floor area to total building size.
8. Ratio of open space on the ground to the maximum height of the structure.

RELATIONSHIP BETWEEN CODES AND ZONING ORDINANCES

Apparent jurisdictional overlap may occur between zoning ordinances and building codes. For example, they could conflict over minimum side-yard requirements. A zoning ordinance may specify the size of a yard or set back from a property line that must be provided for a particular type of building, while the building code may establish a minimum yard dimension that is required in order to provide adequate light and ventilation for a window facing onto a yard. Often these requirements are not the same, and since both requirements must be met, the stricter of the two prevails.

Another type of conflict can occur when a building code does not limit a building's height provided proper fire rated materials are used in its construction, but the town zoning ordinance states that no building can be more than, for example, eight stories high. Or the reverse situation might apply: that is, a town zoning ordinance might permit an eight-story hotel while the building code specifies that hotels may not be more than four stories high if they are of heavy-timber construction. In either case, a solution must be found that satisfies all requirements, and the decisions that result from such conflicts inevitably influence the design of the building.

Apart from building codes and zoning ordinances, the requirements of special-interest agencies can also affect a building. For example, the Board of Health sets up rules for restaurants and hospitals, and the Department of Labor has requirements to protect workers such as mandating guardrails or window ledges to protect window washers. Many of these types of rules will affect the design of buildings. The list of special-interest requirements is enormous, but fortunately their effect on the design of a building is relatively minor, especially when compared with the requirements of the building codes and zoning ordinances. Occasionally, though, a special-interest agency regulation does influence the design of a building.

EXIT REQUIREMENTS

One of the decisive form-givers in any major building is the location of its required means of egress. This is a separate problem from the location of decorative or ceremonial stairs, which codes refer to as "convenient" or "ornamental" stairs.

Although building codes go into minute detail describing exit requirements and the way in which exiting enclosures must be constructed, the five points that follow have a major impact on the overall building design.

1. Use of the building, for example, as an office, store, or school.

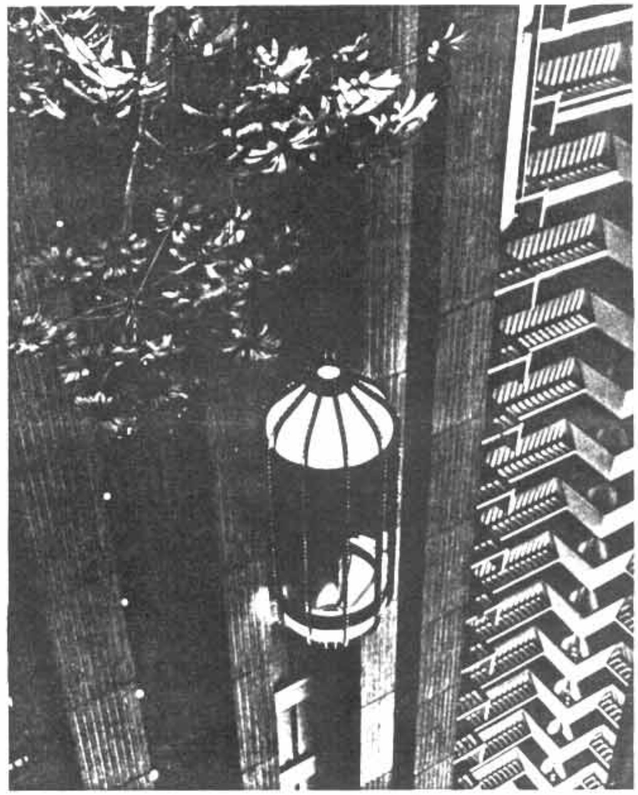
2. Total number of people in the building as a determinant of the required number of separate exits.

3. Limitations on the maximum travel distance permitted to reach an exit enclosure.

4. Provision for a choice of paths to an exit, and a choice of exits in case one exit is blocked.

5. Provision that exits must lead the occupants to a safe area.

Items 1, 2, and 5 are an automatic spellout of the codes. Items 3 and 4 require proper proportioning and shaping of spaces by the architect in order to comply with a specific maximum travel distance and a specific maximum dead-end corridor length. This proportioning can have a dramatic effect on the overall shape of the building. For example, most codes will not permit any dead end corridors in a hospital; therefore, the stairs must be located at the ends of the building.



Elevators can make a major design contribution or be part of a nondescript core, but in either case they may not count as a means of exiting as they could fail during an emergency. Hyatt Regency Hotel, Atlanta, Georgia.

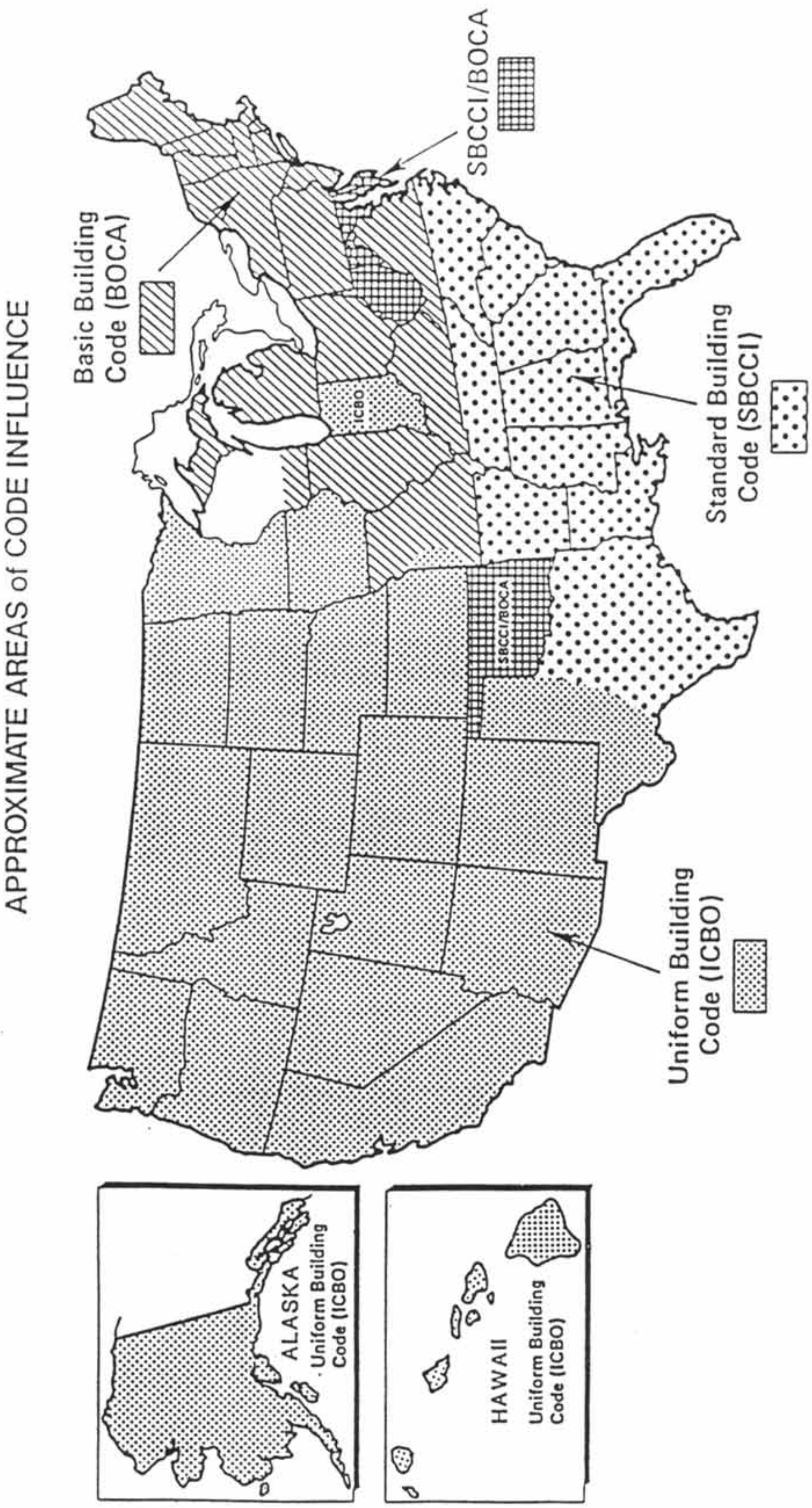


Figure 1. Geographical Influence of Model Codes (After Perry, 1986)

Perry

SEI/ASCE 7-10:
Minimum Design Loads for Buildings and Other Structures

Chapter 1
GENERAL

1.1 SCOPE

This standard provides minimum load requirements for the design of buildings and other structures that are subject to building code requirements. Loads and appropriate load combinations, which have been developed to be used together, are set forth for strength design and allowable stress design. For design strengths and allowable stress limits, design specifications for conventional structural materials used in buildings and modifications contained in this standard shall be followed.

1.2 DEFINITIONS AND NOTATIONS

1.2.1 Definitions

The following definitions apply to the provisions of the entire standard.

ALLOWABLE STRESS DESIGN: A method of proportioning structural members such that elastically computed stresses produced in the members by nominal loads do not exceed specified allowable stresses (also called "working stress design").

AUTHORITY HAVING JURISDICTION: The organization, political subdivision, office, or individual charged with the responsibility of administering and enforcing the provisions of this standard.

BUILDINGS: Structures, usually enclosed by walls and a roof, constructed to provide support or shelter for an intended occupancy.

DESIGN STRENGTH: The product of the nominal strength and a resistance factor.

ESSENTIAL FACILITIES: Buildings and other structures that are intended to remain operational in the event of extreme environmental loading from flood, wind, snow, or earthquakes.

FACTORED LOAD: The product of the nominal load and a load factor.

HIGHLY TOXIC SUBSTANCE: As defined in 29 CFR 1910.1200 Appendix A with Amendments as of February 1, 2000.

IMPORTANCE FACTOR: A factor that accounts for the degree of risk to human life, health, and welfare associated with damage to property or loss of use or functionality.

LIMIT STATE: A condition beyond which a structure or member becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).

LOAD EFFECTS: Forces and deformations produced in structural members by the applied loads.

LOAD FACTOR: A factor that accounts for deviations of the actual load from the nominal load, for uncertainties in the analysis that transforms the load into a load effect, and for the probability that more than one extreme load will occur simultaneously.

LOADS: Forces or other actions that result from the weight of all building materials, occupants and their possessions, environmental effects, differential movement, and restrained dimensional changes. Permanent loads are those loads in which variations over time are rare or of small magnitude. All other loads are variable loads (see also "nominal loads").

NOMINAL LOADS: The magnitudes of the loads specified in this standard for dead, live, soil, wind, snow, rain, flood, and earthquake.

NOMINAL STRENGTH: The capacity of a structure or member to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

OCCUPANCY: The purpose for which a building or other structure, or part thereof, is used or intended to be used.

OTHER STRUCTURES: Structures, other than buildings, for which loads are specified in this standard.

P-DELTA EFFECT: The second order effect on shears and moments of frame members induced by axial loads on a laterally displaced building frame.

RESISTANCE FACTOR: A factor that accounts for deviations of the actual strength from the nominal strength and the manner and consequences of failure (also called "strength reduction factor").

RISK CATEGORY: A categorization of buildings and other structures for determination of flood, wind, snow, ice, and earthquake loads based on the risk associated with unacceptable performance. See Table 1.5-1.

STRENGTH DESIGN: A method of proportioning structural members such that the computed forces produced in the members by the factored loads do not

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent a low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life.	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure.	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where their quantity exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.	
Buildings and other structures designated as essential facilities.	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community.	
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity exceeds a threshold quantity established by the authority having jurisdiction to be dangerous to the public if released and is sufficient to pose a threat to the public if released.[a]	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the substances is commensurate with the risk associated with that Risk Category.

exceed the member design strength (also called "load and resistance factor design").

TEMPORARY FACILITIES: Buildings or other structures that are to be in service for a limited time and have a limited exposure period for environmental loadings.

TOXIC SUBSTANCE: As defined in 29 CFR 1910.1200 Appendix A with Amendments as of February 1, 2000.

1.1.2 Symbols and Notations

F_x A minimum design lateral force applied to level x of the structure and used for purposes of evaluating structural integrity in accordance with Section 1.4.2.

W_x The portion of the total dead load of the structure, D, located or assigned to Level x.

D Dead load.

L Live load.

L_r Roof live load.

N Notional load used to evaluate conformance with minimum structural integrity criteria.

R Rain load.

S Snow load.

1.3 BASIC REQUIREMENTS

1.3.1 Strength and Stiffness

Buildings and other structures, and all parts thereof, shall be designed and constructed with adequate strength and stiffness to provide structural stability, protect nonstructural components and systems from unacceptable damage, and meet the serviceability requirements of Section 1.3.2.

Acceptable strength shall be demonstrated using one or more of the following procedures:

a. the Strength Procedures of Section 1.3.1.1,
b. the Allowable Stress Procedures of Section 1.3.1.2, or
c. subject to the approval of the authority having jurisdiction for individual projects, the Performance-Based Procedures of Section 1.3.1.3.

It shall be permitted to use alternative procedures for different parts of a structure and for different load combinations, subject to the limitations of Chapter 2. Where resistance to extraordinary events is considered, the procedures of Section 2.5 shall be used.

1.3.1.1 Strength Procedures

Structural and nonstructural components and their connections shall have adequate strength to resist the applicable load combinations of Section 2.3 of this Standard without exceeding the applicable strength limit states for the materials of construction.

1.3.1.2 Allowable Stress Procedures

Structural and nonstructural components and their connections shall have adequate strength to resist the applicable load combinations of Section 2.4 of this Standard without exceeding the applicable allowable stresses for the materials of construction.

1.3.1.3 Performance-Based Procedures

Structural and nonstructural components and their connections shall be demonstrated by analysis or by a combination of analysis and testing to provide a reliability not less than that expected for similar components designed in accordance with the Strength Procedures of Section 1.3.1.1 when subject to the influence of dead, live, environmental, and other loads. Consideration shall be given to uncertainties in loading and resistance.

1.3.1.3.1 Analysis Analysis shall employ rational methods based on accepted principles of engineering mechanics and shall consider all significant sources of deformation and resistance. Assumptions of stiffness, strength, damping, and other properties of components and connections incorporated in the analysis shall be based on approved test data or referenced Standards.

1.3.1.3.2 Testing Testing used to substantiate the performance capability of structural and nonstructural components and their connections under load shall accurately represent the materials, configuration, construction, loading intensity, and boundary conditions anticipated in the structure. Where an approved industry standard or practice that governs the testing of similar components exists, the test program and determination of design values from the test program shall be in accordance with those industry standards and practices. Where such standards or practices do not exist, specimens shall be constructed to a scale similar to that of the intended application unless it can be demonstrated that scale effects are not significant to the indicated performance. Evaluation of test results shall be made on the basis of the values obtained from not less than 3 tests, provided that the deviation of any value obtained from any single test does not vary from the average value for all tests by more than 15%. If such deviaton from the average value for any test exceeds 15%, then additional tests shall be performed until the deviation of any test from the average value does not exceed 15% or a minimum of 6 tests have been performed. No test shall be eliminated unless a rationale for its exclusion is given. Test reports shall document the location, the time and date of the test, the characteristics of the tested specimen, the laboratory facilities, the test configuration, the applied loading and deformation under load, and the occurrence of any damage sustained by the specimen, together with the loading and deformation at which such damage occurred.

1.3.1.3.3 Documentation The procedures used to demonstrate compliance with this section and the results of analysis and testing shall be documented in one or more reports submitted to the authority having jurisdiction and to an independent peer review.

1.3.1.3.4 Peer Review The procedures and results of analysis, testing, and calculation used to demonstrate compliance with the requirements of this section shall be subject to an independent peer review approved by the authority having jurisdiction. The peer review shall comprise one or more persons having the necessary expertise and knowledge to evaluate compliance, including knowledge of the expected performance, the structural and component behavior, the particular loads considered, structural analysis of the type performed, the materials of construction, and laboratory testing of elements and components to determine structural resistance and performance characteristics. The review shall include the assumptions, criteria, procedures, calculations, analytical models, test setup, test data, final drawings, and reports. Upon satisfactory completion, the peer review shall submit a letter to the authority having jurisdiction indicating the scope of their review and their findings.

1.3.2 Serviceability

Structural systems, and members thereof, shall be designed to have adequate stiffness to limit deflections, lateral drift, vibration, or any other deformations that adversely affect the intended use and performance of buildings and other structures.

Chapter 2
COMBINATIONS OF LOADS

2.1 GENERAL

Buildings and other structures shall be designed using the provisions of either Section 2.3 or 2.4. Where elements of a structure are designed by a particular material standard or specification, they shall be designed exclusively by either Section 2.3 or 2.4.

2.2 SYMBOLS

A_k = load or load effect arising from extra ordinary event A
D = dead load
D_i = weight of ice
E = earthquake load
F = load due to fluids with well-defined pressures and maximum heights
F_a = flood load
H = load due to lateral earth pressure, ground water pressure, or pressure of bulk materials
L = live load
L_r = roof live load
R = rain load
S = snow load
T = self-straining load
W = wind load
W_i = wind-on-ice determined in accordance with Chapter 10

2.3 COMBINING FACTORED LOADS USING STRENGTH DESIGN

2.3.1 Applicability

The load combinations and load factors given in Section 2.3.2 shall be used only in those cases in which they are specifically authorized by the applicable material design standard.

2.3.2 Basic Combinations

Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

EXCEPTIONS:

1. The load factor on L in combinations 3, 4, and 5 is permitted to equal 0.5 for all occupancies in which L_o in Table 4-1 is less than or equal to 100 psf, with the exception of garages or areas occupied as places of public assembly.
2. In combinations 2, 4, and 5, the companion load S shall be taken as either the flat roof snow load (p_f) or the sloped roof snow load (p_s).

Where fluid loads F are present, they shall be included with the same load factor as dead load D in combinations 1 through 5 and 7.

Where load H are present, they shall be included as follows:

1. where the effect of H adds to the primary variable load effect, include H with a load factor of 1.6;
2. where the effect of H resists the primary variable load effect, include H with a load factor of 0.9 where the load is permanent or a load factor of 0 for all other conditions.

Effects of one or more loads not acting shall be investigated. The most unfavorable effects from both wind and earthquake loads shall be investigated, where appropriate, but they need not be considered to act simultaneously. Refer to Section 12.4 for specific definition of the earthquake load effect E.[1]

Each relevant strength limit state shall be investigated.

2.3.3 Load Combinations Including Flood Load

When a structure is located in a flood zone (Section 5.3.1), the following load combinations shall be considered in addition to the basic combinations in Section 2.3.2:

1. In V-Zones or Coastal A-Zones, $1.0W$ in combinations 4 and 6 shall be replaced by $1.0W + 2.0F_a$.
2. In noncoastal A-Zones, $1.0W$ in combinations 4 and 6 shall be replaced by $0.5W + 1.0F_a$.

[1]The same E from Sections 1.4 and 12.4 is used for both Sections 2.3.2 and 2.4.1. Refer to the Chapter 11 Commentary for the Seismic Provisions.

2.3.4. Load Combinations Including Atmospheric Ice Loads

When a structure is subjected to atmospheric ice and wind-on-ice loads, the following load combinations shall be considered:

1. $0.5(L_r$ or S or $R)$ in combination 2 shall be replaced by $0.2D_i + 0.5S$.
2. $1.0W + 0.5(L_r$ or S or $R)$ in combination 4 shall be replaced by $D_i + W_i + 0.5S$.
3. $1.0W$ in combination 6 shall be replaced by $D_i + W_i$.

2.3.5 Load Combinations Including Self-Straining Loads

Where applicable, the structural effects of load T shall be considered in combination with other loads. The load factor on load T shall be established considering the uncertainty associated with the likely magnitude of the load, the probability that the maximum effect of T will occur simultaneously with other applied loadings, and the potential adverse consequences if the effect of T is greater than assumed. The load factor on T shall not have a value less than 1.0.

2.3.6 Load Combinations for Nonspecified Loads

Where approved by the Authority Having Jurisdiction, the Responsible Design Professional is permitted to determine the combined load effect for strength design using a method that is consistent with the method on which the load combination requirements in Section 2.3.2 are based. Such a method must be probability-based and must be accompanied by documentation regarding the analysis and collection of supporting data that is acceptable to the Authority Having Jurisdiction.

2.4 COMBINING NOMINAL LOADS USING ALLOWABLE STRESS DESIGN

2.4.1 Basic Combinations

Loads listed herein shall be considered to act in the following combinations; whichever produces the most unfavorable effect in the building, foundation, or structural member being considered. Effects of one or more loads not acting shall be considered.

1. D
2. $D + L$
3. $D + (L_r$ or S or $R)$
4. $D + 0.75L + 0.75(L_r$ or S or $R)$
5. $D + (0.6W$ or $0.7E)$

6a. $D + 0.75L + 0.75(0.6W) + 0.75(L_r$ or S or $R)$

6b. $D + 0.75L + 0.75(0.7E) + 0.75S$

7. $0.6D + 0.6W$
8. $0.6D + 0.7E$

EXCEPTIONS:

1. In combinations 4 and 6, the companion load S shall be taken as either the flat roof snow load (p_f) or the sloped roof snow load (p_s).
2. For nonbuilding structures, in which the wind load is determined from force coefficients, C_f, identified in Figures 29.5-1, 29.5-2 and 29.5-3 and the projected area contributing wind force to a foundation element exceeds 1,000 square feet on either a vertical or a horizontal plane, it shall be permitted to replace W with $0.9W$ in combination 7 for design of the foundation, excluding anchorage of the structure to the foundation.
3. It shall be permitted to replace $0.6D$ with $0.9D$ in combination 8 for the design of Special Reinforced Masonry Shear Walls, where the walls satisfy the requirement of Section 14.4.2.

Where fluid loads F are present, they shall be included in combinations 1 through 6 and 8 with the same factor as that used for dead load D.

Where load H is present, it shall be included as follows:

1. where the effect of H adds to the primary variable load effect, include H with a load factor of 1.0;
2. where the effect of H resists the primary variable load effect, include H with a load factor of 0.6 where the load is permanent or a load factor of 0 for all other conditions.

The most unfavorable effects from both wind and earthquake loads shall be considered, where appropriate, but they need not be assumed to act simultaneously. Refer to Section 1.4 and 12.4 for the specific definition of the earthquake load effect E.[2]

Increases in allowable stress shall not be used with the loads or load combinations given in this standard unless it can be demonstrated that such an increase is justified by structural behavior caused by rate or duration of load.

[2]The same E from Sections 1.4 and 12.4 is used for both Sections 2.3.2 and 2.4.1. Refer to the Chapter 11 Commentary for the Seismic Provisions.

2.4.2 Load Combinations Including Flood Load

When a structure is located in a flood zone, the following load combinations shall be considered in addition to the basic combinations in Section 2.4.1:

1. In V-Zones or Coastal A-Zones (Section 5.3.1), $1.5F_a$ shall be added to other loads in combinations 5, 6, and 7, and E shall be set equal to zero in 5 and 6.
2. In non-coastal A-Zones, $0.75F_a$ shall be added to combinations 5, 6, and 7, and E shall be set equal to zero in 5 and 6.

2.4.3 Load Combinations Including Atmospheric Ice Loads

When a structure is subjected to atmospheric ice and wind-on-ice loads, the following load combinations shall be considered:

1. $0.7D_i$ shall be added to combination 2.
2. (L_r or S or R) in combination 3 shall be replaced by $0.7D_i + 0.7W_i + S$.
3. $0.6W$ in combination 7 shall be replaced by $0.7D_i + 0.7W_i$.

2.4.4 Load Combinations Including Self-Straining Loads

Where applicable, the structural effects of load T shall be considered in combination with other loads. Where the maximum effect of load T is unlikely to occur simultaneously with the maximum effects of other variable loads, it shall be permitted to reduce the magnitude of T considered in combination with these other loads. The fraction of T considered in combination with other loads shall not be less than 0.75.

2.5 LOAD COMBINATIONS FOR EXTRAORDINARY EVENTS

2.5.1 Applicability

Where required by the owner or applicable code, strength and stability shall be checked to ensure that structures are capable of withstanding the effects of extraordinary (i.e., low-probability) events, such as fires, explosions, and vehicular impact without disproportionate collapse.

2.5.2 Load Combinations

2.5.2.1 Capacity

For checking the capacity of a structure or structural element to withstand the effect of an extraordinary event, the following gravity load combination shall be considered:

$$(0.9 \text{ or } 1.2)D + A_k + 0.5L + 0.2S \qquad (2.5\text{-}1)$$

in which A_k = the load or load effect resulting from extraordinary event A.

2.5.2.2 Residual Capacity

For checking the residual load-carrying capacity of a structure or structural element following the occurrence of a damaging event, selected load-bearing elements identified by the Responsible Design Professional shall be notionally removed, and the capacity of the damaged structure shall be evaluated using the following gravity load combination:

$$(0.9 \text{ or } 1.2)D + 0.5L + 0.2(L_r \text{ or } S \text{ or } R) \qquad (2.5\text{-}2)$$

2.5.3 Stability Requirements

Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of second-order effects is permitted.

Building Material Weights
AISC Manual of Load and Resistance Factor Design, 3rd ed.

Table 17-12. Weights and Specific Gravities

Substance	Weight lb per cu ft	Specific Gravity
ASHLAR, MASONRY		
Granite, syenite, gneiss	165	2.3–3.0
Limestone, marble	160	2.3–2.8
Sandstone, bluestone	140	2.1–2.4
MORTAR RUBBLE MASONRY		
Granite, syenite, gneiss	155	2.2–2.8
Limestone, marble	150	2.2–2.6
Sandstone, bluestone	130	2.0–2.2
DRY RUBBLE MASONRY		
Granite, syenite, gneiss	130	1.9–2.3
Limestone, marble	125	1.9–2.1
Sandstone, bluestone	110	1.8–1.9
BRICK MASONRY		
Pressed brick	140	2.2–2.3
Common brick	120	1.8–2.0
Soft brick	100	1.5–1.7
CONCRETE MASONRY		
Cement, stone, sand	144	2.2–2.4
Cement, slag, etc.	130	1.9–2.3
Cement, cinder, etc.	100	1.5–1.7
VARIOUS BUILDING MATERIALS		
Ashes, cinders	40–45	–
Cement, portland, loose	90	–
Cement, portland, set	183	2.7–3.2
Lime, gypsum, loose	53–64	–
Mortar, set	103	1.4–1.9
Slags, bank slag	67–72	–
Slags, bank screenings	98–117	–
Slags, machine slag	96	–
Slags, slag sand	49–55	–
EARTH, ETC., EXCAVATED		
Clay, dry	63	–
Clay, damp, plastic	110	–
Clay and gravel, dry	100	–
Earth, dry, loose	76	–
Earth, dry, packed	95	–
Earth, moist, loose	78	–
Earth, moist, packed	96	–
Earth, mud, flowing	108	–
Earth, mud, packed	115	–
Riprap, limestone	80–85	–
Riprap, sandstone	90	–
Riprap, shale	105	–
Sand, gravel, dry, loose	90–105	–
Sand, gravel, dry, packed	100–120	–
Sand, gravel, wet	118–120	–
EXCAVATIONS IN WATER		
Sand or gravel	60	–
Sand or gravel and clay	65	–
Clay	80	–
River mud	90	–
Soil	70	–
Stone riprap	65	–

Substance	Weight lb per cu ft	Specific Gravity
MINERALS		
Asbestos	153	2.1–2.8
Barytes	281	4.50
Basalt	184	2.7–3.2
Bauxite	159	2.55
Borax	109	1.7–1.8
Chalk	137	1.8–2.6
Clay, marl	137	1.8–2.6
Dolomite	181	2.9
Feldspar, orthoclase	159	2.5–2.6
Gneiss, serpentine	159	2.4–2.7
Granite, syenite	175	2.5–3.1
Greenstone, trap	187	2.8–3.2
Gypsum, alabaster	159	2.3–2.8
Hornblende	187	3.0
Limestone, marble	165	2.5–2.8
Magnesite	187	3.0
Phosphate rock, apatite	200	3.2
Porphyry	172	2.6–2.9
Pumice, natural	40	0.37–0.90
Quartz, flint	165	2.5–2.8
Sandstone, bluestone	147	2.2–2.5
Shale, slate	175	2.7–2.9
Soapstone, talc	169	2.6–2.8
STONE, QUARRIED, PILED		
Basalt, granite, gneiss	96	–
Limestone, marble, quartz	95	–
Sandstone	82	–
Shale	92	–
Greenstone, hornblende	107	–
BITUMINOUS SUBSTANCES		
Asphaltum	81	1.1–1.5
Coal, anthracite	97	1.4–1.7
Coal, bituminous	84	1.2–1.5
Coal, lignite	78	1.1–1.4
Coal, peat, turf, dry	47	0.65–0.85
Coal, charcoal, pine	23	0.28–0.44
Coal, charcoal, oak	33	0.47–0.57
Coal, coke	75	1.0–1.4
Graphite	131	1.9–2.3
Paraffin	56	0.87–0.91
Petroleum	54	0.87
Petroleum, refined	50	0.79–0.82
Petroleum, benzine	46	0.73–0.75
Petroleum, gasoline	42	0.66–0.69
Pitch	69	1.07–1.15
Tar, bituminous	75	1.20
COAL AND COKE, PILED		
Coal, anthracite	47–58	–
Coal, bituminous, lignite	40–54	–
Coal, peat, turf	20–26	–
Coal charcoal	10–14	–
Coal coke	23–32	–

The specific gravities of solids and liquids refer to water at 4°C, those of gases to air at 0°C and 760 mm pressure. The weights per cubic foot are derived from average specific gravities, except where stated that weights are for bulk, heaped, or loose material, etc.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

Table 17-12 (cont.). Weights and Specific Gravities

Substance	Weight lb per cu ft	Specific Gravity
METALS, ALLOYS, ORES		
Aluminum, cast, hammered	165	2.55–2.75
Brass, cast, rolled	534	8.4–8.7
Bronze, 7.9 to 14% Sn	509	7.4–8.9
Bronze, aluminum	481	7.7
Copper, cast, rolled	556	8.8–9.0
Copper ore, pyrites	262	4.1–4.3
Gold, cast, hammered	1205	19.25–19.3
Iron, cast, pig	450	7.2
Iron, wrought	485	7.6–7.9
Iron, speigel-eisen	468	7.5
Iron, ferro-silicon	437	6.7–7.3
Iron ore, hematite	325	5.2
Iron ore, hematite in bank	160–180	–
Iron ore, hematite loose	130–160	–
Iron ore, limonite	237	3.6–4.0
Iron ore, magnetite	315	4.9–5.2
Iron slag	172	2.5–3.0
Lead	710	11.37
Lead ore, galena	465	7.3–7.6
Magnesium, alloys	112	1.74–1.83
Manganese	475	7.2–8.0
Manganese ore, pyrolusite	259	3.7–4.6
Mercury	849	13.6
Monel Metal	556	8.8–9.0
Nickel	565	8.9–9.2
Platinum, cast, hammered	1330	21.1–21.5
Silver, cast, hammered	656	10.4–10.6
Steel, rolled	490	7.85
Tin, cast, hammered	459	7.2–7.5
Tin ore, cassiterite	418	6.4–7.0
Zinc, cast, rolled	440	6.9–7.2
Zinc ore, blende	253	3.9–4.2
VARIOUS SOLIDS		
Cereals, oats ... bulk	32	–
Cereals, barley ... bulk	39	–
Cereals, corn, rye ... bulk	48	–
Cereals, wheat ... bulk	48	–
Hay and Straw ... bales	20	–
Cotton, Flax, Hemp	93	1.47–1.50
Fats	58	0.90–0.97
Flour, loose	28	0.40–0.50
Flour, pressed	47	0.70–0.80
Glass, common	156	2.40–2.60
Glass, plate or crown	161	2.45–2.72
Glass, crystal	184	2.90–3.00
Leather	59	0.86–1.02
Paper	58	0.70–1.15
Potatoes, piled	42	–
Rubber, caoutchouc	59	0.92–0.96
Rubber goods	94	1.0–2.0
Salt, granulated, piled	48	–
Saltpeter	67	–
Starch	96	1.53
Sulphur	125	1.93–2.07
Wool	82	1.32

Substance	Weight lb per cu ft	Specific Gravity
TIMBER, U.S. SEASONED		
Moisture content by weight:		
Seasoned timber 15 to 20%		
Green timber up to 50%		
Ash, white, red	40	0.62–0.65
Cedar, white, red	22	0.32–.038
Chestnut	41	0.66
Cypress	30	0.48
Fir, Douglas spruce	32	0.51
Fir, eastern	25	0.40
Elm, white	45	0.72
Hemlock	29	0.42–0.52
Hickory	49	0.74–0.84
Locust	46	0.73
Maple, hard	43	0.68
Maple, white	33	0.53
Oak, chestnut	54	0.86
Oak, live	59	0.95
Oak, red, black	41	0.65
Oak, white	46	0.74
Pine, Oregon	32	0.51
Pine, red	30	0.48
Pine, white	26	0.41
Pine, yellow, long-leaf	44	0.70
Pine, yellow, short-leaf	38	0.61
Poplar	30	0.48
Redwood, California	26	0.42
Spruce, white, black	27	0.40–0.46
Walnut, black	38	0.61
Walnut, white	26	0.41
VARIOUS LIQUIDS		
Alcohol, 100%	49	0.79
Acids, muriatic 40%	75	1.20
Acids, nitric 91%	94	1.50
Acids, sulphuric 87%	112	1.80
Lye, soda 66%	106	1.70
Oils, vegetable	58	0.91–0.94
Oils, mineral, lubricants	57	0.90–0.93
Water, 4°C max. density	62.428	1.0
Water, 100°C	59.830	0.9584
Water, ice	56	0.88–0.92
Water, snow, fresh fallen	8	.125
Water, sea water	64	1.02–1.03
GASES		
Air, 0°C 760 mm	.08071	1.0
Ammonia	.0478	0.5920
Carbon dioxide	.1234	1.5291
Carbon monoxide	.0781	0.9673
Gas, illuminating	.028–.036	0.35–0.45
Gas, natural	.038–.039	0.47–0.48
Hydrogen	.00559	0.0693
Nitrogen	.0784	0.9714
Oxygen	.0892	1.1056

The specific gravities of solids and liquids refer to water at 4°C, those of gases to air at 0°C and 760 mm pressure. The weights per cubic foot are derived from average specific gravities, except where stated that weights are for bulk, heaped, or loose material, etc.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

Table 17-13. Weights of Building Materials

Materials	Weight lb per sq ft
CEILINGS	
Channel suspended system	1
Lathing and plastering	See Partitions
Acoustical fiber tile	1
FLOORS	
Steel deck	See Manufacturer
Concrete-Reinforced 1 in.	
Stone	12 1/2
Slag	11 1/2
Lightweight	6 to 10
Concrete-Plain 1 in.	
Stone	12
Slag	11
Lightweight	3 to 9
Fills 1 inch	
Gypsum	6
Sand	8
Cinders	4
Finishes	
Terrazzo 1 in.	13
Ceramic or Quarry Tile 3/4-in.	10
Linoleum 1/4-in.	1
Mastic 3/4-in.	9
Hardwood 7/8-in.	4
Softwood 3/4-in.	2 1/2
ROOFS	
Copper or tin	1
Corrugated steel	See Manufactuer
3-ply ready roofing	1
3-ply felt and gravel	5 1/2
5-ply felt and gravel	6
Shingles	
Wood	2
Asphalt	3
Clay tile	9 to 14
Slate 1/4 in.	10
Sheathing	
Wood 3/4 in.	3
Gypsum 1 in.	4
Insulation 1 in.	
Loose	1/2
Poured	2
Rigid	1 1/2

Materials	Weight lb per sq ft
PARTITIONS	
Clay tile	
3 in.	17
4 in.	18
6 in.	28
8 in.	34
10 in.	40
Gypsum block	
2 in.	9 1/2
3 in.	10 1/2
4 in.	12 1/2
5 in.	14
6 in.	18 1/2
Wood studs 2×4	
12–16 in. o.c.	2
Steel partitions	4
Plaster 1 in.	
Cement	10
Gypsum	5
Lathing	
Metal	1/2
Gypsum board 1/2 in.	2
WALLS	
Brick	
4 in.	40
8 in.	80
12 in.	120
Hollow concrete block (Heavy aggregate)	
4 in.	30
6 in.	43
8 in.	55
12 1/2 in.	80
Hollow concrete block (Light aggregate)	
4 in.	21
6 in.	30
8 in.	38
12 in.	55
Clay tile (Load bearing)	
4 in.	25
6 in.	30
8 in.	33
12 in.	45
Stone 4 in.	55
Glass block 4 in.	18
Window, Glass, Frame, & Sash	8
Curtain walls	See Manufacturer
Structural glass 1 in.	15
Corrugated Cement Asbestos 1/4 in.	3

For weights of other materials used in building construction, see Table 17-12.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

Table 17-14. Weights and Measures United States System

LINEAR MEASURE

Inches	Feet	Yards	Rods	Furlongs	Miles
1.0 =	.08333 =	.02778 =	.0050505 =	.00012626 =	.00001578
12.0 =	1.0 =	.33333 =	.0606061 =	.00151515 =	.00018939
36.0 =	3.0 =	1.0 =	.1818182 =	.00454545 =	.00056818
198.0 =	16.5 =	5.5 =	1.0 =	.025 =	.003125
7,920.0 =	660.0 =	220.0 =	40.0 =	1.0 =	.125
63,360.0 =	5,280.0 =	1,760.0 =	320.0 =	8.0 =	1.0

SQUARE AND LAND MEASURE

Sq. Inches	Square Feet	Square Yards	Square Rods	Acres	Sq. Miles
1.0 =	.006944 =	.000772			
144.0 =	1.0 =	.111111			
1,296.0 =	9.0 =	1.0 =	.03306 =	.000207	
39,204.0 =	272.25 =	30.25 =	1.0 =	.00625 =	.0000098
	43,560.0 =	4,840.0 =	160.0 =	1.0 =	.0015625
		3,097,600.0 =	102,400.0 =	640.0 =	1.0

AVOIRDUPOIS WEIGHTS

Grains	Drams	Ounces	Pounds	Tons
1.0 =	.03657 =	.002286 =	.000143 =	.0000000714
27.34375 =	1.0 =	.0625 =	.003906 =	.00000195
437.5 =	16.0 =	1.0 =	.0625 =	.00003125
7,000.0 =	256.0 =	16.0 =	1.0 =	.0005
14,000,000.0 =	512,000.0 =	32,000.0 =	2,000.0 =	1.0

DRY MEASURE

Pints	Quarts	Pecks	Cubic Feet	Bushels
1.0 =	.5 =	.0625 =	.01945 =	.01563
2.0 =	1.0 =	.125 =	.03891 =	.03125
16.0 =	8.0 =	1.0 =	.31112 =	.25
51.42627 =	25.71314 =	3.21414 =	1.0 =	.80354
64.0 =	32.0 =	4.0 =	1.2445 =	1.0

LIQUID MEASURE

Gills	Pints	Quarts	U.S. Gallons	Cubic Feet
1.0 =	.25 =	.125 =	.03125 =	.00418
4.0 =	1.0 =	.5 =	.125 =	.01671
8.0 =	2.0 =	1.0 =	.250 =	.03342
32.0 =	8.0 =	4.0 =	1.0 =	.1337
			7.48052 =	1.0

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

Structural Load Requirements
International Building Code (2012)

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
1. Apartments (see residential)	—	—
2. Access floor systems Office use Computer use	 50 100	 2,000 2,000
3. Armories and drill rooms	150[m]	—
4. Assembly areas Fixed seats (fastened to floor) Follow spot, projections and control rooms Lobbies Movable seats Stage floors Platforms (assembly) Other assembly areas	 60[m] 50 100[m] 100[m] 150[m] 100[m] 100[m]	—
5. Balconies and decks[h]	Same as occupancy served	—
6. Catwalks	40	300
7. Cornices	60	—
8. Corridors First floor Other floors	 100 Same as occupancy served except as indicated	—
9. Dining rooms and restaurants	100[m]	—
10. Dwellings (see residential)	—	—
11. Elevator machine room grating (on area of 2 inches by 2 inches)	—	300
12. Finish light floor plate construction (on area of 1 inch by 1 inch)	—	200
13. Fire escapes On single-family dwellings only	100 40	—
14. Garages (passenger vehicles only) Trucks and buses	40[m] See Section 1607.7	Note a
15. Handrails, guards and grab bars	See Section 1607.8	
16. Helipads	See Section 1607.6	
17. Hospitals Corridors above first floor Operating rooms, laboratories Patient rooms	 80 60 40	 1,000 1,000 1,000
18. Hotels (see residential)	—	—
19. Libraries Corridors above first floor Reading rooms Stack rooms	 80 60 150[b, m]	 1,000 1,000 1,000
20. Manufacturing Heavy Light	 250[m] 125[m]	 3,000 2,000
21. Marquees	75	—
22. Office buildings Corridors above first floor File and computer rooms shall be designed for heavier loads based on anticipated occupancy Lobbies and first-floor corridors Offices	 80 — 100 50	 2,000 — 2,000 2,000

TABLE 1607.1—continued
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
23. Penal institutions Cell blocks Corridors	 40 100	—
24. Recreational uses: Bowling alleys, poolrooms and similar uses Dance halls and ballrooms Gymnasiums Reviewing stands, grandstands and bleachers Stadiums and arenas with fixed seats (fastened to floor)	 75[m] 100[m] 100[m] 100[c, m] 60[c, m]	—
25. Residential One- and two-family dwellings Uninhabitable attics without storage[i] Uninhabitable attics with storage[i, j, k] Habitable attics and sleeping areas[k] All other areas Hotels and multifamily dwellings Private rooms and corridors serving them Public rooms[m] and corridors serving them	 10 20 30 40 40 100	—
26. Roofs All roof surfaces subject to maintenance workers Awnings and canopies: Fabric construction supported by a skeleton structure All other construction Ordinary flat, pitched, and curved roofs (that are not occupiable) Where primary roof members are exposed to a work floor, at single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs: Over manufacturing, storage warehouses, and repair garages All other primary roof members Occupiable roofs: Roof gardens Assembly areas All other similar areas	 5 nonreducible 20 20 100 100[m] Note 1	300 2,000 300 Note 1
27. Schools Classrooms Corridors above first floor First-floor corridors	 40 80 100	 1,000 1,000 1,000
28. Scuttles, skylight ribs and accessible ceilings	—	200
29. Sidewalks, vehicular drive ways and yards, subject to trucking	250[d, m]	8,000[e]

TABLE 1607.1—continued
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
30. Stairs and exits One- and two-family dwellings All other	 40 100	 300[f] 300[f]
31. Storage warehouses (shall be designed for heavier loads if required for anticipated storage) Heavy Light	 250[m] 125[m]	—
32. Stores Retail First floor Upper floors Wholesale, all floors	 100 75 125[m]	 1,000 1,000 1,000
33. Vehicle barriers	See Section 1607.8.3	
34. Walkways and elevated platforms (other than exitways)	60	—
35. Yards and terraces, pedestrians	100[m]	—

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm^2,
1 square foot = 0.0929 m^2,
1 pound per square foot = 0.0479 kN/m^2, 1 pound = 0.004448 kN,
1 pound per cubic foot = 16 kg/m^3.

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 1607.1 or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of 4.5 inches by 4.5 inches; (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds per wheel.

b. The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:
1. The nominal bookstack unit height shall not exceed 90 inches;
2. The nominal shelf depth shall not exceed 12 inches for each face; and
3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.

c. Design in accordance with ICC 300.

d. Other uniform loads in accordance with an approved method containing provisions for truck loadings shall also be considered where appropriate.

e. The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.

f. The minimum concentrated load on stair treads shall be applied on an area of 2 inches by 2 inches. This load need not be assumed to act concurrently with the uniform load.

g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official (see Section 1608).

h. See Section 1604.8.3 for decks attached to exterior walls.

i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.

Live Loads & Allowed Reductions

1607.10 Reduction in uniform live loads. Except for uniform live loads at roofs, all other minimum uniformly distributed live loads, L_o, in Table 1607.1 are permitted to be reduced in accordance with Section 1607.10.1 or 1607.10.2. Uniform live loads at roofs are permitted to be reduced in accordance with Section 1607.12.2.

1607.10.1 Basic uniform live load reduction. Subject to the limitations of Sections 1607.10.1.1 through 1607.10.1.3 and Table 1607.1, members for which a value of $K_{LL}A_T$ is 400 square feet (37.16 m²) or more are permitted to be designed for a reduced uniformly distributed live load, L, in accordance with the following equation:

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \qquad \textbf{(Equation 16-23)}$$

For SI: $L = L_o\left(0.25 + \frac{4.57}{\sqrt{K_{LL}A_T}}\right)$

where:

L = Reduced design live load per square foot (m²) of area supported by the member.

L_o = Unreduced design live load per square foot (m²) of area supported by the member (see Table 1607.1).

K_{LL}= Live load element factor (see Table 1607.10.1).

A_T = Tributary area, in square feet (m²).

L shall not be less than $0.50L_o$ for members supporting one floor and L shall not be less than $0.40L_o$ for members supporting two or more floors.

TABLE 1607.10.1
LIVE LOAD ELEMENT FACTOR, K_{LL}

ELEMENT	K_{LL}
Interior columns Exterior columns without cantilever slabs	4 4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs Edge beams without cantilever slabs Interior beams	2 2 2
All other members not identified above including: Edge beams with cantilever slabs Cantilever beams One-way slabs Two-way slabs Members without provisions for continuous shear transfer normal to their span	1

1607.10.1.1 One-way slabs. The tributary area, A_T, for use in Equation 16-23 for one-way slabs shall not exceed an area defined by the slab span times a width normal to the span of 1.5 times the slab span.

1607.10.1.2 Heavy live loads. Live loads that exceed 100 psf (4.79 kN/m²) shall not be reduced.

Exceptions:

1. The live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than L as calculated in Section 1607.10.1.
2. For uses other than storage, where *approved*, additional live load reductions shall be permitted where shown by the *registered design professional* that a rational approach has been used and that such reductions are warranted.

1607.10.1.3 Passenger vehicle garages. The live loads shall not be reduced in passenger vehicle garages.

Exception: The live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than L as calculated in Section 1607.10.1.

1607.10.2 Alternative uniform live load reduction. As an alternative to Section 1607.10.1 and subject to the limitations of Table 1607.1, uniformly distributed live loads are permitted to be reduced in accordance with the following provisions. Such reductions shall apply to slab systems, beams, girders, columns, piers, walls and foundations.

1. A reduction shall not be permitted where the live load exceeds 100 psf (4.79 kN/m²) except that the design live load for members supporting two or more floors is permitted to be reduced by a maximum of 20 percent.

   Exception: For uses other than storage, where *approved*, additional live load reductions shall be permitted where shown by the *registered design professional* that a rational approach has been used and that such reductions are warranted.

2. A reduction shall not be permitted in passenger vehicle parking garages except that the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent.
3. For live loads not exceeding 100 psf (4.79 kN/m²), the design live load for any structural member supporting 150 square feet (13.94 m²) or more is permitted to be reduced in accordance with Equation 16-24.
4. For one-way slabs, the area, A, for use in Equation 16-24 shall not exceed the product of the slab span and a width normal to the span of 0.5 times the slab span.

$$R = 0.08(A - 150) \qquad \textbf{(Equation 16-24)}$$

For SI: $R = 0.861(A - 13.94)$

Such reduction shall not exceed the smallest of:

1. 40 percent for horizontal members;
2. 60 percent for vertical members; or
3. R as determined by the following equation.

$$R = 23.1(1 + D/L_o) \qquad \textbf{(Equation 16-25)}$$

where:

A = Area of floor supported by the member, square feet (m²).

D = Dead load per square foot (m²) of area supported.

L_o = Unreduced live load per square foot (m²) of area supported.

R = Reduction in percent.

1607.11 Distribution of floor loads. Where uniform floor live loads are involved in the design of structural members arranged so as to create continuity, the minimum applied loads shall be the full dead loads on all spans in combination with the floor live loads on spans selected to produce the greatest *load effect* at each location under consideration. Floor live loads are permitted to be reduced in accordance with Section 1607.10.

Minimum Roof Loads

1607.12 Roof loads. The structural supports of roofs and marquees shall be designed to resist wind and, where applicable, snow and earthquake loads, in addition to the dead load of construction and the appropriate live loads as prescribed in this section, or as set forth in Table 1607.1. The live loads acting on a sloping surface shall be assumed to act vertically on the horizontal projection of that surface.

1607.12.1 Distribution of roof loads. Where uniform roof live loads are reduced to less than 20 psf (0.96 kN/m^2) in accordance with Section 1607.12.2.1 and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the most unfavorable *load effect*. See Section 1607.12.2 for reductions in minimum roof live loads and Section 7.5 of ASCE 7 for partial snow loading.

1607.12.2 General. The minimum uniformly distributed live loads of roofs and marquees, L_o, in Table 1607.1 are permitted to be reduced in accordance with Section 1607.12.2.1.

1607.12.2.1 Ordinary roofs, awnings and canopies. Ordinary flat, pitched and curved roofs, and awnings and canopies other than of fabric construction supported by a skeleton structure, are permitted to be designed for a reduced uniformly distributed roof live load, L_r, as specified in the following equations or other controlling combinations of loads as specified in Section 1605, whichever produces the greater *load effect*.

In structures such as greenhouses, where special scaffolding is used as a work surface for workers and materials during maintenance and repair operations, a lower roof load than specified in the following equations shall not be used unless *approved* by the *building official*. Such structures shall be designed for a minimum roof live load of 12 psf (0.58 kN/m^2).

$L_r = L_o R_1 R_2$ **(Equation 16-26)**

where: $12 \le L_r \le 20$

For SI: $L_r = L_o R_1 R_2$

where: $0.58 \le L_r \le 0.96$

L_o = Unreduced roof live load per square foot (m^2) of horizontal projection supported by the member (see Table 1607.1).

L_r = Reduced roof live load per square foot (m^2) of horizontal projection supported by the member.

The reduction factors R_1 and R_2 shall be determined as follows:

$R_1 = 1$ for $A_t \le 200$ square feet (18.58 m^2) **(Equation 16-27)**

$R_1 = 1.2 - 0.001A_t$ for 200 square feet $< A_t <$ 600 square feet **(Equation 16-28)**

For SI: $1.2 - 0.011A_t$ for 18.58 square meters $< A_t <$ 55.74 square meters

$R_1 = 0.6$ for $A_t \ge 600$ square feet (55.74 m^2) **(Equation 16-29)**

where:

A_t = Tributary area (span length multiplied by effective width) in square feet (m^2) supported by the member, and

$R_2 = 1$ for $F \le 4$ **(Equation 16-30)**

$R_2 = 1.2 - 0.05F$ for $4 < F < 12$ **(Equation 16-31)**

$R_2 = 0.6$ for $F \ge 12$ **(Equation 16-32)**

where:

F = For a sloped roof, the number of inches of rise per foot (for SI: $F = 0.12 \times$ slope, with slope expressed as a percentage), or for an arch or dome, the rise-to-span ratio multiplied by 32.

1607.12.3 Occupiable roofs. Areas of roofs that are occupiable, such as roof gardens, or for assembly or other similar purposes, and marquees are permitted to have their uniformly distributed live loads reduced in accordance with Section 1607.10.

1607.12.3.1 Landscaped roofs. The uniform design live load in unoccupied landscaped areas on roofs shall be 20 psf (0.958 kN/m^2). The weight of all landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil.

1607.12.4 Awnings and canopies. Awnings and canopies shall be designed for uniform live loads as required in Table 1607.1 as well as for snow loads and wind loads as specified in Sections 1608 and 1609.

Minimum Snow Loads

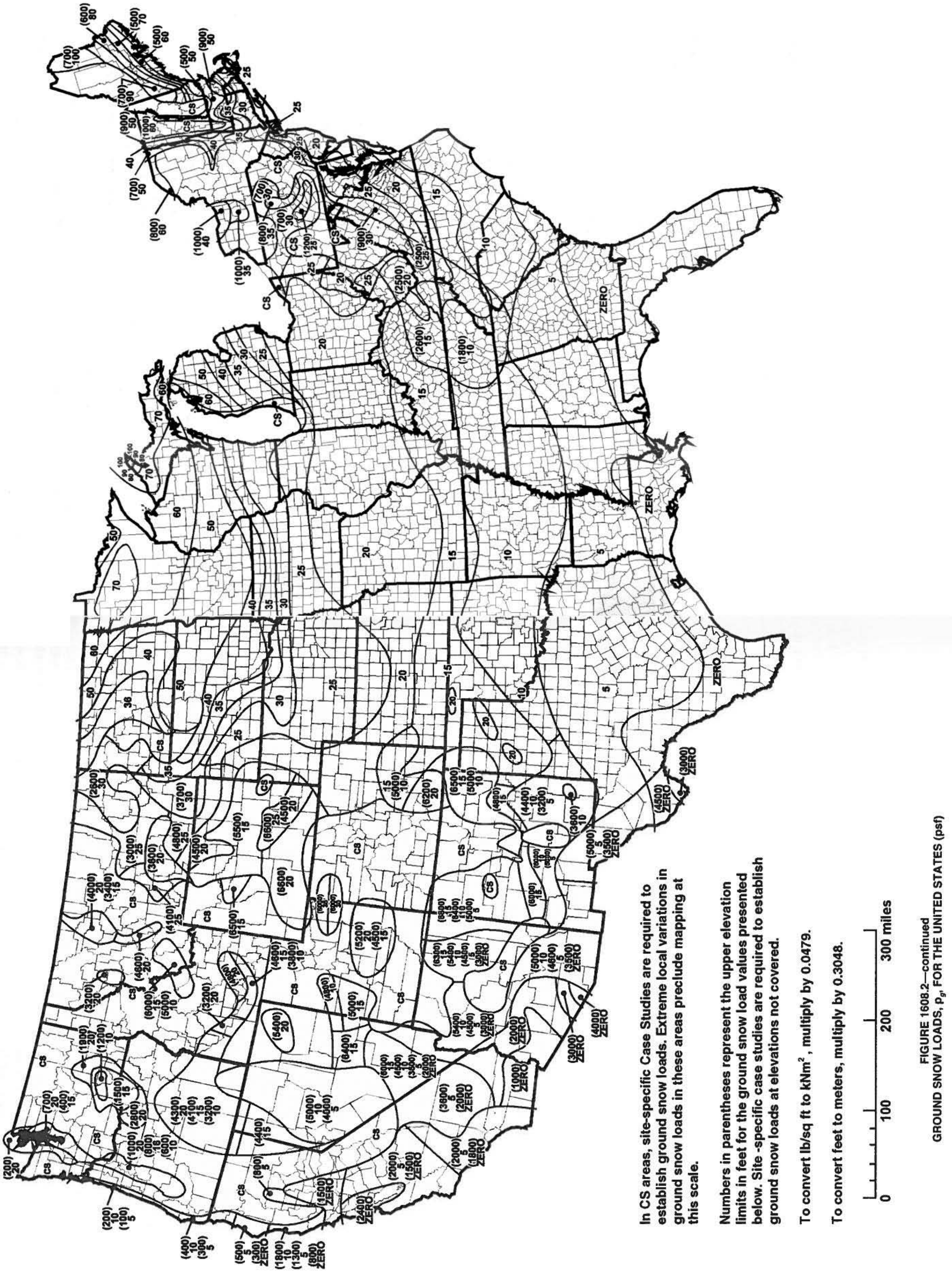


FIGURE 1608.2—continued
GROUND SNOW LOADS, p_g, FOR THE UNITED STATES (psf)

Documentation of Loads

SECTION 1603
CONSTRUCTION DOCUMENTS

1603.1 General. *Construction documents* shall show the size, section and relative locations of structural members with floor levels, column centers and offsets dimensioned. The design loads and other information pertinent to the structural design required by Sections 1603.1.1 through 1603.1.9 shall be indicated on the *construction documents*.

Exception: *Construction documents* for buildings constructed in accordance with the *conventional light-frame construction* provisions of Section 2308 shall indicate the following structural design information:

1. Floor and roof live loads.
2. Ground snow load, P_g.
3. Ultimate design wind speed, V_{ult}, (3-second gust), miles per hour (mph) (km/hr) and nominal design wind speed, V_{asd}, as determined in accordance with Section 1609.3.1 and wind exposure.
4. *Seismic design category* and *site class*.
5. Flood design data, if located in *flood hazard areas* established in Section 1612.3.
6. Design load-bearing values of soils.

1603.1.1 Floor live load. The uniformly distributed, concentrated and impact floor live load used in the design shall be indicated for floor areas. Use of live load reduction in accordance with Section 1607.10 shall be indicated for each type of live load used in the design.

1603.1.2 Roof live load. The roof live load used in the design shall be indicated for roof areas (Section 1607.12).

1603.1.3 Roof snow load data. The ground snow load, P_g, shall be indicated. In areas where the ground snow load, P_g, exceeds 10 pounds per square foot (psf) (0.479 kN/m^2), the following additional information shall also be provided, regardless of whether snow loads govern the design of the roof:

1. Flat-roof snow load, P_f.
2. Snow exposure factor, C_e.
3. Snow load importance factor, *I*.
4. Thermal factor, C_t.

1603.1.4 Wind design data. The following information related to wind loads shall be shown, regardless of whether wind loads govern the design of the lateral force-resisting system of the structure:

1. Ultimate design wind speed, V_{ult}, (3-second gust), miles per hour (km/hr) and nominal design wind speed, V_{asd}, as determined in accordance with Section 1609.3.1.
2. *Risk category*.
3. Wind exposure.Where more than one wind exposure is utilized, the wind exposure and applicable wind direction shall be indicated.
4. The applicable internal pressure coefficient.
5. Components and cladding. The design wind pressures in terms of psf (kN/m^2) to be used for the design of exterior component and cladding materials not specifically designed by the *registered design professional*.

1603.1.5 Earthquake design data. The following information related to seismic loads shall be shown, regardless of whether seismic loads govern the design of the lateral force-resisting system of the structure:

1. *Risk category*.
2. Seismic importance factor, I_e.
3. Mapped spectral response acceleration parameters, S_S and S_1.
4. *Site class*.
5. Design spectral response acceleration parameters, S_{DS} and S_{D1}.
6. *Seismic design category*.
7. Basic seismic force-resisting system(s).
8. Design base shear(s).
9. Seismic response coefficient(s), C_S.
10. Response modification coefficient(s), *R*.
11. Analysis procedure used.

1603.1.6 Geotechnical information. The design load-bearing values of soils shall be shown on the *construction documents*.

1603.1.7 Flood design data. For buildings located in whole or in part in *flood hazard areas* as established in Section 1612.3, the documentation pertaining to design, if required in Section 1612.5, shall be included and the following information, referenced to the datum on the community's Flood Insurance Rate Map (FIRM), shall be shown, regardless of whether flood loads govern the design of the building:

1. In *flood hazard areas* not subject to high-velocity wave action, the elevation of the proposed lowest floor, including the basement.
2. In *flood hazard areas* not subject to high-velocity wave action, the elevation to which any nonresidential building will be dry flood proofed.
3. In *flood hazard areas* subject to high-velocity wave action, the proposed elevation of the bottom of the lowest horizontal structural member of the lowest floor, including the basement.

1603.1.8 Special loads. Special loads that are applicable to the design of the building, structure or portions thereof shall be indicated along with the specified section of this code that addresses the special loading condition.

1603.1.9 Systems and components requiring special inspections for seismic resistance. *Construction documents* or specifications shall be prepared for those systems and components requiring *special inspection* for seismic resistance as specified in Section 1705.11 by the *registered design professional* responsible for their design and shall be submitted for approval in accordance with Section 107.1. Reference to seismic standards in lieu of detailed drawings is acceptable.

Building Code Requirements for Masonry Structures (2011)

CHAPTER 2
ALLOWABLE STRESS DESIGN OF MASONRY

CODE

2.1 — General

2.1.1 *Scope*

This chapter provides requirements for allowable stress design of masonry. Masonry design in accordance with this chapter shall comply with the requirements of Chapter 1, Sections 2.1.2 through 2.1.7, and either Section 2.2 or 2.3.

2.1.2 *Load combinations*

When the legally adopted building code does not provide allowable stress load combinations, structures and members shall be designed to resist the combinations of load specified by the building official.

2.1.3 *Design strength*

2.1.3.1 Project drawings shall show the specified compressive strength of masonry, f'_m, for each part of the structure.

2.1.3.2 Each portion of the structure shall be designed based on the specified compressive strength of masonry, f'_m, for that part of the work.

2.1.3.3 Computed stresses shall not exceed the allowable stress requirements of this Chapter.

2.1.4 *Anchor bolts embedded in grout*

2.1.4.1 *Design requirements* — Anchor bolts shall be designed using either the provisions of Section 2.1.4.2 or, for headed and bent-bar anchor bolts, by the

COMMENTARY

2.1 — General

2.1.1 *Scope*

Historically, a one-third increase in allowable stress has been permitted for load combinations that include wind or seismic loads. The origin and the reason for the one-third stress increase are unclear [2.1]. From a structural reliability standpoint, the one-third stress increase is a poor way to handle load combination effects. Therefore, the one-third stress increase is no longer permitted in this Code. The allowable stresses of this Chapter should not be increased by one-third for wind and load combinations.

2.1.2 *Load combinations*

When there is no legally adopted building code or the legally adopted building code does not have allowable stress load combinations, possible sources of allowable stress load combinations are ASCE 7[2.2] and IBC[2.3].

2.1.3 *Design strength*

The structural adequacy of masonry construction requires that the compressive strength of masonry equal or exceed the specified strength. The specified compressive strength f'_m on which design is based for each part of the structure must be shown on the project drawings.

The 1995, 1999, 2002, and 2005 editions of the Code contained provisions to permit use of strength-level load combinations in allowable stress design, to compensate for lack of service-level load combinations in previously referenced load standards. This procedure, which enabled the calculation of 'pseudo-strengths' on the basis of allowable stresses, is no longer included in the Code because recent editions of ASCE 7 include both service-level and strength-level load combinations. The 2005 edition of the Code provides guidance for using strength-level load combinations whenever the legally adopted building code does not provide service-level load combinations.

2.1.4 *Anchor bolts embedded in grout*

Allowable Stress Design anchor bolt provisions were obtained by calibrating corresponding Strength Design provisions to produce similar results. See Code

Code Requirements for Steel Construction, AISC 14th ed.

CHAPTER B

DESIGN REQUIREMENTS

The general requirements for the analysis and design of steel structures that are applicable to all chapters of the specification are given in this chapter.

The chapter is organized as follows:

B1. General Provisions
B2. Loads and Load Combinations
B3. Design Basis
B4. Classification of Sections for Local Buckling
B5. Fabrication, Erection and Quality Control
B6. Evaluation of Existing Structures

B1. GENERAL PROVISIONS

The design of members and *connections* shall be consistent with the intended behavior of the framing system and the assumptions made in the *structural analysis*. Unless restricted by the *applicable building code*, *lateral load* resistance and *stability* may be provided by any combination of members and connections.

B2. LOADS AND LOAD COMBINATIONS

The loads and load combinations shall be as stipulated by the *applicable building code*. In the absence of a building code, the loads and load combinations shall be those stipulated in SEI/ASCE 7. For design purposes, the *nominal loads* shall be taken as the *loads* stipulated by the applicable building code.

User Note: For LRFD designs, the load combinations in SEI/ASCE 7, Section 2.3 apply. For ASD designs, the load combinations in SEI/ASCE 7, Section 2.4 apply.

B3. DESIGN BASIS

Designs shall be made according to the provisions for *Load and Resistance Factor Design* (LRFD) or to the provisions for *Allowable Strength Design* (ASD).

1. Required Strength

The *required strength* of structural members and *connections* shall be determined by *structural analysis* for the appropriate load combinations as stipulated in Section B2.

Design by *elastic*, *inelastic* or *plastic analysis* is permitted. Provisions for inelastic and plastic analysis are as stipulated in Appendix 1, Inelastic Analysis and Design. The provisions for moment redistribution in continuous beams in Appendix 1, Section 1.3 are permitted for elastic analysis only.

2. Limit States

Design shall be based on the principle that no applicable strength or serviceability *limit state* shall be exceeded when the structure is subjected to all appropriate load combinations.

3. Design for Strength Using Load and Resistance Factor Design (LRFD)

Design according to the provisions for *Load and Resistance Factor Design* (LRFD) satisfies the requirements of this Specification when the *design strength* of each *structural component* equals or exceeds the *required strength* determined on the basis of the *LRFD load combinations*. All provisions of this Specification, except for those in Section B3.4, shall apply.

Design shall be performed in accordance with Equation B3-1:

$$R_u \leq \phi R_n \qquad \text{(B3-1)}$$

where

R_u = required strength (LRFD)
R_n = *nominal strength*, specified in Chapters B through K
ϕ = *resistance factor*, specified in Chapters B through K
ϕR_n = design strength

4. Design for Strength Using Allowable Strength Design (ASD)

Design according to the provisions for *Allowable Strength Design* (ASD) satisfies the requirements of this Specification when the *allowable strength* of each *structural component* equals or exceeds the *required strength* determined on the basis of the *ASD load combinations*. All provisions of this Specification, except those of Section B3.3, shall apply.

Design shall be performed in accordance with Equation B3-2:

$$R_a \leq R_n / \Omega \qquad \text{(B3-2)}$$

where

R_a = required strength (ASD)
R_n = *nominal strength*, specified in Chapters B through K
Ω = safety factor, specified in Chapters B through K
R_n/Ω = allowable strength

Code Requirements for Structural Concrete, ACI 318-11

CHAPTER 9 — STRENGTH AND SERVICEABILITY REQUIREMENTS

CODE

9.1 — General

9.1.1 — Structures and structural members shall be designed to have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations as are stipulated in this Code.

9.1.2 — Members also shall meet all other requirements of this Code to ensure adequate performance at service load levels.

9.1.3 — Design of structures and structural members using the load factor combinations and strength reduction factors of Appendix C shall be permitted. Use of load factor combinations from this chapter in conjunction with strength reduction factors of Appendix C shall not be permitted.

9.2 — Required strength

9.2.1 — Required strength U shall be at least equal to the effects of factored loads in Eq. (9-1) through (9-7). The effect of one or more loads not acting simultaneously shall be investigated.

$$U = 1.4D \qquad (9\text{-}1)$$

$$U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) \qquad (9\text{-}2)$$

$$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.5W) \qquad (9\text{-}3)$$

$$U = 1.2D + 1.0W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R) \qquad (9\text{-}4)$$

$$U = 1.2D + 1.0E + 1.0L + 0.2S \qquad (9\text{-}5)$$

COMMENTARY

R9.1 — General

In the 2002 Code, the factored load combinations and strength reduction factors of the 1999 Code were revised and moved to Appendix C. The 1999 combinations were replaced with those of SEI/ASCE 7-02.[9.1] The strength reduction factors were replaced with those of the 1999 Appendix C, except that the factor for flexure was increased. In the 2011 Code, the factored load combinations were revised for consistency with ASCE/SEI 7-10.[9.2]

The changes were made to further unify the design profession on one set of load factors and combinations, and to facilitate the proportioning of concrete building structures that include members of materials other than concrete. When used with the strength reduction factors in 9.3, the designs for gravity loads will be comparable to those obtained using the strength reduction and load factors of the 1999 and earlier Codes. For combinations with lateral loads, some designs will be different, but the results of either set of load factors are considered acceptable.

Chapter 9 defines the basic strength and serviceability conditions for proportioning structural concrete members.

The basic requirement for strength design may be expressed as follows:

$$\text{Design Strength} \geq \text{Required Strength}$$

$$\phi(\text{Nominal Strength}) \geq U$$

In the strength design procedure, the margin of safety is provided by multiplying the service load by a load factor and the nominal strength by a strength reduction factor.

R9.2 — Required strength

The required strength U is expressed in terms of factored loads, or related internal moments and forces. Factored loads are the loads specified in the general building code multiplied by appropriate load factors.

The factor assigned to each load is influenced by the degree of accuracy to which the load effect usually can be calculated and the variation that might be expected in the load during the lifetime of the structure. Dead loads, because they are more accurately determined and less variable, are assigned a lower load factor than live loads. Load factors also account for variability in the structural analysis used to compute moments and shears.

Code Requirements for Structural Concrete, ACI 318-11 (continued)

CODE

$$U = 0.9D + 1.0W \tag{9-6}$$

$$U = 0.9D + 1.0E \tag{9-7}$$

except as follows:

(a) The load factor on the live load L in Eq. (9-3) to (9-5) shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where L is greater than 100 lb/ft^2.

(b) Where W is based on service-level wind loads, **1.6*W*** shall be used in place of **1.0*W*** in Eq. (9-4) and (9-6), and **0.8*W*** shall be used in place of **0.5*W*** in Eq. (9-3).

(c) Where E is based on service-level forces, **1.4*E*** shall be used in place of **1.0*E*** in Eq. (9-5) and (9-7).

COMMENTARY

The Code gives load factors for specific combinations of loads. In assigning factors to combinations of loading, some consideration is given to the probability of simultaneous occurrence. While most of the usual combinations of loadings are included, it should not be assumed that all cases are covered.

Due regard is to be given to sign in determining U for combinations of loadings, as one type of loading may produce effects of opposite sense to that produced by another type. The load combinations with **0.9*D*** are specifically included for the case where a higher dead load reduces the effects of other loads. The loading case may also be critical for tension-controlled column sections. In such a case, a reduction in axial load and an increase in moment may result in a critical load combination.

Consideration should be given to various combinations of loading to determine the most critical design condition. This is particularly true when strength is dependent on more than one load effect, such as strength for combined flexure and axial load or shear strength in members with axial load.

If unusual circumstances require greater reliance on the strength of particular members than encountered in usual practice, some reduction in the stipulated strength reduction factors ϕ or increase in the stipulated load factors may be appropriate for such members.

In 2011, the Code removed the weight of soil and other fill materials as part of the definition of H. Consistent with ASCE/SEI 7-10, the weight of these materials is part of dead load, D. The load factors for D are appropriate provided the unit weight and thickness of earth or other fill materials are well controlled. If the weight of earth stabilizes the structure, a load factor of zero may be appropriate.

R9.2.1(a) — The load modification factor of 9.2.1(a) is different than the live load reductions based on the loaded area that may be allowed in the legally adopted general building code. The live load reduction, based on loaded area, adjusts the nominal live load (L_0 in ASCE/SEI 7) to L. The live load reduction as specified in the legally adopted general building code can be used in combination with the 0.5 load factor specified in 9.2.1(a).

R9.2.1(b) — ASCE/SEI 7-10 has converted wind loads to strength level, and reduced the wind load factor to 1.0. ACI 318 requires use of the previous load factor for wind loads, 1.6, when service-level wind loads are used. For serviceability checks, the commentary to Appendix C of ASCE/SEI 7-10 provides service-level wind loads, W_a.

R9.2.1(c) — In 1993, ASCE 7[9.3] converted earthquake forces to strength level, and reduced the earthquake load factor to 1.0. Model building codes[9.4-9.6] followed. ACI 318 requires use of the previous load factor for earthquake effects, approximately 1.4, when service-level earthquake effects are used.

Examples: Load Tracing and Factored Loads

EXAMPLE (pg. 129 **with corrections and additions)**

Assume that the average dead plus live load on the structure shown in Figure 3.15 is 60 lbs/ft^2. **Determine** the reactions for Beam D. This is the same structure as shown in Figure 3.1.
^ E, B and A *Assuming all beams are weightless!*

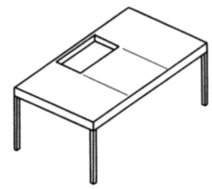
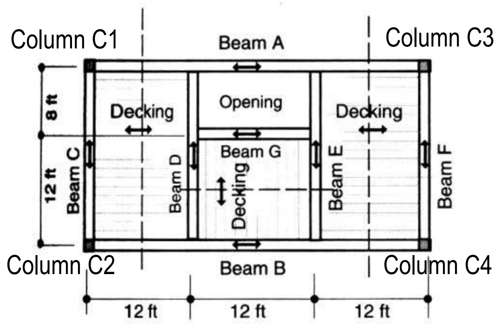


Figure 3.1

Solution:

Note carefully the directions of the decking span. Beam D carries floor loads from the decking to the left (see the contributory area and load strip), but not to the right, since the



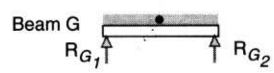
Live and dead load

Assume $w_{DL+LL} = 60 \text{ lbs/ft}^2$

Beam G carries distributed loads only

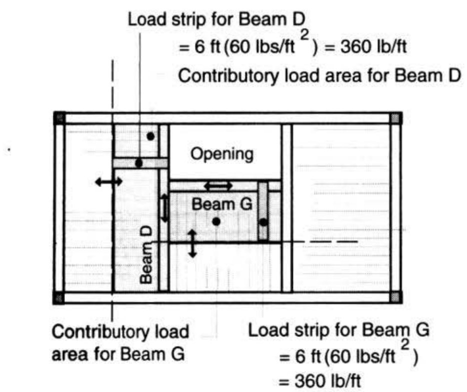
Find reactions for Beam G

$w = 6 \text{ ft}(60 \text{ lbs/ft}^2) = 360 \text{ lb/ft}$

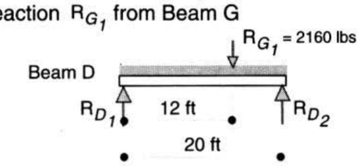


$R_{G_1} = wL/2 = (360 \text{ lb/ft})(12 \text{ ft})/2 = 2160 \text{ lbs}$

$R_{G_2} = wL/2 = (360 \text{ lb/ft})(12 \text{ ft})/2 = 2160 \text{ lbs}$



Beam D (and E) carries both distributed loads and the reaction R_{G_1} from Beam G



$\Sigma M_{D_1} = 0$

$-(12 \text{ ft})(2160 \text{ lb}) - (360 \text{ lb/ft})(20 \text{ ft})(20 \text{ ft}/2) + 20R_{D_2} = 0$

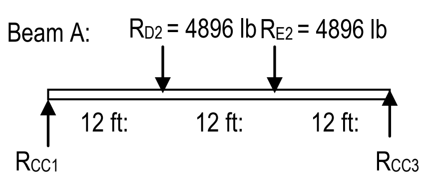
$R_{D_2} = 4896 \text{ lb} = R_{E2}$

$\Sigma F_y = 0$

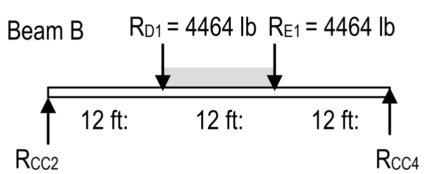
$R_{D_1} + R_{D_2} = (360 \text{ lb/ft})(20 \text{ ft}) + 2160 \text{ lb}$

$R_{D_1} = 4464 \text{ lb} = R_{E1}$

FIGURE 3.15 Load modeling and reaction determination.



By symmetry; $R_{CC1} = R_{CC3} = (4893 \text{ lb} + 4896 \text{ lb})/2 = 4896 \text{ lb}$



By symmetry; $R_{CC2} = R_{CC4} = (4464 \text{ lb} + 4464 \text{ lb})/2 + (6 \text{ ft})(60 \text{ lb/ft}^2)(12 \text{ ft})/2 = 6624 \text{ lb}$

Additional loads are transferred to the column from the reactions on Beams C and F:

$R_{C1} = R_{C2} = R_{F1} = R_{F2} = wL/2 = (6 \text{ ft})(60 \text{ lb/ft}^2)(20 \text{ ft})/2 = 3600 \text{ lb}$

center decking runs parallel to Beam D and is not carried by it. Beam D also picks up the end of Beam G and thus also "carries" the reactive force from Beam G. It is therefore necessary to analyze Beam G first to determine the magnitude of this force. The analysis appears in Figure 3.15. The reactive force from Beam G of 2160 lbs is then treated as a downward force acting on Beam D. The load model for Beam D thus consists of distributed forces from the decking plus the 2160-lb force. It is then analyzed by means of the equations of statics to obtain reactive forces of 4896 lbs and 4464 lbs at its ends.

C1 = 4896 lb + 3600 lb = 8,496 lb

C2 = 6624 lb + 3600 lb = 10,224 lb

C3 = 4896 lb + 3600 lb = 8,496 lb

C4 = 6624 lb + 3600 lb = 10,224 lb

Example 2

Determine the controlling load combinations(s) using AISC-LRFD for a building column subject to the following service or nominal (unfactored) axial compressive loads: D = 30 k, L = 50 k, L_r = 10 k, W = 25 k, E = 40 k

Using a spreadsheet analysis:

LRFD (ASCE-7)		FACTORED LOAD
$1.4D$		
$\quad 1.4D$	=	42 kips
$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$		
$\quad 1.2D + 1.6L + 0.5L_r$	=	121
$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$		
$\quad 1.2D + 1.6L_r + L$	=	102
$\quad 1.2D + 1.6L_r + 0.5W$	=	64.5
$\quad 1.2D + 1.6L_r - 0.5W$	=	39.5
$1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$		
$\quad 1.2D + 1.0W + L + 0.5L_r$	=	116
$\quad 1.2D - 1.0W + L + 0.5L_r$	=	66
$1.2D + 1.0E + L + 0.2S$		
$\quad 1.2D + 1.0E + L$	=	126
$\quad 1.2D - 1.0E + L$	=	46
$0.9D + 1.0W$		
$\quad 0.9D + 1.0W$	=	52
$\quad 0.9D - 1.0W$	=	2
$0.9D + 1.0E$		
$\quad 0.9D + 1.0E$	=	67
$\quad 0.9D - 1.0E$	=	-13
Critical Factored Load		126 kips (C) -13 kips (T)

Example 3

EXAMPLE 2–4

Determine factored loads for the beam shown in Figure 2–16.

Solution

For the left half of the beam:

$$w_{u1} = 1.2w_D + 1.6w_L$$
$$w_{u1} = 1.2 \times 1.0 + 1.6 \times 2.0 = 4.4 \text{ kip/ft}$$

For the right half of the beam:

$$w_{u2} = 1.2w_D + 1.6w_L$$
$$w_{u2} = 1.2 \times 1.0 + 1.6 \times 0 = 1.2 \text{ kip/ft}$$

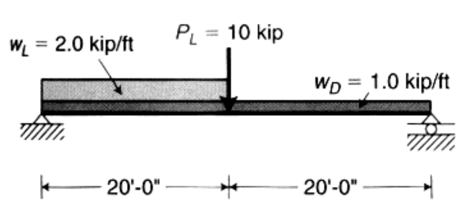


FIGURE 2–16 Example 2–4 (service loads).

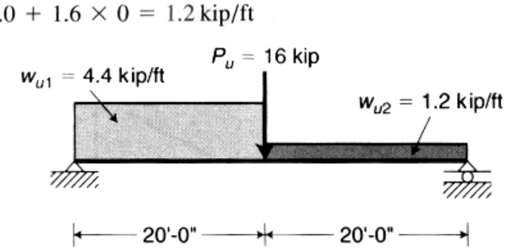


FIGURE 2–17 Example 2–4 (factored loads).

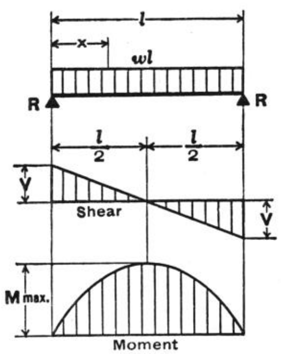
The concentrated load is a live load only:

$$P_u = 1.2P_D + 1.6P_L$$
$$P_u = 1.2 \times 0 + 1.6 \times 10 = 16 \text{ kip}$$

The factored loads on the beam are shown in Figure 2–17.

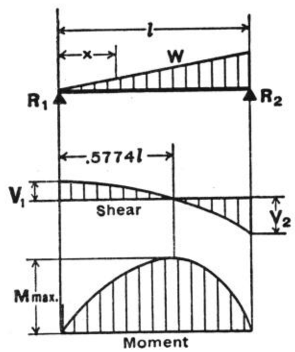
BEAM DIAGRAMS AND FORMULAS For Various Static Loading Conditions, AISC ASD 8th ed.

1. SIMPLE BEAM—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load		$= wl$
$R = V$		$= \frac{wl}{2}$
V_x		$= w\left(\frac{l}{2} - x\right)$
M max.	(at center)	$= \frac{wl^2}{8}$
M_x		$= \frac{wx}{2}(l-x)$
Δmax.	(at center)	$= \frac{5wl^4}{384EI}$
Δ_x		$= \frac{wx}{24EI}(l^3 - 2lx^2 + x^3)$

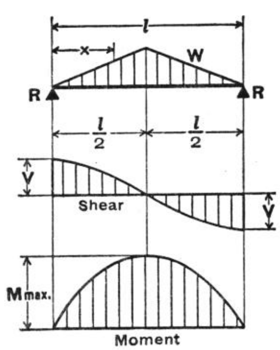
2. SIMPLE BEAM—LOAD INCREASING UNIFORMLY TO ONE END



Total Equiv. Uniform Load		$= \frac{16W}{9\sqrt{3}} = 1.0264W$
$R_1 = V_1$		$= \frac{W}{3}$
$R_2 = V_2$ max.		$= \frac{2W}{3}$
V_x		$= \frac{W}{3} - \frac{Wx^2}{l^2}$
M max.	$\left(\text{at } x = \frac{l}{\sqrt{3}} = .5774l\right)$	$= \frac{2Wl}{9\sqrt{3}} = .1283\,Wl$
M_x		$= \frac{Wx}{3l^2}(l^2 - x^2)$
Δmax.	$\left(\text{at } x = l\sqrt{1-\sqrt{\frac{8}{15}}} = .5193l\right)$	$= .01304\,\frac{Wl^3}{EI}$
Δ_x		$= \frac{Wx}{180EIl^2}(3x^4 - 10l^2x^2 + 7l^4)$

$W = \frac{wl}{2}$

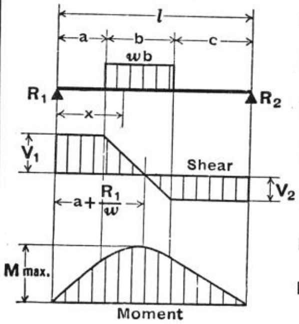
3. SIMPLE BEAM—LOAD INCREASING UNIFORMLY TO CENTER



Total Equiv. Uniform Load		$= \frac{4W}{3}$
$R = V$		$= \frac{W}{2}$
V_x	$\left(\text{when } x < \frac{l}{2}\right)$	$= \frac{W}{2l^2}(l^2 - 4x^2)$
M max.	(at center)	$= \frac{Wl}{6}$
M_x	$\left(\text{when } x < \frac{l}{2}\right)$	$= Wx\left(\frac{1}{2} - \frac{2x^2}{3l^2}\right)$
Δmax.	(at center)	$= \frac{Wl^3}{60EI}$
Δ_x	$\left(\text{when } x < \frac{l}{2}\right)$	$= \frac{Wx}{480EIl^2}(5l^2 - 4x^2)^2$

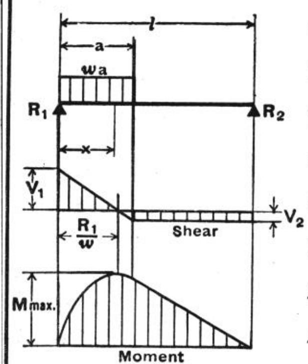
$W = \frac{wl}{2}$

4. SIMPLE BEAM—UNIFORM LOAD PARTIALLY DISTRIBUTED



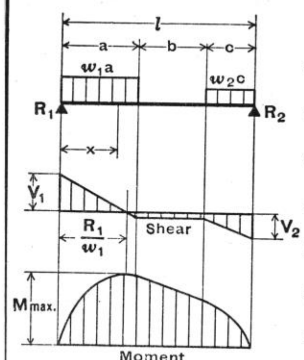
$R_1 = V_1$	(max. when $a < c$)	$= \frac{wb}{2l}(2c + b)$
$R_2 = V_2$	(max. when $a > c$)	$= \frac{wb}{2l}(2a + b)$
V_x	(when $x > a$ and $< (a + b)$)	$= R_1 - w(x - a)$
M max.	$\left(\text{at } x = a + \frac{R_1}{w}\right)$	$= R_1\left(a + \frac{R_1}{2w}\right)$
M_x	(when $x < a$)	$= R_1 x$
M_x	(when $x > a$ and $< (a + b)$)	$= R_1 x - \frac{w}{2}(x - a)^2$
M_x	(when $x > (a + b)$)	$= R_2(l - x)$

5. SIMPLE BEAM—UNIFORM LOAD PARTIALLY DISTRIBUTED AT ONE END



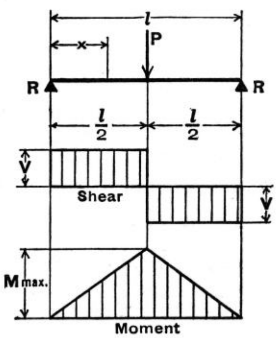
$R_1 = V_1$ max.		$= \frac{wa}{2l}(2l - a)$
$R_2 = V_2$		$= \frac{wa^2}{2l}$
V_x	(when $x < a$)	$= R_1 - wx$
M max.	$\left(\text{at } x = \frac{R_1}{w}\right)$	$= \frac{R_1^2}{2w}$
M_x	(when $x < a$)	$= R_1 x - \frac{wx^2}{2}$
M_x	(when $x > a$)	$= R_2(l - x)$
Δ_x	(when $x < a$)	$= \frac{wx}{24EIl}\left(a^2(2l-a)^2 - 2ax^2(2l-a) + lx^3\right)$
Δ_x	(when $x > a$)	$= \frac{wa^2(l-x)}{24EIl}(4xl - 2x^2 - a^2)$

6. SIMPLE BEAM—UNIFORM LOAD PARTIALLY DISTRIBUTED AT EACH END



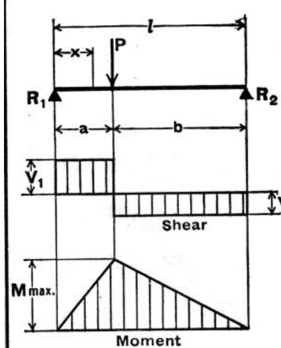
$R_1 = V_1$		$= \frac{w_1 a(2l - a) + w_2 c^2}{2l}$
$R_2 = V_2$		$= \frac{w_2 c(2l - c) + w_1 a^2}{2l}$
V_x	(when $x < a$)	$= R_1 - w_1 x$
V_x	(when $x > a$ and $< (a + b)$)	$= R_1 - w_1 a$
V_x	(when $x > (a + b)$)	$= R_2 - w_2(l - x)$
M max.	$\left(\text{at } x = \frac{R_1}{w_1} \text{ when } R_1 < w_1 a\right)$	$= \frac{R_1^2}{2w_1}$
M max.	$\left(\text{at } x = l - \frac{R_2}{w_2} \text{ when } R_2 < w_2 c\right)$	$= \frac{R_2^2}{2w_2}$
M_x	(when $x < a$)	$= R_1 x - \frac{w_1 x^2}{2}$
M_x	(when $x > a$ and $< (a + b)$)	$= R_1 x - \frac{w_1 a}{2}(2x - a)$
M_x	(when $x > (a + b)$)	$= R_2(l - x) - \frac{w_2(l - x)^2}{2}$

7. SIMPLE BEAM—CONCENTRATED LOAD AT CENTER



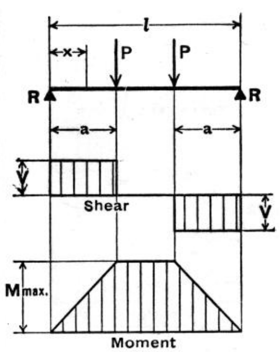
Total Equiv. Uniform Load		$= 2P$
$R = V$		$= \frac{P}{2}$
M max.	(at point of load)	$= \frac{Pl}{4}$
M_x	(when $x < \frac{l}{2}$)	$= \frac{Px}{2}$
Δmax.	(at point of load)	$= \frac{Pl^3}{48EI}$
Δ_x	(when $x < \frac{l}{2}$)	$= \frac{Px}{48EI}(3l^2 - 4x^2)$

8. SIMPLE BEAM—CONCENTRATED LOAD AT ANY POINT



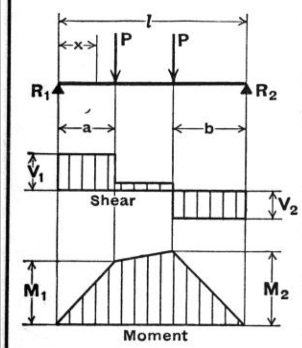
Total Equiv. Uniform Load		$= \frac{8\,Pab}{l^2}$
$R_1 = V_1$	(max. when $a < b$)	$= \frac{Pb}{l}$
$R_2 = V_2$	(max. when $a > b$)	$= \frac{Pa}{l}$
M max.	(at point of load)	$= \frac{Pab}{l}$
M_x	(when $x < a$)	$= \frac{Pbx}{l}$
Δmax.	(at $x = \sqrt{\frac{a(a+2b)}{3}}$ when $a > b$)	$= \frac{Pab(a+2b)\sqrt{3a(a+2b)}}{27\,EI\,l}$
Δ_a	(at point of load)	$= \frac{Pa^2b^2}{3EI\,l}$
Δ_x	(when $x < a$)	$= \frac{Pbx}{6EI\,l}(l^2 - b^2 - x^2)$

9. SIMPLE BEAM—TWO EQUAL CONCENTRATED LOADS SYMMETRICALLY PLACED



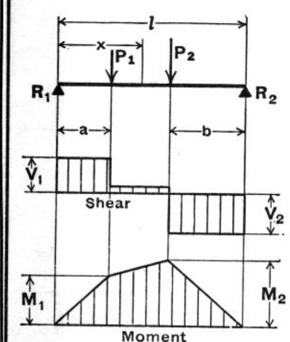
Total Equiv. Uniform Load		$= \frac{8\,Pa}{l}$
$R = V$		$= P$
M max.	(between loads)	$= Pa$
M_x	(when $x < a$)	$= Px$
Δmax.	(at center)	$= \frac{Pa}{24EI}(3l^2 - 4a^2)$
Δ_x	(when $x < a$)	$= \frac{Px}{6EI}(3la - 3a^2 - x^2)$
Δ_x	(when $x > a$ and $< (l - a)$)	$= \frac{Pa}{6EI}(3lx - 3x^2 - a^2)$

10. SIMPLE BEAM—TWO EQUAL CONCENTRATED LOADS UNSYMMETRICALLY PLACED



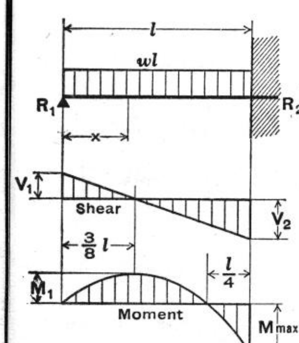
$R_1 = V_1$	(max. when $a < b$)	$= \frac{P}{l}(l - a + b)$
$R_2 = V_2$	(max. when $a > b$)	$= \frac{P}{l}(l - b + a)$
V_x	(when $x > a$ and $< (l - b)$)	$= \frac{P}{l}(b - a)$
M_1	(max. when $a > b$)	$= R_1 a$
M_2	(max. when $a < b$)	$= R_2 b$
M_x	(when $x < a$)	$= R_1 x$
M_x	(when $x > a$ and $< (l - b)$)	$= R_1 x - P(x - a)$

11. SIMPLE BEAM—TWO UNEQUAL CONCENTRATED LOADS UNSYMMETRICALLY PLACED



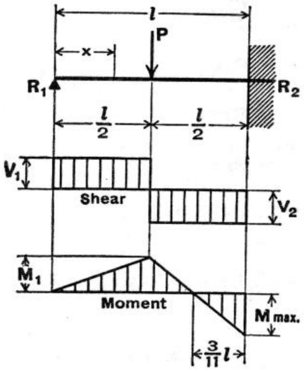
$R_1 = V_1$		$= \frac{P_1(l - a) + P_2 b}{l}$
$R_2 = V_2$		$= \frac{P_1 a + P_2(l - b)}{l}$
V_x	(when $x > a$ and $< (l - b)$)	$= R_1 - P_1$
M_1	(max. when $R_1 < P_1$)	$= R_1 a$
M_2	(max. when $R_2 < P_2$)	$= R_2 b$
M_x	(when $x < a$)	$= R_1 x$
M_x	(when $x > a$ and $< (l - b)$)	$= R_1 x - P_1(x - a)$

12. BEAM FIXED AT ONE END, SUPPORTED AT OTHER—UNIFORMLY DISTRIBUTED LOAD



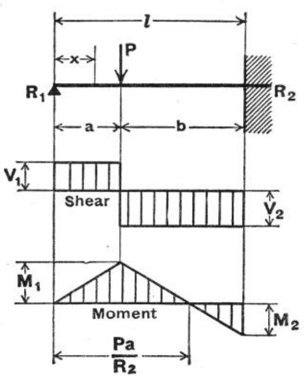
Total Equiv. Uniform Load		$= wl$
$R_1 = V_1$		$= \frac{3wl}{8}$
$R_2 = V_2$ max.		$= \frac{5wl}{8}$
V_x		$= R_1 - wx$
M max.		$= \frac{wl^2}{8}$
M_1	(at $x = \frac{3}{8}l$)	$= \frac{9}{128}wl^2$
M_x		$= R_1 x - \frac{wx^2}{2}$
Δmax.	(at $x = \frac{l}{16}(1 + \sqrt{33}) = .4215l$)	$= \frac{wl^4}{185EI}$
Δ_x		$= \frac{wx}{48EI}(l^3 - 3lx^2 + 2x^3)$

13. BEAM FIXED AT ONE END, SUPPORTED AT OTHER—CONCENTRATED LOAD AT CENTER



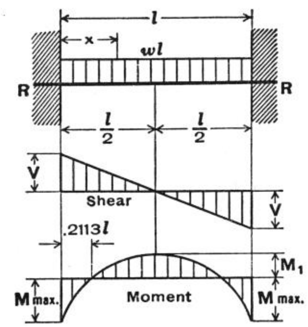
Total Equiv. Uniform Load	$= \frac{3P}{2}$
$R_1 = V_1$	$= \frac{5P}{16}$
$R_2 = V_2$ max.	$= \frac{11P}{16}$
M max. (at fixed end)	$= \frac{3Pl}{16}$
M_1 (at point of load)	$= \frac{5Pl}{32}$
M_x $\left(\text{when } x < \frac{l}{2}\right)$	$= \frac{5Px}{16}$
M_x $\left(\text{when } x > \frac{l}{2}\right)$	$= P\left(\frac{l}{2} - \frac{11x}{16}\right)$
Δmax. $\left(\text{at } x = l\sqrt{\frac{1}{5}} = .4472l\right)$	$= \frac{Pl^3}{48EI\sqrt{5}} = .009317\,\frac{Pl^3}{EI}$
Δx (at point of load)	$= \frac{7Pl^3}{768EI}$
Δx $\left(\text{when } x < \frac{l}{2}\right)$	$= \frac{Px}{96EI}(3l^2 - 5x^2)$
Δx $\left(\text{when } x > \frac{l}{2}\right)$	$= \frac{P}{96EI}(x - l)^2(11x - 2l)$

14. BEAM FIXED AT ONE END, SUPPORTED AT OTHER—CONCENTRATED LOAD AT ANY POINT



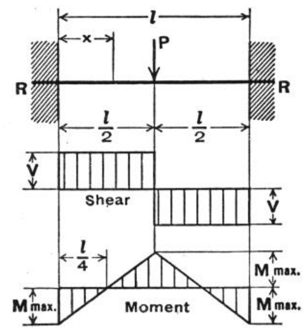
$R_1 = V_1$	$= \frac{Pb^2}{2l^3}(a + 2l)$
$R_2 = V_2$	$= \frac{Pa}{2l^3}(3l^2 - a^2)$
M_1 (at point of load)	$= R_1 a$
M_2 (at fixed end)	$= \frac{Pab}{2l^2}(a + l)$
M_x (when $x < a$)	$= R_1 x$
M_x (when $x > a$)	$= R_1 x - P(x - a)$
Δmax. $\left(\text{when } a < .414l \text{ at } x = l\,\frac{l^2 + a^2}{3l^2 - a^2}\right)$	$= \frac{Pa}{3EI}\,\frac{(l^2 - a^2)^3}{(3l^2 - a^2)^2}$
Δmax. $\left(\text{when } a > .414l \text{ at } x = l\sqrt{\frac{a}{2l + a}}\right)$	$= \frac{Pab^2}{6EI}\sqrt{\frac{a}{2l + a}}$
Δa (at point of load)	$= \frac{Pa^2b^3}{12EIl^3}(3l + a)$
Δx (when $x < a$)	$= \frac{Pb^2x}{12EIl^3}(3al^2 - 2lx^2 - ax^2)$
Δx (when $x > a$)	$= \frac{Pa}{12EIl^3}(l - x)^2(3l^2x - a^2x - 2a^2l)$

15. BEAM FIXED AT BOTH ENDS—UNIFORMLY DISTRIBUTED LOADS



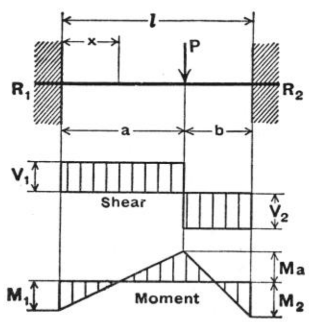
Total Equiv. Uniform Load	$= \frac{2wl}{3}$
$R = V$	$= \frac{wl}{2}$
V_x	$= w\left(\frac{l}{2} - x\right)$
M max. (at ends)	$= \frac{wl^2}{12}$
M_1 (at center)	$= \frac{wl^2}{24}$
M_x	$= \frac{w}{12}(6lx - l^2 - 6x^2)$
Δmax. (at center)	$= \frac{wl^4}{384EI}$
Δx	$= \frac{wx^2}{24EI}(l - x)^2$

16. BEAM FIXED AT BOTH ENDS—CONCENTRATED LOAD AT CENTER



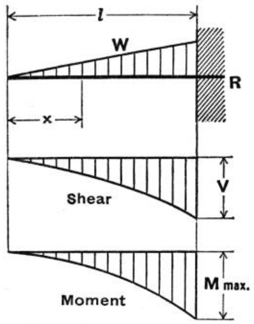
Total Equiv. Uniform Load	$= P$
$R = V$	$= \frac{P}{2}$
M max. (at center and ends)	$= \frac{Pl}{8}$
M_x $\left(\text{when } x < \frac{l}{2}\right)$	$= \frac{P}{8}(4x - l)$
Δmax. (at center)	$= \frac{Pl^3}{192EI}$
Δx $\left(\text{when } x < \frac{l}{2}\right)$	$= \frac{Px^2}{48EI}(3l - 4x)$

17. BEAM FIXED AT BOTH ENDS—CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1$ (max. when $a < b$)	$= \frac{Pb^2}{l^3}(3a + b)$
$R_2 = V_2$ (max. when $a > b$)	$= \frac{Pa^2}{l^3}(a + 3b)$
M_1 (max. when $a < b$)	$= \frac{Pab^2}{l^2}$
M_2 (max. when $a > b$)	$= \frac{Pa^2b}{l^2}$
M_a (at point of load)	$= \frac{2Pa^2b^2}{l^3}$
M_x (when $x < a$)	$= R_1x - \frac{Pab^2}{l^2}$
Δmax. $\left(\text{when } a > b \text{ at } x = \frac{2al}{3a + b}\right)$	$= \frac{2Pa^3b^2}{3EI\,(3a + b)^2}$
Δa (at point of load)	$= \frac{Pa^3b^3}{3EIl^3}$
Δx (when $x < a$)	$= \frac{Pb^2x^2}{6EIl^3}(3al - 3ax - bx)$

18. CANTILEVER BEAM—LOAD INCREASING UNIFORMLY TO FIXED END



Total Equiv. Uniform Load . . . . . $= \frac{8}{3}W$

$R = V$ . . . . $= W$

$W = \frac{wl}{2}$

V_x . . . . $= W\frac{x^2}{l^2}$

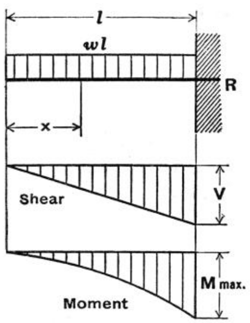
M max. (at fixed end) . . . . . . $= \frac{Wl}{3}$

M_x . . . . $= \frac{Wx^3}{3l^2}$

Δmax. (at free end) . . . . . . $= \frac{Wl^3}{15EI}$

Δx . . . . $= \frac{W}{60EIl^2}(x^5 - 5l^4x + 4l^5)$

19. CANTILEVER BEAM—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load . . . . . $= 4wl$

$R = V$ . . . . $= wl$

V_x . . . . $= wx$

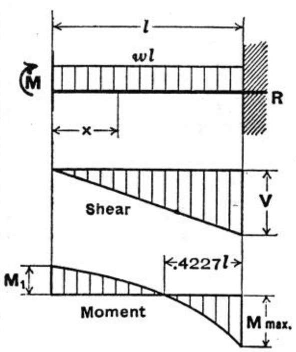
M max. (at fixed end) . . . . . . $= \frac{wl^2}{2}$

M_x . . . . $= \frac{wx^2}{2}$

Δmax. (at free end) . . . . . . $= \frac{wl^4}{8EI}$

Δx . . . . $= \frac{w}{24EI}(x^4 - 4l^3x + 3l^4)$

20. BEAM FIXED AT ONE END, FREE TO DEFLECT VERTICALLY BUT NOT ROTATE AT OTHER—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load . . . . . $= \frac{8}{3}wl$

$R = V$ . . . . $= wl$

V_x . . . . $= wx$

M max. (at fixed end) . . . . . . $= \frac{wl^2}{3}$

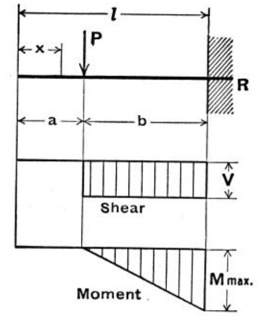
M_1 (at deflected end) . . . . . $= \frac{wl^2}{6}$

M_x . . . . $= \frac{w}{6}(l^2 - 3x^2)$

Δmax. (at deflected end) . . . . . $= \frac{wl^4}{24EI}$

Δx . . . . $= \frac{w(l^2 - x^2)^2}{24EI}$

21. CANTILEVER BEAM—CONCENTRATED LOAD AT ANY POINT



Total Equiv. Uniform Load . . . . . $= \frac{8Pb}{l}$

$R = V$ . . . . $= P$

M max. (at fixed end) . . . . . . $= Pb$

M_x (when $x > a$) . . . . . . $= P(x - a)$

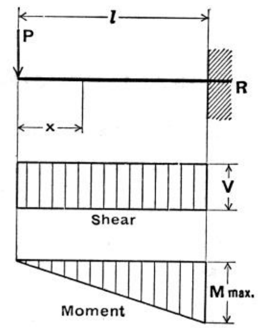
Δmax. (at free end) . . . . . . $= \frac{Pb^2}{6EI}(3l - b)$

Δa (at point of load) . . . . . $= \frac{Pb^3}{3EI}$

Δx (when $x < a$) . . . . . . $= \frac{Pb^2}{6EI}(3l - 3x - b)$

Δx (when $x > a$) . . . . . . $= \frac{P(l - x)^2}{6EI}(3b - l + x)$

22. CANTILEVER BEAM—CONCENTRATED LOAD AT FREE END



Total Equiv. Uniform Load . . . . . $= 8P$

$R = V$ . . . . $= P$

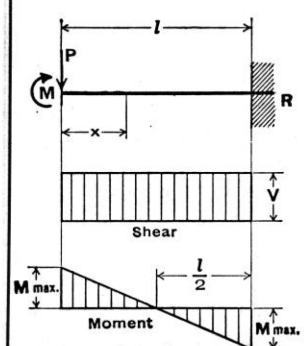
M max. (at fixed end) . . . . . . $= Pl$

M_x . . . . $= Px$

Δmax. (at free end) . . . . . $= \frac{Pl^3}{3EI}$

Δx . . . . $= \frac{P}{6EI}(2l^3 - 3l^2x + x^3)$

23. BEAM FIXED AT ONE END, FREE TO DEFLECT VERTICALLY BUT NOT ROTATE AT OTHER—CONCENTRATED LOAD AT DEFLECTED END



Total Equiv. Uniform Load . . . . . $= 4P$

$R = V$ . . . . $= P$

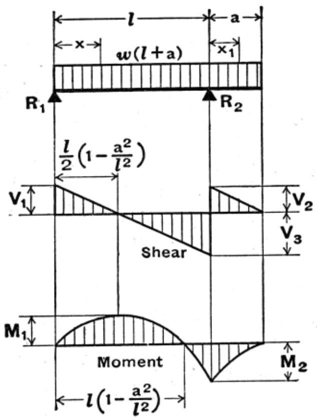
M max. (at both ends) . . . . . $= \frac{Pl}{2}$

M_x . . . . $= P\left(\frac{l}{2} - x\right)$

Δmax. (at deflected end) . . . . . $= \frac{Pl^3}{12EI}$

Δx . . . . $= \frac{P(l - x)^2}{12EI}(l + 2x)$

24. BEAM OVERHANGING ONE SUPPORT—UNIFORMLY DISTRIBUTED LOAD



$R_1 = V_1 = \frac{w}{2l}(l^2 - a^2)$

$R_2 = V_2 + V_3 = \frac{w}{2l}(l + a)^2$

$V_2 = wa$

$V_3 = \frac{w}{2l}(l^2 + a^2)$

V_x (between supports) $= R_1 - wx$

V_{x_1} (for overhang) $= w(a - x_1)$

M_1 $\left(\text{at } x = \frac{l}{2}\left[1 - \frac{a^2}{l^2}\right]\right) = \frac{w}{8l^2}(l + a)^2(l - a)^2$

M_2 (at R_2) $= \frac{wa^2}{2}$

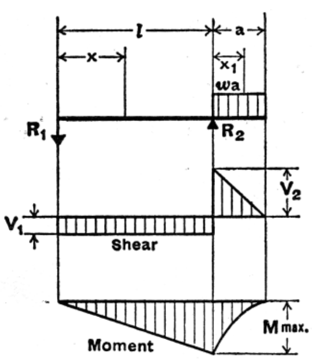
M_x (between supports) $= \frac{wx}{2l}(l^2 - a^2 - xl)$

M_{x_1} (for overhang) $= \frac{w}{2}(a - x_1)^2$

Δx (between supports) $= \frac{wx}{24EIl}(l^4 - 2l^2x^2 + lx^3 - 2a^2l^2 + 2a^2x^2)$

Δx_1 (for overhang) $= \frac{wx_1}{24EI}(4a^2l - l^3 + 6a^2x_1 - 4ax_1^2 + x_1^3)$

25. BEAM OVERHANGING ONE SUPPORT—UNIFORMLY DISTRIBUTED LOAD ON OVERHANG



$R_1 = V_1 = \frac{wa^2}{2l}$

$R_2 = V_1 + V_2 = \frac{wa}{2l}(2l + a)$

$V_2 = wa$

V_{x_1} (for overhang) $= w(a - x_1)$

M max. (at R_2) $= \frac{wa^2}{2}$

M_x (between supports) $= \frac{wa^2x}{2l}$

M_{x_1} (for overhang) $= \frac{w}{2}(a - x_1)^2$

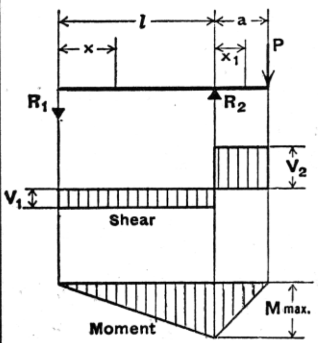
$\Delta max.$ $\left(\text{between supports at } x = \frac{l}{\sqrt{3}}\right) = \frac{wa^2l^2}{18\sqrt{3}EI} = .03208\,\frac{wa^2l^2}{EI}$

$\Delta max.$ (for overhang at $x_1 = a$) $= \frac{wa^3}{24EI}(4l + 3a)$

Δx (between supports) $= \frac{wa^2x}{12EIl}(l^2 - x^2)$

Δx_1 (for overhang) $= \frac{wx_1}{24EI}(4a^2l + 6a^2x_1 - 4ax_1^2 + x_1^3)$

26. BEAM OVERHANGING ONE SUPPORT—CONCENTRATED LOAD AT END OF OVERHANG



$R_1 = V_1 = \frac{Pa}{l}$

$R_2 = V_1 + V_2 = \frac{P}{l}(l + a)$

$V_2 = P$

M max. (at R_2) $= Pa$

M_x (between supports) $= \frac{Pax}{l}$

M_{x_1} (for overhang) $= P(a - x_1)$

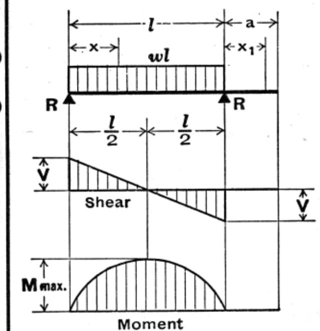
$\Delta max.$ $\left(\text{between supports at } x = \frac{l}{\sqrt{3}}\right) = \frac{Pal^2}{9\sqrt{3}EI} = .06415\,\frac{Pal^2}{EI}$

$\Delta max.$ (for overhang at $x_1 = a$) $= \frac{Pa^2}{3EI}(l + a)$

Δx (between supports) $= \frac{Pax}{6EIl}(l^2 - x^2)$

Δx_1 (for overhang) $= \frac{Px_1}{6EI}(2al + 3ax_1 - x_1^2)$

27. BEAM OVERHANGING ONE SUPPORT—UNIFORMLY DISTRIBUTED LOAD BETWEEN SUPPORTS



Total Equiv. Uniform Load $= wl$

$R = V = \frac{wl}{2}$

$V_x = w\left(\frac{l}{2} - x\right)$

M max. (at center) $= \frac{wl^2}{8}$

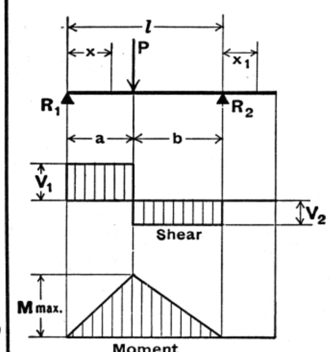
$M_x = \frac{wx}{2}(l - x)$

$\Delta max.$ (at center) $= \frac{5wl^4}{384EI}$

$\Delta x = \frac{wx}{24EI}(l^3 - 2lx^2 + x^3)$

$\Delta x_1 = \frac{wl^3x_1}{24EI}$

28. BEAM OVERHANGING ONE SUPPORT—CONCENTRATED LOAD AT ANY POINT BETWEEN SUPPORTS



Total Equiv. Uniform Load $= \frac{8Pab}{l^2}$

$R_1 = V_1$ (max. when $a < b$) $= \frac{Pb}{l}$

$R_2 = V_2$ (max. when $a > b$) $= \frac{Pa}{l}$

M max. (at point of load) $= \frac{Pab}{l}$

M_x (when $x < a$) $= \frac{Pbx}{l}$

$\Delta max.$ $\left(\text{at } x = \sqrt{\frac{a(a+2b)}{3}} \text{ when } a > b\right) = \frac{Pab\,(a + 2b)\,\sqrt{3a\,(a + 2b)}}{27EIl}$

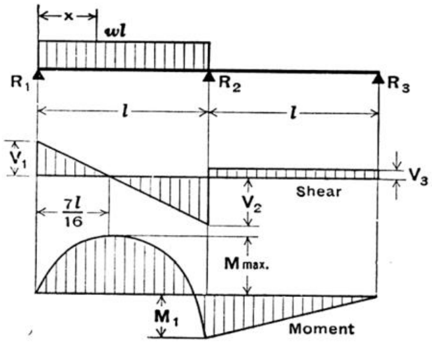
Δa (at point of load) $= \frac{Pa^2b^2}{3EIl}$

Δx (when $x < a$) $= \frac{Pbx}{6EIl}(l^2 - b^2 - x^2)$

Δx (when $x > a$) $= \frac{Pa\,(l - x)}{6EIl}(2lx - x^2 - a^2)$

$\Delta x_1 = \frac{Pabx_1}{6EIl}(l + a)$

29. CONTINUOUS BEAM—TWO EQUAL SPANS—UNIFORM LOAD ON ONE SPAN



Total Equiv. Uniform Load $= \frac{49}{64} wl$

$R_1 = V_1 \ldots\ldots = \frac{7}{16} wl$

$R_2 = V_2 + V_3 \ldots\ldots = \frac{5}{8} wl$

$R_3 = V_3 \ldots\ldots = -\frac{1}{16} wl$

$V_2 \ldots\ldots = \frac{9}{16} wl$

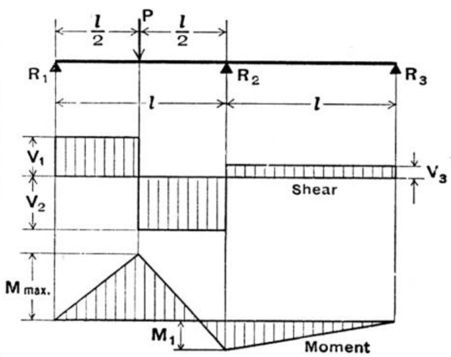
M max. $\left(\text{at } x = \frac{7}{16} l\right) \ldots = \frac{49}{512} wl^2$

M_1 (at support R_2) $\ldots = \frac{1}{16} wl^2$

M_x (when $x < l$) $\ldots = \frac{wx}{16}(7l - 8x)$

Δ Max. (0.472 l from R_1) $= 0.0092\, wl^4/EI$

30. CONTINUOUS BEAM—TWO EQUAL SPANS—CONCENTRATED LOAD AT CENTER OF ONE SPAN



Total Equiv. Uniform Load $= \frac{13}{8} P$

$R_1 = V_1 \ldots\ldots = \frac{13}{32} P$

$R_2 = V_2 + V_3 \ldots\ldots = \frac{11}{16} P$

$R_3 = V_3 \ldots\ldots = -\frac{3}{32} P$

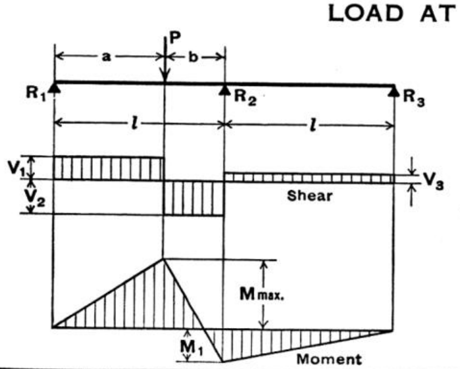
$V_2 \ldots\ldots = \frac{19}{32} P$

M max. (at point of load) $= \frac{13}{64} Pl$

M_1 (at support R_2) $= \frac{3}{32} Pl$

Δ Max. (0.480 l from R_1) $= 0.015\, Pl^3/EI$

31. CONTINUOUS BEAM—TWO EQUAL SPANS—CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1 \ldots\ldots = \frac{Pb}{4l^3}\left(4l^2 - a(l + a)\right)$

$R_2 = V_2 + V_3 \ldots\ldots = \frac{Pa}{2l^3}\left(2l^2 + b(l + a)\right)$

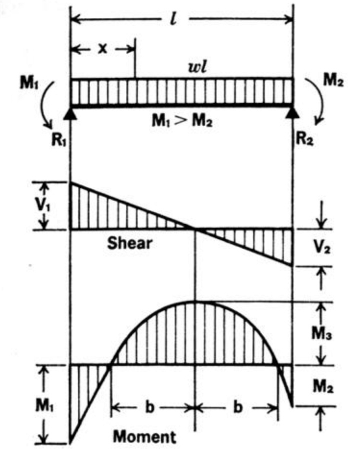
$R_3 = V_3 \ldots\ldots = -\frac{Pab}{4l^3}(l + a)$

$V_2 \ldots\ldots = \frac{Pa}{4l^3}\left(4l^2 + b(l + a)\right)$

M max. (at point of load) $= \frac{Pab}{4l^3}\left(4l^2 - a(l + a)\right)$

M_1 (at support R_2) $= \frac{Pab}{4l^2}(l + a)$

32. BEAM—UNIFORMLY DISTRIBUTED LOAD AND VARIABLE END MOMENTS



$R_1 = V_1 = \frac{wl}{2} + \frac{M_1 - M_2}{l}$

$R_2 = V_2 = \frac{wl}{2} - \frac{M_1 - M_2}{l}$

$V_x = w\left(\frac{l}{2} - x\right) + \frac{M_1 - M_2}{l}$

$M_3 \left(\text{at } x = \frac{l}{2} + \frac{M_1 - M_2}{wl}\right)$

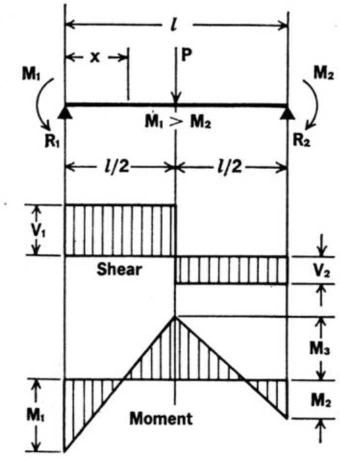
$= \frac{wl^2}{8} - \frac{M_1 + M_2}{2} + \frac{(M_1 - M_2)^2}{2wl^2}$

$M_x = \frac{wx}{2}(l - x) + \left(\frac{M_1 - M_2}{l}\right) x - M_1$

$b \left(\begin{array}{l}\text{To locate} \\ \text{inflection points}\end{array}\right) = \sqrt{\frac{l^2}{4} - \left(\frac{M_1 + M_2}{w}\right) + \left(\frac{M_1 - M_2}{wl}\right)^2}$

$\Delta_x = \frac{wx}{24EI}\left[x^3 - \left(2l + \frac{4M_1}{wl} - \frac{4M_2}{wl}\right) x^2 + \frac{12M_1}{w} x + l^3 - \frac{8M_1 l}{w} - \frac{4M_2 l}{w}\right]$

33. BEAM—CONCENTRATED LOAD AT CENTER AND VARIABLE END MOMENTS



$R_1 = V_1 = \frac{P}{2} + \frac{M_1 - M_2}{l}$

$R_2 = V_2 = \frac{P}{2} - \frac{M_1 - M_2}{l}$

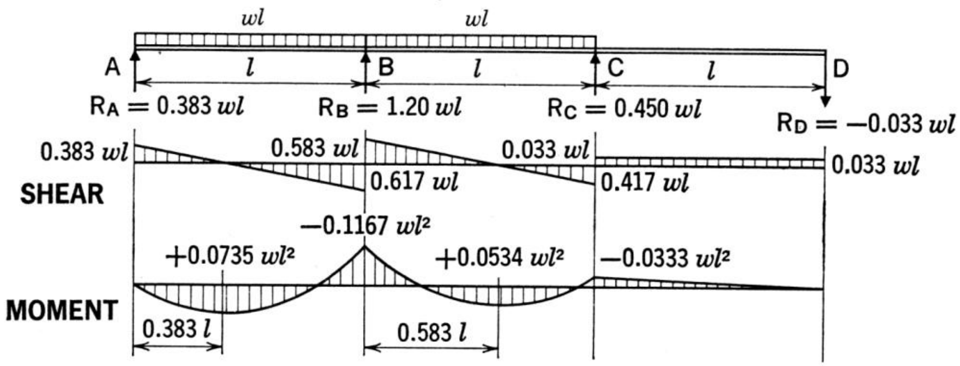
M_3 (At center) $= \frac{Pl}{4} - \frac{M_1 + M_2}{2}$

$M_x \left(\text{When } x < \frac{l}{2}\right) = \left(\frac{P}{2} + \frac{M_1 - M_2}{l}\right) x - M_1$

$M_x \left(\text{When } x > \frac{l}{2}\right) = \frac{P}{2}(l - x) + \frac{(M_1 - M_2)x}{l} - M_1$

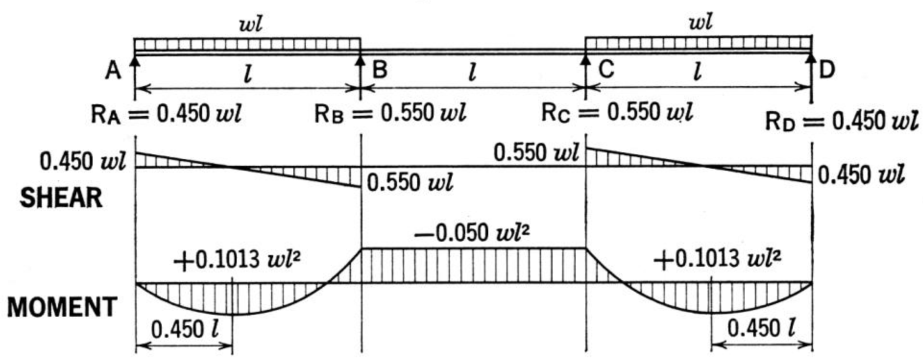
$\Delta_x \left(\text{When } x < \frac{l}{2}\right) = \frac{Px}{48EI}\left(3l^2 - 4x^2 - \frac{8(l - x)}{Pl}\left[M_1(2l - x) + M_2(l + x)\right]\right)$

34. CONTINUOUS BEAM—THREE EQUAL SPANS—ONE END SPAN UNLOADED



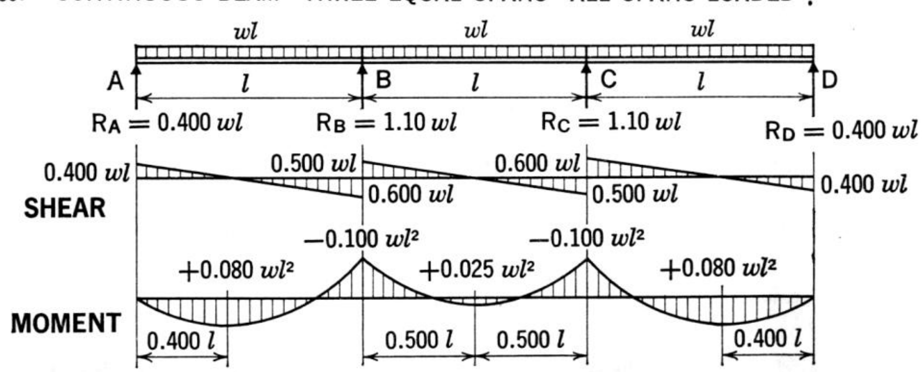
Δ Max. (0.430 l from A) = 0.0059 wl^4/EI

35. CONTINUOUS BEAM—THREE EQUAL SPANS—END SPANS LOADED



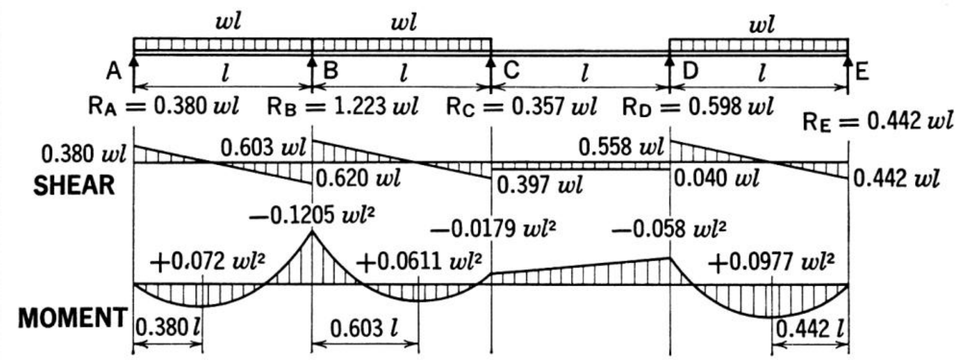
Δ Max. (0.479 l from A or D) = 0.0099 wl^4/EI

36. CONTINUOUS BEAM—THREE EQUAL SPANS—ALL SPANS LOADED



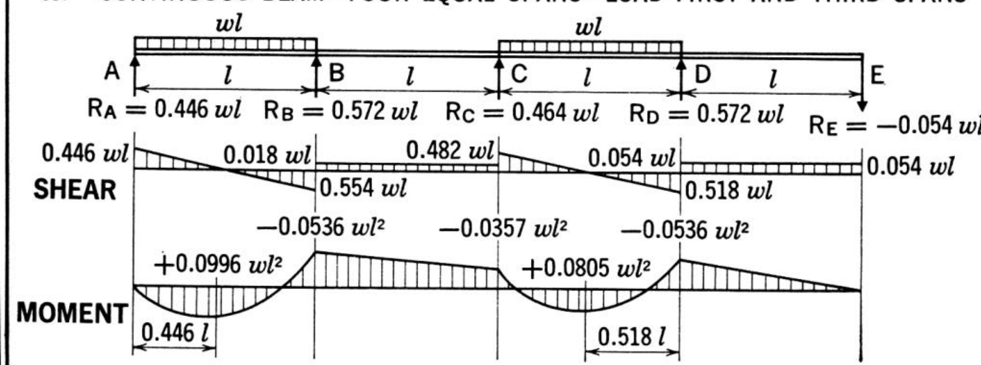
Δ Max. (0.446 l from A or D) = 0.0069 wl^4/EI

37. CONTINUOUS BEAM—FOUR EQUAL SPANS—THIRD SPAN UNLOADED



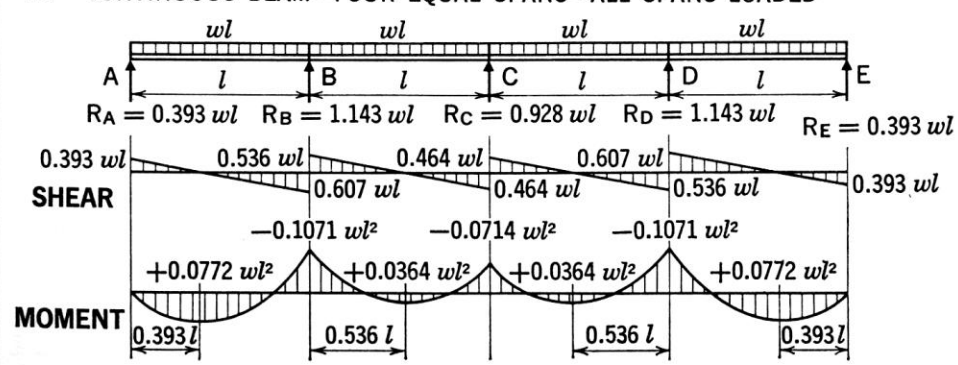
Δ Max. (0.475 l from E) = 0.0094 wl^4/EI

38. CONTINUOUS BEAM—FOUR EQUAL SPANS—LOAD FIRST AND THIRD SPANS



Δ Max. (0.477 l from A) = 0.0097 wl^4/EI

39. CONTINUOUS BEAM—FOUR EQUAL SPANS—ALL SPANS LOADED



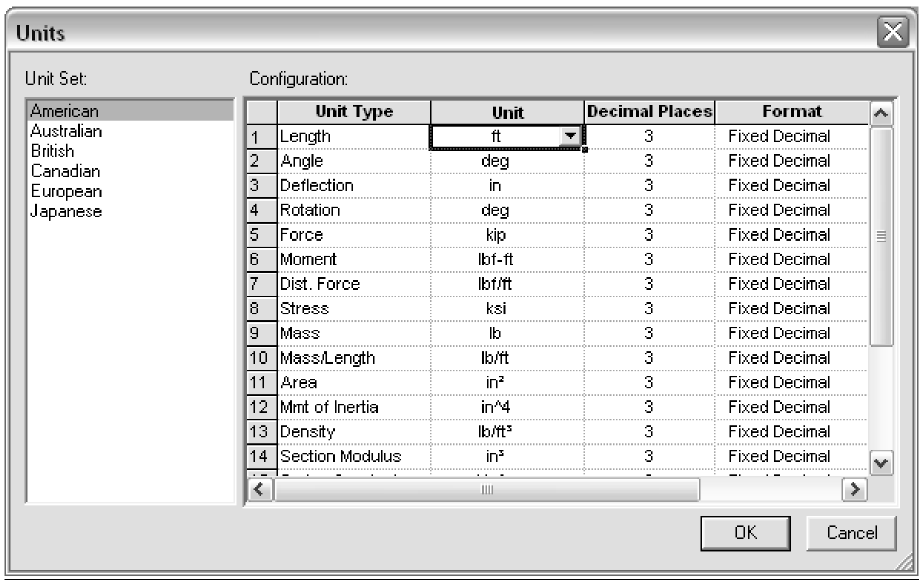
Δ Max. (0.440 l from A and E) = 0.0065 wl^4/EI

Beam Analysis using Multiframe

1. The software is on the computers in the College of Architecture in Programs under the Windows Start menu (see https://wikis.arch.tamu.edu/display/HELPDESK/Computer+Accounts for lab locations). Multiframe is under the Multiframe [current version #] menu. Or it can be downloaded from the web site: http://www.formsys.com/academic/multiframe

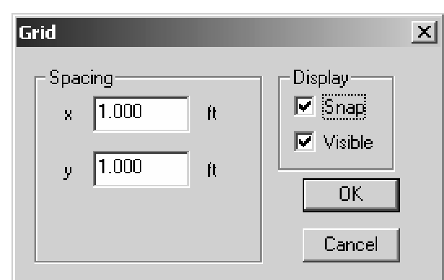
2. There are tutorials available on line at http://www.formsys.com/mflearning that list the tasks and order in greater detail. The first task is to define the unit system:

   - Choose Units… from the View menu. Unit sets are available, but specific units can also be selected by double clicking on a unit or format and making a selection from the menu.

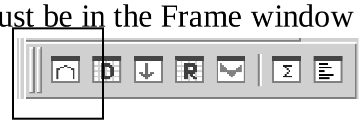


3. To see the scale of the geometry, a grid option is available:

   - Choose Grid… from the View menu



4. To create the geometry, you must be in the Frame window (default). The symbol is the frame in the window toolbar:

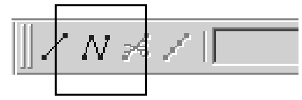


   The Member toolbar shows ways to create members:

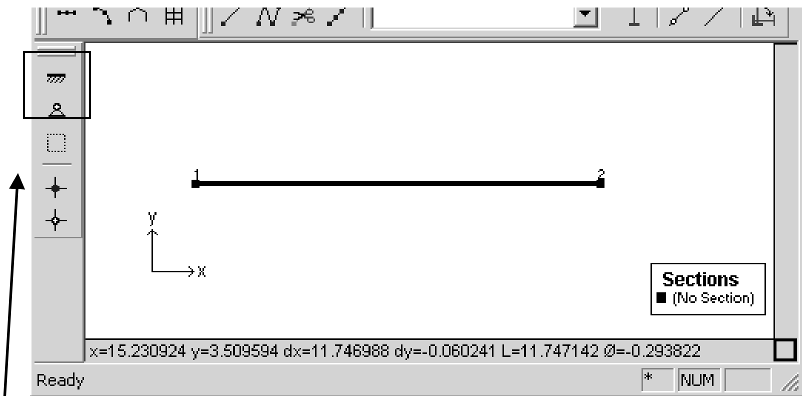
   The Generate toolbar has convenient tools to create typical structural shapes.

   - To create a beam with supports at one or both ends, use the add member button:

- Select a starting point and ending point with the cursor. The location of the cursor and the segment length is displayed at the bottom of the geometry window.
- To create a beam with supports NOT at the ends, use the add connected members button to create segments between supports and ends

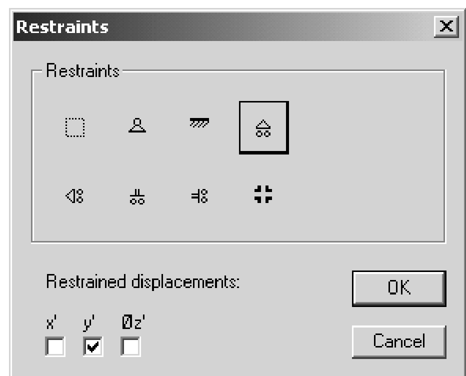


- Select a starting point and ending point with the cursor. The location of the cursor and the segment length is displayed at the bottom of the geometry window. The ESC button will end the segmented drawing.
- The geometry can be set precisely by selecting the beam member, bringing up the specific menu (right click), choosing Member Properties to set the length.



- The support types can be set by selecting the joint (drag) and using the Joint Toolbar (pin shown), or the Frame / Joint Restraint ... menu (right click).

  NOTE: If the support appears at both ends of the beam, you had the beam selected rather than the joint. Select the joint to change the support for and right click to select the joint restraints menu or select the correct support on the joint toolbar.



The support forces will be determined in the analysis.

5. All members must have sections assigned (see section 6.) in order to calculate reactions and deflections. To use a standard steel section **proceed to step 6.** For custom sections, the section information must be entered. To define a section:

- Choose Edit Sections / Add Section… from the Edit menu
- Type a name for your new section
- Choose group <u>Frame</u> from the group names provided so that the section will remain with the file data
- Choose a shape. The Flat Bar shape is a rectangular section.
- Enter the cross section data.

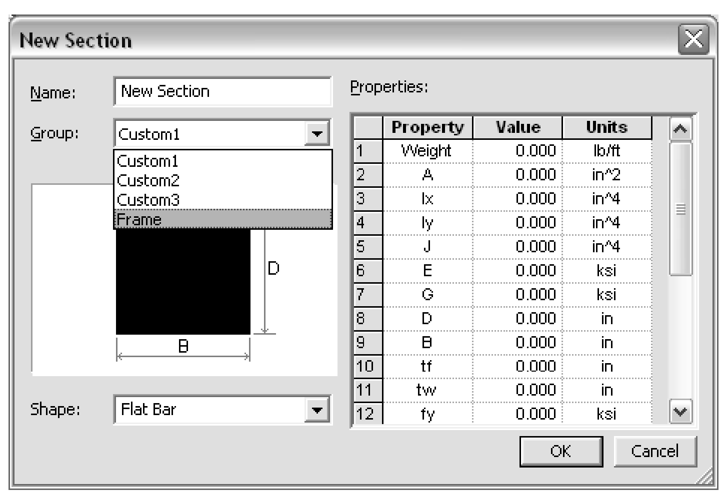
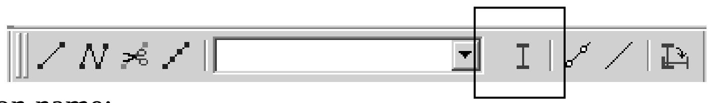


Table values 1-9 must have values for a Flat Bar, but not all are used for every analysis. A recommendation is to put the value of 1 for those properties you don't know or care about. Properties like t_f, t_w, etc. refer to wide flange sections.

- Answer any query. If the message says there is an error, the section will not be created until the error is corrected.

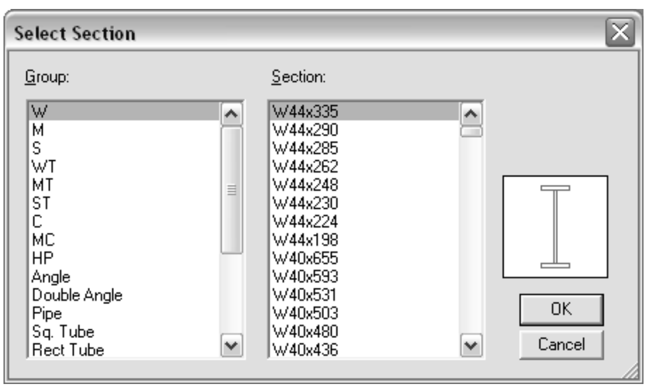
6. The standard sections library loaded is for the United States. If another section library is needed, use the Open Sections Library... command under the file menu, choose the library folder, and select the SectionsLibrary.slb file.

   Select the members (drag to make bold) and assign sections with the Section button on the Member toolbar:

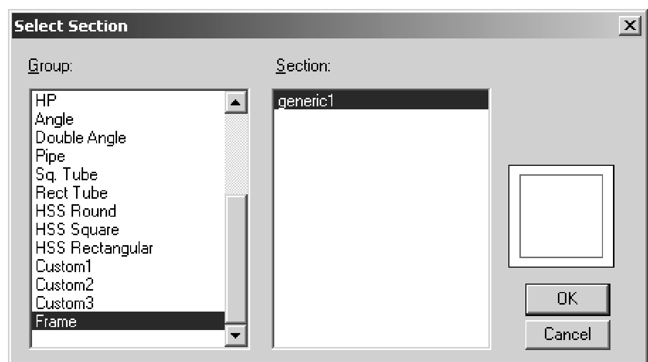


- Choose the group name and section name:

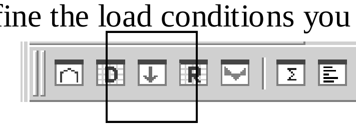
(STANDARD SHAPES)



(CUSTOM)

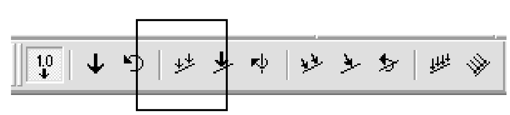


7. The beam geometry is complete, and in order to define the load conditions you must be in the Load window represented by the green arrow:

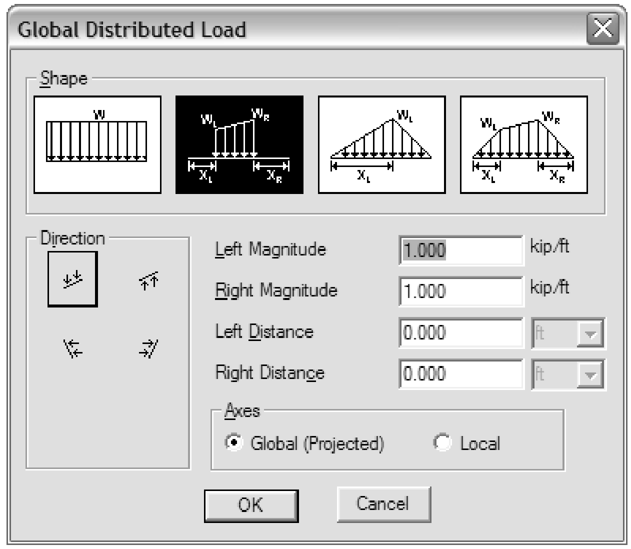


8. The Load toolbar allows a joint to be loaded with a force or a moment in global coordinates, shown by the first two buttons after the display numbers button. It allows a member to be loaded with a distributed load, concentrated load or moment (next three buttons) in global coordinates, as well as loading with distributed or single force or moment in the local coordinate system (next three buttons). It allows a load panel to be loaded with a distributed load in global or local coordinates (last two buttons)..

- Choose the member to be loaded (drag) and select the load type (here shown for global distributed loading):

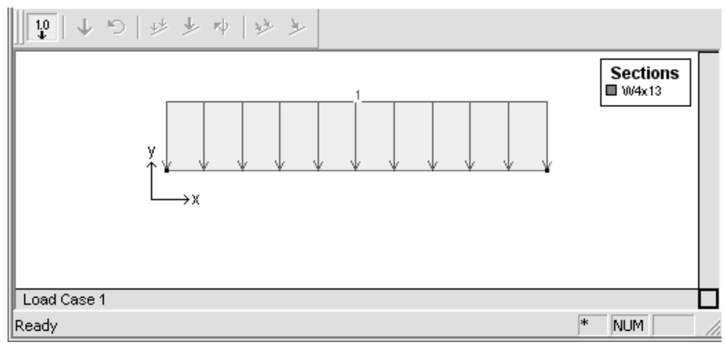


- Choose the distribution type and direction. Note that the arrow shown is the direction of the loading. There is no need to put in negative values for downward loading.
- Enter the values of the load and distances (if any). Distances can be entered as a function of the length , i.e. L/2, L/4...

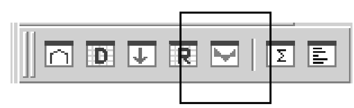


NOTE: Do not put support reactions as applied loads. The analysis will determine the reaction values.

Multiframe4D will automatically generate a grouping called a Load Case named Load Case 1 when a load is created. All additional loads will be added to this load case unless a new load case is defined (Add case under the Case menu).

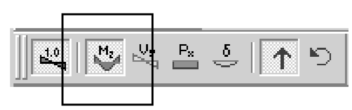


9. In order to run the analysis after the geometry, member properties and loading has been defined:
   - Choose Linear from the Analyze menu
10. If the analysis is successful, you can view the results in the Plot window represented by the red moment diagram:

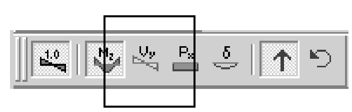


11. The Plot toolbar allows the numerical values to be shown (1.0 button), the reaction arrows to be shown (brown up arrow) and reaction moments to be shown (brown curved arrow):

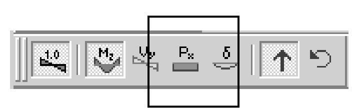
   - To show the moment diagram, Choose the red Moment button



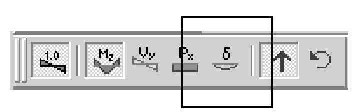
   - To show the shear diagram, Choose the green Shear button



   - To show the axial force diagram, Choose the purple Axial Force button



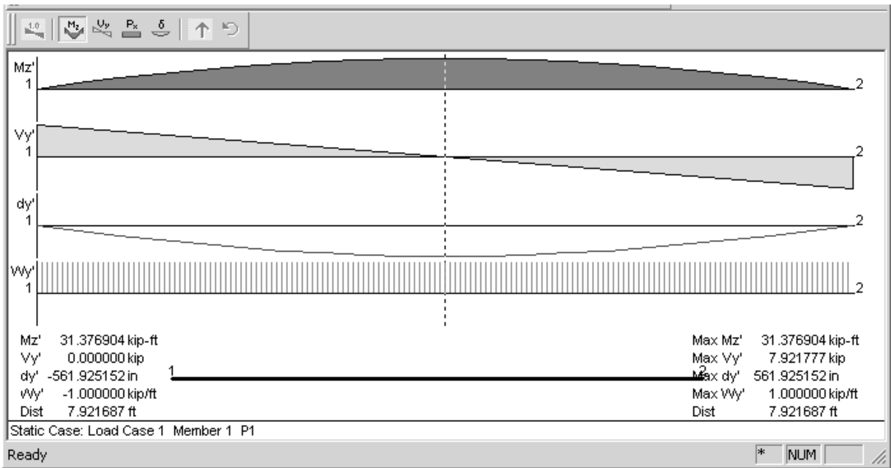
   - To show the deflection diagram, Choose the blue Deflection button



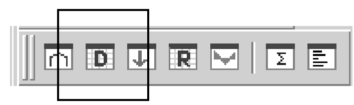
   - To animate the deflection diagram, Choose Animate... from the Display menu. You can also save the animation to a .avi file by checking the box.

- To plot the bending moment on the "top" choose Preferences from the Edit menu and under the Presentation tab Draw moments on the compression face

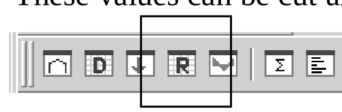
- To see exact values of shear, moment and deflection, double click on the member and move the vertical cross hair with the mouse. The ESC key will return you to the window.

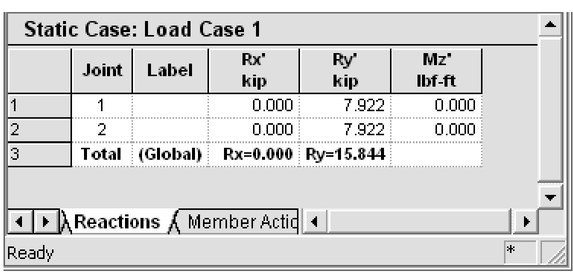


12. The Data window (D) allows you to view all data "entered" for the geometry, sections and loading. These values can be edited.



13. The Results window (R) allows you to view all results of the analysis including displacements, reactions, member forces (actions) and stresses. These values can be cut and pasted into other Windows programs such as Word or Excel.



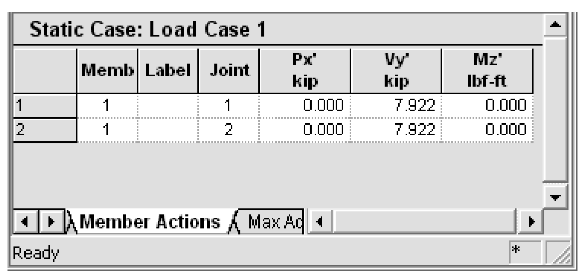


Static Case: Load Case 1

	Joint	Label	Rx' kip	Ry' kip	Mz' lbf-ft
1	1		0.000	7.922	0.000
2	2		0.000	7.922	0.000
3	Total	(Global)	Rx=0.000	Ry=15.844	

Reactions / Member Actic

Ready



Static Case: Load Case 1

	Memb	Label	Joint	Px' kip	Vy' kip	Mz' lbf-ft
1	1		1	0.000	7.922	0.000
2	1		2	0.000	7.922	0.000

Member Actions / Max Ac

Ready

NOTE: Px' refers to the axial load (P) in the local axis x direction (x'). Vy' refers to the shear perpendicular to the local x axis, and Mz' refers to the bending moment.

14. To save the file Choose Save from the File menu.

15. To load an existing file Choose Open... from the File menu.

16. To print a plot Choose Print Window... from the File menu. As an alternative, you may copy the plot (Ctrl+c) and paste it in a word processing document (Ctrl+v).

Examples:
Beams (V, M, Stresses and Design)

<u>Example 1</u>

Example Problem 9.5: Section Modulus (Figures 9.26 to 9.28)

Two C10×15.3 steel channels are placed back to back to form a 10"-deep beam. Determine the permissible P if $F_b = 30$ ksi. Assume A572 grade 50 steel.

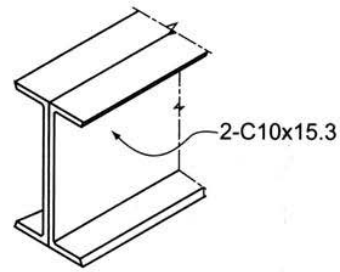


Figure 9.26 Two steel channels.

Solution:

$$I_x = 67.4 \text{ in.}^4 \times 2 = 134.8 \text{ in.}^4$$

$$M_{\text{max}} = \tfrac{1}{2}(5)(5) + (P/2)(5)$$

$$M_{\text{max}} = 12.5 + 2.5P$$

$$= (12.5 \text{ k-ft.} + 2.5P) \times (12 \ ^{\text{in.}}/_{\text{ft.}})$$

$$f = \frac{Mc}{I} = \frac{M}{S}; \quad \therefore M = F_b \times S_x$$

$$S_x = 2 \times 13.5 \text{ in.}^3 = 27 \text{ in.}^3$$

Equating both M_{max} equations:

$$M = (30 \text{ k/in.}^2) \times (27 \text{ in.}^3) = 810 \text{ k-in.}$$

$$(12.5 \text{ k-ft.} + 2.5P)(12 \ ^{\text{in.}}/_{\text{ft.}}) = 810 \text{ k-in.}$$

Dividing both sides of the equation by 12 in./ft.:

$$(12.5 \text{ k-ft.}) + (2.5 \text{ ft.})P + 67.5 \text{ k-ft.}$$

$$2.5P = 55 \text{ k}$$

$$\therefore P = 22 \text{ k}$$

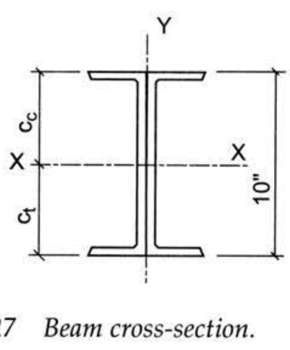


Figure 9.27 Beam cross-section.

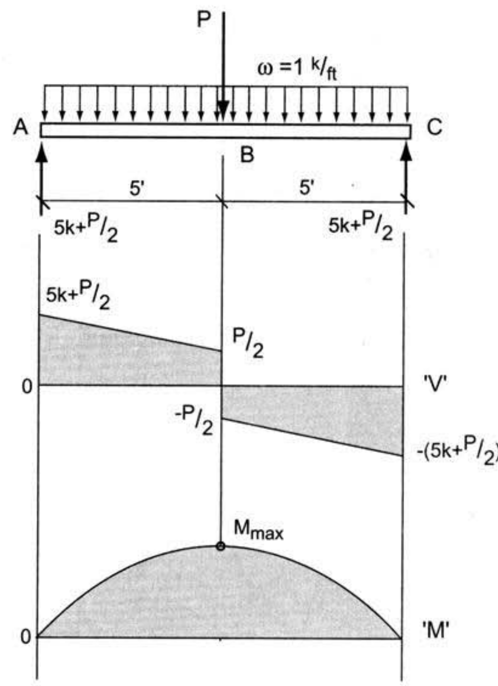


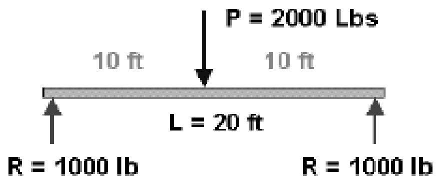
Figure 9.28 Load, V, and M diagrams.

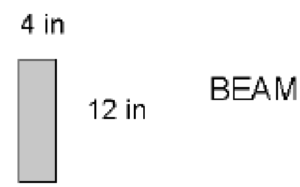
Example 2 **From eStructures v1.1, Schodek and Pollalis, 2000 Harvard College**

Beam Analysis STEP 1

BEAM ANALYSIS

Determine the bending and shear stresses in the timber beam shown. Also determine the deflections present. Is the beam adequately sized? Assume that the allowable bending stresses is $F_{b\,allowable}$ = 1500 lbs/in^2, the allowable shear stress is $F_{v\,allowable}$ = 150 lbs/in^2, and the allowable deflection is L /360. Also assume that the allowable stress in bearing is f_{bg} = 400 lbs/in^2 and E = 1.6 x 10^6 lbs/in^2



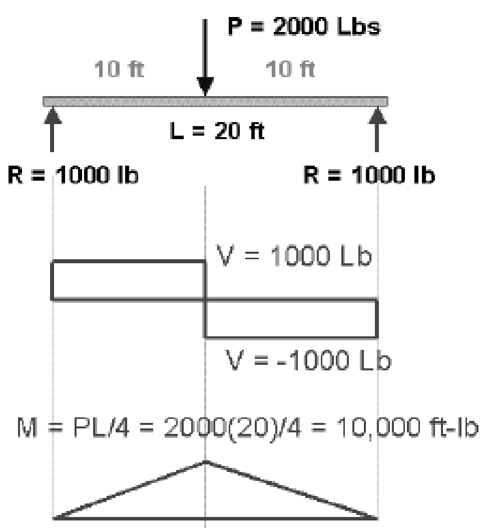


CHECK BENDING, SHEAR, BEARING STRESSES AND DEFLECTIONS

Reference: eStructures v1.1, Shodek & Pollalis, 2000
Simple Beams, Beam Analysis

Beam Analysis STEP 2

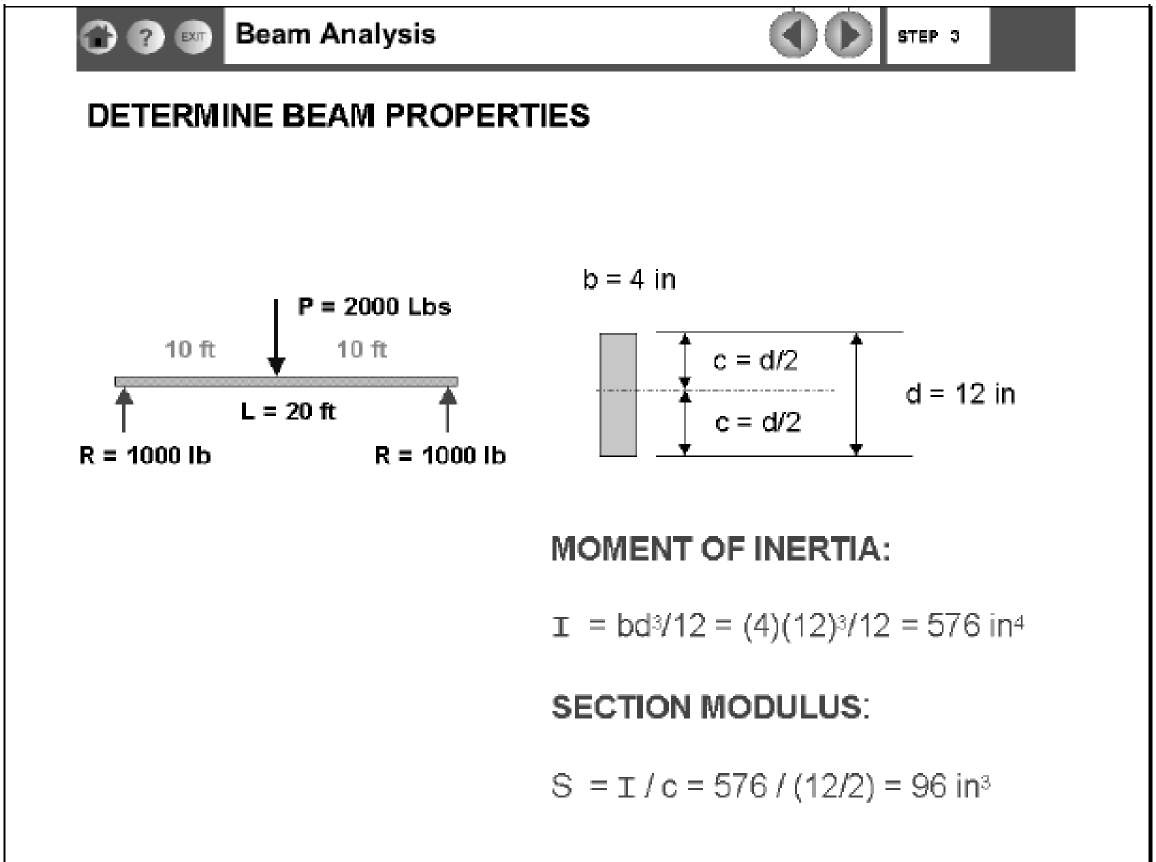
DRAW SHEAR AND MOMENT DIAGRAMS

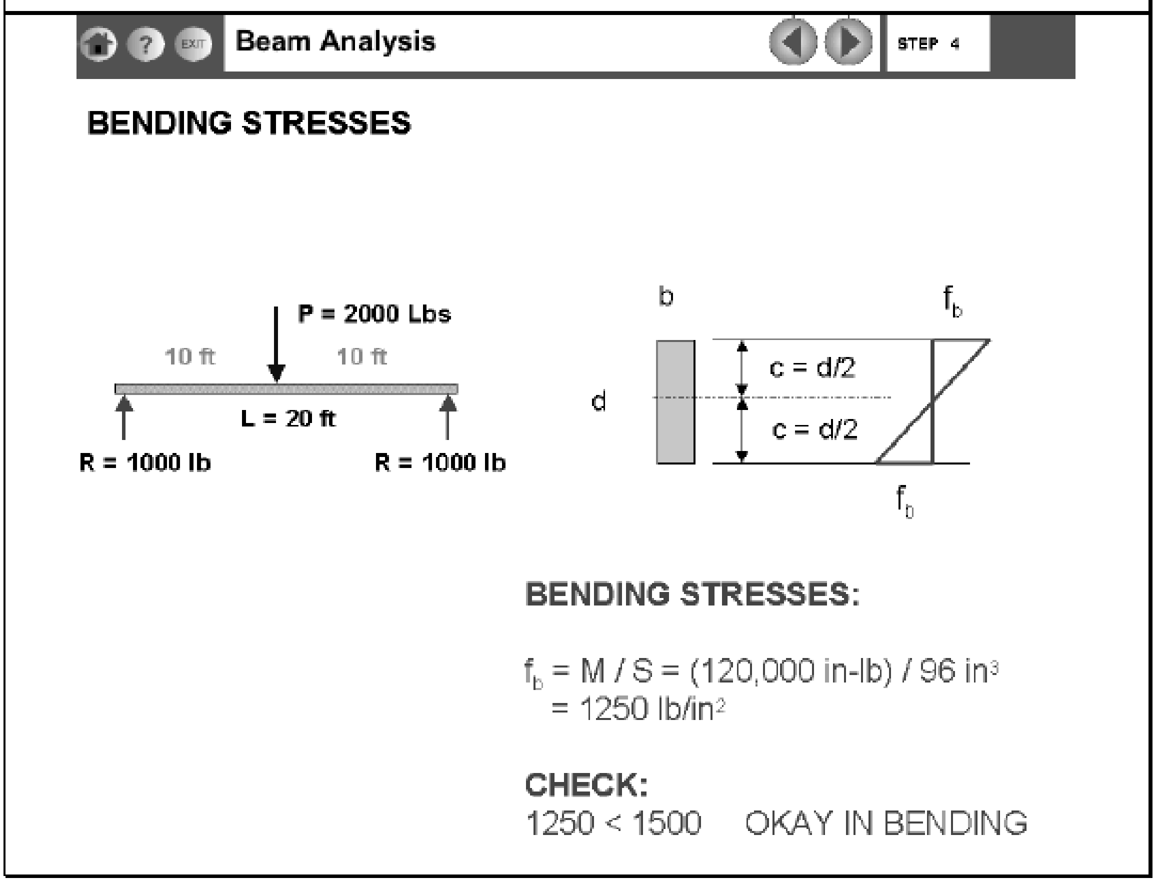


SHEAR AND MOMENT DIAGRAMS:

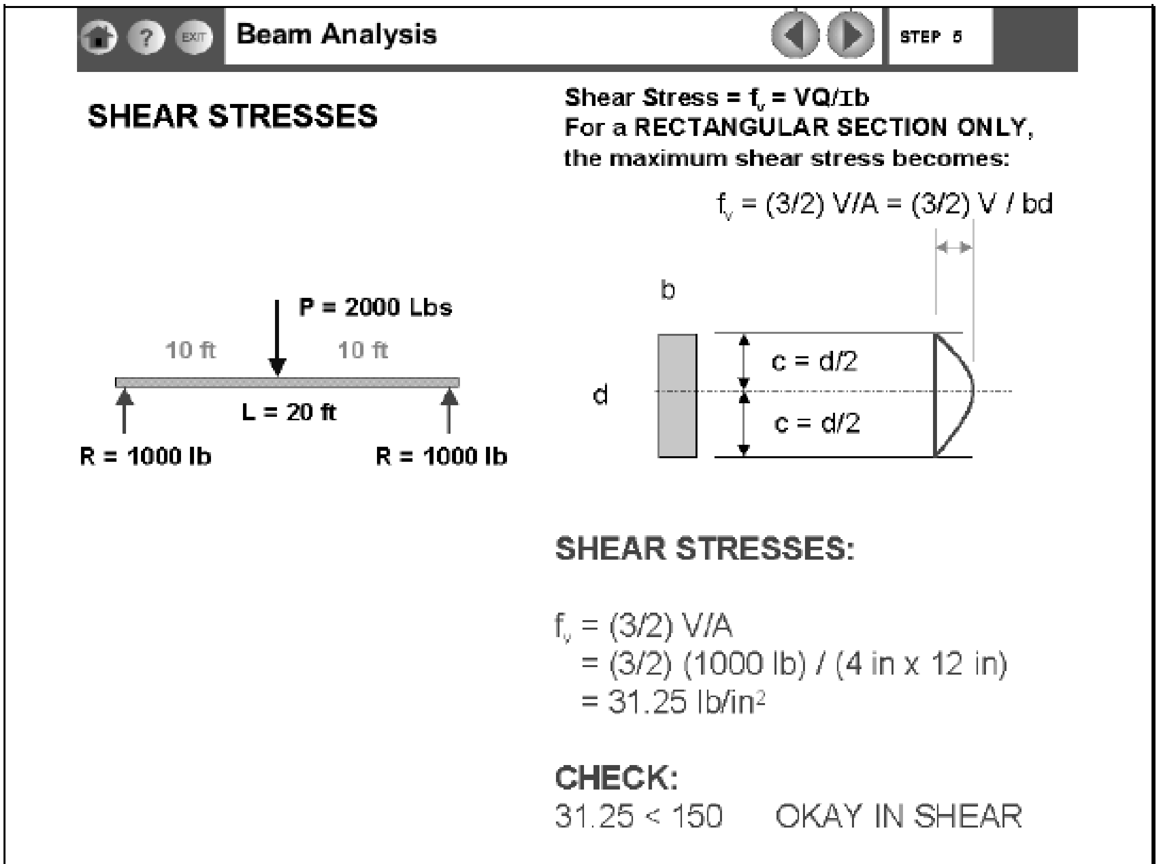
Maximum Shear Force:
= 1000 lbs
Maximum Bending Moment:
= 10,000 ft-lbs = 120, 000 in-lbs

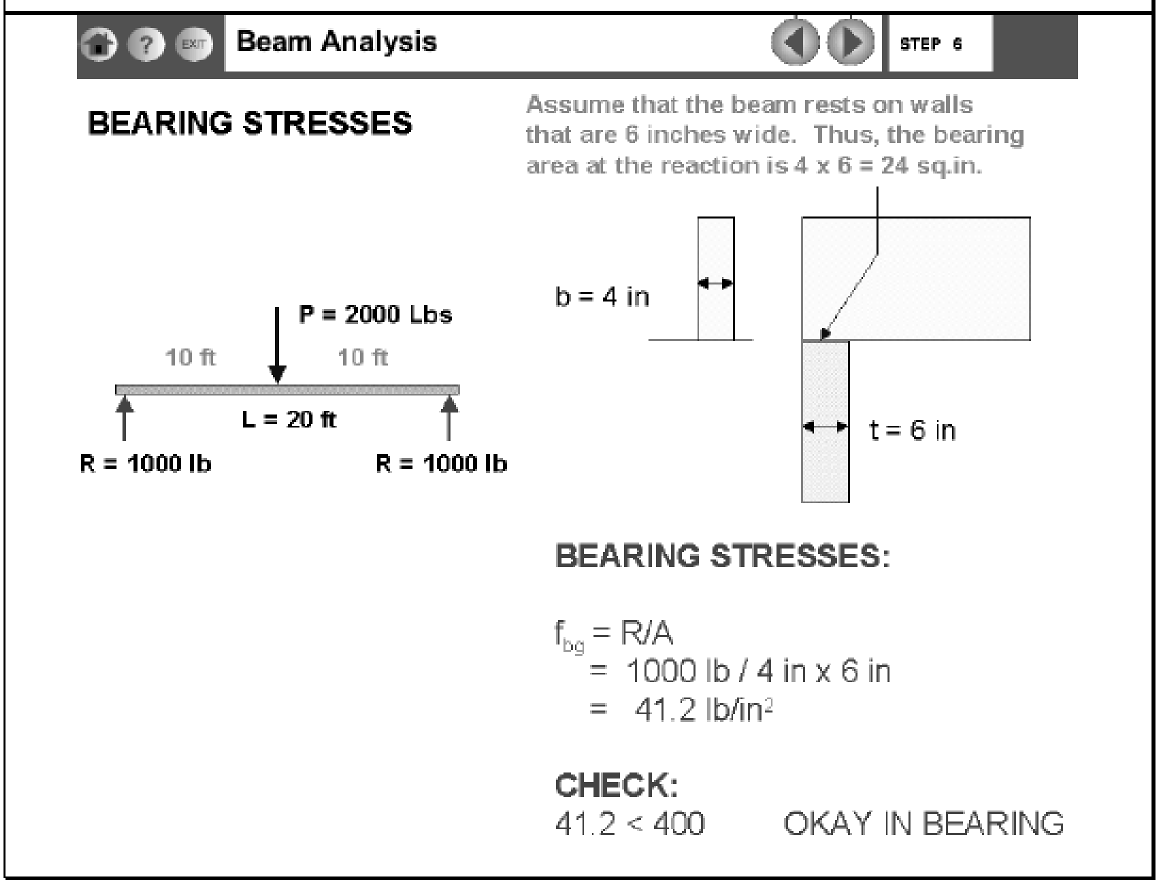
Example 2 (continued)





Example 2 (continued)



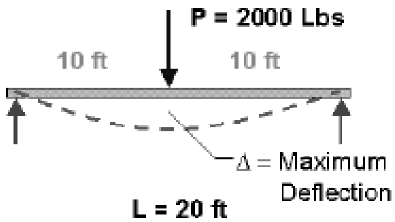


Example 2 (continued)

Beam Analysis STEP 7

DEFLECTIONS

For a simply supported beam with a concentrated load, the maximum deflection is given by $\Delta = PL^3/48EI$:



$$\Delta = PL^3/48EI$$

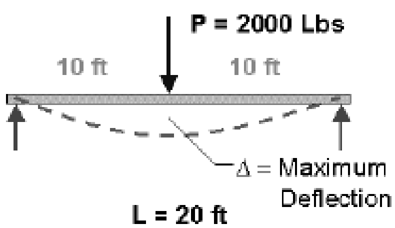
$$= \frac{(2000\ \text{lb})(20\ \text{ft} \times 12\ \text{in/ft})^3}{48\ (1.6 \times 10^6\ \text{lb/in}^2)(576\ \text{in}^4)}$$

$$= 0.625\ \text{inches}$$

Beam Analysis STEP 8

DEFLECTIONS

For a simply supported beam with a concentrated load, the maximum deflection is given by $\Delta = PL^3/48EI$:



$$\Delta = PL^3/48EI$$

$$= \frac{(2000\ \text{lb})(20\ \text{ft} \times 12\ \text{in/ft})^3}{48\ (1.6 \times 10^6\ \text{lb/in}^2)(576\ \text{in}^4)}$$

$$= 0.625\ \text{inches}$$

COMPARE ACTUAL DEFLECTION TO ALLOWABLE DEFLECTION:

$\Delta_{actual} = 0.625$ in

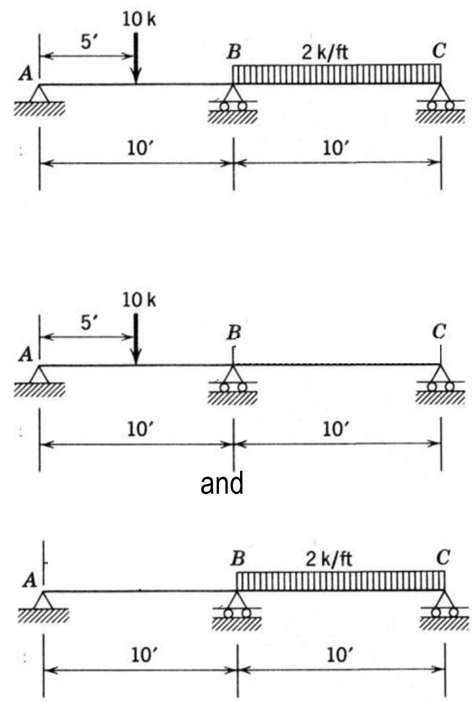
$\Delta_{a|allowable}$: L / 360 = (20 ft x 12 in/ft)/360 = 0.67 in.

$\Delta_{actual} < \Delta_{a\ allowable}$

Deflections are okay!

Example 3

Using an "approximate" method of analysis (specifically beam diagrams and formulas with superpositioning), find reactions, shears, and moments present in the structure. Verify the solution using a computer-based structural analysis program (Multiframe4D).



SOLUTION:

The load cases can be divided into the two shown which correspond to beam diagrams 30 and 29 (mirrored).

Because the maximum moments **do not** occur at the same place, find the reactions to add up and construct the V & M diagrams. The moment diagram should look like the two diagrams (with one flipped) "added" together:

Diagram 30:

$$R_1 = \frac{13}{32}P = \frac{13}{32}(10k) = 4.06k \qquad R_2 = \frac{11}{16}P = \frac{11}{16}(10k) = 6.875k$$

$$R_3 = -\frac{3}{32}P = -\frac{3}{32}(10k) = -0.9375k$$

Diagram 29:

$$R_1(was\ R_3) = -\frac{1}{16}wl = -\frac{1}{16}(2\ {}^{k}\!/\!_{ft})10\ ft = -1.25k \qquad R_2 = \frac{5}{8}wl = \frac{5}{8}(2\ {}^{k}\!/\!_{ft})10\ ft = 12.5k$$

$$R_3(was\ R_1) = \frac{7}{16}wl = \frac{7}{16}(2\ {}^{k}\!/\!_{ft})10\ ft = 8.75k$$

Reaction sums:

R_1 = 4.06 + -1.25 = 2.81 k R_2 = 6.875+12.5=19.375k R_3 = -0.9375+8.75=7.8125k

Shear calculations:

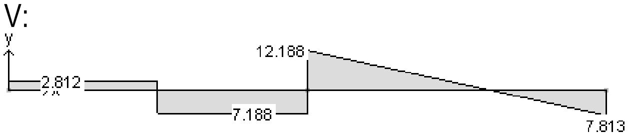
V_A = 0 and 2.81k $V_{at\ 5ft}$ = 2.81k and 2.81-10=-7.19k V_B = -7.19k and -7.19+19.375=12.185k
V_C = 12.185-2k/ft(10ft)=-7.8125 and -7.815+7.815=0k

Moment shapes:

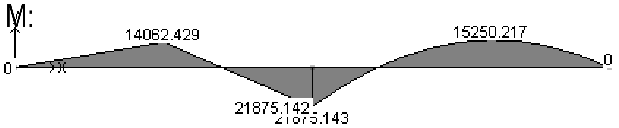
M_A = 0 $M_{at\ 5ft}$ =0+2.81k(5ft) = 14.05k-ft M_B = 14.05-7.19k(5ft) = -21.9k-ft
location of cross over = 12.185k/(2k/ft) = 6.0925ft: $M_{at\ 6.1\ ft\ from\ B}$ = -21.9+12.185k(6.0925ft)/2 = 15.218 k-ft
M_C = 15.218 – 7.8125k(3.9075ft)/2 = 0

MULTIFRAME4D:

V:



M:

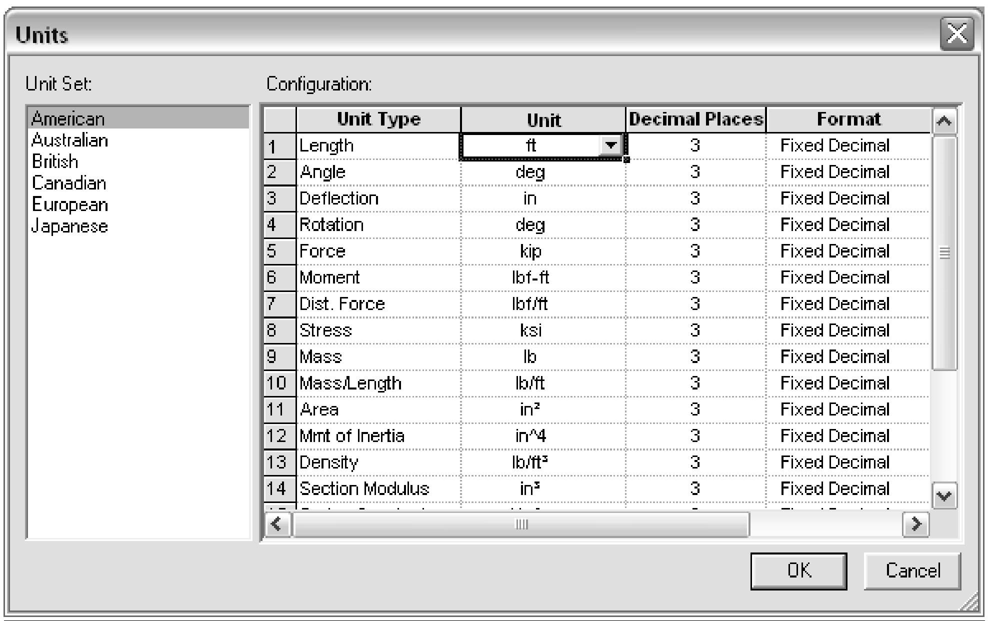


Truss Analysis using Multiframe

1. The software is on the computers in the College of Architecture in Programs under the Windows Start menu (see https://wikis.arch.tamu.edu/pages/HELPDESK/Computer+Accounts for lab locations). Multiframe is under the Multiframe [current version #] menu. Or it can be downloaded from the web site: http://www.formsys.com/academic/multiframe

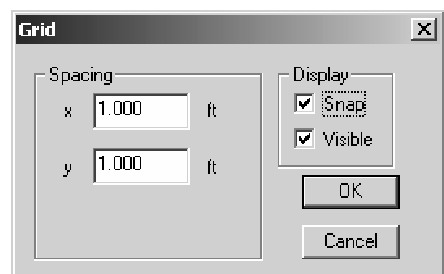
2. There are tutorials available on line at http://www.formsys.com/mflearning that list the tasks and order in greater detail. The first task is to define the unit system:

   - Choose Units… from the View menu. Unit sets are available, but specific units can also be selected by double clicking on a unit or format and making a selection from the menu.

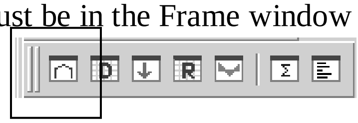


3. To see the scale of the geometry, a grid option is available:

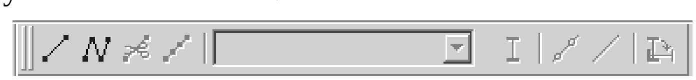
   - Choose Grid… from the View menu



4. To create the geometry, you must be in the Frame window (default). The symbol is the frame in the window toolbar:

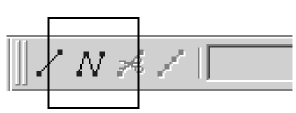


   The Member toolbar shows ways to create members:



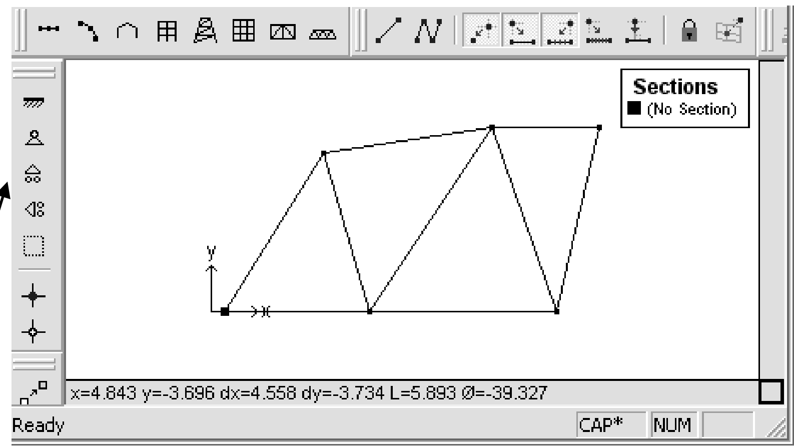
   The Generate toolbar has convenient tools to create typical structural shapes.

   - To create a truss, use the add connected members button:



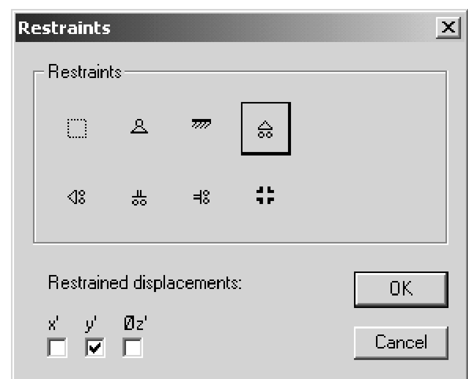
- Select a starting point and ending point with the cursor. The location of the cursor and the segment length is displayed at the bottom of the geometry window. The ESC button will end the segmented drawing. Continue to use the add connected members button. Any time the cursor is over an existing joint, the joint will be highlighted by a red circle.

- The geometry can be set precisely by selecting the joint (drag), and bringing up the joint properties menu (right click) to set the coordinates.



- The support types can be set by selecting the joint (drag) and using the Joint Toolbar (pin shown), or the Frame / Joint Restraint ... menu (right click).

NOTE: If the support appears at both ends of the member, you had the member selected rather than the joint. Select the joint to change support for and right click to select the joint restraints menu or select the correct support on the joint toolbar.



The support forces will be determined in the analysis.

5. All members must have sections assigned (see section 6.) in order to calculate reactions and deflections. To use a standard steel section **proceed to step 6.** For custom sections, the section information must be entered. To define a section:

- Choose Edit Sections / Add Section… from the Edit menu

- Type a name for your new section

- Choose group <u>Frame</u> from the group names provided so that the section will remain with the file data

- Choose a shape. The Flat Bar shape is a rectangular section.

- Enter the cross section data.

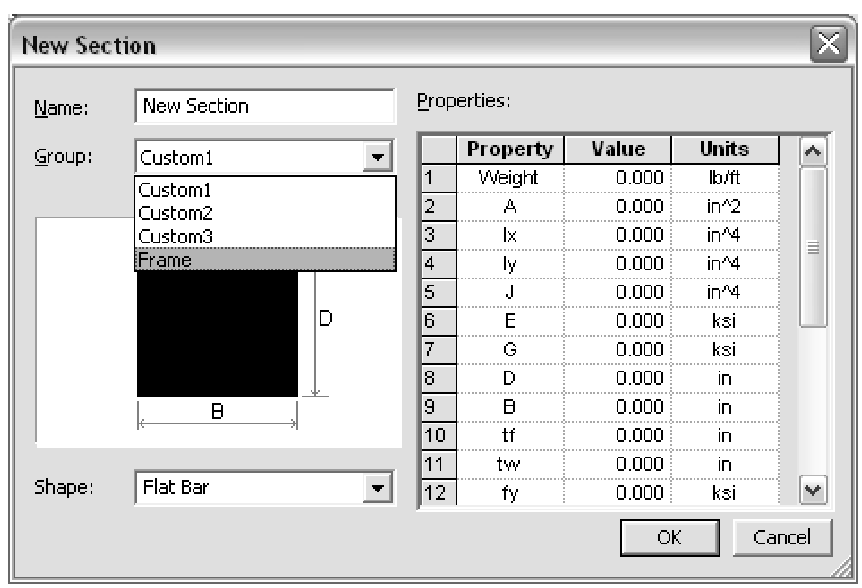


Table values 1-9 must have values for a Flat Bar, but not all are used for every analysis. A recommendation is to put the value of 1 for those properties you don't know or care about. Properties like t_f, t_w, etc. refer to wide flange sections.

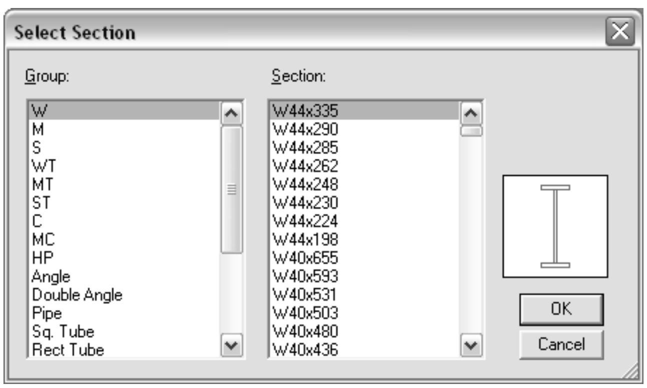
- Answer any query. If the message says there is an error, the section will not be created until the error is corrected.

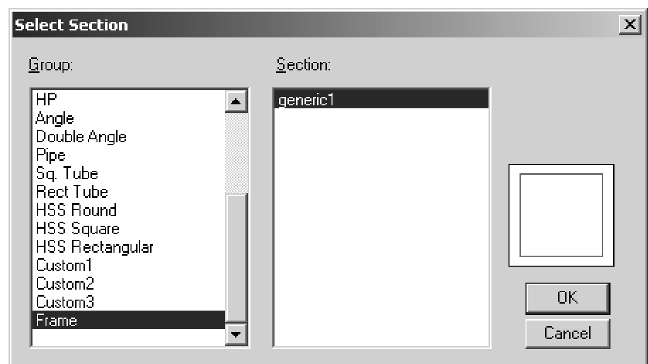
6. The standard sections library loaded is for the United States. If another section library is needed, use the Open Sections Library... command under the file menu, choose the library folder, and select the SectionsLibrary.slb file.

   Select the members (drag to make bold) and assign sections with the Section button on the Member toolbar:

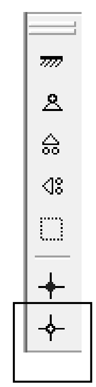
- Choose the group name and section name:

(STANDARD SHAPES) (CUSTOM)

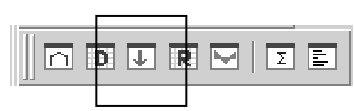




7. In order for Multiframe4D to recognize that the truss members are two-force bodies, all joints must be highlighted and assigned as pins with the Pinned Joints button on the Joint toolbar:

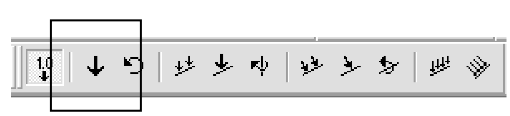


8. The truss geometry is complete, and in order to define the load conditions you must be in the Load window represented by the green arrow:



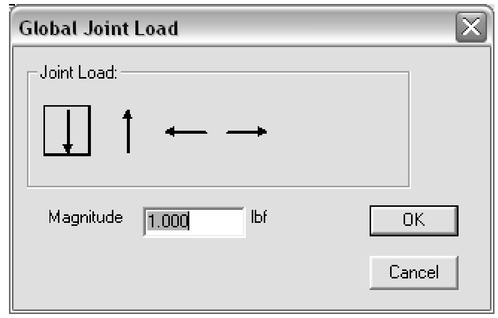
9. The Load toolbar allows a joint to be loaded with a force or a moment in global coordinates, shown by the first two buttons after the display numbers button. It allows a member to be loaded with a distributed load, concentrated load or moment (next three buttons) in global coordinates, as well as loading with distributed or single force or moment in the local coordinate system (next three buttons). It allows a load panel to be loaded with a distributed load in global or local coordinates (last two buttons).

- Choose the joint to be loaded (drag) and select the load type (here shown for point loading):



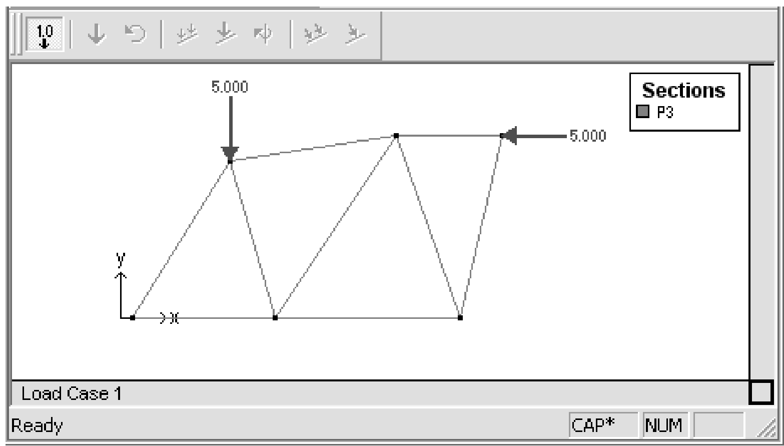
- Choose the direction by the arrow shown. There is no need to put in negative values for downward loading.

- Enter the values of the load



NOTE: Do not put support reactions as applied loads. The analysis will determine the reaction values.

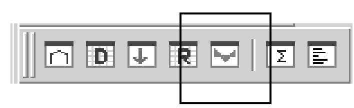
Multiframe2D will automatically generate a grouping called a Load Case named Load Case 1 when a load is created. All additional loads will be added to this load case unless a new load case is defined (Add case under the Case menu).



10. In order to run the analysis after the geometry, member properties and loading has been defined:

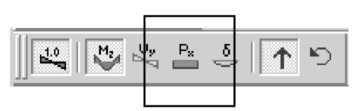
    - Choose Linear from the Analyze menu

11. If the analysis is successful, you can view the results in the Plot window represented by the red moment diagram:

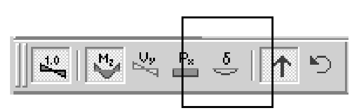


12. The Plot toolbar allows the numerical values to be shown (1.0 button), the reaction arrows to be shown (brown up arrow) and reaction moments to be shown (brown curved arrow):

- To show the axial force diagram, Choose the purple Axial Force button. Tensile members will have “T” by the value (if turned on), while compression members will have “C” by the value

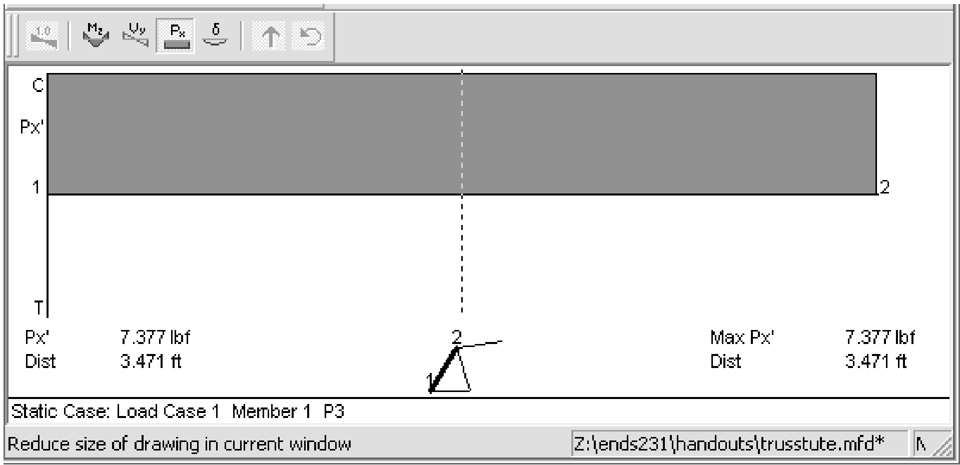


- To show the deflection diagram, Choose the blue Deflection button



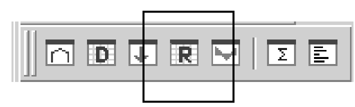
- To animate the deflection diagram, Choose Animate... from the Display menu. You can also save the animation to a .avi file by checking the box.

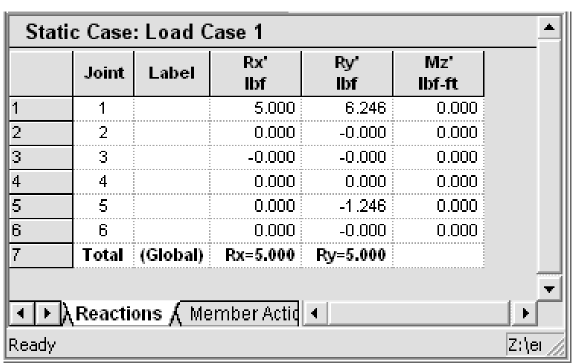
- To see exact values of axial load and deflection, double click on the member and move the vertical cross hair with the mouse. The ESC key will return you to the window.



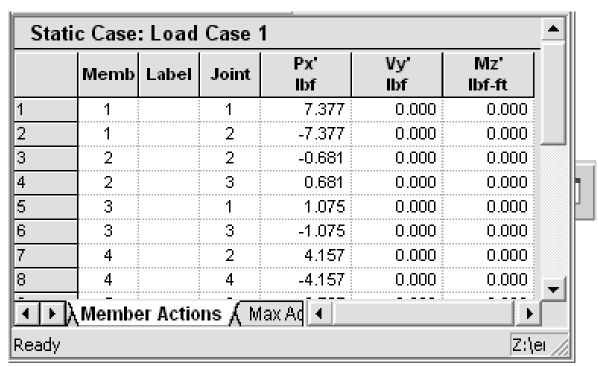
13. The Data window (D) allows you to view all data "entered" for the geometry, sections and loading. These values can be edited.

14. The Results window (R) allows you to view all results of the analysis including displacements, reactions, member forces (actions) and stresses. These values can be cut and pasted into other Windows programs such as Word or Excel.




Static Case: Load Case 1

	Joint	Label	Rx' lbf	Ry' lbf	Mz' lbf-ft
1	1		5.000	6.246	0.000
2	2		0.000	-0.000	0.000
3	3		-0.000	-0.000	0.000
4	4		0.000	0.000	0.000
5	5		0.000	-1.246	0.000
6	6		0.000	-0.000	0.000
7	Total	(Global)	Rx=5.000	Ry=5.000	


Static Case: Load Case 1

	Memb	Label	Joint	Px' lbf	Vy' lbf	Mz' lbf-ft
1	1		1	7.377	0.000	0.000
2	1		2	-7.377	0.000	0.000
3	2		2	-0.681	0.000	0.000
4	2		3	0.681	0.000	0.000
5	3		1	1.075	0.000	0.000
6	3		3	-1.075	0.000	0.000
7	4		2	4.157	0.000	0.000
8	4		4	-4.157	0.000	0.000

NOTE: Px' refers to the axial load (P) in the local axis x direction (x').

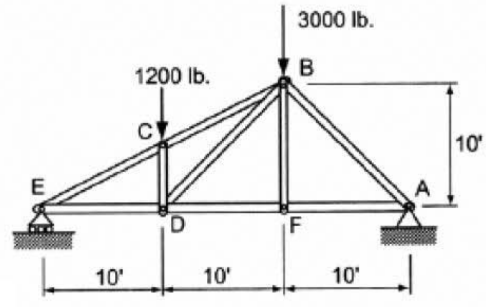
15. To save the file Choose Save from the File menu.
16. To load an existing file Choose Open... from the File menu.
17. To print a plot Choose Print Window... from the File menu. As an alternative, you may copy the plot (Ctrl+c) and paste it in a word processing document (Ctrl+v).

Examples:
Trusses and Columns

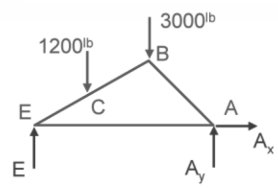
Example 1

Example Problem 4.1 (Method of Joints)

An asymmetrical roof truss, shown in Figure 4.4, supports two vertical roof loads. Determine the support reactions at each end, then, using the method of joints, solve for all member forces. Summarize the results of all member forces on a FBD (this diagram is referred to as a *force summation diagram*).



1. FBD

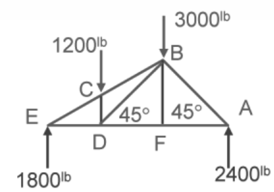


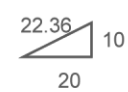
2. solve for support forces

$$\Sigma F_x = A_x = \boxed{0}$$

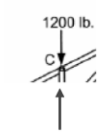
$$\Sigma M_A = 3000^{lb}\cdot 10^{ft} + 1200^{lb}\cdot 20^{ft} - E\cdot 30^{ft} = 0 \qquad E = \frac{54000^{lb-ft}}{30^{ft}} = \boxed{1800^{lb}}$$

$$\Sigma F_y = 1800^{lb} - 1200^{lb} - 3000^{lb} + A_y = 0 \qquad A_y = \boxed{2400^{lb}}$$





3. look for special cases:

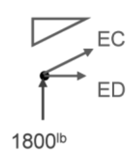


C, so CE = BC and CD = -1200lb

F, so DF = AF and BF = 0

4. choose a joint with 2 or less unknowns: E or A will work (C won't)

E:



$$\Sigma F_y = 1800^{lb} + EC\left(\frac{10}{22.36}\right) = 0$$

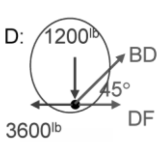
$$EC = -1800^{lb}\left(\frac{22.36}{10}\right) = \boxed{-4025^{lb}} = BC$$

$$\Sigma F_x = ED + (-4025^{lb})\left(\frac{20}{22.36}\right) = 0 \qquad ED = 4025^{lb}\left(\frac{20}{22.36}\right) = \boxed{3600^{lb}}$$

need BD, AB, (AF or DF) which leaves joints B, D & A (F won't work)

5. choose a joint with 2 or less unknowns: B, D or A will work (F won't)

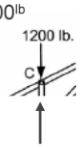
D:



$$\Sigma F_y = -1200^{lb} + BD\sin 45 = 0 \quad BD = \frac{1200^{lb}}{\sin 45} = \boxed{1697^{lb}}$$

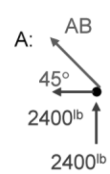
$$\Sigma F_x = -3600^{lb} + DF + (1697^{lb})\cos 45 = 0$$

$$DF = \boxed{2400^{lb}} = AF$$



6. last joint needs only one equation

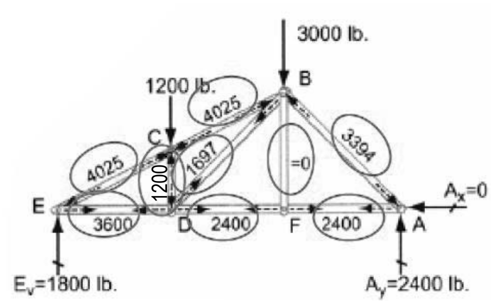
A:



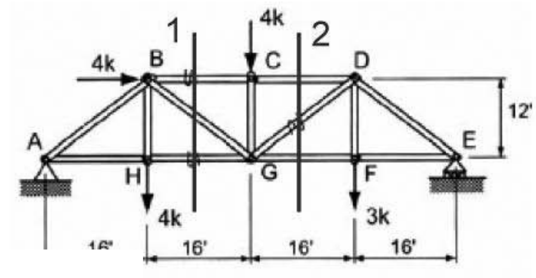
$$\Sigma F_y = 2400^{lb} + AB\sin 45 = 0$$

$$AB = \frac{-2400^{lb}}{\sin 45} = \boxed{-3394^{lb}}$$

$$(\Sigma F_x = -2400^{lb} - (-3394^{lb})\cos 45 = 0)\ \surd$$



Example 2

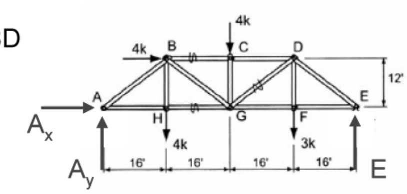


Example Problem 4.3 (Method of Sections)

A 64-foot parallel chord truss (Figure 4.30) supports horizontal and vertical loads as shown. Using the method of sections, determine the member forces *BC*, *HG*, and *GD*.

1. look for sections

2. FBD



3. solve for support forces

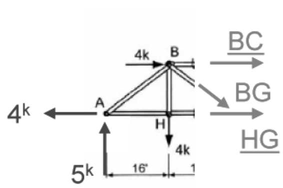
$$\sum F_x = A_x + 4^k = 0 \qquad A_x = \boxed{-4^k}$$

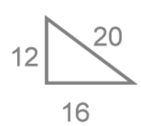
$$\sum F_y = A_y - 4^k - 4^k - 3^k + E = 0$$

$$\sum M_A = -4^k \cdot 12^{ft} - 4^k \cdot 16^{ft} - 4^k \cdot 32^{ft} - 3^k \cdot 48^{ft} + E \cdot 64^{ft} = 0$$

$$E = \frac{384^{k-ft}}{64^{ft}} = \boxed{6^k} \qquad \text{and sub:} \quad A_y = \boxed{5^k}$$

4. draw section





5. look for intersection for summing moments (B or G)

6. write equilibrium equations

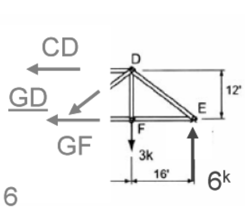
$$\sum M_B = HG \cdot 12^{ft} - 5^k \cdot 16^{ft} - 4^k \cdot 12^{ft} = 0 \qquad HG = \frac{128^{k-ft}}{12^{ft}} = \boxed{10.67^k}$$

$$\sum M_G = 4^k \cdot 16^{ft} - 5^k \cdot 32^{ft} - 4^k \cdot 12^{ft} - BC \cdot 12^{ft} = 0$$

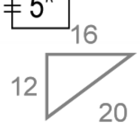
$$BC = \frac{144^{k-ft}}{-12^{ft}} = \boxed{-12^k}$$

$$(\sum F_y = 5^k - 4^k - BG\left(\frac{12}{20}\right) = 0 \qquad BG = -1.67^k\,)$$

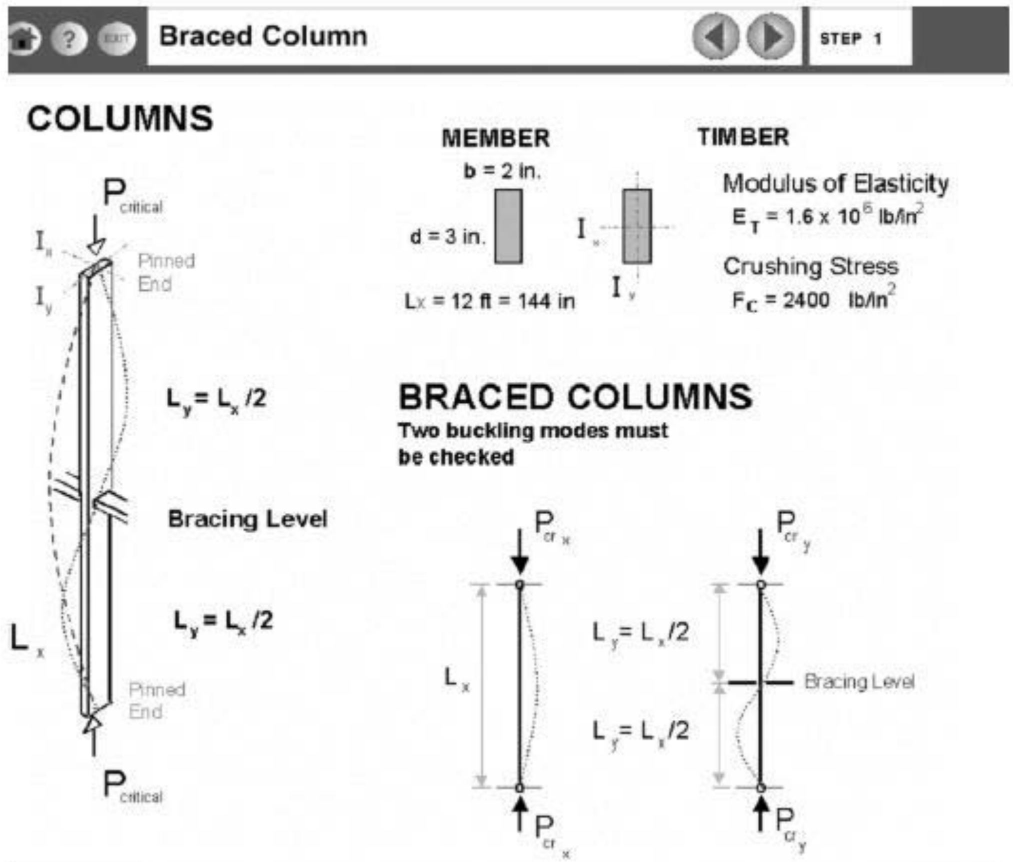
7. repeat with other section

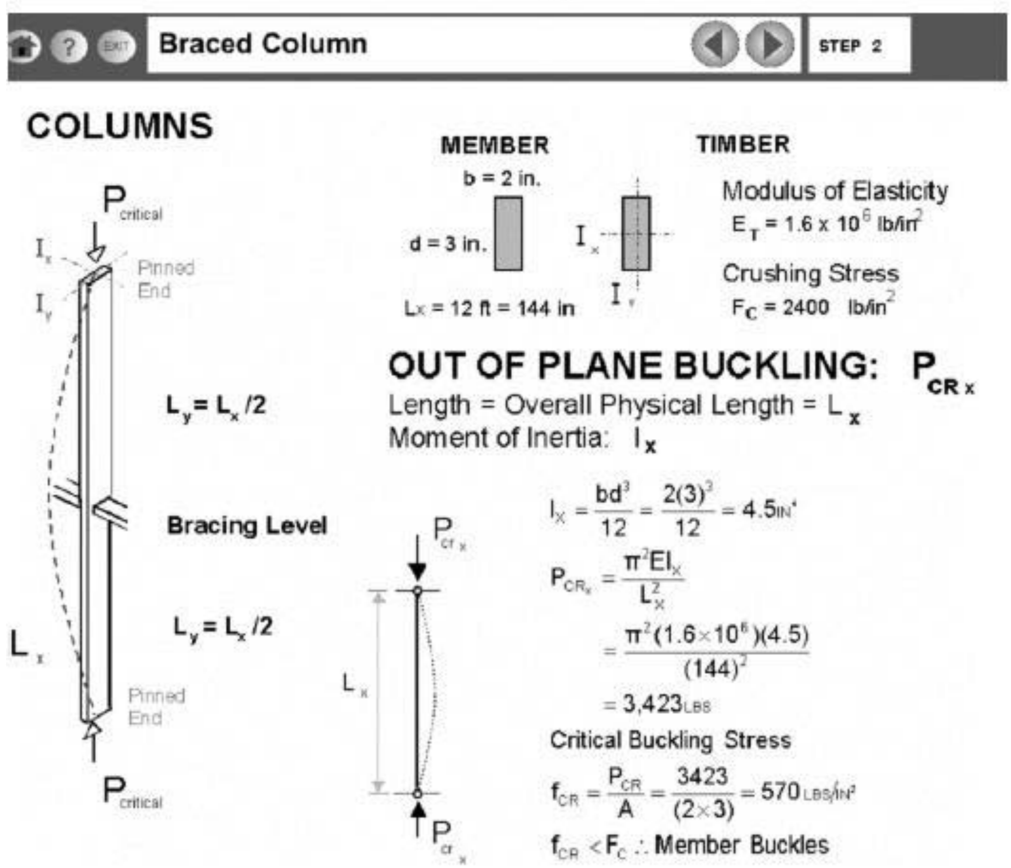


$$\sum F_y = 6^k - 3^k - GD\left(\frac{12}{20}\right) = 0 \qquad GD = \boxed{5^k}$$

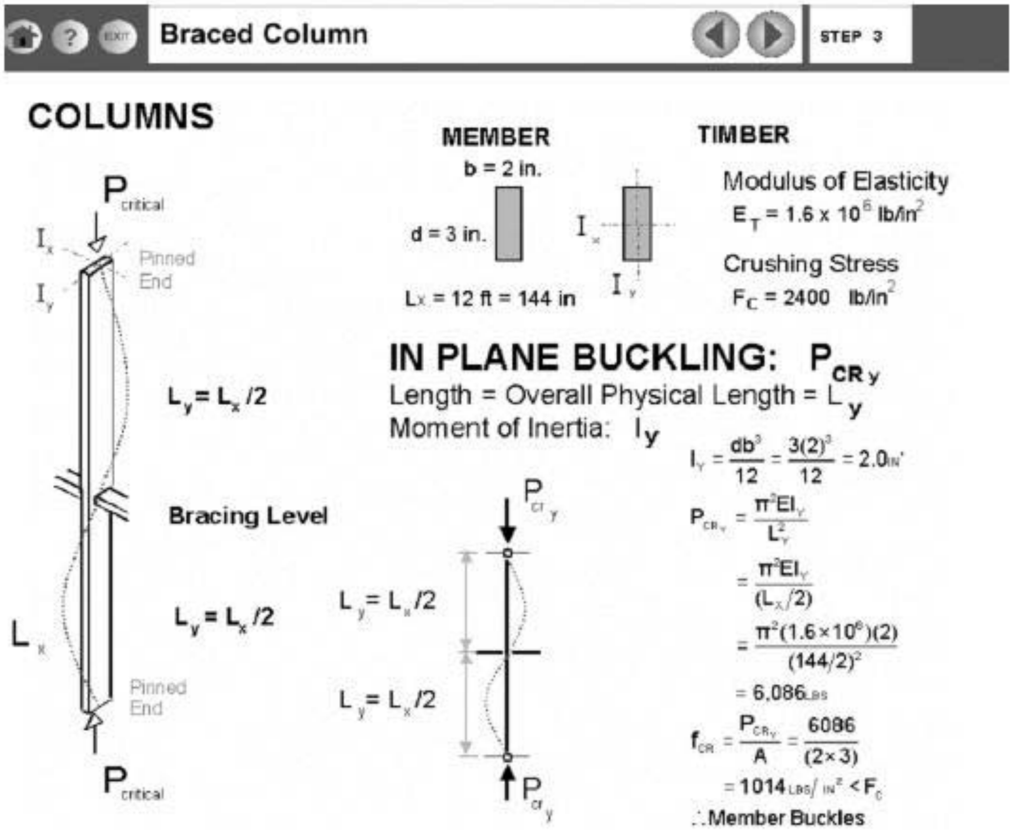


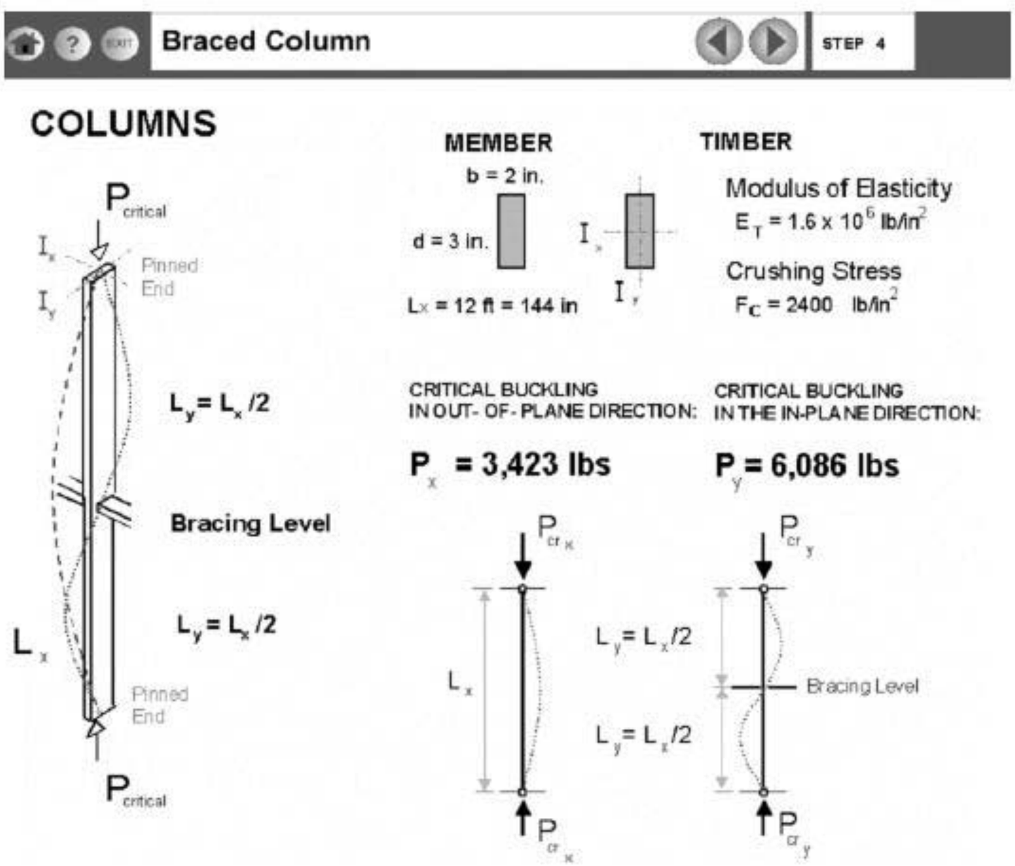
Example 3 **From eStructures v1.1, Schodek and Pollalis, 2000 Harvard College**



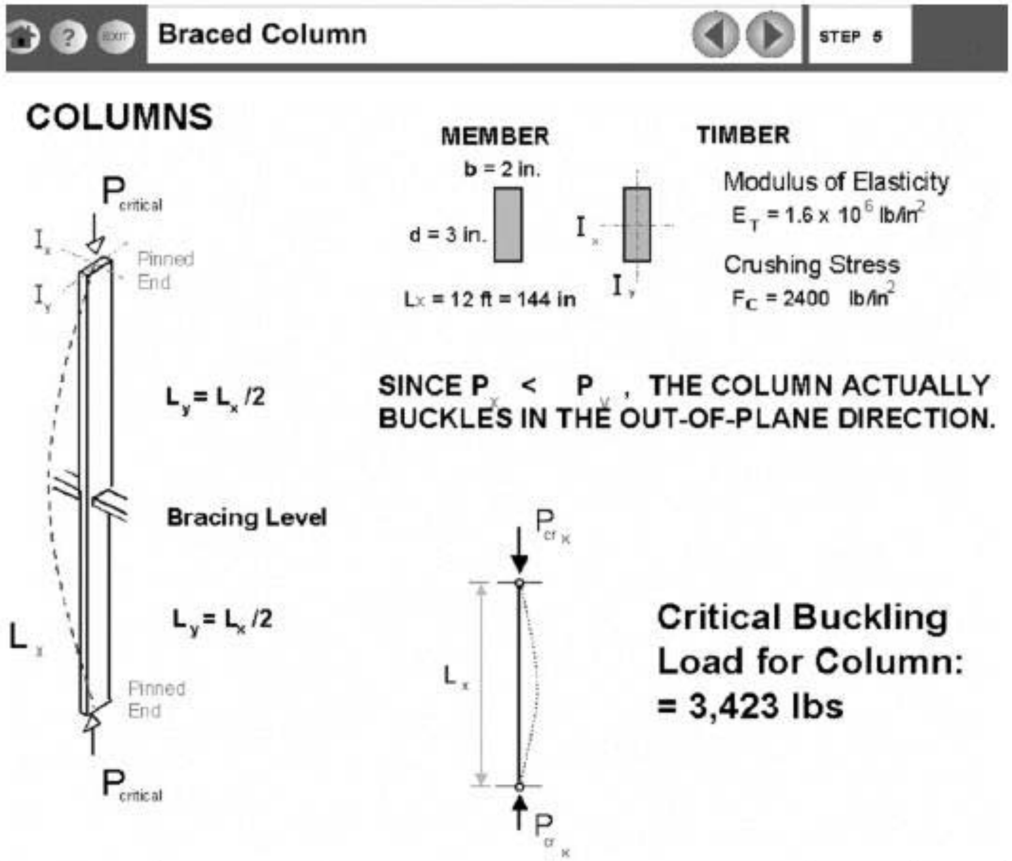


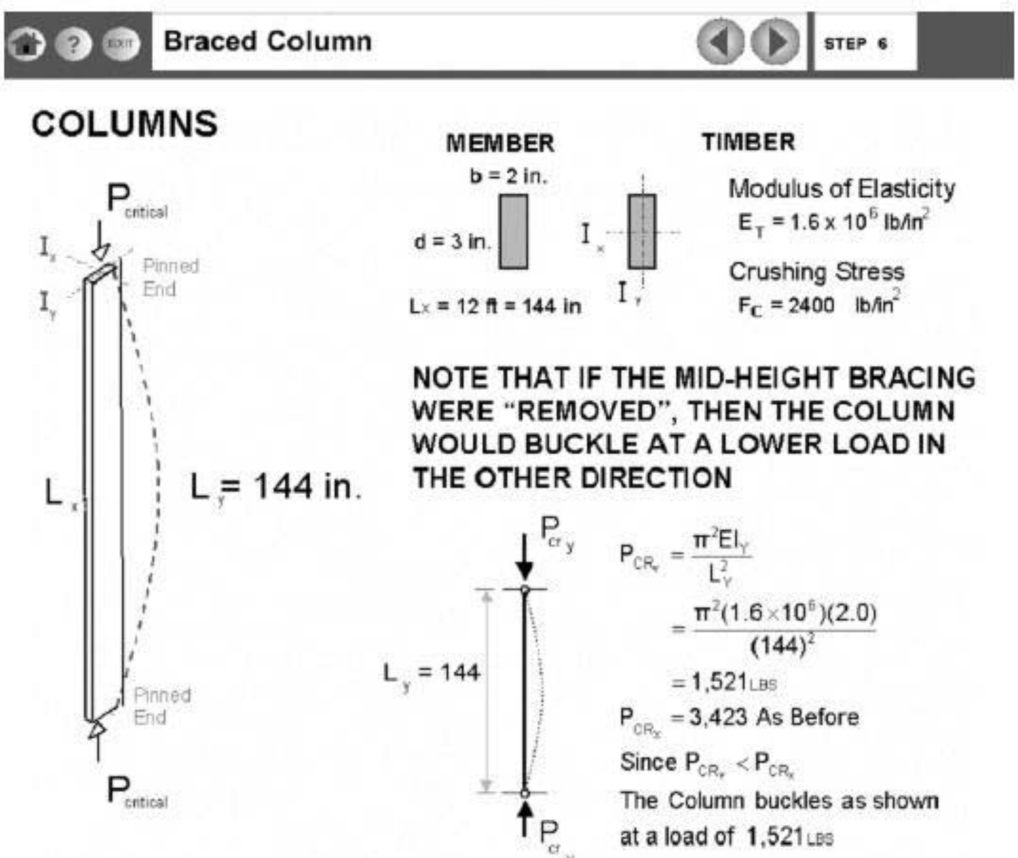
Example 3 (continued)





Example 3 (continued)





Technical Notes on Brick Construction

31 REVISED

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Brick Industry Association 11490 Commerce Park Drive, Reston, Virginia 20191

BRICK MASONRY ARCHES INTRODUCTION

Abstract: The masonry arch is one of the oldest structural elements. Brick masonry arches have been used for hundreds of years. This *Technical Notes* is an introduction to brick masonry arches. Many of the different types of brick masonry arches are discussed and a glossary of arch terms is provided. Material selection, proper construction methods, detailing and arch construction recommendations are discussed to ensure proper structural support, durability and weather resistance of the brick masonry arch.

Key Words: arch, brick, reinforced, unreinforced.

INTRODUCTION

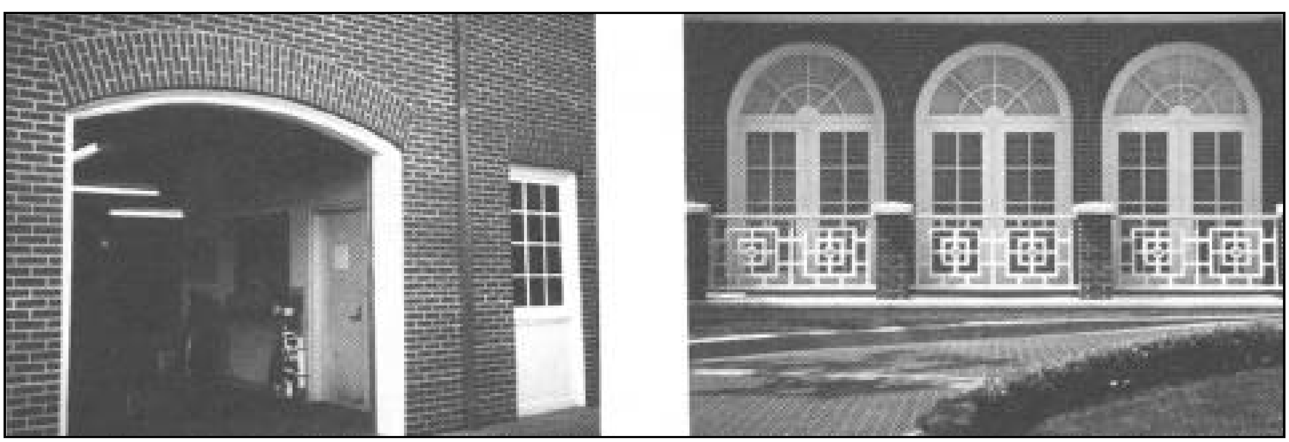
In the latter part of the 19th century, an arch was discovered in the ruins of Babylonia. Archeologists estimate that the arch was constructed about the year 1400 B.C. Built of well-baked, cigar-shaped brick and laid with clay mortar, this arch is probably the oldest known to man. The Chinese, Egyptians and others also made use of the arch before the Christian era. Later, more elaborate arches, vaults and domes with complicated forms and intersections were constructed by Roman builders during the Middle Ages.

The brick arch is the consummate example of form following function. Its aesthetic appeal lies in the variety of forms which can be used to express unity, balance, proportion, scale and character. Its structural advantage results from the fact that under uniform load, the induced stresses are principally compressive. Because brick masonry has greater resistance to compression than tension, the masonry arch is frequently the most efficient structural element to span openings.

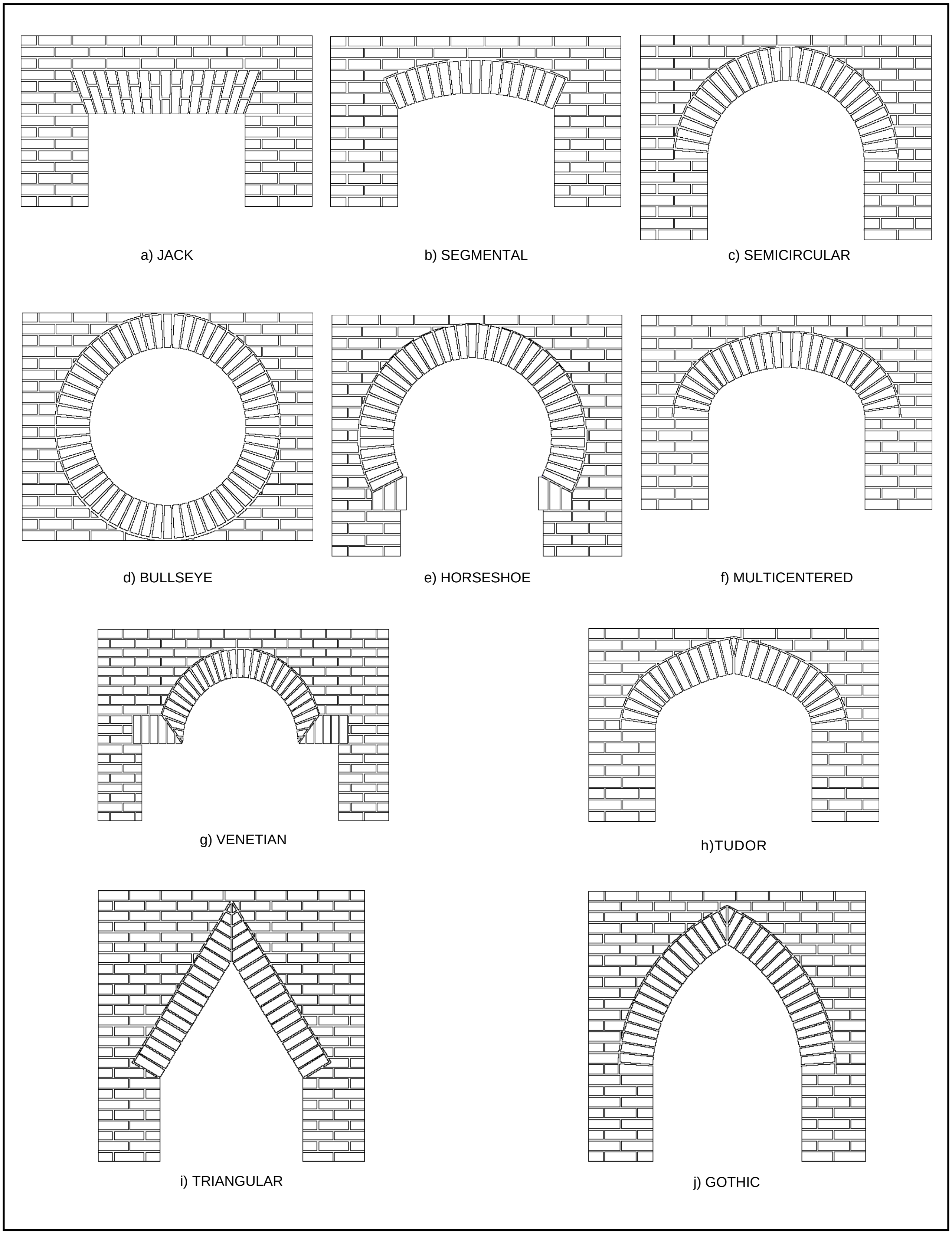
This *Technical Notes* addresses the detailing and construction of brick masonry arches. The common types of brick masonry arches are presented, along with proper arch terminology. Methods of selecting the type and configuration of brick masonry arches most appropriate for the application are discussed. Proper material selection and construction methods are recommended. Other *Technical Notes* in this series discuss the structural design of brick masonry arches and lintels.

ARCH TYPES AND TERMINOLOGY

Many arch forms have been developed during the centuries of use, ranging from the jack arch through the circular, elliptical and parabolic to the Gothic arch. Fig-



Structural Brick Arches
FIG. 1



Arch Types
FIG. 2

ure 1 depicts examples of structural masonry arches used in contemporary construction. An arch is normally classified by the curve of its intrados and by its function, shape or architectural style. Figure 2 illustrates some of the many different brick masonry arch types. Jack, segmental, semicircular and multicentered arches are the most common types used for building arches. For very long spans and for bridges, semicircular arches are often used because of their structural efficiency.

Mainly due to their variety of components and elements, arches have developed their own set of terminology. Following is a glossary of arch terminology. Figure 3 illustrates many of the terms defined in this glossary. *Technical Notes* in this series will use this terminology.

Abutment: The masonry or combination of masonry and other structural members which support one end of the arch at the skewback.
Arch: A form of construction in which masonry units span an opening by transferring vertical loads laterally to adjacent voussoirs and, thus, to the abutments. Some common arch types are as follows:

Blind - An arch whose opening is filled with masonry.

Bullseye - An arch whose intrados is a full circle. Also known as a *Circular* arch.

Elliptical - An arch with two centers and continually changing radii.

Fixed - An arch whose skewback is fixed in position and inclination. Masonry arches are fixed arches by nature of their construction.

Gauged - An arch formed with tapered voussoirs and thin mortar joints.

Gothic - An arch with relatively large rise-to-span ratio, whose sides consist of arcs of circles, the centers of which are at the level of the spring line. Also referred to as a *Drop*, *Equilateral* or *Lancet* arch, depending upon whether the spacings of the centers are respectively less than, equal to or more than the clear span.

Horseshoe - An arch whose intrados is greater than a semicircle and less than a full circle. Also known as an *Arabic* or *Moorish* arch.

Jack - A flat arch with zero or little rise.

Multicentered - An arch whose curve consists of several arcs of circles which are normally tangent at their intersections.

Relieving - An arch built over a lintel, jack arch or smaller arch to divert loads, thus relieving the lower arch or lintel from excessive loading. Also known as a *Discharging* or *Safety* arch.

Segmental - An arch whose intrados is circular but less than a semicircle.

Semicircular - An arch whose intrados is a semicircle (half circle).

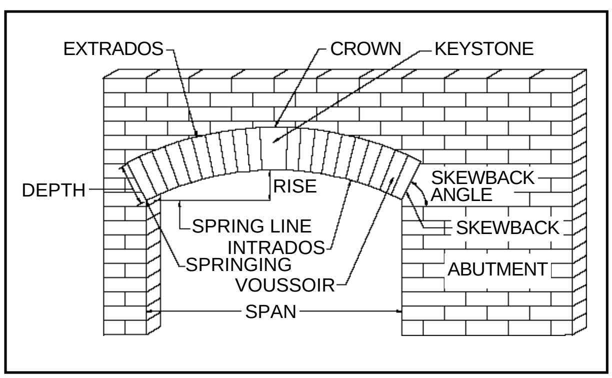
Slanted - A flat arch which is constructed with a keystone whose sides are sloped at the same angle as the skewback and uniform width brick and mortar joints.

Triangular - An arch formed by two straight, inclined sides.

Tudor - A pointed, four-centered arch of medium rise-to-span ratio whose four centers are all beneath the extrados of the arch.

Venetian - An arch formed by a combination of jack arch at the ends and semicircular arch at the middle. Also known as a *Queen Anne* arch.

Camber: The relatively small rise of a jack arch.
Centering: Temporary shoring used to support an arch until the arch becomes self-supporting.
Crown: The apex of the arch's extrados. In symmetrical arches, the crown is at the midspan.
Depth: The dimension of the arch at the skewback which is perpendicular to the arch axis, except that the depth of a jack arch is taken to be the vertical dimension of the arch at the springing.
Extrados: The curve which bounds the upper edge of the arch.
Intrados: The curve which bounds the lower edge of the arch. The distinction between soffit and intrados is that the intrados is a line, while the soffit is a surface.
Keystone: The voussoir located at the crown of the arch. Also called the *key*.
Label Course: A ring of projecting brickwork that forms the extrados of the arch.
Rise: The maximum height of the arch soffit above the level of its spring line.
Skewback: The surface on which the arch joins the supporting abutment.
Skewback Angle: The angle made by the skewback from horizontal.
Soffit: The surface of an arch or vault at the intrados.
Span: The horizontal clear dimension between abutments.
Spandrel: The masonry contained between a horizontal line drawn through the crown and a vertical line drawn through the upper most point of the skewback.
Springing: The point where the skewback intersects the intrados.
Springer: The first voussoir from a skewback.
Spring Line: A horizontal line which intersects the springing.
Voussoir: One masonry unit of an arch.



Arch Terms
FIG. 3

STRUCTURAL FUNCTION OF ARCHES

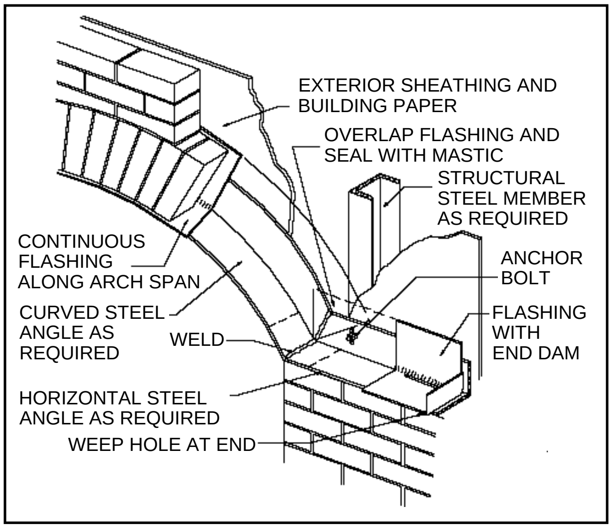
The brick masonry arch has been used to span openings of considerable length in many different applications. Structural efficiency is attributed to the curvature of the arch, which transfers vertical loads laterally along the arch to the abutments at each end. The transfer of vertical forces gives rise to both horizontal and vertical reactions at the abutments. The curvature of the arch and the restraint of the arch by the abutments cause a combination of flexural stress and axial compression. The arch depth, rise and configuration can be manipulated to keep stresses primarily compressive. Brick masonry is very strong in compression, so brick masonry arches can support considerable load.

Historically, arches have been constructed with unreinforced masonry. Most brick masonry arches continue to be built with unreinforced masonry. The structural design of unreinforced brick masonry arches is discussed in *Technical Notes* 31A. Very long span arches and arches with a small rise may require steel reinforcement to resist tensile stresses. Also, reduction in abutment size and arch thickness for economy may require incorporation of reinforcement for adequate load resistance. Refer to the *Technical Notes* 17 Series for more information on reinforced brick masonry. Elaborate and intricate arches are sometimes prefabricated to avoid the complexity of on-site shoring. Most prefabricated brick masonry arches are reinforced. Prefabricated arches are built off site and transported to the job or built at the site. Cranes are often used to lift the arch into place in the wall. Such fabrication, handling and transportation should be considered in the structural design of the arch. Refer to *Technical Notes* 40 for a discussion of prefabricated brick masonry.

If an unreinforced or reinforced brick masonry arch is not structurally adequate, the arch will require support. Typically, this support is provided by a steel angle. This is the most common means of supporting brick masonry arches in modern construction. The steel angle is bent to the curvature of the intrados of the arch. Curved sections of steel angle are welded to horizontal steel angles to form a continuous support. The angle either bears on the brickwork abutments or is attached to a structural member behind the wall. One example is shown in Fig. 4. When an arch is supported by a steel angle, the angle is designed to support the entire weight of brick masonry loading the arch, and the structural resistance of the arch is neglected. Consult *Technical Notes* 31B Revised for a discussion of the structural design of steel angle lintels.

WEATHER RESISTANCE

Water penetration resistance is a primary concern in most applications of the building arch. In the past, the mass of a multi-wythe brick masonry arch was sufficient to resist water penetration. Today, thinner wall sections are used to minimize material use for economy and efficiency. Still, the arch must provide an effective weather resistant facade. Some arch applications do not require provisions for water penetration and insulation. For example, arch arcades and arches supported by porch columns typically do not conceal a direct path for water migration to the interior of the building they serve and may not require insulation. If this is the case, provisions for weather resistance need not be included in the arch design and detailing.

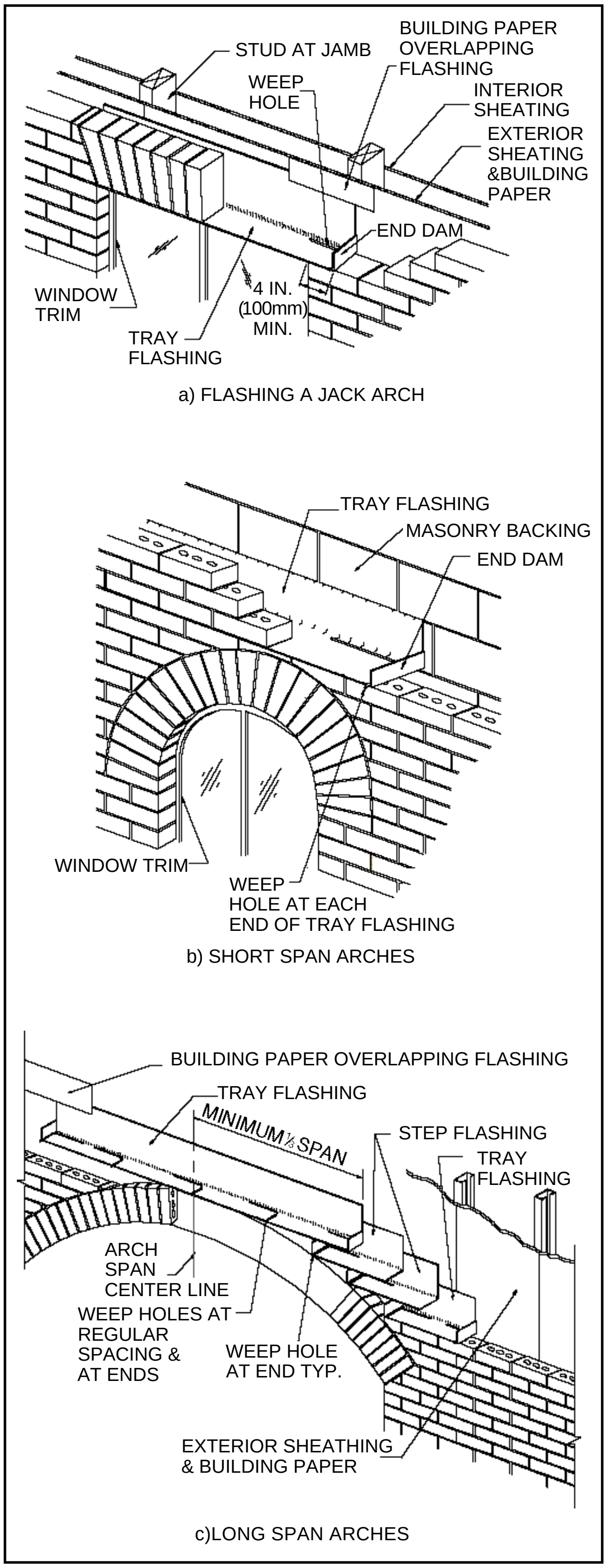


Arch Supported by Curved Steel Angle
FIG. 4

Preventing water entry at an arch in an exterior building wall is just as important as at any other wall opening. Water penetration resistance can be provided by using a barrier wall system or a drainage wall system. Refer to *Technical Notes* 7 *Revised* for definitions and discussion of barrier and drainage wall systems. A drainage wall system, such as a brick veneer or cavity wall, is the most common brick masonry wall system used today. For either wall system, the arch should be flashed, with weep holes provided above all flashing locations.

Flashing and Weep Holes

Installation of flashing and weep holes around an arch can be difficult. Installation of flashing is easiest with jack arches because they are flat or nearly flat. Flashing should be installed below the arch and above the window framing or steel angle lintel. Flashing should extend a minimum of 4 in. (100 mm) past the wall opening at either end and should be turned up to form end dams. This is often termed tray flashing. Weep holes should be provided at both ends of the flashing and should be placed at a maximum spacing of 24 in. (600 mm) on centers along the arch span, or 16 in. (400 mm) if rope wicks are used. An example of flashing a jack arch in this manner is shown in Fig. 5a. Attachment of the flashing to the backing and formation of end dams should follow standard procedures. If the arch is constructed with reinforced brick masonry, flashing and weep holes can be placed in the first masonry course above the arch.



Flashing Arches
FIG. 5

Installation of flashing with other arch types, such as segmental and semicircular arches, can be more difficult. This is because most rigid flashing materials are hard to bend around an arch with tight curvature. If the arch span is less than about 3 ft (0.9 m), one section of tray flashing can be placed in the first horizontal mortar joint above the keystone, as illustrated in Fig. 5b. For arch spans greater than 3 ft (0.9 m), flashing can be bent along the curve of the arch with overlapping sections, as illustrated in Fig. 4. Alternately, a combination of stepped and tray flashing can be used, as shown in Fig. 5c. To form a step, the end nearest the arch should be turned up to form an end dam, while the opposite end is laid flat. A minimum of No. 15 building paper or equivalent moisture resistant protection should be installed on the exterior face of the backing over the full height of the arch and abutments. The building paper or equivalent should overlap the arch flashing.

The design of a structural masonry arch should include consideration of the effect of flashing on the strength of the arch. Flashing acts as a bond break. If flashing is installed above the arch, the loading on the arch will likely be increased, and the structural resistance of the arch will be reduced. Installation of flashing at the abutments will affect their structural resistance and should also be considered. Consult *Technical Notes* 31A for a more extensive discussion of arch loads and structural resistance of brick masonry arches.

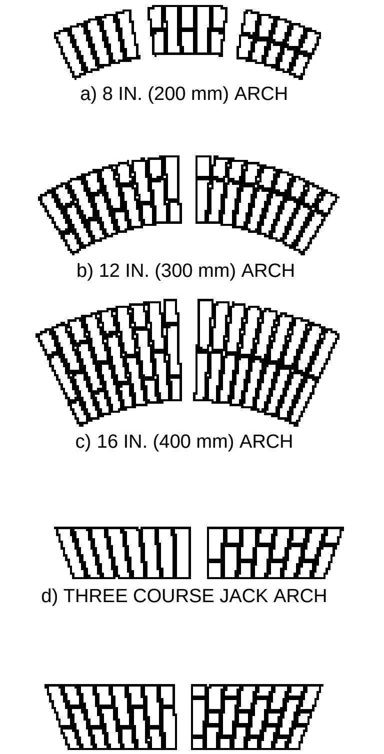
DETAILING CONSIDERATIONS

The brick masonry arch should serve its structural purpose and also provide an attractive architectural element to complement its surrounding structure. Careful consideration should be given to the options available for the arch, soffit and skewback. Proper configuration of the abutments and location of expansion joints should be considered for any arch design.

Arch

Arches can be configured in a variety of arch depths, brick sizes and shapes and bonding patterns. The arch is normally composed of an odd number of units for aesthetic purposes. Some of the more common arch configurations are illustrated in Fig. 6. Arch voussoirs are typically laid in radial orientation and are most often of similar size and color to the surrounding brickwork. However, the arch can be formed with brick which are thinner or wider than the surrounding brickwork and of a different color for variation. Another variation is to project or recess rings of multiple-ring arches to provide shadow lines or a label course.

Brick masonry arches are constructed with two different types of units. The first is tapered or wedge-shaped brick. These brick are tapered in the appropriate manner to obtain mortar joints of uniform thickness along the arch depth. The second is uncut, rectangular brick. When rectangular brick are used, the mortar joints are tapered to obtain the desired arch curvature. In some cases, a combination of these is used. For example, a slanted arch is formed with a tapered keystone and rectangular brick. This arch is similar to a jack



Typical Arch Configurations
FIG. 6

arch, but can be more economical because it requires only one special-shaped brick.

Selection of tapered or rectangular brick can be determined by the arch type, arch dimensions and by the appearance desired. Some arch types require more unique shapes and sizes of brick if uniform mortar joint thickness is desired. For example, the brick in a traditional jack arch or elliptical arch are all different sizes and shapes from the abutment to the keystone. Conversely, the voussoirs of a semicircular arch are all the same size and shape. Arch types with many different brick shapes and sizes should be special ordered from the brickmanufacturer rather than cut in the field.

The arch span should also be considered when selecting the arch brick. For short arch spans, use of tapered brick is recommended to avoid excessively wide mortar joints at the extrados. Larger span arches require less taper of the voussoirs and, consequently, can be formed with rectangular brick and tapered mortar joints. The thickness of mortar joints between arch brick should be a maximum of ¾ in. (19 mm) and a minimum of ⅛ in. (3 mm). When using mortar joints thinner than ¼ in. (6 mm), consideration should be given to the use of very uniform brick that meet the dimensional tolerance limits of ASTM C 216, Type FBX, or the use of gauged brickwork. Refer to Table 1 for determination of the minimum segmental and semicircular arch radii permitted for rectangular brick and tapered mortar joints. Typically, the use of tapered brick and uniform thickness mortar joints will be more aesthetically appealing.

TABLE 1

Minimum Radius for Uncut Arch Brick[1,2]

Nominal Face Dimensions of Arch Brick, in. (height by width)	**Minimum Permissible Radius of Arch to Intrados, ft**
4 x 2⅔	3.3
8 x 2⅔	6.7
12 x 2⅔	10.0
16 x 2⅔	13.3
4 x 3⅕	4.0
8 x 3⅕	8.0
12 x 3⅕	12.0
16 x 3⅕	16.0
4 x 4	5.2
8 x 4	10.3
12 x 4	15.5
16 x 4	20.7

[1]Based on ¼ in. (6 mm) mortar joint width at the intrados and ½ in. (13 mm) mortar joint width at the extrados. If the mortar joint thickness at theextra dos is ¾ in. (19 mm), divide minimum radius value by 2.

[2]1 in. = 25.4 mm; 1 ft = 0.3 m

Depth. The arch depth will depend upon the size and orientation of the brick used to form the arch. Typically, the arch depth is a multiple of the brick's width. For structural arches, a minimum arch depth is determined from the structural requirements. If the arch is supported by a lintel, any arch depth may be used.

The depth of the arch should also be detailed based on the scale of the arch in relation to the scale of the building and surrounding brickwork. To provide proper visual balance and scale, the arch depth should increase with increasing arch span. Because aesthetics of an arch are subjective, there are no hard rules for this. However, the following rules-of-thumb will help provide an arch with proper scale. For segmental and semicircular arches, the arch depth should equal or exceed 1 in. (25 mm) for every foot (300 mm) of arch span or 4 in. (100 mm), whichever is greater. For jack arches, the arch depth should equal or exceed 4 in. (100 mm) plus 1 in. (25 mm) for every foot (300 mm) of arch span or 8 in. (200 mm), whichever is greater. For example, the minimum arch depth for an 8 ft (2.4 m) span should be 8 in. (200 mm) for segmental arches and 12 in. (300 mm) for jack arches.

The depth of jack arches will also be a function of the coursing of the surrounding brick masonry. The springing and the extrados of the jack arch should coincide with horizontal mortar joints in the surrounding brick masonry. Typically, the depth of a jack arch will equal the height of 3, 4 or 5 courses of the surrounding brickwork, depending upon the course height.

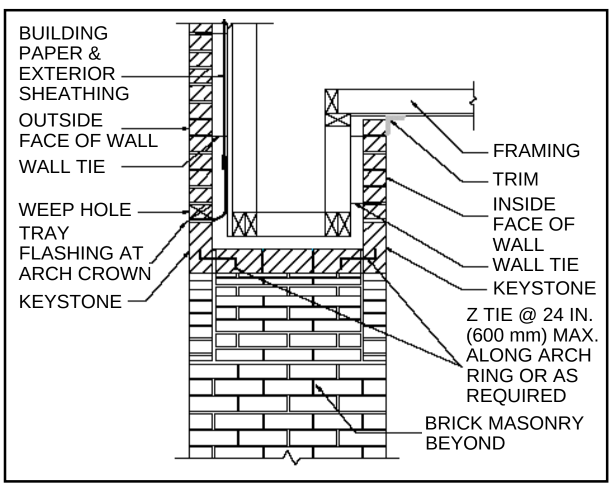
Keystone. The keystone may be a single brick, multiple brick, stone, precast concrete or terra cotta. Avoid using a keystone which is much taller than the adjacent voussoirs. A rule-of-thumb is that the keystone should not extend above adjacent arch brick by more than one-third the arch depth. When a keystone is used that is larger than adjacent arch brick or formed with different material, one option is to use springers that match the keystone.

The use of a large keystone has its basis in both purpose and visual effect. With most arch types, the likely location of the first crack when the arch fails is at the mortar joint nearest to the midspan of the arch. Use of a large keystone at this point moves the first mortar joint further from the midspan and increases the resistance to cracking at this point. Aesthetically, a large keystone adds variation of scale and can introduce other masonry materials in the facade for additional color and texture.

If the keystone is formed with more than one masonry unit, avoid placing the smaller unit at the bottom. Such units are more likely to slip when the arch settles under load. Also, it is preferred to have the arch crown (the top of the keystone) coincident with a horizontal mortar joint in the surrounding brickwork to give the arch a neater appearance.

Soffit

A brick masonry soffit is one attractive feature of a structural brick masonry arch. Many bonding patterns and arrangements can be used to form the arch soffit. Deep soffits are common on building arcades or arched entranceways. In this case, it is common to form a U-shaped wall section, as illustrated in Fig. 7. The arches on either wall face should be bonded to the brick masonry forming the soffit. Bonding pattern or metal ties should be used to tie the brick masonry forming the soffit together structurally and to tie the arches on either wall face to the soffit. If metal ties are used to bond the masonry, corrosion resistant box or Z metal wire ties should be placed along the arch span at a maximum spacing of 24 in. (600 mm) on center.

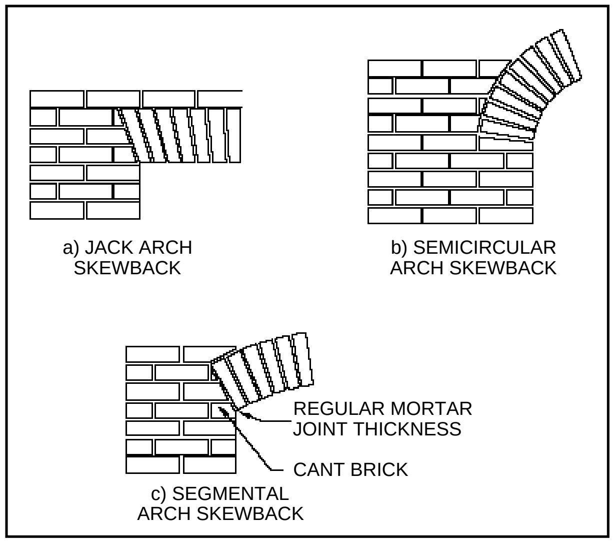


Structural Arch Soffit Option
FIG. 7

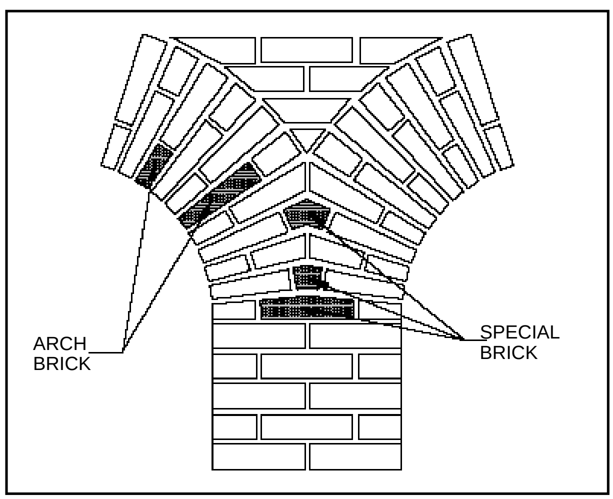
Structural resistance of the arch should be evaluated at sections through the soffit, the exterior wall face and the interior wall face. Deeper soffits may require an increase in arch depth. If the arch is structural, connection of the brick masonry forming the soffit to interior framing members with wall ties or connectors may not be required.

Skewback

For flat arches and arch types that have horizontal skewbacks, such as jack and semicircular arches, respectively, the most desirable spring line location is coincident with a bed joint in the abutment. For other arch types, it is preferred to have the spring line pass about midway through a brick course in the abutment, as illustrated in Fig. 8, to avoid a thick mortar joint at the springing. The brick in the abutment at the spring-



Skewback Options
FIG. 8



Option For Intersecting Arches
FIG. 9

ing should be cut or be a special cant-shaped brick. This allows vertical alignment with the brick beneath, producing more accurate alignment of the arch.

When two arches are adjacent, such as with a two-bay garage or building arcades, intersection of the arches may occur at the skewback. Attention should be given to proper bonding of the arches for both visual appeal and structural bonding. Creation of a vertical line between arches should be avoided. Rather, special shape brick should be used to mesh the two arches properly. One example is illustrated in Fig. 9.

Abutments

An arch abutment can be a column, wall or combination of wall and shelf angle. Failure of an abutment occurs from excessive lateral movement of the abutment or exceeding the flexural, compressive or shear strength of the abutment. Lateral movement of the abutment is due to the horizontal thrust of the arch. Thrust develops in all arches and the thrust force is greater for flatter arches. The thrust should be resisted so that lateral movement of the abutment does not cause failure in the arch. If the abutment is formed by a combination of brickwork and a non-masonry structural member, rigidity of the non-masonry structural member and rigidity of the ties are very important. Adjustable ties or single or double wire ties are recommended. Corrugated ties should not be used in this application because they do not provide adequate axial stiffness. Consult *Technical Notes* 31A for further discussion of abutment and tie stiffness requirements.

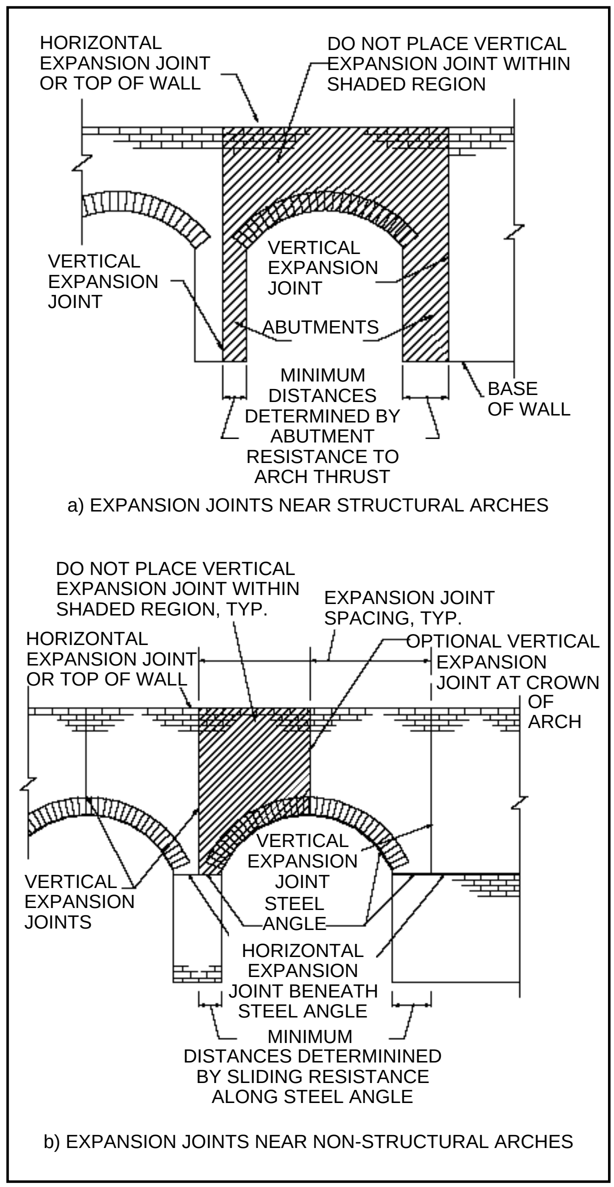
Lateral Bracing

In addition to gravity loads, out-of-plane loads should be considered when designing a masonry arch. The arch should have adequate resistance to out-of-plane loads or lateral bracing should be provided. In veneer construction, lateral bracing is provided by the backing through the use of wall ties. Arches which are not laterally braced may require increased masonry thickness or reinforcement to carry loads perpendicular to the arch plane in addition to vertical loads.

Expansion Joints

Thermal and moisture movements of brick masonry are controlled by the use of expansion joints. Expansion joints avoid cracking of the brickwork and also reduce the size of wall sections. Reduction of wall size has a very important effect upon the performance of structural brick masonry arches. The state of stress in a structural brick arch and the surrounding masonry is very sensitive to the relative movements of the abutments. If an inadequate number of expansion joints are provided, the differential movement of abutments can cause cracking and downward displacement of brick in the masonry arch and surrounding masonry. Proper size and spacing of expansion joints is discussed in *Technical Notes* 18A Revised.

If the arch is structural, care should be taken not to affect the integrity of the arch by detailing expansion joints too close to the arch and its abutments. Vertical expansion joints should not be placed in the masonry directly above a structural arch. This region of masonry is in compression, so an expansion joint will cause displacement when centering is removed and possible collapse of the arch and surrounding brickwork. In addition, vertical expansion joints should not be placed in close proximity to the springing. The expansion joint will reduce the effective width of the abutment and its ability to resist horizontal thrust from the arch. If the arch is non-structural, placement of expansion joints may be at the arch crown and also at a sufficient distance away from the springing to avoid sliding. While permitted, placement of an expansion joint at the arch crown is not preferred because it disrupts ones tradi-



Expansion Joints Near Arches
FIG. 10

tional view of the arch as a structural element. Refer to Fig. 10 for suggested expansion joint locations for structural and non-structural arches.

Detailing of expansion joints can be difficult with very long span arches or runs of multiple arches along an arcade. Structural analysis of the arch should consider the location of expansion joints. For the particular case of multiple arches closely spaced, vertical expansion joints should be detailed at a sufficient distance away from the end arches so that horizontal arch thrusts are adequately resisted by the abutments to avoid overturning of the abutments. For long arcades, expansion joints should also be placed along the centerline of abutments between arches when necessary. In this case, horizontal thrusts from adjacent arches will not be counteracting, so the effective abutment length should be halved and overturning of each half of the abutment should be checked. Refer to *Technical Notes* 31A for further discussion of abutment design for adequate stiffness.

MATERIAL SELECTION

To provide a weather resistant barrier and maintain its structural resistance, the arch must be constructed with durable materials. The strength of an arch depends upon the compressive strength and the flexural tensile strength of the masonry. Selection of brick and mortar should consider these properties.

Brick

Solid or hollow clay brick may be used to form the arch and the surrounding brickwork. Solid brick should comply with the requirements of ASTM C 216 Specification for Facing Brick. Hollow brick should comply with the requirements of ASTM C 652 Specification for Hollow Brick. Refer to *Technical Notes* 9 Series for a discussion of brick selection and classification. The compressive strength of masonry is related to the compressive strength of the brick, the mortar type and the grout strength. For structural arches, brick should be selected with consideration of the required compressive strength of masonry. Typically, compressive strength of the brick masonry will not limit the design of the arch.

Tapered voussoirs can be cut from rectangular units at the job site or special ordered from the brick manufacturer. Before specifying manufactured special arch shapes, the designer should determine the availability of special shapes for the arch type and brick color and texture desired. Many brick manufacturers produce tapered arch brick for the more common arch types as part of their regular stock of special shapes. Be sure to contact the manufacturer as early as possible if special shapes are needed. In many instances, production of the special shapes may require a color matching process and adequate lead time for the manufacturer.

Mortar

Mortar used to construct brick masonry arches should meet the requirements of ASTM C 270 Standard Specification for Masonry Mortar. Consult *Technical Notes* 8 Series for a discussion of mortar types and kinds for brick masonry. For structural arches, the flexural tensile strength of the masonry should be considered when selecting the mortar. The flexural tensile strength of the masonry will affect the load resistance of the arch and the abutments.

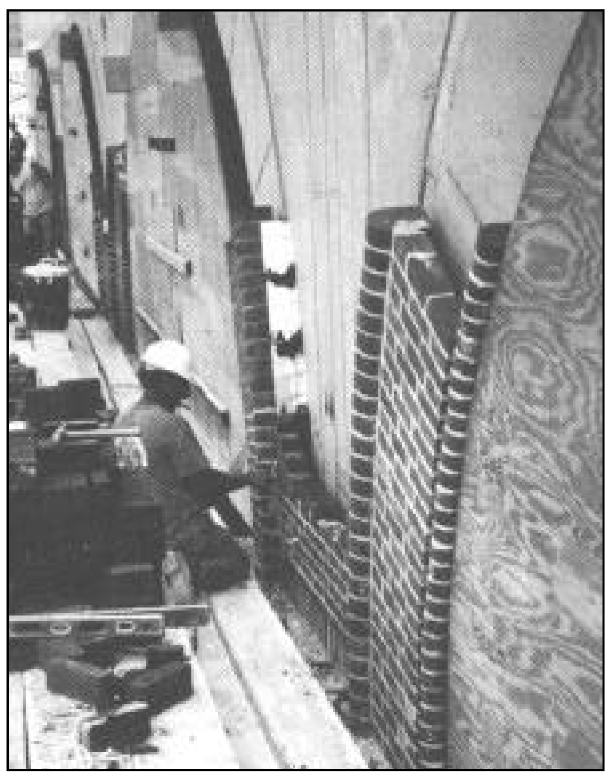
CONSTRUCTION AND WORKMANSHIP

The proper performance of a brick masonry arch depends upon proper methods of construction and attention to workmanship. Layout of the arch prior to construction will help avoid poor spacing of voussoirs, which results in thicker mortar joints and unsymmetrical arches. Some arch applications, such as barrel vaults and domes, can be entirely self-supporting, even during construction. However, most applications of the masonry arch used today require proper shoring and bracing.

Centering

Both structural and non-structural arches should be properly supported throughout construction. Brick masonry arches are constructed with the aid of temporary shoring, termed centering, or permanent supports, such as a structural steel angle.

Centering is used to carry the weight of a brick masonry arch and the loads being supported by the arch until the arch itself has gained sufficient strength. The term "centering" is used because the shoring is marked for proper positioning of the brick forming the arch.



Centering
FIG. 11

Centering is typically provided by wood construction. An example of centering for an arch is shown in Fig. 11. Careful construction of the centering will ensure a more pleasing arch appearance and avoid layout problems, such as an uneven number of brick to either side of the keystone.

Immediately after placement of the keystone, very slight downward displacement of the centering, termed easing, can be performed to cause the arch voussoirs to press against one another and compress the mortar joints between them. Easing helps to avoid separation cracks in the arch. In no case should centering be removed until it is certain that the masonry is capable of carrying all imposed loads. Premature removal of the centering may result in collapse of the arch.

Centering should remain in place for at least seven days after construction of the arch. Longer curing periods may be required when the arch is constructed in cold weather conditions and when required for structural reasons. The arch loading and the structural resistance of the arch will depend upon the amount of brickwork surrounding the arch, particularly the brick masonry within spandrel areas. Appropriate time of removal of centering for a structural arch should be determined with consideration of the assumptions made in the structural analysis of the arch. It may be necessary to wait until the brickwork above the arch has also cured before removing the centering.

Workmanship

All mortar joints should be completely filled, especially in a structural member such as an arch. If hollow brick are used to form the arch, it is very important that all face shells and end webs are completely filled with mortar. Brick masonry arches are sometimes constructed with the units laid in a soldier orientation. It may be difficult to lay units in a soldier position and also obtain completely filled mortar joints. This is especially true for an arch with tapered mortar joints. In such cases, the use of two or more rings of arch brick laid in rowlock orientation can help ensure full mortar joints.

SUMMARY

This *Technical Notes* is an introduction to brick masonry arches. A glossary of arch terms has been provided. Many different types of brick masonry arches are described and illustrated. Proper detailing of brick masonry arches for appearance, structural support and weather resistance is discussed. Material selection and proper construction practices are explained. Other *Technical Notes* in this Series discuss the structural design of arches.

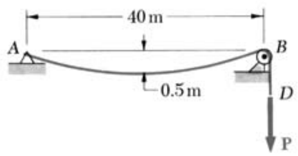
The information and suggestions contained in this *Technical Notes* are based on the available data and the experience of the engineering staff of the Brick Institute of America. The information contained herein must be used in conjunction with good technical judgment and a basic understanding of the properties of brick masonry. Final decisions on the use of the information contained in this *Technical Notes* are not within the purview of the Brick Institute of America and must rest with the project architect, engineer and owner.

REFERENCES

1. *Brickwork Arch Detailing*, Ibstock Building Products, Butterworth & Co. (Publishers) Ltd., London, England, 1989, 114 pp.
2. Lynch, G., *Gauged Brickwork, A Technical Handbook*, Gower Publishing Company, Aldershot, Hants, England, 1990, 115 pp.
3. Trimble, B.E., and Borchelt, J.G., "Jack Arches in Masonry Construction," *The Construction Specifier*, Construction Specifications Institute, Alexandria, VA, January 1991, pp. 62-65.

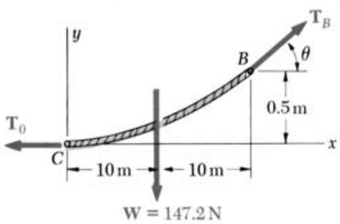
Examples:
Cables and Arches

Example 1



SAMPLE PROBLEM 7.9

A light cable is attached to a support at A, passes over a small pulley at B, and supports a load **P**. Knowing that the sag of the cable is 0.5 m and that the mass per unit length of the cable is 0.75 kg/m, determine (*a*) the magnitude of the load **P**, (*b*) the slope of the cable at B, and (*c*) the total length of the cable from A to B. Since the ratio of the sag to the span is small, assume the cable to be parabolic. Also, neglect the weight of the portion of cable from B to D.



a. **Load P.** We denote by C the lowest point of the cable and draw the free-body diagram of the portion CB of cable. Assuming the load to be uniformly distributed along the horizontal, we write

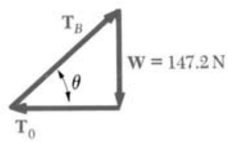
$$w = (0.75 \text{ kg/m})(9.81 \text{ m/s}^2) = 7.36 \text{ N/m}$$

The total load for the portion CB of the cable is

$$W = wx_B = (7.36 \text{ N/m})(20 \text{ m}) = 147.2 \text{ N}$$

and is applied halfway between C and B. Summing moments about B, we write

$$+\circlearrowleft \Sigma M_B = 0: \qquad (147.2 \text{ N})(10 \text{ m}) - T_0(0.5 \text{ m}) = 0 \qquad T_0 = 2944 \text{ N}$$



From the force triangle we obtain

$$T_B = \sqrt{T_0^2 + W^2}$$
$$= \sqrt{(2944 \text{ N})^2 + (147.2 \text{ N})^2} = 2948 \text{ N}$$

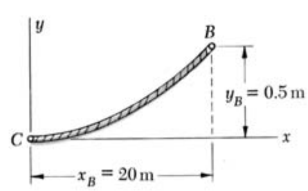
Since the tension on each side of the pulley is the same, we find

$$P = T_B = 2948 \text{ N} \blacktriangleleft$$

b. **Slope of Cable at B.** We also obtain from the force triangle

$$\tan\theta = \frac{W}{T_0} = \frac{147.2 \text{ N}}{2944 \text{ N}} = 0.05$$

$$\theta = 2.9° \blacktriangleleft$$



c. **Length of Cable.** Applying Eq. (7.10) between C and B, we write

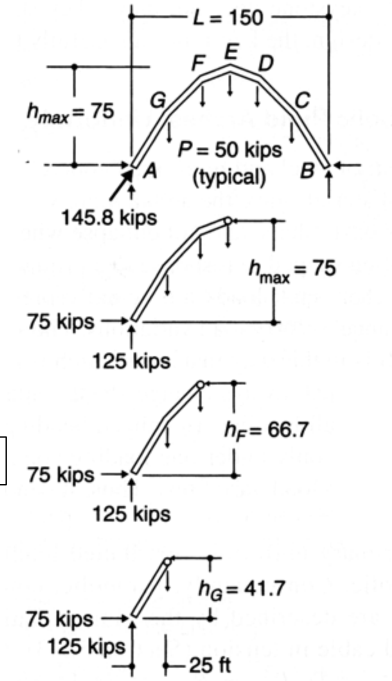
$$s_B = x_B\left[1 + \frac{2}{3}\left(\frac{y_B}{x_B}\right)^2 + \cdots\right]$$
$$= (20 \text{ m})\left[1 + \frac{2}{3}\left(\frac{0.5 \text{ m}}{20 \text{ m}}\right)^2 + \cdots\right] = 20.00833 \text{ m}$$

The total length of the cable between A and B is twice this value,

$$\text{Length} = 2s_B = 40.0167 \text{ m} \blacktriangleleft$$

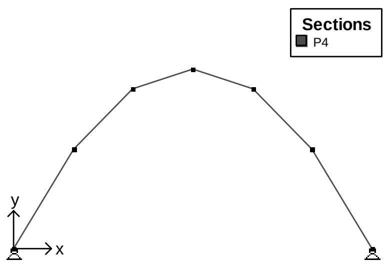
Example 2 (Figure 5.19(b))

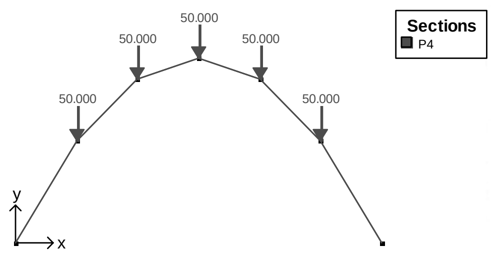
Using Multiframe4D, verify the axial force, shear and bending moment for the funcular shape with P = 50,000 lb, L = 150 ft, and h_{max} = 75 ft.



Joint Coordinates (ft)

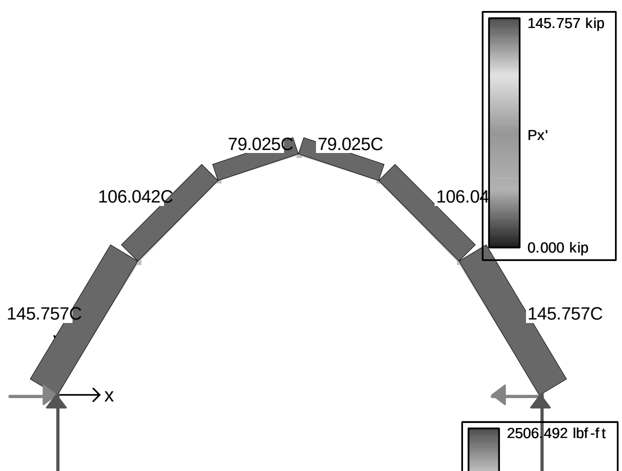
Joint	Label	x	y	z
1		0.000	0.000	0.000
2		25.000	41.700	0.000
3		50.000	66.700	0.000
4		75.000	75.000	0.000
5		100.000	66.700	0.000
6		125.000	41.700	0.000
7		150.000	0.000	0.000

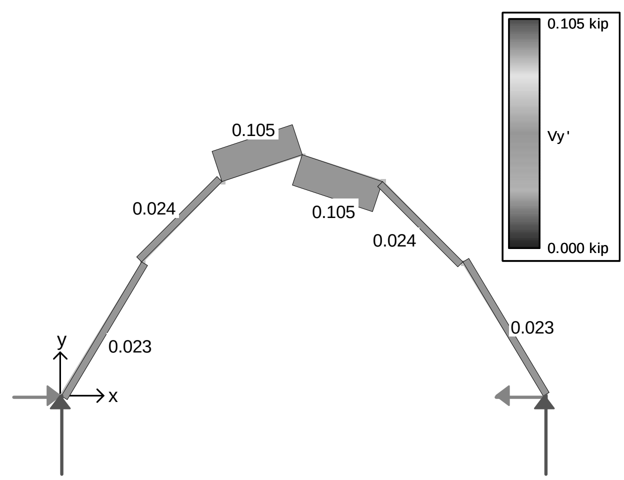


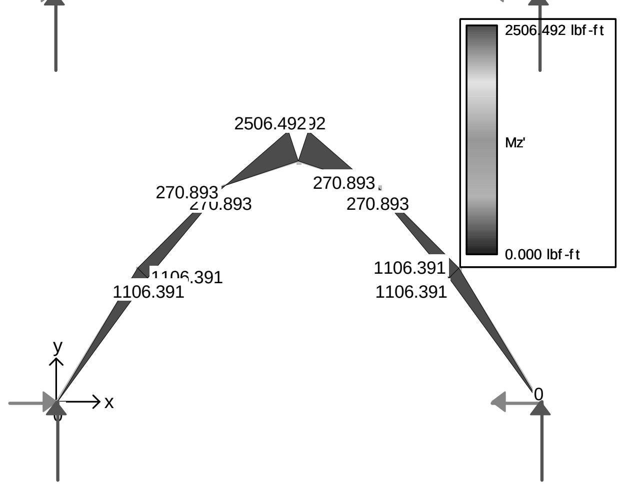


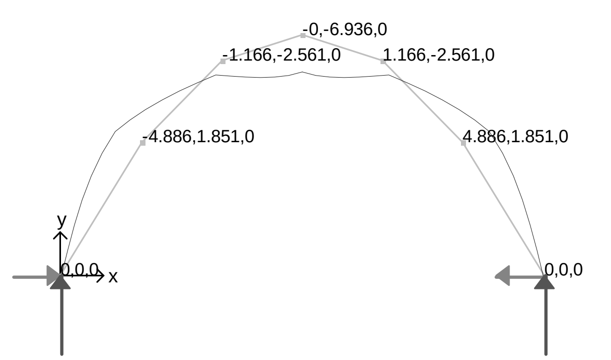
Joint Reactions (kip,lbf-ft)

Joint Label	Load Case	Rx'	Ry'	Rz'
1	Load Case 1	74.967	125.000	0.000
2	Load Case 1	-0.000	-0.000	0.000
3	Load Case 1	0.000	0.000	0.000
4	Load Case 1	-0.000	0.000	0.000
5	Load Case 1	0.000	-0.000	0.000
6	Load Case 1	-0.000	0.000	0.000
7	Load Case 1	-74.967	125.000	0.000









Simplified Frame Analysis (Portal Method)
from Simplified Design, 3rd ed., Portland Cement Association, 2004

Chapter 2
Simplified Frame Analysis

2.1 INTRODUCTION

The final design of the structural components in a building frame is based on maximum moment, shear, axial load, torsion and/or other load effects, as generally determined by an elastic frame analysis (ACI 8.3). For building frames of moderate size and height, preliminary and final designs will often be combined. Preliminary sizing of members, prior to analysis, may be based on designer experience, design aids, or simplified sizing expressions suggested in this book.

Analysis of a structural frame or other continuous construction is usually the most time consuming part of the total design. For gravity load analysis of continuous one-way systems (beams and slabs), the approximate moments and shears given by ACI 8.3.3 are satisfactory within the span and loading limitations stated. For cases when ACI 8.3.3 is not applicable, a two-cycle moment distribution method is accurate enough. The speed and accuracy of the method can greatly simplify the gravity load analysis of building frames with usual types of construction, spans, and story heights. The method isolates one floor at a time and assumes that the far ends of the upper and lower columns are fixed. This simplifying assumption is permitted by ACI 8.8.3.

For lateral load analysis of a sway frame, the Portal Method may be used. It offers a direct solution for the moments and shears in the beams (or slabs) and columns, without having to know the member sizes or stiffnesses.

The simplified methods presented in this chapter for gravity load analysis and lateral load analysis are considered to provide sufficiently accurate results for buildings of moderate size and height. However, determinations of load effects by computer analysis or other design aids are equally applicable for use with the simplified design procedures presented in subsequent chapters of this book.

2.2 LOADING

2.2.1 Service Loads

The first step in the frame analysis is the determination of design (service) loads and lateral forces (wind and seismic) as called for in the general building code under which the project is to be designed and constructed. For the purposes of this book, design live loads (and permissible reductions in live loads) and wind loads are based on *Minimum Design Loads for Buildings and Other Structures*, ASCE7-02.[2.1] References to specific ASCE Standard requirements are noted (ASCE 4.2 refers to ASCE 7-02, Section 4.2). For a specific project, however, the governing general building code should be consulted for any variances from ASCE 7-02.

Design dead loads include member self-weight, weight of fixed service equipment (plumbing, electrical, etc.) and, where applicable, weight of built-in partitions. The latter may be accounted for by an equivalent uniform load of not less than 20 psf, although this is not specifically defined in the ASCE Standard (see ASCE Commentary Section 3.2).

Design live loads will depend on the intended use and occupancy of the portion or portions of the building being designed. Live loads include loads due to movable objects and movable partitions temporarily supported by the building during maintenance. In ASCE Table 4-1, uniformly distributed live loads range from 40 psf for residential use to 250 psf for heavy manufacturing and warehouse storage. Portions of buildings, such as library floors and file rooms, require substantially heavier live loads. Live loads on a roof include maintenance equipment, workers, and materials. Also, snow loads, ponding of water, and special features, such as landscaping, must be included where applicable.

Occasionally, concentrated live loads must be included; however, they are more likely to affect individual supporting members and usually will not be included in the frame analysis (see ASCE 4.3).

Design wind loads are usually given in the general building code having jurisdiction. For both example buildings here, the calculation of wind loads is based on the procedure presented in ASCE 6.0. Design for seismic loads is discussed in Chapter 11.

2.6 LATERAL LOAD ANALYSIS

For frames without shear walls, the lateral load effects must be resisted by the "sway" frame. For low-to-moderate height buildings, lateral load analysis of a sway frame can be performed by either of two simplified methods: the Portal Method or the Joint Coefficient Method. Both methods can be considered to satisfy the elastic frame analysis requirements of the code (ACI 8.3). The two methods differ in overall approach. The Portal Method considers a vertical slice through the entire building along each row of column lines. The method is well suited to the range of building size and height considered in this book, particularly to buildings with a regular rectangular floor plan. The Joint Coefficient Method considers a horizontal slice through the entire building, one floor at a time. The method can accommodate irregular floor plans, and provision is made to adjust for a lateral loading that is eccentric to the centroid of all joint coefficients (centroid of resistance). The Joint Coefficient Method considers member stiffnesses, whereas the Portal Method does not.

The Portal Method is presented in this book because of its simplicity and its intended application to buildings of regular shape. If a building of irregular floor plan is encountered, the designer is directed to Reference 2.2 for details of the Joint Coefficient Method.

2.6.1 Portal Method

The Portal Method considers a two-dimensional frame consisting of a line of columns and their connecting horizontal members (slab-beams), with each frame extending the full height of the building. The frame is considered to be a series of portal units. Each portal unit consists of two story-high columns with connecting slab-beams. Points of contraflexure are assumed at mid-length of beams and mid-height of columns. Figure 2-11 illustrates the portal unit concept applied to the top story of a building frame, with each portal unit shown separated (but acting together).

The lateral load W is divided equally between the three portal units. The shear in the interior columns is twice that in the end columns. In general, the magnitude of shear in the end column is W/2n, and in an interior column it is W/n, where n is the number of bays. For the case shown with equal spans, axial load occurs only in the end columns since the combined tension and compression due to the portal effect results in zero axial loads in the interior columns. Under the assumptions of this method, however, a frame configuration with unequal spans will have axial load in those columns between the unequal spans, as well as in the end columns. The general term for axial load in the end columns in a frame of n bays with unequal spans is:

$$\frac{Wh}{2n\ell_1} \text{and} \frac{Wh}{2n\ell_n}, \ell_n \ = \text{length of bay n}$$

The axial load in the first interior column is:

$$\frac{Wh}{2n\ell_1} - \frac{Wh}{2n\ell_2}$$

and, in the second interior column:

$$\frac{Wh}{2n\ell_2} - \frac{Wh}{2n\ell_3}$$

Column moments are determined by multiplying the column shear with one-half the column height. Thus, for joint B in Fig. 2-11, the column moment is (W/3) (h/2) = Wh/6. The column moment Wh/6 must be balanced by equal moments in beams BA and BC, as shown in Fig. 2-12.

Note that the balancing moment is divided equally between the horizontal members without considering their relative stiffnesses. The shear in beam AB or BC is determined by dividing the beam end moment by one-half the beam length, $(Wh/12)(\ell/2) = Wh/6\ell$.

The process is continued throughout the frame taking into account the story shear at each floor level.

2.6.2 Examples: Wind Load Analyses for Buildings #1 and #2

For Building #1, determine the moments, shears, and axial forces using the Portal Method for an interior frame resulting from wind loads acting in the N-S direction. The wind loads are determined in Section 2.2.1.2.

Moments, shears, and axial forces are shown directly on the frame diagram in Fig. 2-13. The values can be easily determined by using the following procedure:

(1) Determine the shear forces in the columns:

For the end columns:

3rd story: V = 12.0 kips/6 = 2.0 kips
2nd story V = (12.0 kips + 23.1 kips)/6 = 5.85 kips
1st story: V = (12.0 kips + 23.1 kips + 21.7 kips)/6 = 9.50 kips

The shear forces in the interior columns are twice those in the end columns.

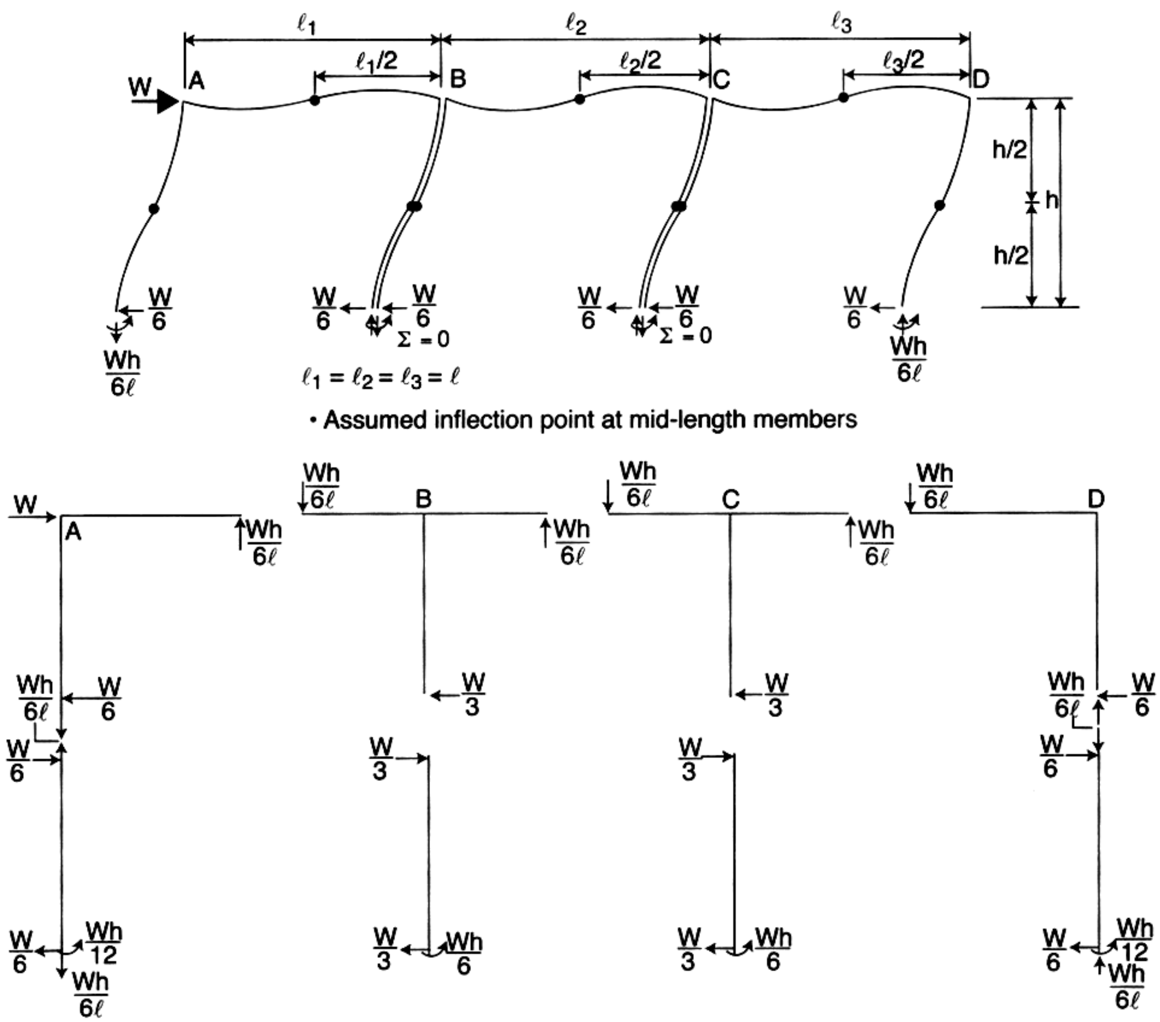


Figure 2-11 Portal Method

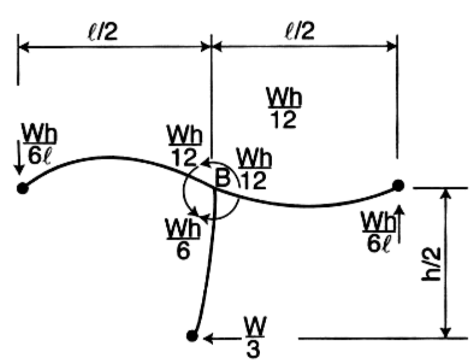


Figure 2-12 Joint Detail

(2) Determine the axial loads in the columns:

For the end columns, the axial loads can be obtained by summing moments about the column inflection points at each level. For example, for the 2nd story columns:

$$\Sigma M = 0 : 12(13 + 6.5) + 23.1\ (6.5) - P\ (90) = 0$$
$$P = 4.27 \text{ kips}$$

For this frame, the axial forces in the interior columns are zero.

(3) Determine the moments in the columns:

The moments can be obtained by multiplying the column shear force by one-half of the column length.

For example, for an exterior column in the 2nd story:

$$M = 5.85(13/2) = 38.03 \text{ ft-kips}$$

(4) Determine the shears and the moments in the beams:
These quantities can be obtained by satisfying equilibrium at each joint. Free-body diagrams for the 2nd story are shown in Fig. 2-14.

As a final check, sum moments about the base of the frame:

$$\Sigma M = 0{:}\ \ 12.0(39) + 23.1(26) + 21.7(13) - 10.91(90) - 2(61.53 + 123.07) = 0 \qquad \text{(checks)}$$

In a similar manner, the wind load analyses for an interior frame of Building #2 (5-story flat plate), in both the N-S and E-W directions are shown in Figs. 2-15 and 2-16, respectively. The wind loads are determined in Section 2.2.1.1.

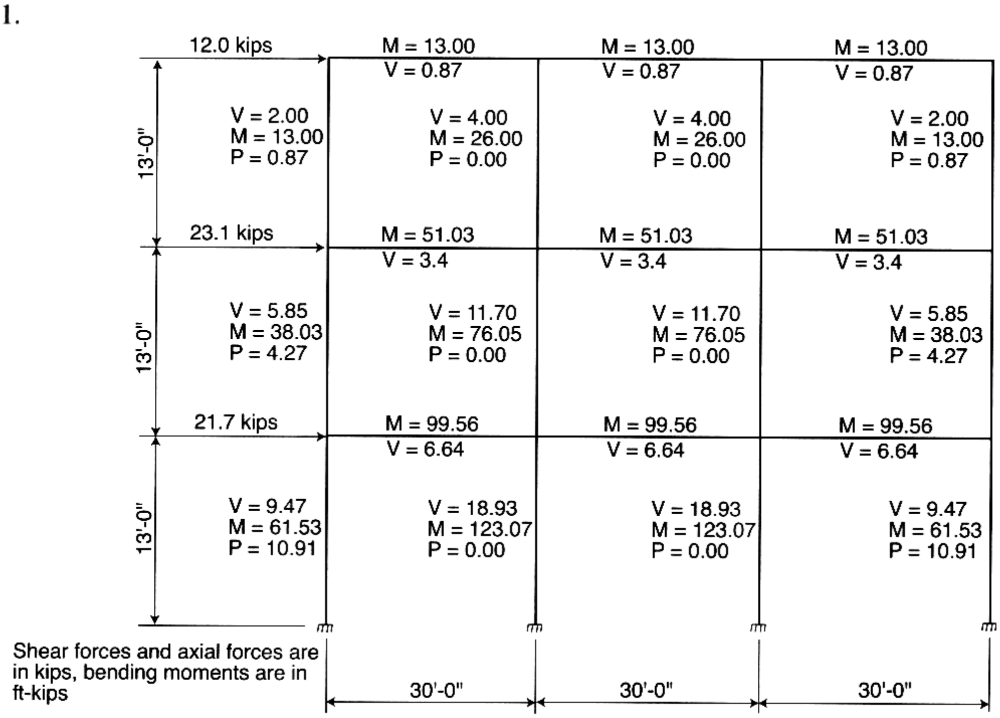


Figure 2-13 Shear, Moments and Axial Forces Resulting from Wind Loads for an Interior Frame of Building #1 in the N-S Direction, using the Portal Method

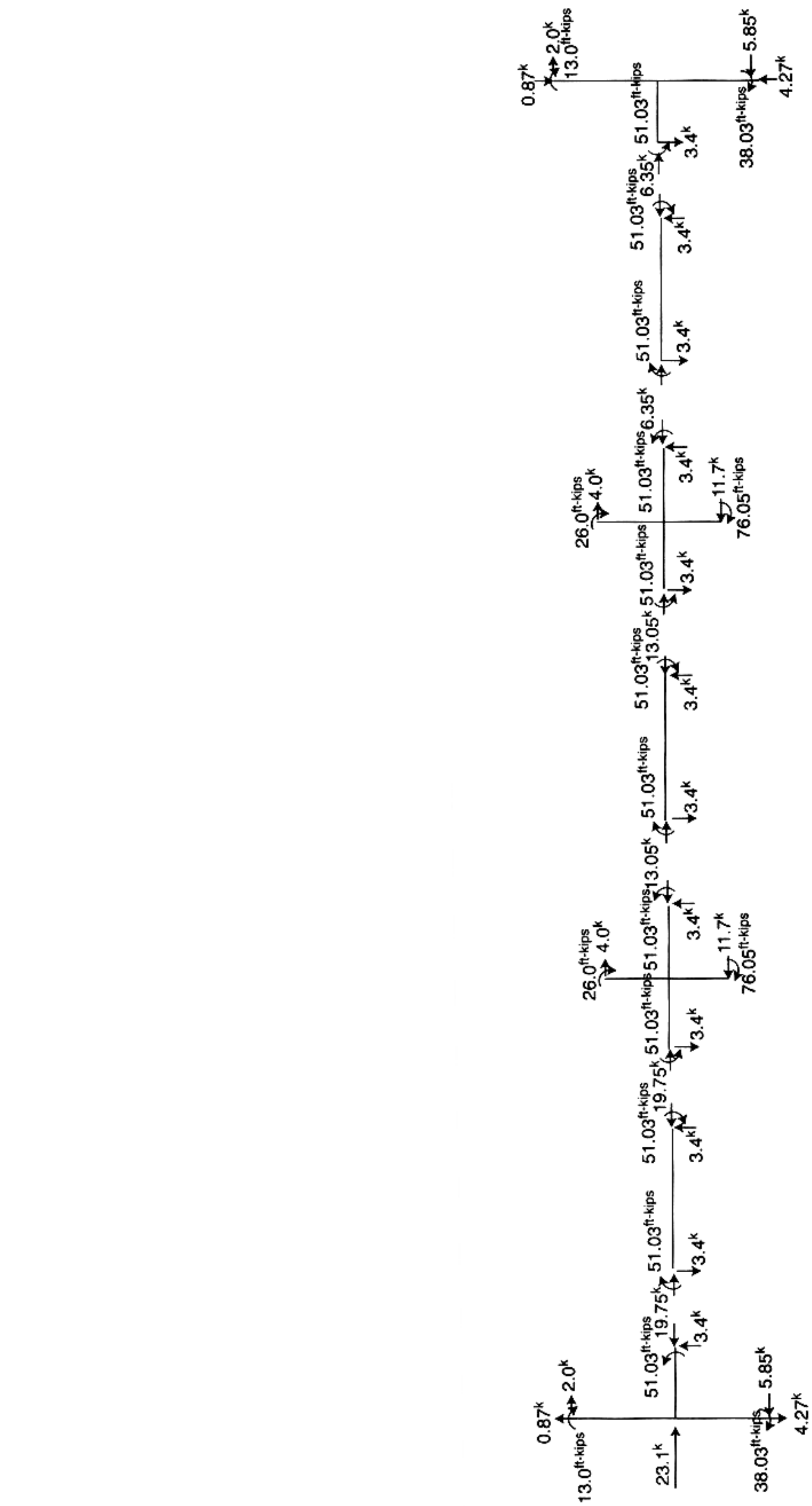


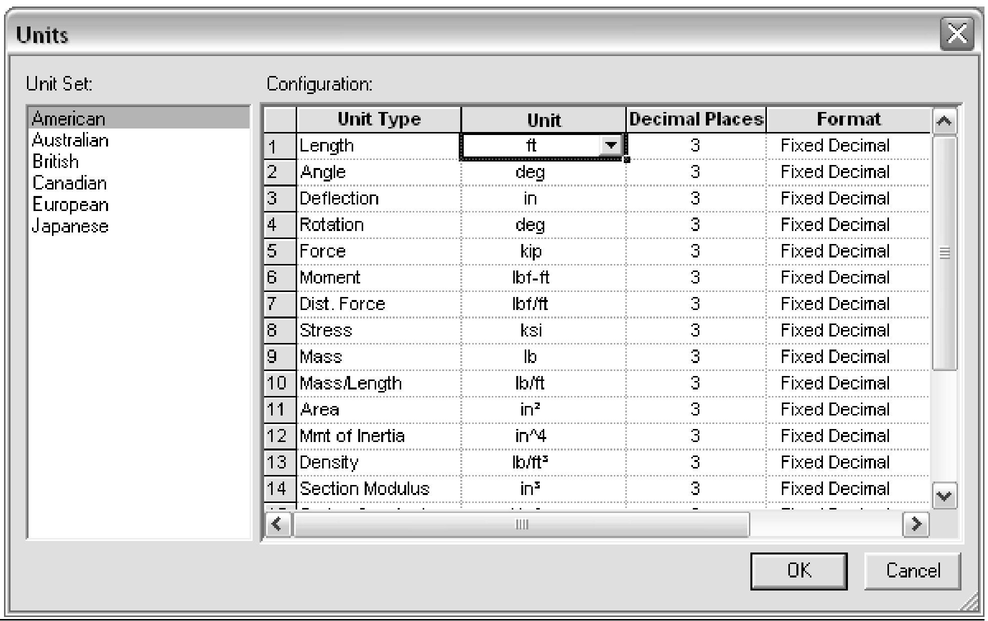
Figure 2-14 Shear Forces, Axial Forces, and Bending Moments at 2nd Story of Building #1

Frame Analysis Using Multiframe

1. The software is on the computers in the College of Architecture in Programs under the Windows Start menu (see https://wikis.arch.tamu.edu/display/HELPDESK/Computer+Accounts for lab locations). Multiframe is under the Multiframe [current version #] menu. Or it can be downloaded from the web site: http://www.formsys.com/academic/multiframe

2. There are tutorials available on line at http://www.formsys.com/mflearning that list the tasks and order in greater detail. The first task is to define the unit system:

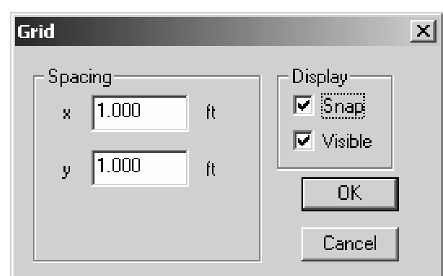
   - Choose Units… from the View menu. Unit sets are available, but specific units can also be selected by double clicking on a unit or format and making a selection from the menu.



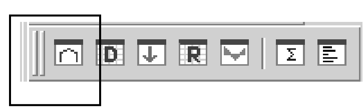
	Unit Type	Unit	Decimal Places	Format
1	Length	ft	3	Fixed Decimal
2	Angle	deg	3	Fixed Decimal
3	Deflection	in	3	Fixed Decimal
4	Rotation	deg	3	Fixed Decimal
5	Force	kip	3	Fixed Decimal
6	Moment	lbf-ft	3	Fixed Decimal
7	Dist. Force	lbf/ft	3	Fixed Decimal
8	Stress	ksi	3	Fixed Decimal
9	Mass	lb	3	Fixed Decimal
10	Mass/Length	lb/ft	3	Fixed Decimal
11	Area	in²	3	Fixed Decimal
12	Mmt of Inertia	in^4	3	Fixed Decimal
13	Density	lb/ft³	3	Fixed Decimal
14	Section Modulus	in³	3	Fixed Decimal

3. To see the scale of the geometry, a grid option is available:

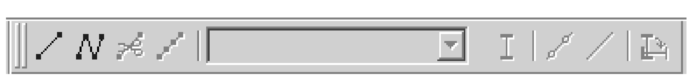
   - Choose Grid… from the View menu



4. To create the geometry, you must be in the Frame window (default). The symbol is the frame in the window toolbar:

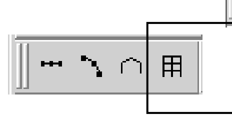


   The Member toolbar shows ways to create members:

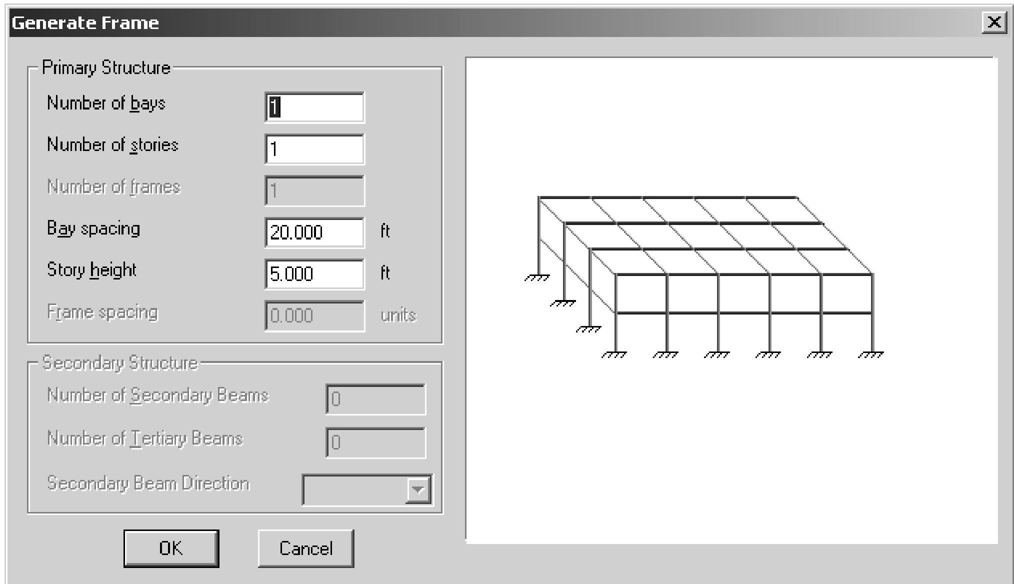


   The Generate toolbar has convenient tools to create typical structural shapes.

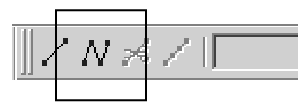
   - To create a frame, use the multi-bay frame button:



- Enter the number of bays (horizontally), number of stories (vertically) and the corresponding spacings:



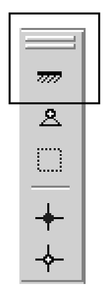
- If the frame does not have regular bays, use the add connected members button to create segments:

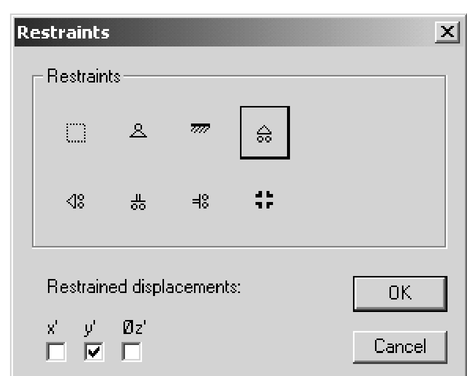


- Select a starting point and ending point with the cursor. The location of the cursor and the segment length is displayed at the bottom of the geometry window. The ESC button will end the segmented drawing.

- The geometry can be set precisely by selecting the joint (drag), and bringing up the joint properties menu (right click) to set the coordinates.

- The support types can be set by selecting the joint (drag) and using the Joint Toolbar (fixed shown), or the Frame / Joint Restraint ... menu (right click).





NOTE: If the support appears at both ends of the member, you had the member selected rather than the joint. Select the joint to change support for and right click to select the joint restraints menu or select the correct support on the joint toolbar.

The support forces will be determined in the analysis.

5. All members must have sections assigned (see section 6.) in order to calculate reactions and deflections. To use a standard steel section **proceed to step 6.** For custom sections the section information must be entered. To define a section:

    - Choose Edit Sections / Add Section… from the Edit menu
    - Type a name for your new section
    - Choose group <u>Frame</u> from the group names provided so that the section will remain with the file data
    - Choose a shape. The Flat Bar shape is a rectangular section.
    - Enter the cross section data.

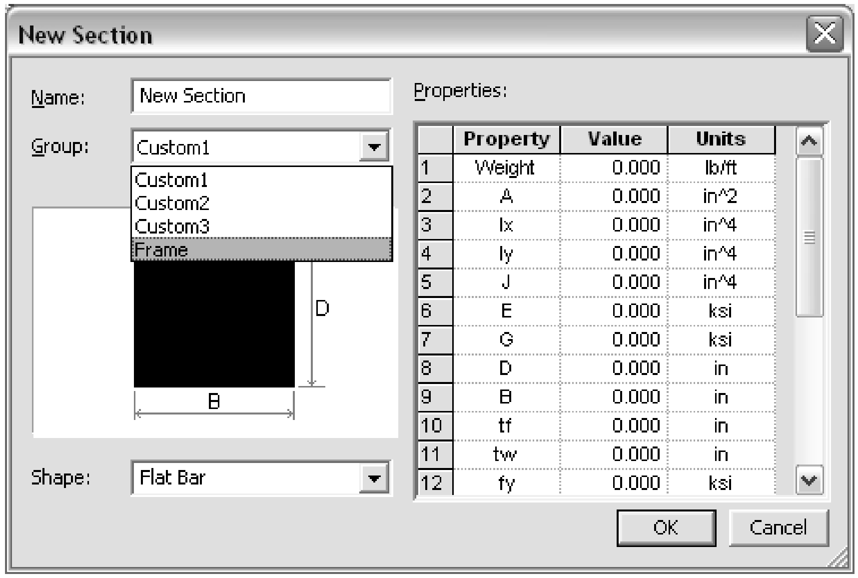
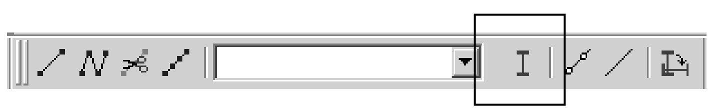


Table values 1-9 must have values for a Flat Bar, but not all are used for every analysis. A recommendation is to put the value of 1 for those properties you don't know or care about. Properties like t_f, t_w, etc. refer to wide flange sections.

- Answer any query. If the message says there is an error, the section will not be created until the error is corrected.

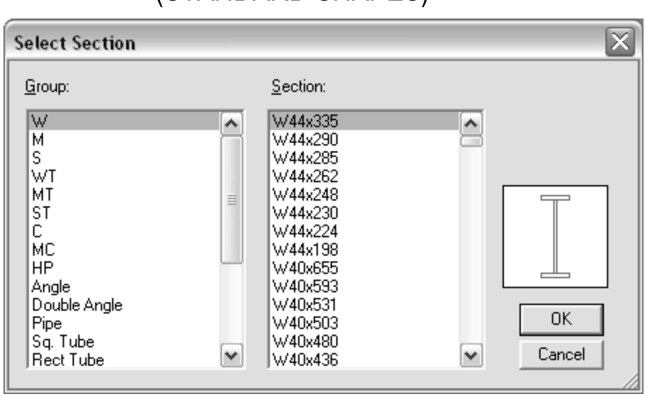
6. The standard sections library loaded is for the United States. If another section library is needed, use the Open Sections Library... command under the file menu, choose the library folder, and select the SectionsLibrary.slb file.

    Select the members (drag to make bold) and assign sections with the Section button on the Member toolbar:

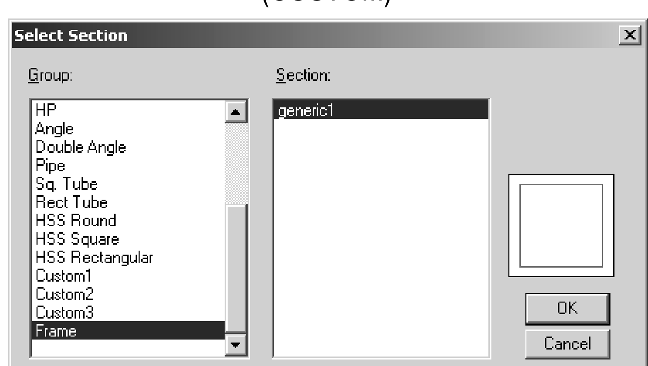


- Choose the group name and section name:

(STANDARD SHAPES)

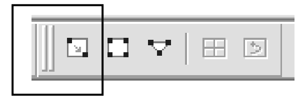


(CUSTOM)

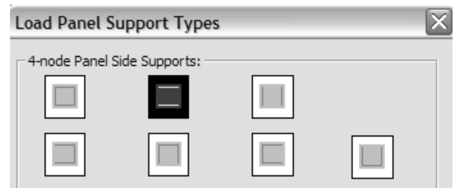


7. If there is an area that has a uniformly distributed load, load panels may be defined in the Frame window. Because the loaded area may not be visible in the current view, choose the View button at the lower left of the Frame window. The options for view are shown. (See 3D Frames, last page.)

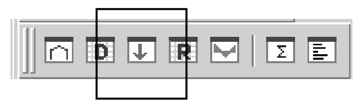
   - Choose the panel type (rectangular, 4-node, or 3-node) from the menu and select the corners. If the area is rectangular, only the opposite corners need to be selected.



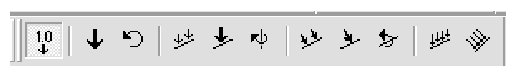
   - Select the panel and from the pop-up menu, or the Frame menu, specify the load panel supports. The default supports are on all sides. If the panel is one way, chose the corresponding picture



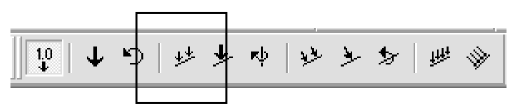
8. The frame geometry is complete, and in order to define the load conditions you must be in the Load window represented by the green arrow:



9. The Load toolbar allows a joint to be loaded with a force or a moment in global coordinates, shown by the first two buttons after the display numbers button. It allows a member to be loaded with a distributed load, concentrated load or moment (next three buttons) in global coordinates, as well as loading with distributed or single force or moment in the local coordinate system (next three buttons). It allows a load panel to be loaded with a distributed load in global or local coordinates (last two buttons).



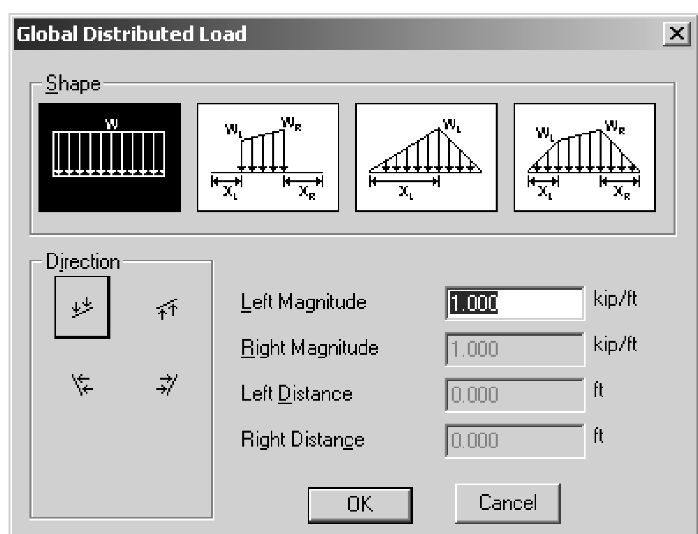
   - Choose the member to be loaded (drag) and select the load type (here shown for global distributed loading):



   - Choose the distribution type and direction. Note that the arrow shown is the direction of the loading. There is no need to put in negative values for downward loading.

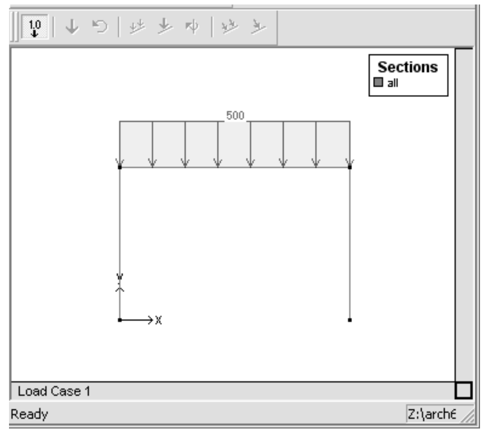
   - Enter the values of the load and distances (if any). Distances can be entered as a function of the length , i.e. L/2, L/4...

   - Area load units may have to be changed in the View Units dialogue.



NOTE:* <u>*Do not*</u> *put support reactions as applied loads. The analysis will determine the reaction values.

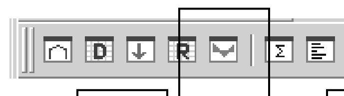
Multiframe4D will automatically generate a grouping called a Load Case named Load Case 1 when a load is created. All additional loads will be added to this load case unless a new load case is defined (Add case under the Case menu).



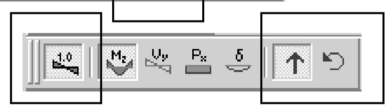
10. In order to run the analysis after the geometry, member properties and loading has been defined:

    - Choose Linear from the Analyze menu

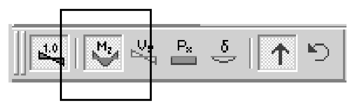
11. If the analysis is successful, you can view the results in the Plot window represented by the red moment diagram:



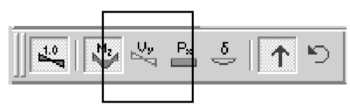
12. The Plot toolbar allows the numerical values to be shown (1.0 button), the reaction arrows to be shown (brown up arrow) and reaction moments to be shown (brown curved arrow):



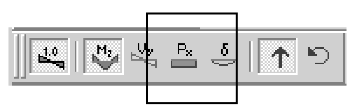
    - To show the moment diagram, Choose the red Moment button



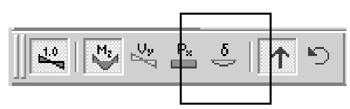
    - To show the shear diagram, Choose the green Shear button



    - To show the axial force diagram, Choose the purple Axial Force button

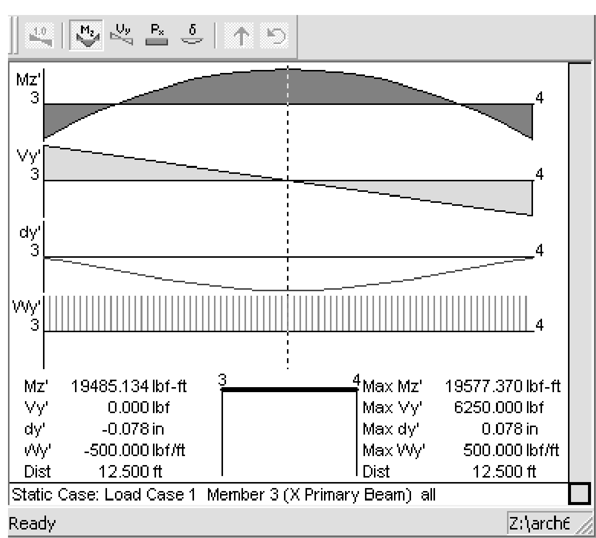


    - To show the deflection diagram, Choose the blue Deflection button

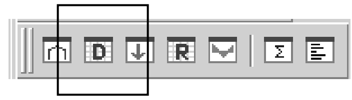


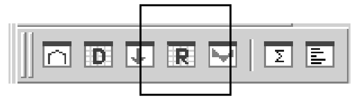
    - To animate the deflection diagram, Choose Animate... from the Display menu. You can also save the animation to a .avi file by checking the box.

    - To see exact values of shear, moment and deflection, double click on the member and move the vertical cross hair with the mouse. The ESC key will return you to the window.



13. The Data window (D) allows you to view all data "entered" for the geometry, sections and loading. These values can be edited.





14. The Results window (R) allows you to view all results of the analysis including displacements, reactions, member forces (actions) and stresses. These values can be cut and pasted into other Windows programs such as Word or Excel.

NOTE: Px' refers to the axial load (P) in the local axis x direction (x'). Vy' refers to the shear perpendicular to the local x axis, and Mz' refers to the bending moment.

Static Case: Load Case 1

	Memb	Label	Joint	Px' lbf	Vy' lbf	Mz' lbf-ft
1	1	Column	1	6250.000	-1786.320	-9725.784
2	1	Column	3	-6250.000	1786.320	-19577.371
3	2	Column	2	6250.000	1786.320	9725.784
4	2	Column	4	-6250.000	-1786.320	19577.371
5	3	X Prima	3	1786.320	6250.000	19577.371
6	3	X Prima	4	-1786.320	6250.000	-19577.371

Member Actions | Max Acti

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15. To save the file Choose Save from the File menu.
16. To load an existing file Choose Open... from the File menu. .
17. To print a plot Choose Print Window... from the File menu. As an alternative, you may copy the plot (Ctrl+c) and paste it in a word processing document (Ctrl+v).

Example of Combined Stresses:

for member 3: M_{max} = 19.6 k-ft, P = 1.76 k

knowing A = 21.46 in^2, I = 796.0 in^4, c = 7.08 in

$$f_{max} = \frac{1.76k}{21.46in^2} + \frac{19.6^{k-ft} \cdot 7.08in}{796in^4} \cdot \frac{12in}{ft} = 0.082ksi + 2.092ksi = 2.174ksi$$

Results window:

Static Case: Load Case 1

	Memb	Label	Joint	Sbz' top ksi	Sbz' bot ksi	Sy' ksi	Sx' ksi	Sx'+Sbz' top ksi	Sx'+Sbz' bot ksi
1	1	Column	1	1.039	-1.039	-1152.461	0.286	1.325	-0.753
2	1	Column	3	-2.092	2.092	-1152.461	0.286	-1.806	2.378
3	2	Column	2	-1.039	1.039	1152.461	0.286	-0.753	1.325
4	2	Column	4	2.092	-2.092	1152.461	0.286	2.378	-1.806
5	3	X Prima	3	-2.092	2.092	4032.245	0.082	-2.011	2.174
6	3	X Prima	4	-2.092	2.092	-4032.245	0.082	-2.011	2.174

Member Stresses | Max Stresses | Member [

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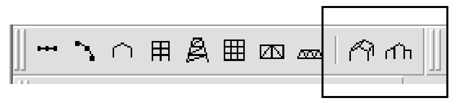
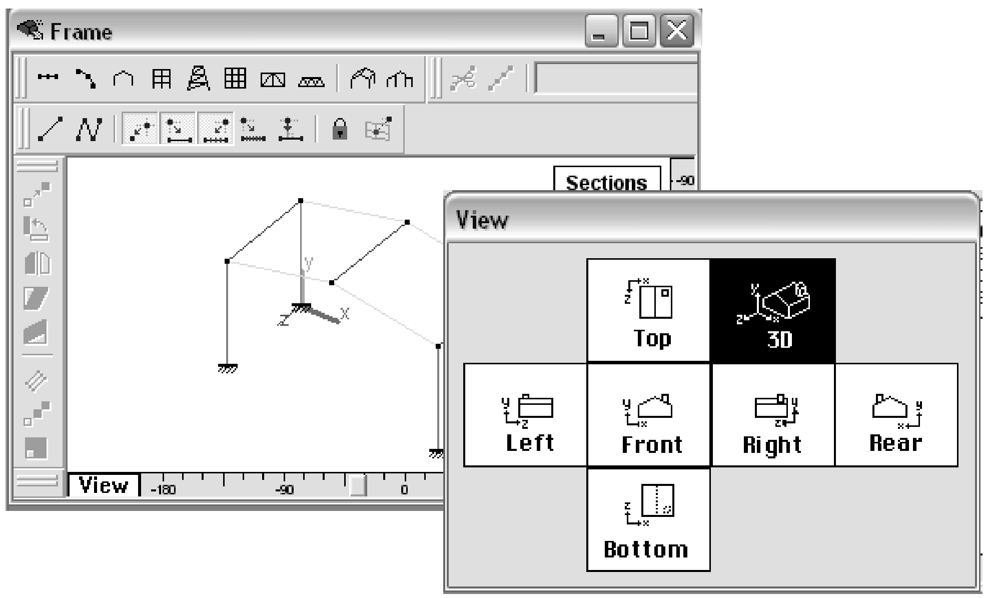
where Sx' refers to the axial stress, Sy' refers to the bending stress around the local vertical axis and Sz' refers to the bending stress around the local horizontal axis.

For 3D Frames:

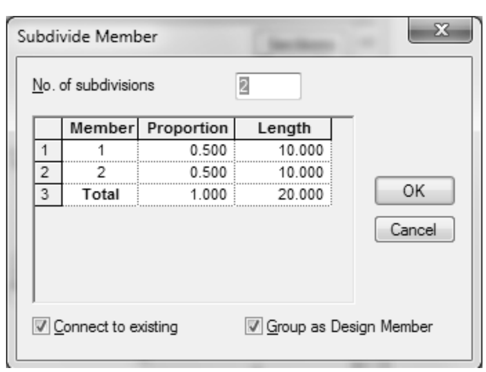
- There is a tutorial in the Help menu (Chapter 1 – 3D Tutorial) that lists the tasks and order in greater detail. It expects that you have been through the 2D tutorial to build on the steps already mastered.
- There are standard 3D frame shapes on the frame toolbar.

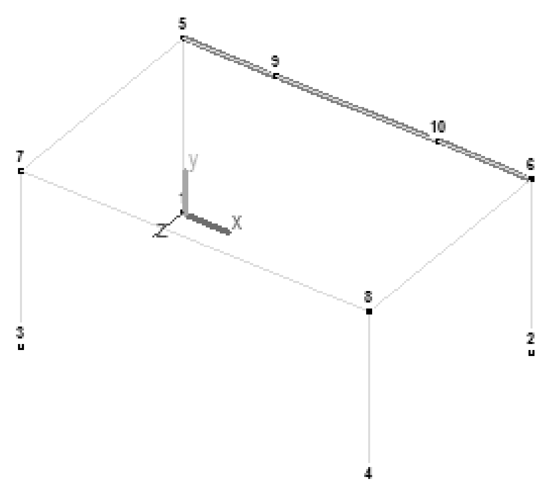


- It is very useful to change the view to isometric with the View Button

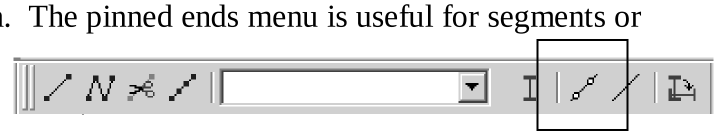


- If you wish to have additional beams supported by the beams of your frame, choose the beam and use the Subdivide Member menu under Geometry. This will make additional joints, but keep the segments together.

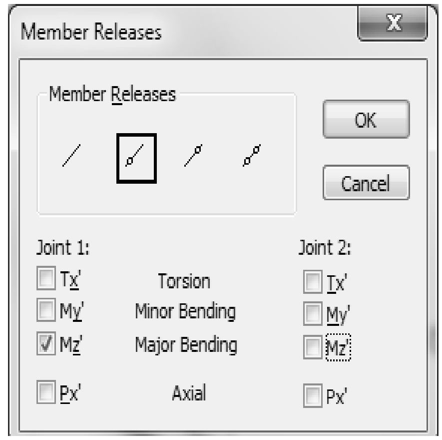




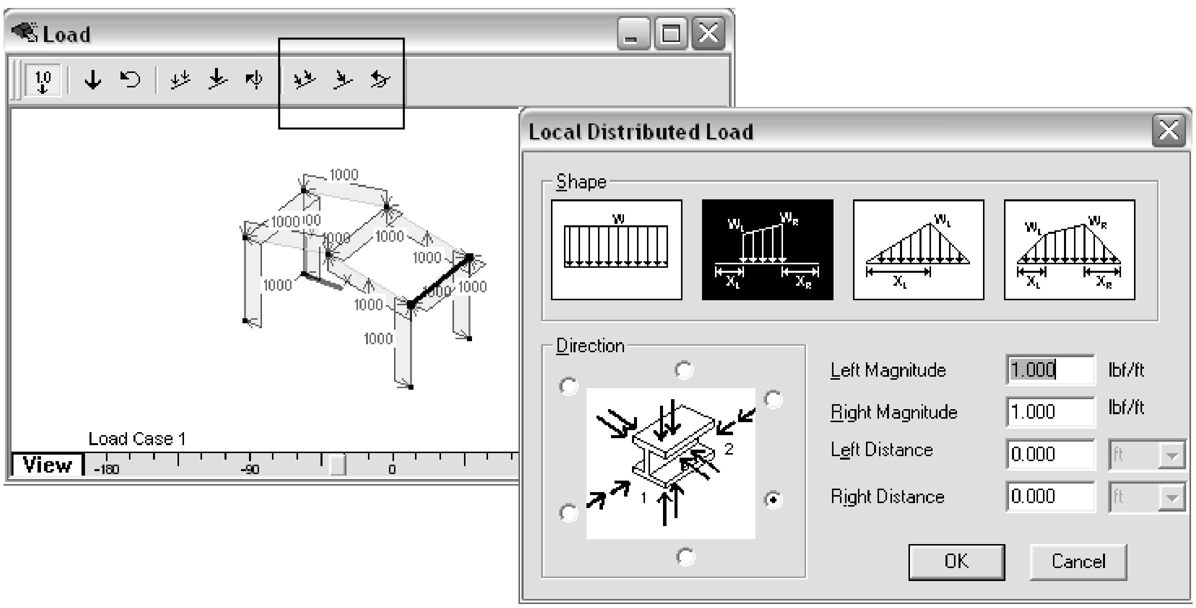
- In order to model a beam end as simply supported, you must release the restraint preventing rotation about the x-x axis of the beam. The pinned ends menu is useful for segments or subdivided members.



Or, by selecting a segment and right clicking for a menu, you can use Member Releases (also under the Frame menu) to release the Major Bending (M'_Z) for one end or both.

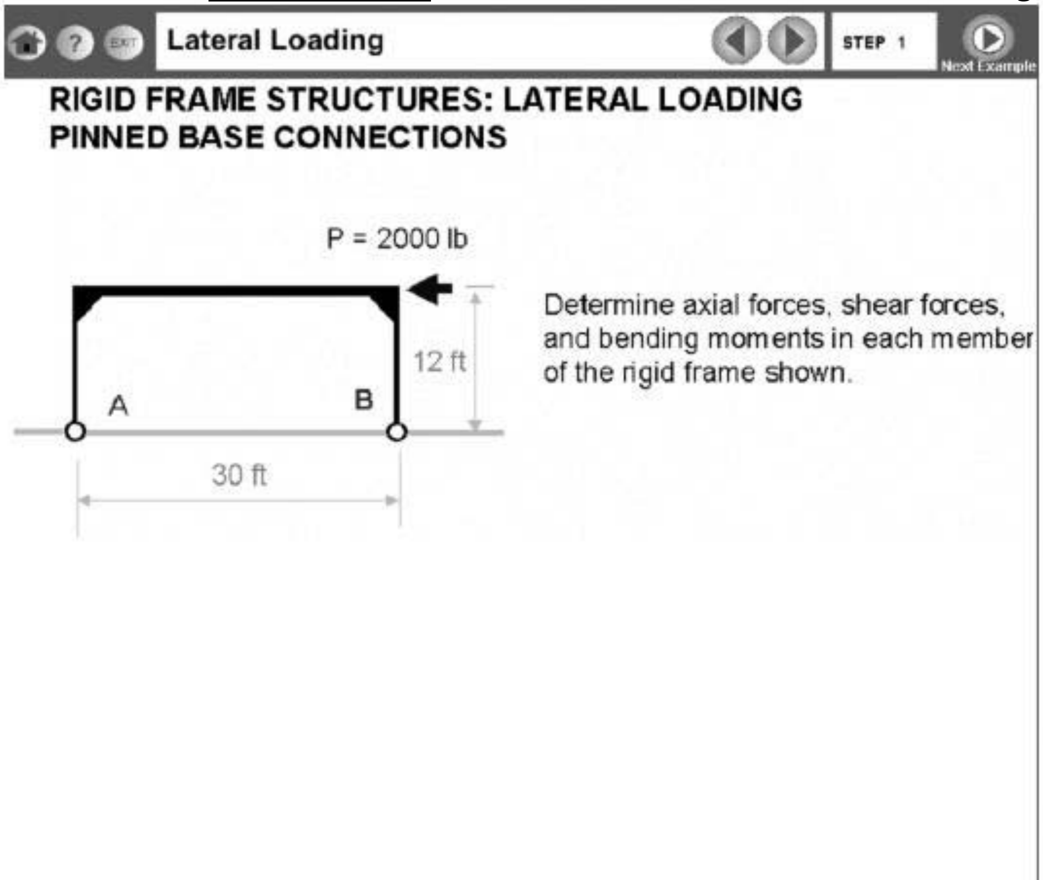


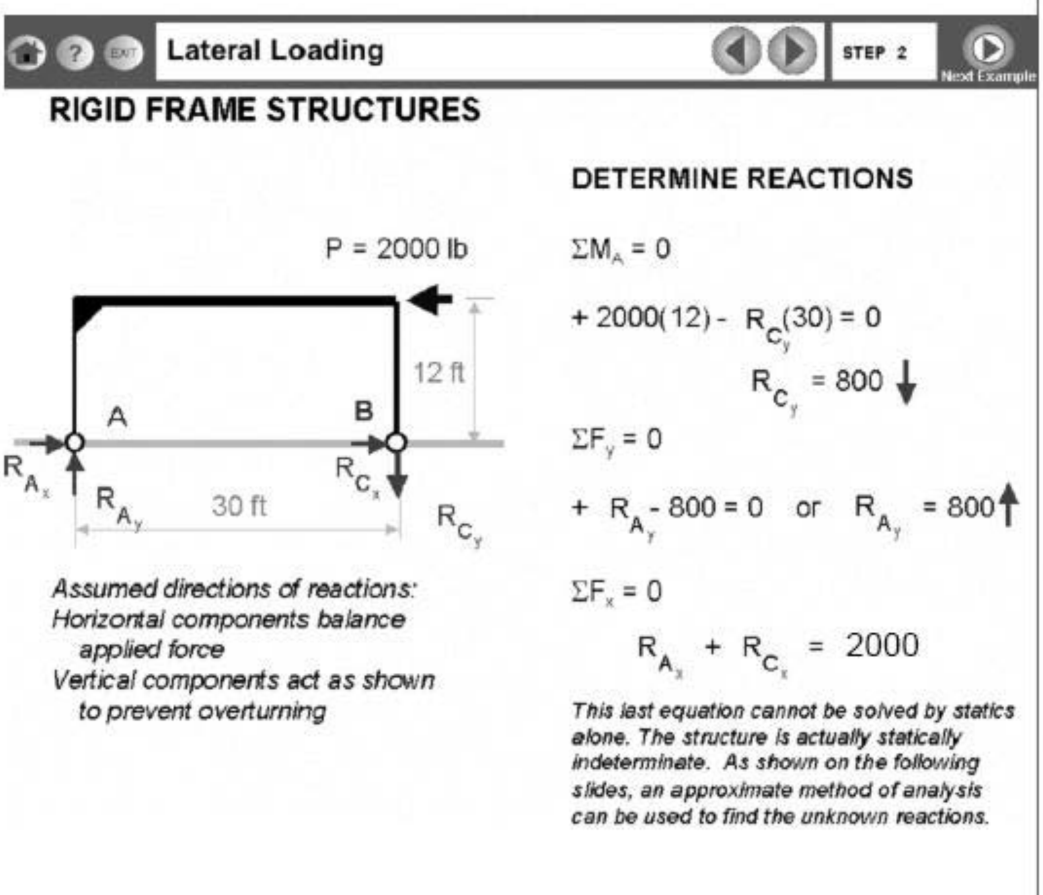
- It is necessary to understand the local member axes to assign the correct load direction. Choosing the *local* loading types will show the member orientation with respect to the load direction.



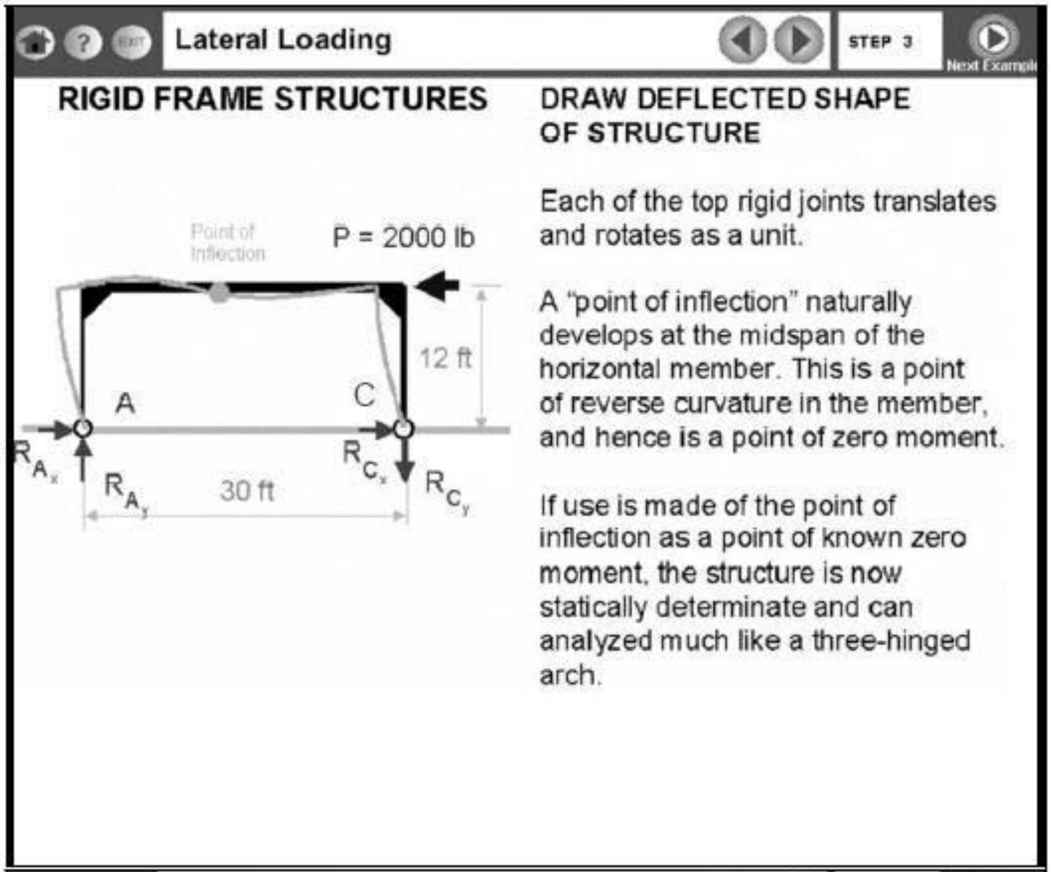
Examples:
Rigid Frames

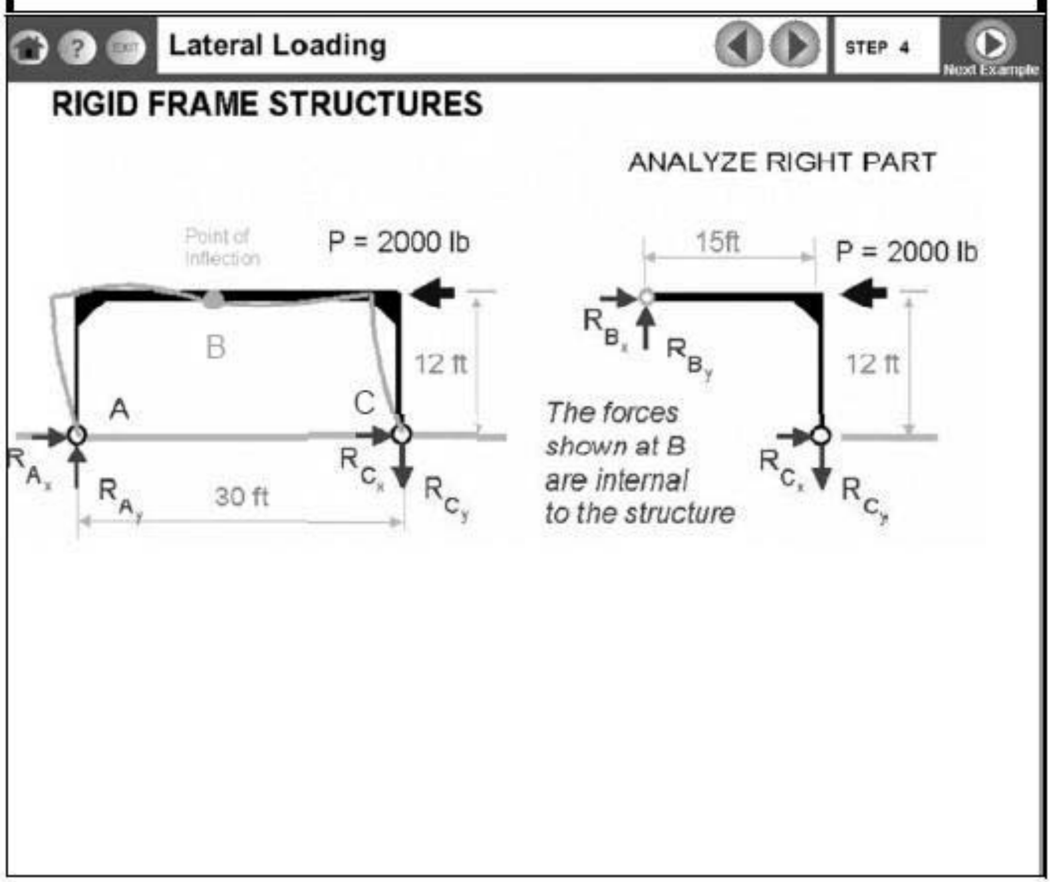
<u>Example 1</u> **From <u>eStructures v1.1</u>, Schodek and Pollalis, 2000 Harvard College**



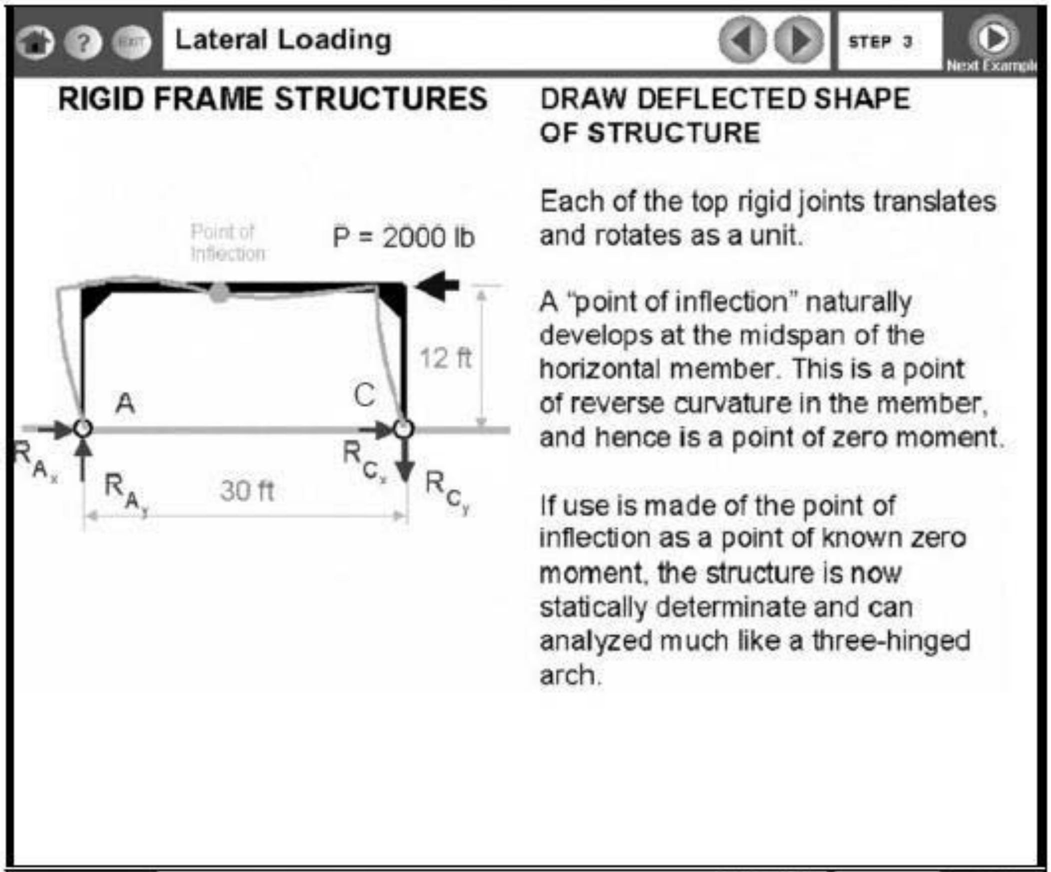


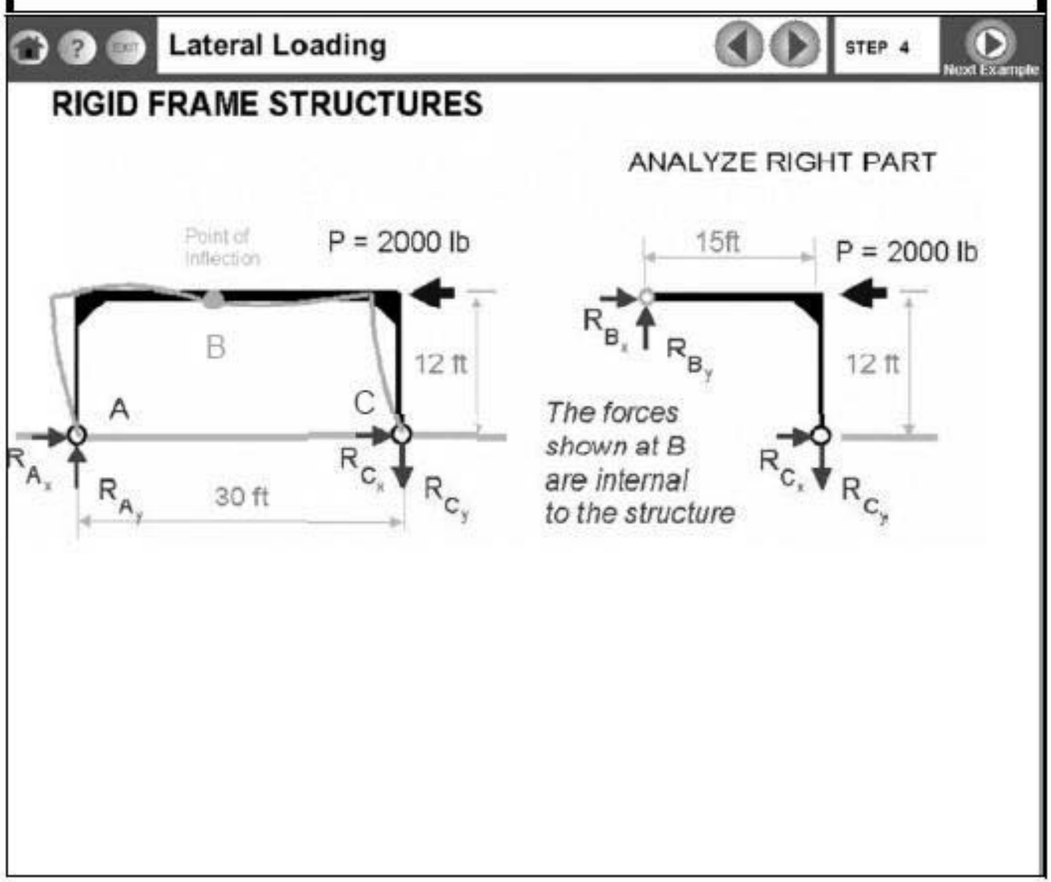
Example 1 (continued)



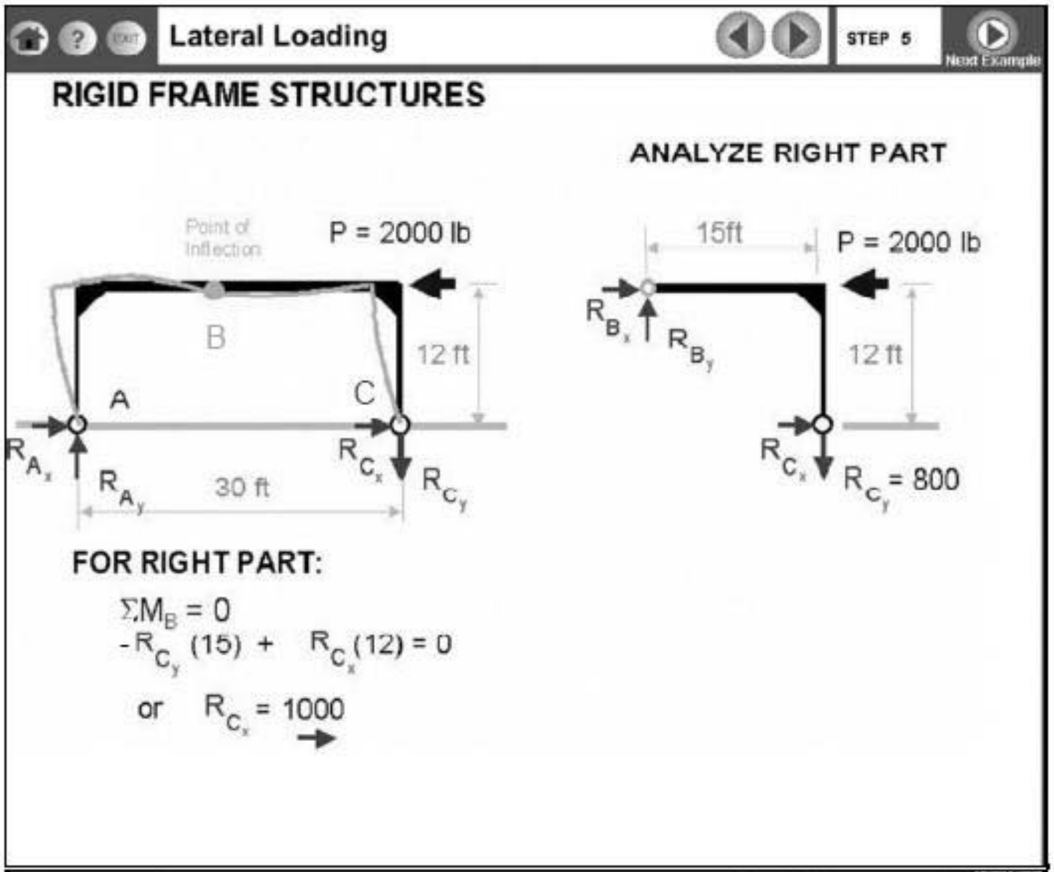


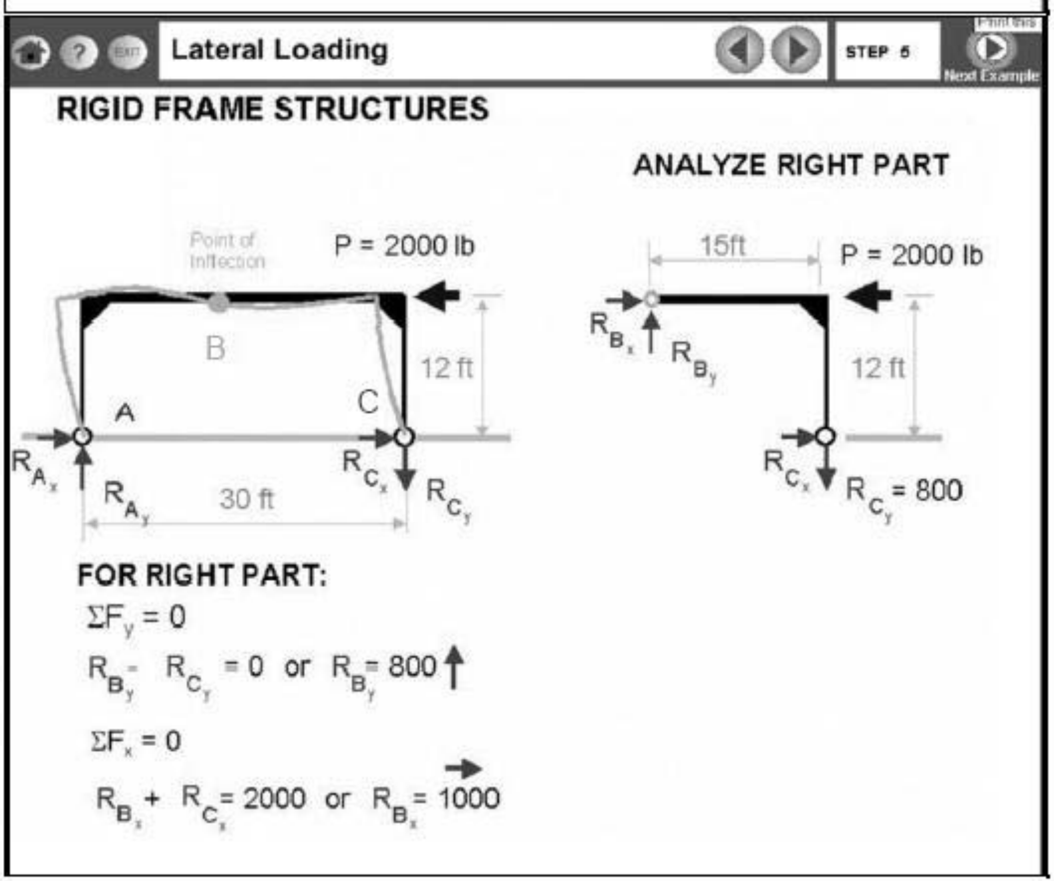
Example 1 (continued)



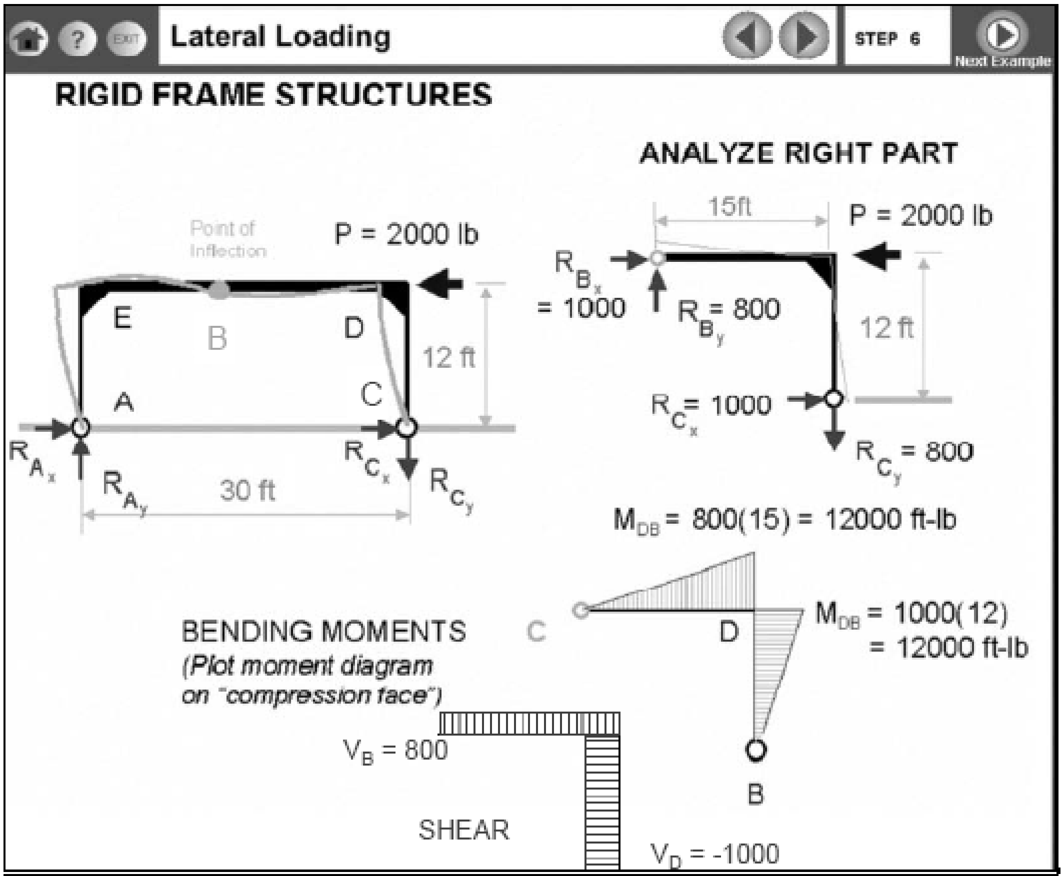


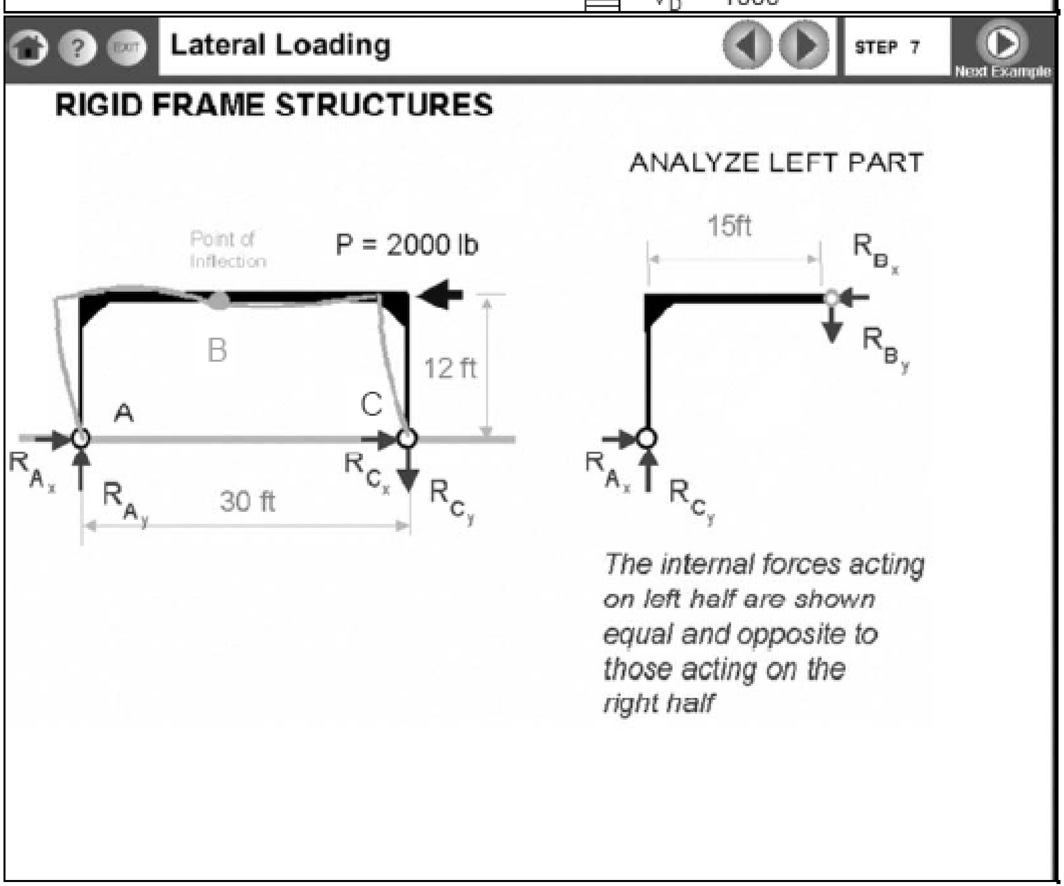
Example 1 (continued)



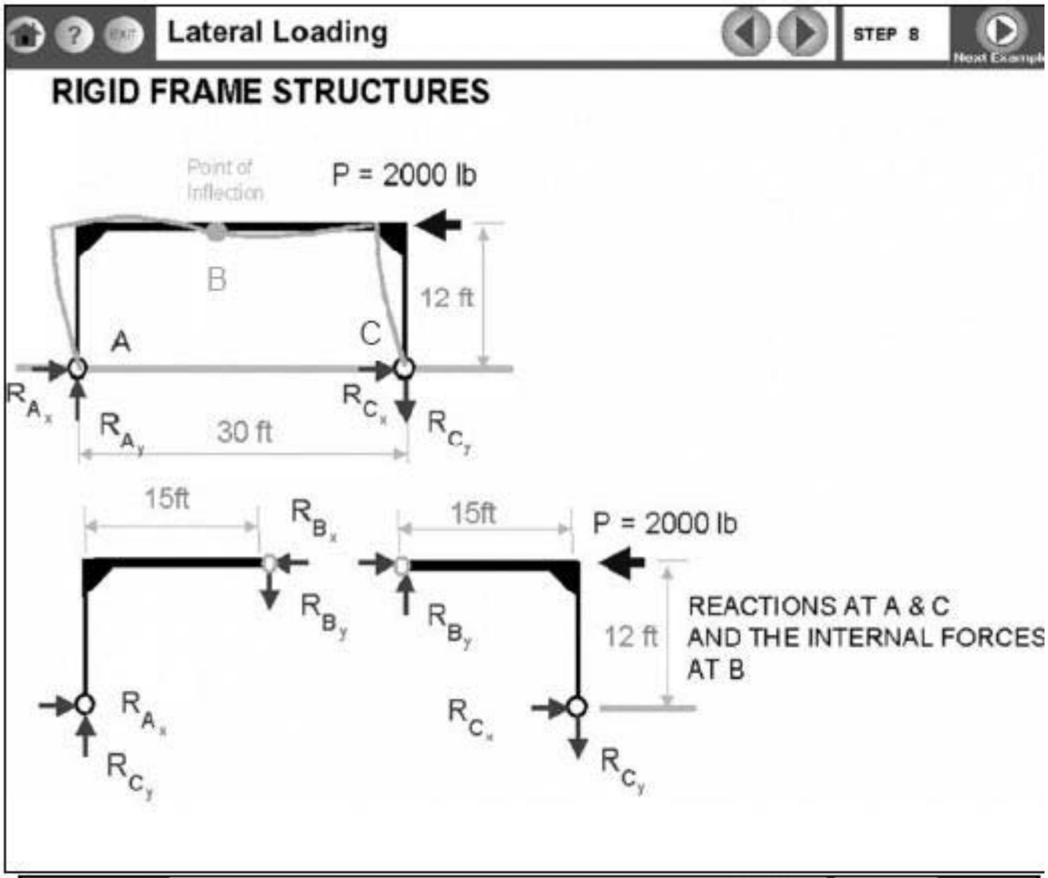


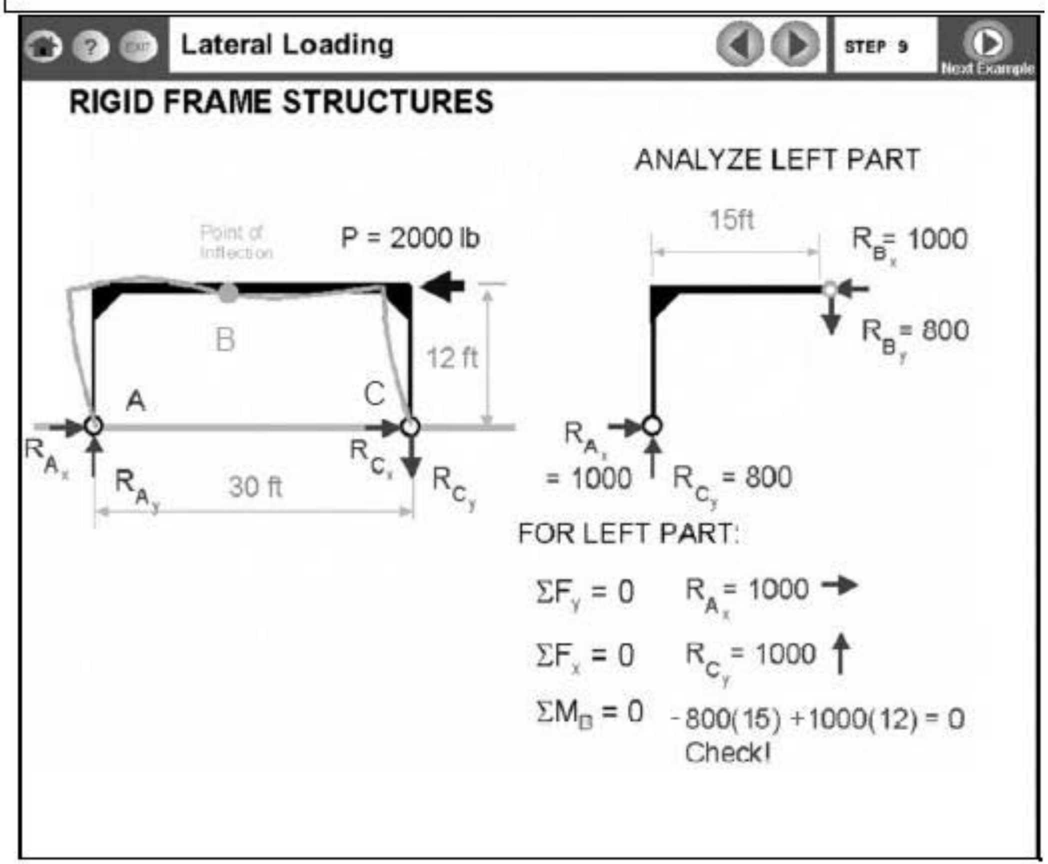
Example 1 (continued)



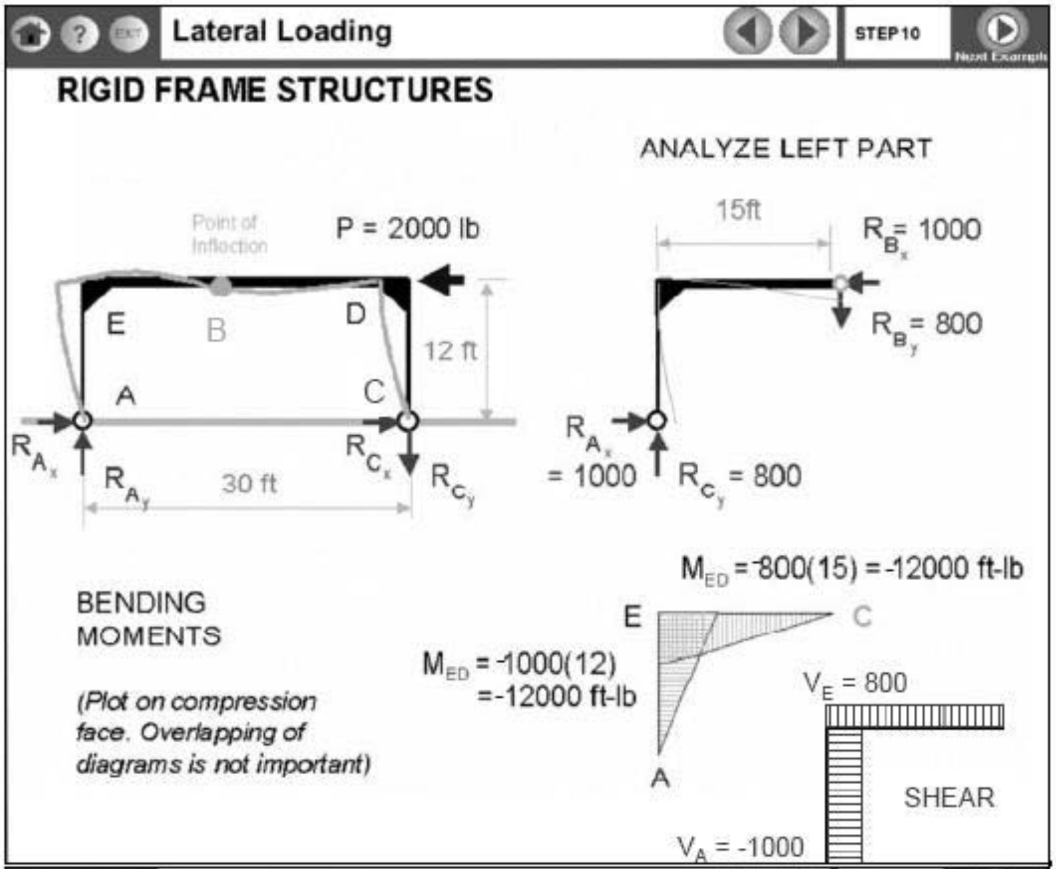


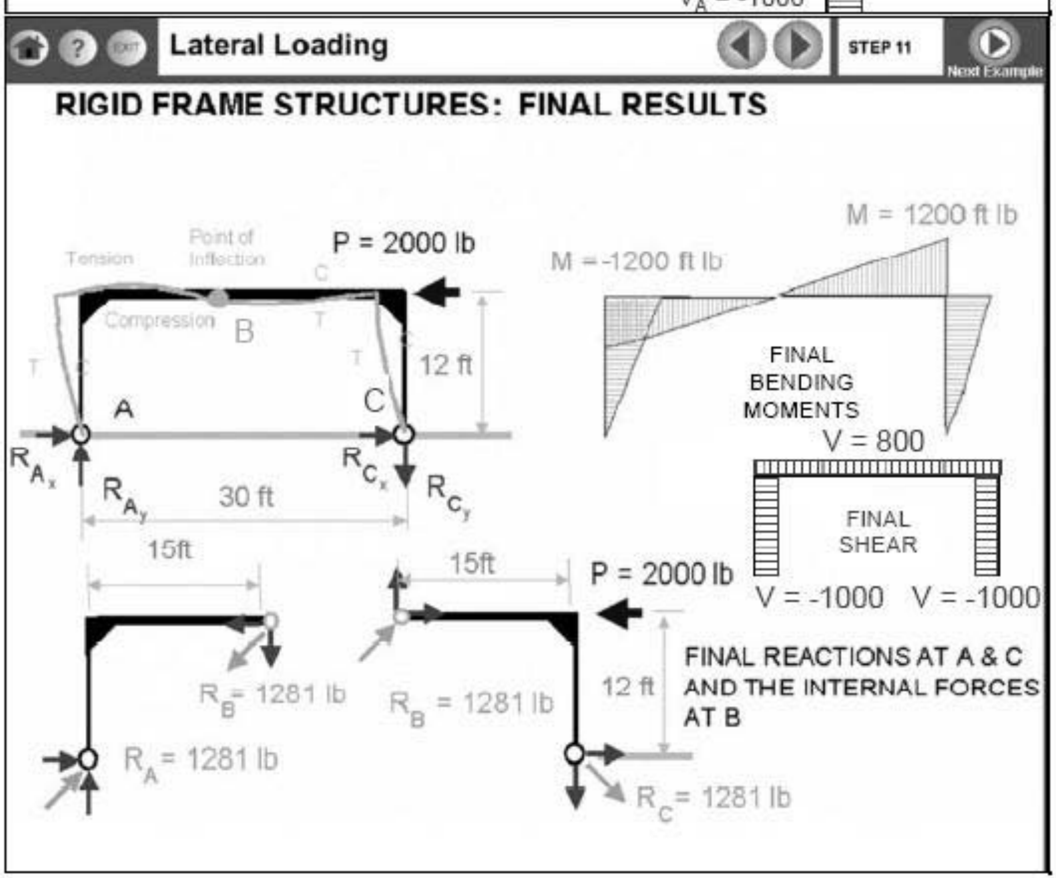
Example 1 (continued)





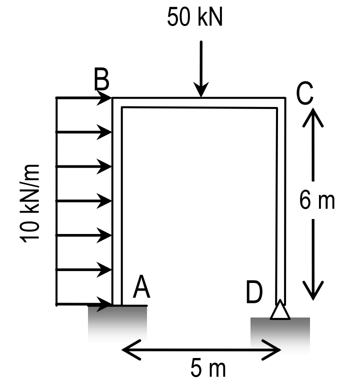
Example 1 (continued)





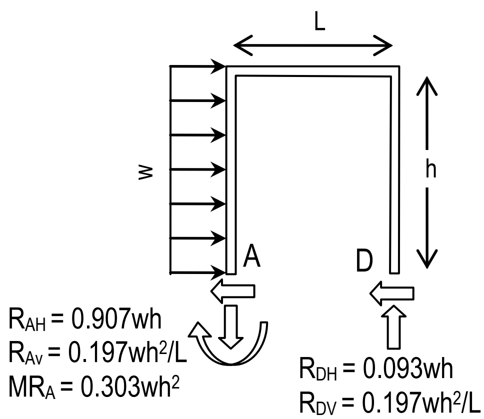
Example 2

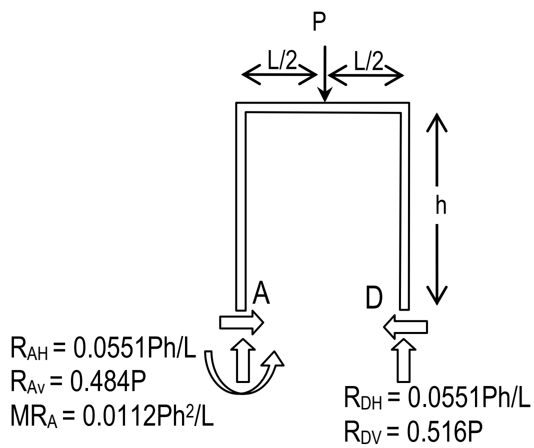
The rigid frame shown at the right has the loading and supports as show. Using superpositioning from approximate analysis methods, draw the shear and bending moment diagrams.



Solution:

Reactions The two loading situations for which approximate reaction values are available are shown below. These values must be calculated *and added together* (allowed by superpositioning).





R_{AH} = -0.907wh + 0.0551Ph/L = $-0.907(10^{kN/m})(6m)+\dfrac{0.0551(50kN)(6m)}{5m}$ = -51.11 kN

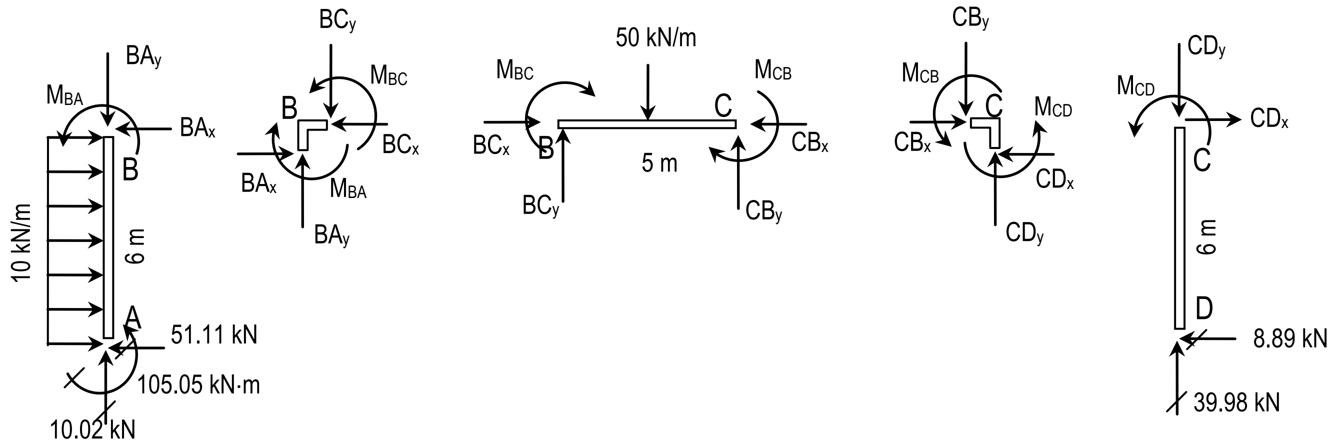
R_{AV} = -0.197wh^2/L + 0.484P = $\dfrac{-0.197(10^{kN/m})(6m)^2}{5m}+0.484(50kN)$ = 10.02 kN

MR_A = -0.303wh^2 + 0.0112Ph^2/L = $-0.303(10^{kN/m})(6m)^2+\dfrac{0.0112(50kN)(6m)^2}{5m}$ = -105.05 kN-m

R_{DH} = -0.093wh - 0.0551Ph/L = $-0.093(10^{kN/m})(6m)-\dfrac{0.0551(50kN)(6m)}{5m}$ = -8.89 kN

R_{DV} = 0.197wh^2/L + 0.516P = $\dfrac{0.197(10^{kN/m})(6m)^2}{5m}+0.516(50kN)$ = 39.98 kN

Member End Forces The free-body diagrams of all the members and joints of the frame are shown below. The unknowns on the members are drawn as anticipated, and the opposite directions are drawn on the joint. We can begin the computation of internal forces with either member AB or CD, both of which have only three unknowns.



Member AB With the magnitudes of reaction forces at A know, the unknowns are at end B of BA_x, BA_y, and M_{BA}, which can get determined by applying $\sum F_x = 0$, $\sum F_y = 0$, and $\sum M_B = 0$. Thus,

$\sum F_x = -51.11kN + 10kN(6m) - BA_x = 0$ BA_x = 8.89 kN, $\sum F_y = 10.02kN - BA_y = 0$ BAy = 10.02 kN

$\sum M_A = 105.05^{kN-m} - 10^{kN/m}(6m)(3m) + 8.89kN(6m) + M_{BA} = 0$ M_{BA} = 21.16kN-m

Joint B Because the forces and moments must be equal and opposite, BC_x = 8.89 kN, BC_y = 10.02 kN and M_{BC} = 21.16 kN·m

Member CD With the magnitudes of reaction forces at D know, the unknowns are at end C of CD_x, CD_y, and M_{CD}, which can get determined by applying $\sum F_x = 0$, $\sum F_y = 0$, and $\sum M_B = 0$. Thus,

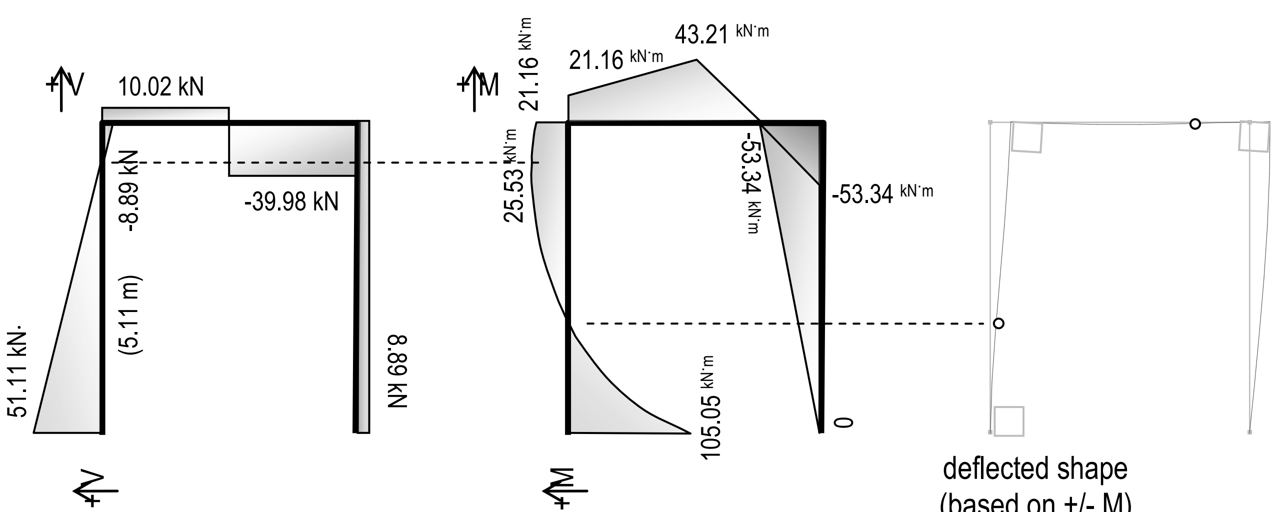
$\sum F_x = -8.89kN + CD_x = 0$ CD_x = 8.89 kN, $\sum F_y = 39.98kN - CD_y = 0$ CD_y = 39.98 kN

$\sum M_D = -8.89kN(6m) + M_{CD} = 0$ M_{DC} = 53.34 kN-m

Joint C Because the forces and moments must be equal and opposite, CB_x = 8.89 kN, CB_y = 39.98 kN and M_{CB} = 53.34 kN-m

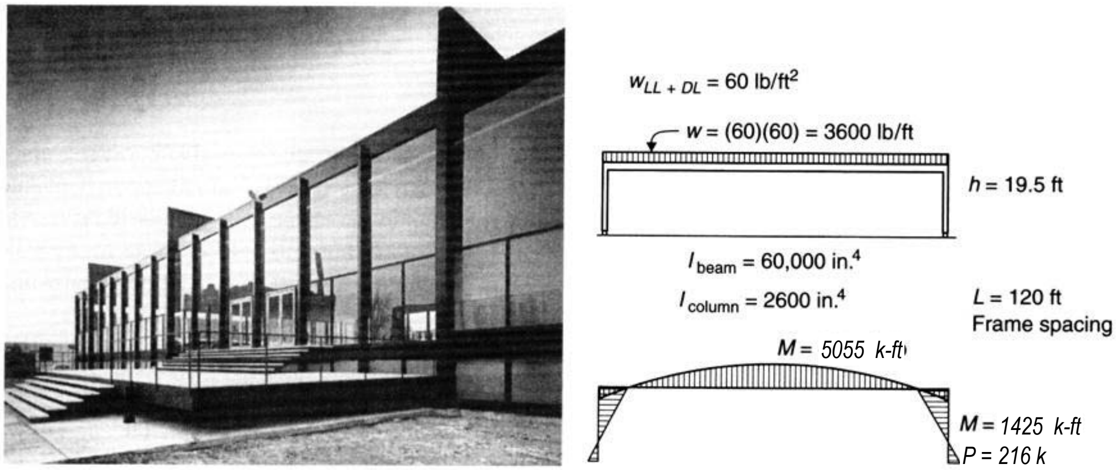
Member BC All forces are known, so equilibrium can be checked.

***(Remember*:** To find the point of zero shear with a distributed load, divide the peak {triangle} shear by the distributed load; ex. 51.11kN/($10^{kN/m}$) = 5.11 m)



Example 3

Using Multiframe4D, verify the bending moment diagram for the example in Figure 9.9:



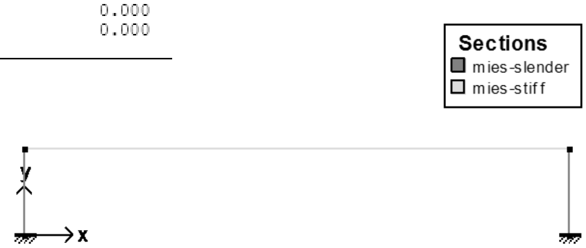
(a) Crown Hall (b) Results of structural analysis

Figure 9.9 The moment distribution illustrates the importance of relative stiffness considerations. The values obtained are quite different from those obtained by estimating points of inflection and using hand calculations.

Joint Coordinates (ft)

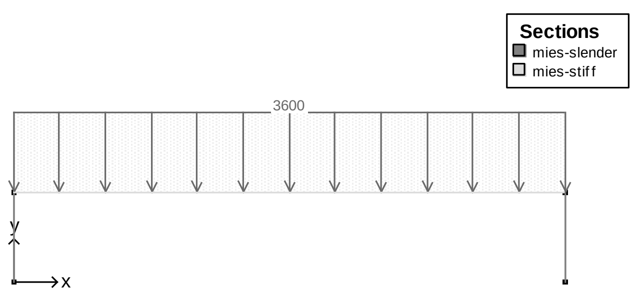
Joint	*Label*	*x*	*y*	*z*
1		0.000	0.000	0.000
2		0.000	19.500	0.000
3		120.000	19.500	0.000
4		120.000	0.000	0.000

Assuming steel (E = 29,000 ksi)

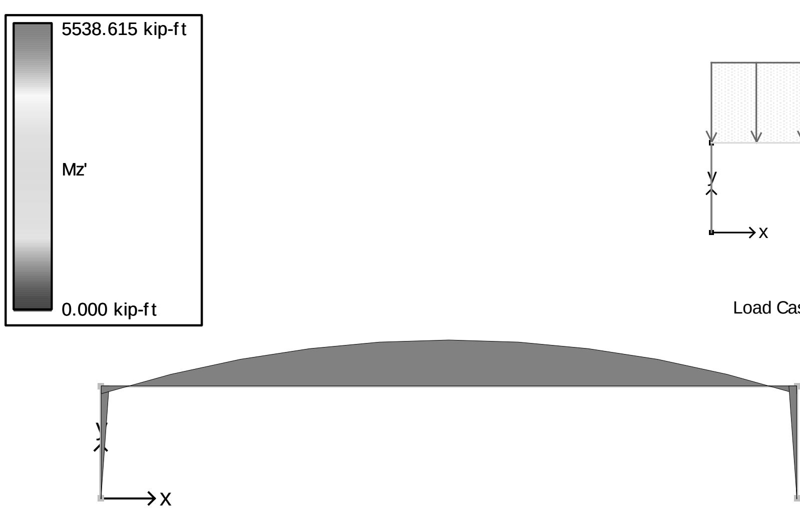


Section Properties

Section	*A* in²	*Ix* in^4	*Ix* in^4
mies-slender	1.000	2380.000	2380.000
mies-stiff	1.000	58700.001	58700.001

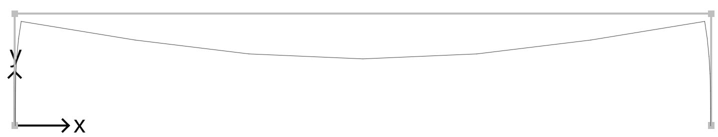


Load Case 1



Example 3 (continued)

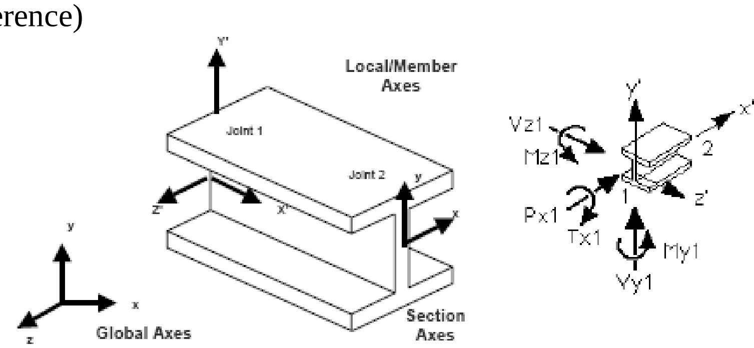
Displacement:



Maximum Actions for all members (column-1, beam-2, column-3):

	Memb	Label	Section	Sign	Px' kip	Vy' kip	Vz' kip	Tx' kip-ft	My' kip-ft	Mz' kip-ft	dy' in	dz' in
1	**1**		**mies-slender**	**+ve**	**216.000**	**0.000**	**0.000**	**0.000**	**0.000**	**1424.716**	**0.486**	**0.000**
2	1		mies-slender	-ve	0.000	-109.079	0.000	0.000	0.000	-702.318	-0.032	0.000
3	1		mies-slender	abs	216.000	109.079	0.000	0.000	0.000	1424.716	0.486	0.000
4	**2**		**mies-stiff**	**+ve**	**109.079**	**216.000**	**0.000**	**0.000**	**0.000**	**1424.716**	**0.000**	**0.000**
5	2		mies-stiff	-ve	0.000	-216.000	0.000	0.000	0.000	-5055.282	-7.326	0.000
6	2		mies-stiff	abs	109.079	216.000	0.000	0.000	0.000	5055.282	7.326	0.000
7	**3**		**mies-slender**	**+ve**	**216.000**	**109.079**	**0.000**	**0.000**	**0.000**	**702.318**	**0.032**	**0.000**
8	3		mies-slender	-ve	0.000	0.000	0.000	0.000	0.000	-1424.716	-0.486	0.000
9	3		mies-slender	abs	216.000	109.079	0.000	0.000	0.000	1424.716	0.486	0.000

(axes orientation reference)



Maximum Stresses for all members (column-1, beam-2, column-3):

	Memb	Label	Section	Sign	Sbz' top ksi	Sbz' bot ksi	Sx' ksi	Sx'+Sbz' top ksi	Sx'+Sbz' bot ksi	dy' in	dz' in
1	**1**		**mies-sl**	**+ve**	**42.494**	**86.203**	**7.714**	**50.208**	**93.917**	**0.486**	**0.000**
2	1		mies-slen	-ve	-86.203	-42.494	0.000	-78.489	-34.780	-0.032	0.000
3	1		mies-slen	abs	86.203	86.203	7.714	78.489	93.917	0.486	0.000
4	**2**		**mies-sti**	**+ve**	**38.237**	**10.776**	**1.283**	**39.521**	**12.060**	**0.000**	**0.000**
5	2		mies-stiff	-ve	-10.776	-38.237	0.000	-9.493	-36.954	-7.326	0.000
6	2		mies-stiff	abs	38.237	38.237	1.283	39.521	36.954	7.326	0.000
7	**3**		**mies-sl**	**+ve**	**86.203**	**42.494**	**7.714**	**93.917**	**50.208**	**0.032**	**0.000**
8	3		mies-slen	-ve	-42.494	-86.203	0.000	-34.780	-78.489	-0.486	0.000
9	3		mies-slen	abs	86.203	86.203	7.714	93.917	78.489	0.486	0.000

Beam-Column stress verification (combined stresses) when d = 24 in, A = 28 in^2. I_x = 2380 in^4:

$$f_{max} = \frac{P}{A} + \frac{M}{S} = \frac{P}{A} + \frac{Mc}{I} = \frac{216k}{28in^2} + \frac{1425^{k-ft} \cdot (24in/2)}{2380in^4} \cdot \frac{12in}{ft} = 7.71ksi + 86.22ksi = 93.93ksi$$

Frame Analysis by Coefficients and Live Load Reduction
from Simplified Design, 3rd ed., Portland Cement Association, 2004

2.2.2 Live Load Reduction for Columns, Beams, and Slabs

Most general building codes permit a reduction in live load for design of columns, beams and slabs to account for the probability that the total floor area "influencing" the load on a member may not be fully loaded simultaneously. Traditionally, the reduced amount of live load for which a member must be designed has been based on a tributary floor area supported by that member. According to ASCE 7-02, the magnitude of live load reduction is based on an influence area rather than a tributary area. The influence area is a function of the tributary area for the structural member. The influence area for different structural members is calculated by multiplying the tributary area for the member A_T, by the coefficients K_{LL} given in Table 2-3, see ASCE 4.8.

The reduced live load L per square foot of floor area supported by columns, beams, and two-way slabs having an influence area ($K_{LL}A_T$) of more than 400 sq ft is:

$$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \qquad \text{ASCE (Eq. 4-1)}$$

where L_o is the unreduced design live load per square foot. The reduced live load cannot be taken less than 50% for members supporting one floor, or less than 40% of the unit live load L_o otherwise. For other limitations on live load reduction, see ASCE 4.8.

Using the above expression for reduced live load, values of the reduction multiplier as a function of influence area are given in Table 2-4.

Table 2-3 Live Load Element Factor K_{LL}

Element	K_{LL}
Interior columns	4
Exterior column without cantilever slabs	4
Edge column with cantilever slabs	3
Corner columns with cantilever slabs	2
Edge beams without cantilever slabs	2
Interior beams	2
All other members not identified above including: Edge beams with cantilever slabs Cantilever beams Two-way slabs	1

The above limitations on permissible reduction of live loads are based on ASCE 4.8. The governing general building code should be consulted for any difference in amount of reduction and type of members that may be designed for a reduced live load.

2.3 FRAME ANALYSIS BY COEFFICIENTS

The ACI Code provides a simplified method of analysis for both one-way construction (ACI 8.3.3) and two-way construction (ACI 13.6). Both simplified methods yield moments and shears based on coefficients. Each method will give satisfactory results within the span and loading limitations stated in Chapter 1. The direct design method for two-way slabs is discussed in Chapter 4.

Table 2-4 Reduction Multiplier (RM) for Live Load = $\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$

Influence Area $K_{LL}A_T$	RM	Influence Area $K_{LL}A_T$	RM
400[a]	1.000	5600	0.450
800	0.780	6000	0.444
1200	0.683	6400	0.438
1600	0.625	6800	0.432
2000	0.585	7200	0.427
2400	0.556	7600	0.422
2800	0.533	8000	0.418
3200	0.515	8400	0.414
3600	0.500[b]	8800	0.410
4000	0.487	9200	0.406
4800	0.467	10000	0.400[c]
5200	0.458		

[a]No live load reduction is permitted for influence area less than 400 sq ft.
[b]Maximum reduction permitted for members supporting one floor only.
[c]Maximum absolute reduction.

2.3.1 Continuous Beams and One-Way Slabs

When beams and one-way slabs are part of a frame or continuous construction, ACI 8.3.3 provides approximate moment and shear coefficients for gravity load analysis. The approximate coefficients may be used as long as all of the conditions illustrated in Fig. 2-2 are satisfied: (1) There must be two or more spans, approximately equal in length, with the longer of two adjacent spans not exceeding the shorter by more than 20 percent; (2) loads must be uniformly distributed, with the service live load not more than 3 times the dead load ($L/D \le 3$); and (3) members must have uniform cross section throughout the span. Also, no redistribution of moments is permitted (ACI 8.4). The moment coefficients defined in ACI 8.3.3 are shown in Figs. 2-3 through 2-6. In all cases, the shear in end span members at the interior support is taken equal to $1.15w_u\ell_n/2$. The shear at all other supports is $w_u/2$ (see Fig. 2-7). $w_u\ell_n$ is the combined factored load for dead and live loads, $w_u = 1.2w_d + 1.6\,w_\ell$. For beams, w_u is the uniformly distributed load per unit length of beam (plf), and the coefficients yield total moments and shears on the beam. For one-way slabs, w_u is the uniformly distributed load per unit area of slab (psf), and the moments and shears are for slab strips one foot in width. The span length ℓ_n is defined as the clear span of the beam or slab. For negative moment at a support with unequal adjacent spans, ℓ_n is the average of the adjacent clear spans. Support moments and shears are at the faces of supports.

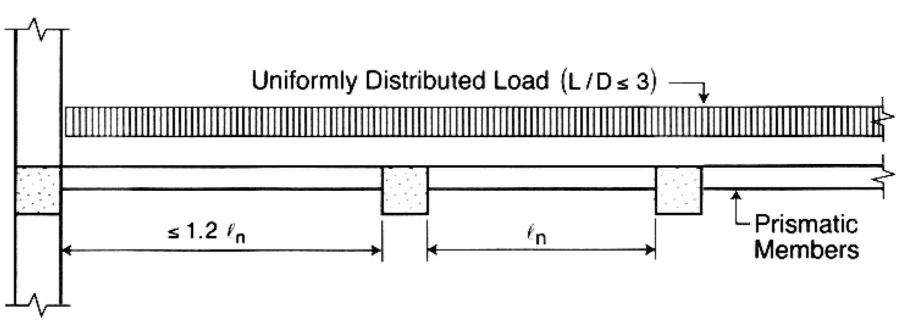


Figure 2-2 Conditions for Analysis by Coefficients (ACI 8.3.3)

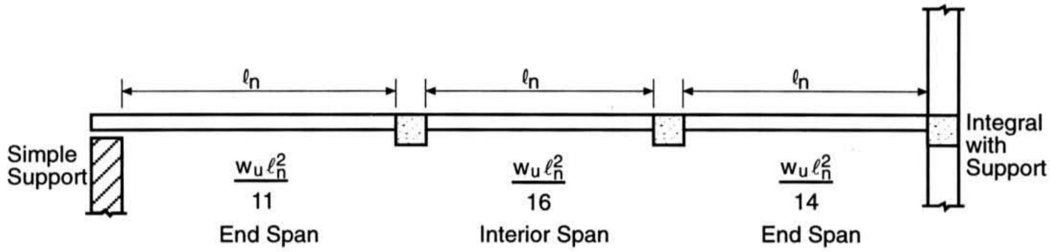


Figure 2-3 Positive Moments—All Cases

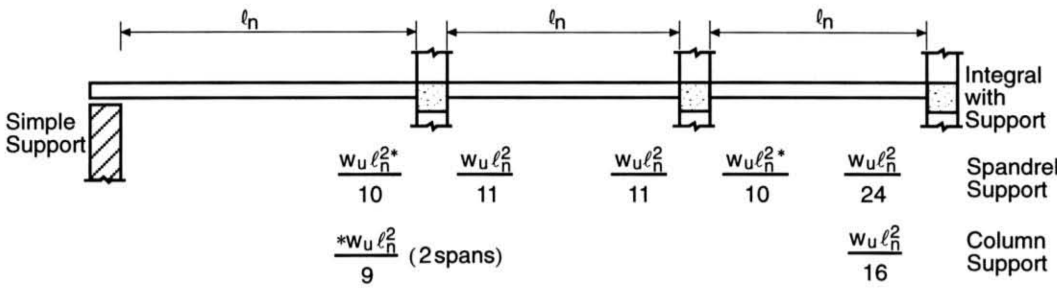


Figure 2-4 Negative Moments—Beams and Slabs

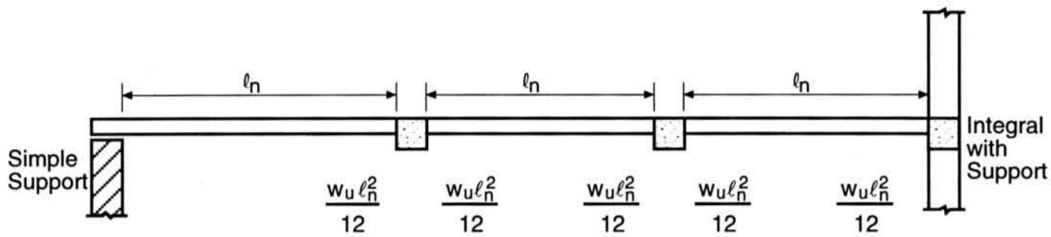


Figure 2-5 Negative Moments—Slabs with spans ≤ 10 ft

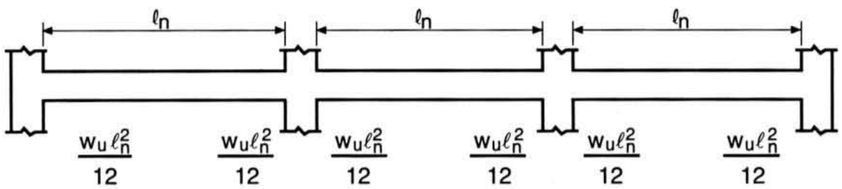


Figure 2-6 Negative Moments—Beams with Stiff Columns ($\Sigma K_c/\Sigma K_b > 8$)

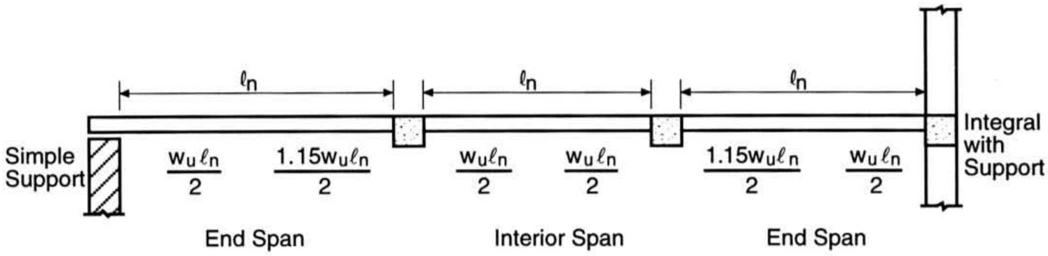


Figure 2-7 End Shears—All Cases

4.3 TWO-WAY SLAB ANALYSIS BY COEFFICIENTS

For gravity loads, ACI Chapter 13 provides two analysis methods for two-way slab systems: 1) the Direct Design Method (ACI 13.6) and the Equivalent Frame Method (ACI 13.7). The Equivalent Frame Method, using member stiffnesses and complex analytical procedures, is not suitable for hand calculations. Only the Direct Design Method, using moment coefficients, will be presented in this Chapter.

Table 4-1 Minimum Thickness for Two-Way Slab Systems

Two-Way Slab System		α_m	β	Minimum h
Flat Plate		—	≤ 2	$\ell_n/30$
Flat Plate with Spandrel Beams [1]	[Min. h = 5 in.]	—	≤ 2	$\ell_n/33$
Flat Slab [2]		—	≤ 2	$\ell_n/33$
Flat Slab [2] with Spandrel Beams[1]	[Min. h = 4 in.]	—	≤ 2	$\ell_n/36$
Two-Way Beam-Supported Slab[3]		≤ 0.2	≤ 2	$\ell_n/30$
		1.0	1	$\ell_n/33$
			2	$\ell_n/36$
		≥ 2.0	1	$\ell_n/37$
			2	$\ell_n/44$
Two-Way Beam-Supported Slab[1,3]		≤ 0.2	≤ 2	$\ell_n/33$
		1.0	1	$\ell_n/36$
			2	$\ell_n/40$
		≥ 2.0	1	$\ell_n/41$
			2	$\ell_n/49$

[1]Spandrel beam-to-slab stiffness ratio $\alpha \geq 0.8$ (ACI 9.5.3.3)

[2]Drop panel length $\geq \ell/3$, depth $\geq 1.25h$ (ACI 13.4.7)

[3]Min. h = 5 in. for $\alpha_m \leq 2.0$; min. h = 3.5 in. for $\alpha_m > 2.0$ (ACI 9.5.3.3)

α is the ratio of flexural stiffness of a beam section ti the slab; α_m is the average α for all beams on edges of a panel

β is the ratio of clear spans in long to short direction

The Direct Design Method applies when all of the conditions illustrated in Fig. 4-4 are satisfied (ACI 13.6.1):

- There must be three or more continuous spans in each direction.
- Slab panels must be rectangular with a ratio of longer to shorter span (c/c of supports) not greater than 2.
- Successive span lengths (c/c of supports) in each direction must not differ by more than one-third of the longer span.
- Columns must not be offset more than 10% of the span (in direction of offset) from either axis between centerlines of successive columns.
- Loads must be due to gravity only and must be uniformly distributed over the entire panel. The live load must not be more than 3 times the dead load (L/D $\leq$ 3). Note that if the live load exceeds one-half the dead load (L/D > 0.5), column-to-slab stiffness ratios must exceed the applicable values given in ACI Table 13.6.10, so that the effects of pattern loading can be neglected. The positive factored moments in panels supported by columns not meeting such minimum stiffness requirements must be magnified by a coefficient computed by ACI Eq. (13-5).
- For two-way slabs, relative stiffnesses of beams in two perpendicular directions must satisfy the minimum and maximum requirements given in ACI 13.6.1.6.
- Redistribution of moments by ACI 8.4 shall not be permitted.

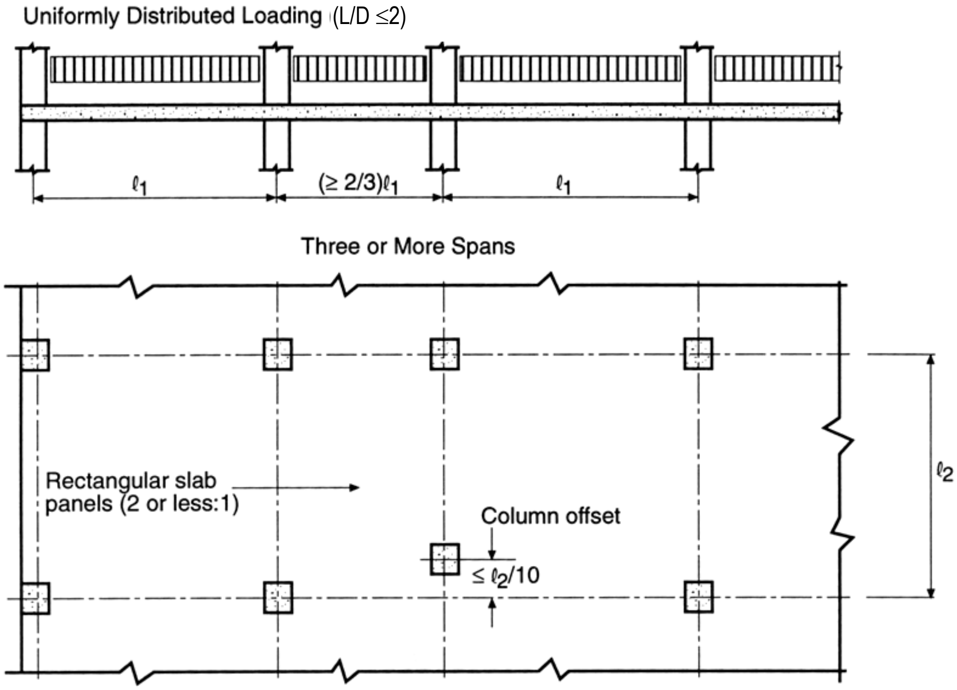


Figure 4-4 Conditions for Analysis by Coefficients

In essence, the Direct Design Method is a three-step analysis procedure. The first step is the calculation of the total design moment M_o for a given panel. The second step involves the distribution of the total moment to the negative and positive moment sections. The third step involves the assignment of the negative and positive moments to the column strips and middle strips.

For uniform loading, the total design moment M_o for a panel is calculated by the simple static moment expression, ACI Eq. (13-3):

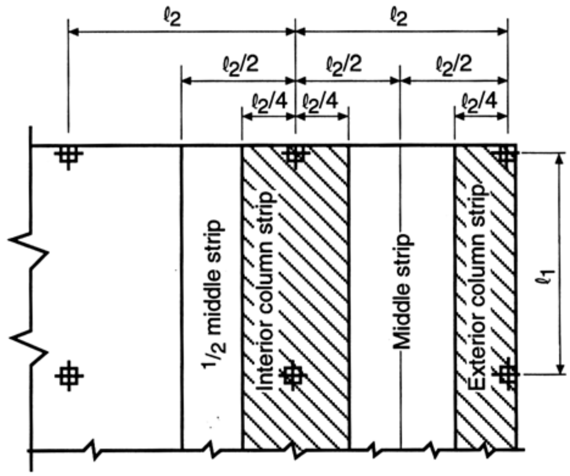
$$M_o = w_u \ell_2 \ell_n^2/8$$

where w_u is the factored combination of dead and live loads (psf), $w_u = 1.2w_d + 1.6w_\ell$. The clear span ℓ_n is defined in a straightforward manner for columns or other supporting elements of rectangular cross section (ACI 12.6.2.5). Note that circular or regular polygon shaped supports shall be treated as square supports with the same area (see ACI Fig. R13.6.2.5). The clear span starts at the face of support. One limitation requires that the clear span never be taken less than 65% of the span center-to-center of supports (ACI 13.6.2.5). The span ℓ_2 is simply the span transverse to ℓ_n; however, when the span adjacent and parallel to an edge is being considered, the distance from edge of slab to panel centerline is used for ℓ_2 in calculation of M_o (ACI 13.6.2.4).

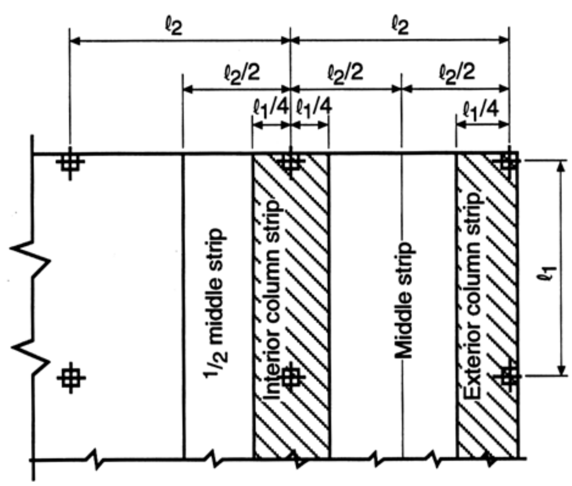
Division of the total panel moment M_o into negative and positive moments, and then into column and middle strip moments, involves direct application of moment coefficients to the total moment M_o. The moment coefficients are a function of span (interior or exterior) and slab support conditions (type of two-way slab system). For design convenience, moment coefficients for typical two-way slab systems are given in Tables 4-2 through 4-6. Tables 4-2 through 4-5 apply to flat plates or flat slabs with various end support conditions. Table 4-6 applies to two-way slabs supported on beams on all four sides. Final moments for the column strip and middle strip are computed directly using the tabulated values. All coefficients were determined using the appropriate distribution factors in ACI 13.6.3 through 13.6.6.

NOTE: The interior column strip is defined by one quarter of the smaller of ℓ_1 and ℓ_2 each side of the column centerline. The exterior column strip is bound by the slab edge and one quarter of the smaller of ℓ_1 and ℓ_2 from the column centerline. The middle strip is the remaining width between column strips.

The column strip and middle strip moments are distributed over an effective slab width as illustrated in Fig. 4-9. The column strip is defined as having a width equal to one-half the transverse or longitudinal span, whichever is smaller (ACI 13.2.1). The middle strip is bounded by two column strips.



(a) Column strip for $\ell_2 \leq \ell_1$

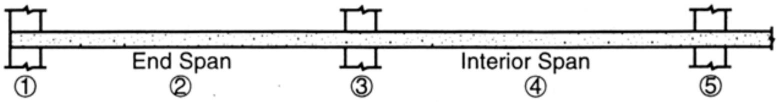


(b) Column strip for $\ell_2 > \ell_1$

Figure 4-9 Definition of Design Strips

Once the slab and beam (if any) moments are determined, design of the slab and beam sections follows the simplified design approach presented in Chapter 3. Slab reinforcement must not be less than that given in Table 3-5, with a maximum spacing of 2h or 18 in. (ACI 13.4).

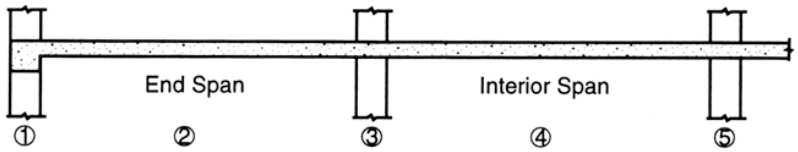
Table 4-2 Flat Plate or Flat Slab Supported Directly on Columns



Slab Moments	End Span: 1 Exterior Negative	End Span: 2 Positive	End Span: 3 First Interior Negative	Interior Span: 4 Positive	Interior Span: 5 Interior Negative
Total Moment	0.26 M_o	0.52 M_o	0.70 M_o	0.35 M_o	0.65 M_o
Column Strip	0.26 M_o	0.31 M_o	0.53 M_o	0.21 M_o	0.49 M_o
Middle Strip	0	0.21 M_o	0.17 M_o	0.14 M_o	0.16 M_o

Note: All negative moments are at face of support.

Table 4-3 Flat Plate or Flat Slab with Spandrel Beams

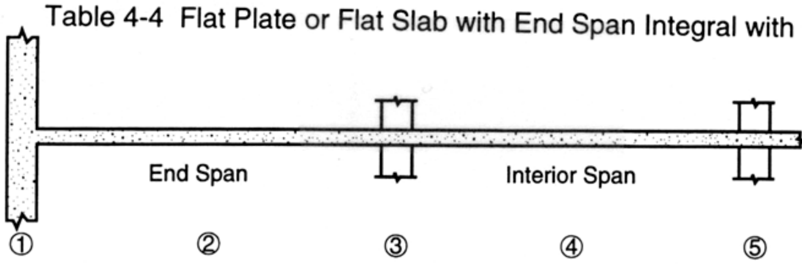


Slab Moments	End Span: 1 Exterior Negative	End Span: 2 Positive	End Span: 3 First Interior Negative	Interior Span: 4 Positive	Interior Span: 5 Interior Negative
Total Moment	0.30 M_o	0.50 M_o	0.70 M_o	0.35 M_o	0.65 M_o
Column Strip	0.23 M_o	0.30 M_o	0.53 M_o	0.21 M_o	0.49 M_o
Middle Strip	0.07 M_o	0.20 M_o	0.17 M_o	0.14 M_o	0.16 M_o

Notes: (1) All negative moments are at face of support.

(2) Torsional stiffness of spandrel beams $\beta_t \geq 2.5$. For values of β_t less than 2.5, exterior negative column strip moment increases to $(0.30 - 0.03\beta_t)\, M_o$.

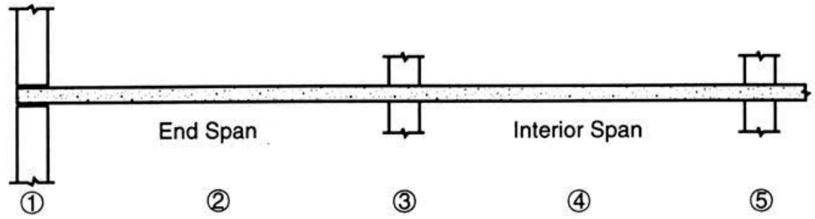
Table 4-4 Flat Plate or Flat Slab with End Span Integral with Wall



Slab Moments	End Span: 1 Exterior Negative	End Span: 2 Positive	End Span: 3 First Interior Negative	Interior Span: 4 Positive	Interior Span: 5 Interior Negative
Total Moment	0.65 M_o	0.35 M_o	0.65 M_o	0.35 M_o	0.65 M_o
Column Strip	0.49 M_o	0.21 M_o	0.49 M_o	0.21 M_o	0.49 M_o
Middle Strip	0.16 M_o	0.14 M_o	0.16 M_o	0.14 M_o	0.16 M_o

Note: All negative moments are at face of support.

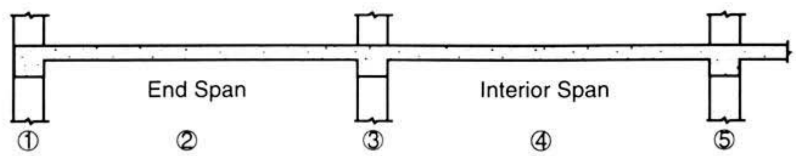
Table 4-5 Flat Plate or Flat Slab with End Span Simply Supported on Wall



Slab Moments	End Span: 1 Exterior Negative	End Span: 2 Positive	End Span: 3 First Interior Negative	Interior Span: 4 Positive	Interior Span: 5 Interior Negative
Total Moment	0	0.63 M_o	0.75 M_o	0.35 M_o	0.65 M_o
Column Strip	0	0.38 M_o	0.56 M_o	0.21 M_o	0.49 M_o
Middle Strip	0	0.25 M_o	0.19 M_o	0.14 M_o	0.16 M_o

Note: All negative moments are at face of support.

Table 4-6 Two-Way Beam-Supported Slab



Span ratio ℓ_2/ℓ_1	Slab Moments	End Span: 1 Exterior Negative	End Span: 2 Positive	End Span: 3 First Interior Negative	Interior Span: 4 Positive	Interior Span: 5 Interior Negative
	Total Moment	0.16 M_o	0.57 M_o	0.70 M_o	0.35 M_o	0.65 M_o
0.5	Column Strip Beam	0.12 M_o	0.43 M_o	0.54 M_o	0.27 M_o	0.50 M_o
	Column Strip Slab	0.02 M_o	0.08 M_o	0.09 M_o	0.05 M_o	0.09 M_o
	Middle Strip	0.02 M_o	0.06 M_o	0.07 M_o	0.03 M_o	0.06 M_o
1.0	Column Strip Beam	0.10 M_o	0.37 M_o	0.45 M_o	0.22 M_o	0.42 M_o
	Column Strip Slab	0.02 M_o	0.06 M_o	0.08 M_o	0.04 M_o	0.07 M_o
	Middle Strip	0.04 M_o	0.14 M_o	0.17 M_o	0.09 M_o	0.16 M_o
2.0	Column Strip Beam	0.06 M_o	0.22 M_o	0.27 M_o	0.14 M_o	0.25 M_o
	Column Strip Slab	0.01 M_o	0.04 M_o	0.05 M_o	0.02 M_o	0.04 M_o
	Middle Strip	0.09 M_o	0.31 M_o	0.38 M_o	0.19 M_o	0.36 M_o

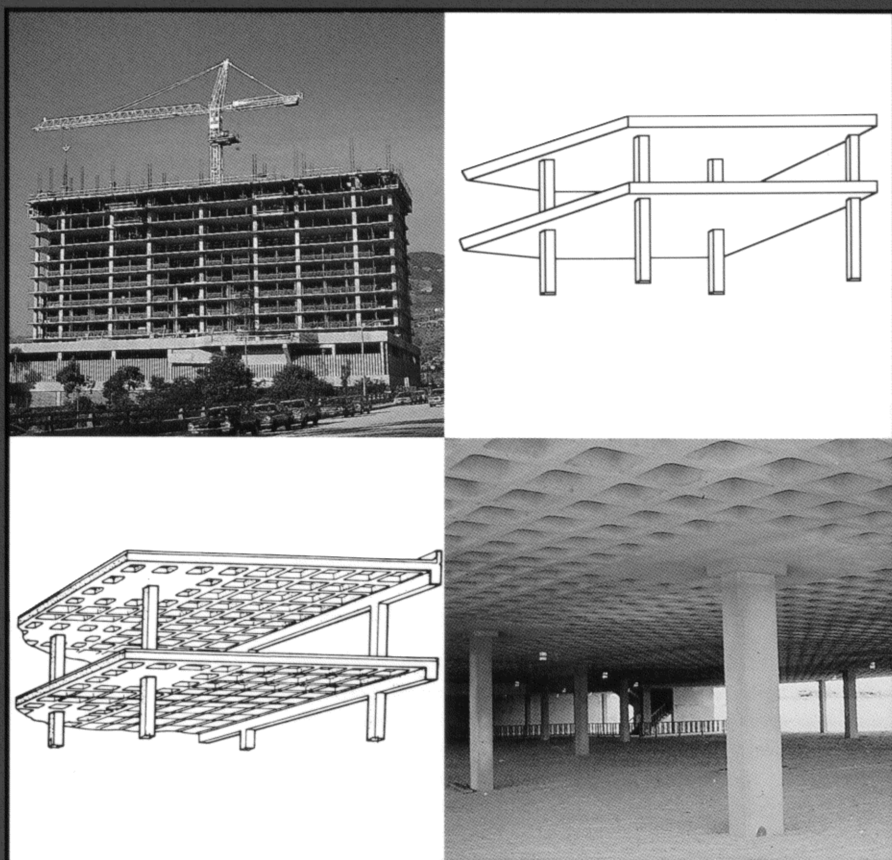
Notes: (1) Beams and slab satisfy stiffness criteria: $\alpha_1\ell_2/\ell_1 \geq 1.0$ and $\beta_t \geq 2.5$.
(2) Interpolate between values shown for different ℓ_2/ℓ_1 ratios.
(3) All negative moments are at face of support.
(4) Concentrated loads applied directly to beams must be accounted for separately.

PORTLAND CEMENT ASSOCIATION

Concrete Floor Systems

GUIDE TO ESTIMATING AND ECONOMIZING

By August W. Domel Jr. and S.K. Ghosh



INTRODUCTION

The main objectives of this publication are to:

- Assist in the selection of the most economical cast-in-place concrete floor system for a given plan layout and a given set of loads;
- Provide a preliminary estimate of material quantities for the floor system; and
- Discuss the effect of different variables in the selection process.

Five different floor systems are considered in this publication. These are the flat plate, the flat slab, the one-way joist, the two-way joist or waffle, and the slab supported on beams on all four sides. Material quantity estimates are given for each floor system for various bay sizes.

Pricing Trends

The total cost to construct a building depends on the use for which the structure is designed, the availability of qualified contractors, and the part of the country in which the structure is built. Figure 1 gives cost comparisons for two different types of uses over the past several years. (The data presented in Figures 1 through 5 and Table 1 were obtained from Means Concrete Cost Data, 1990.) The average price per square foot is considerably greater for office buildings than for apartment buildings. Part of the higher cost is because office buildings are designed with more open spaces which in structural terms means costlier, longer clear spans.

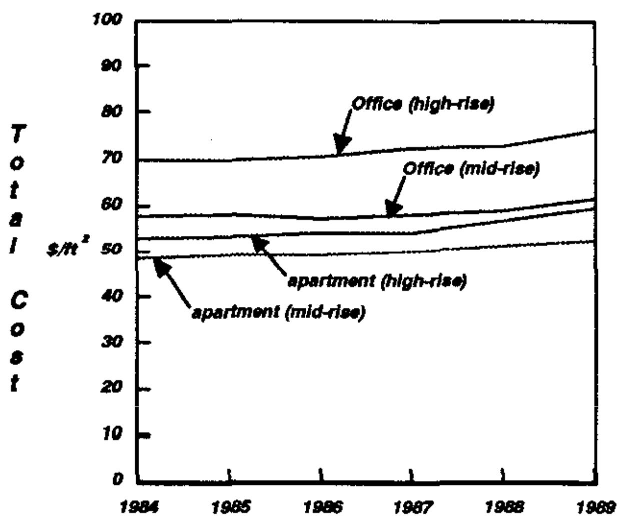


Figure 1 - Price Comparisons for Different Building Types

Table 1 gives cost indices for many major cities in the United States and Canada. The cost index includes both labor and materials, with the value of 100 representing the average cost for 30 major cities. The table shows the wide variation in costs depending on the locale. In Anchorage, Alaska (127.9) or New York City (126.9) the cost of a building can be as much as 60% higher than that of a similar building in Charleston, South Carolina (80.2), Jackson, Mississippi (81) or Sioux Falls, South Dakota (82.2). Figure 2 shows the relative change in costs in current dollars of material and labor over the past 40 years.

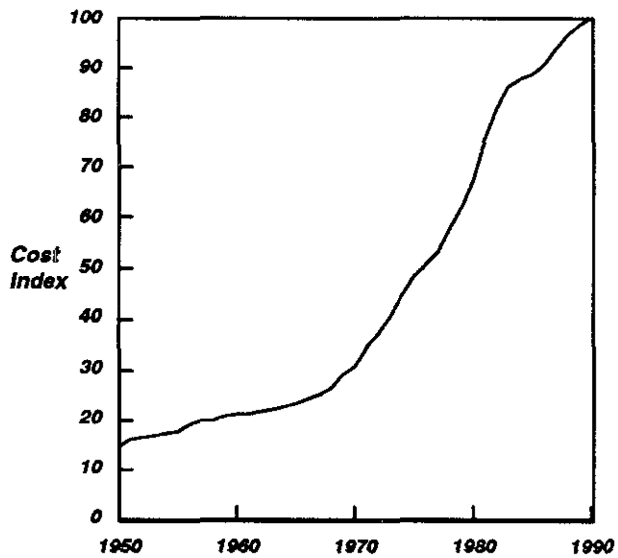


Figure 2 - Annual Construction Cost Comparisons

The majority of the structural cost of a building typically is the cost of the floor system. This is particularly true of low-rise buildings and buildings in low seismic zones. Therefore, it is imperative to select the most economical floor system.

In this publication, estimated quantities are provided for concrete, reinforcing steel and formwork for the five floor systems discussed in the following sections. Prices for labor and material for these items over the past several years are shown in Figures 3 through 5.

Table 1—Relative Construction Costs for Reinforced Concrete

ALABAMA (BIRMINGHAM)	84.0	NEW HAMPSHIRE (MANCHESTER)	90.3
ALASKA (ANCHORAGE)	127.9	NEW JERSEY (NEWARK)	104.9
ARIZONA (PHOENIX)	91.9	NEW MEXICO (ALBUQUERQUE)	91.5
ARKANSAS (LITTLE ROCK)	84.5	NEW YORK (NEW YORK)	126.9
CALIFORNIA (LOS ANGELES)	112.0	NEW YORK (ALBANY)	94.5
CALIFORNIA (SAN FRANCISCO)	126.0	NORTH CAROLINA (CHARLOTTE)	80.8
COLORADO (DENVER)	93.5	OHIO (CLEVELAND)	107.3
CONNECTICUT (HARTFORD)	100.1	OHIO (CINCINNATI)	95.3
DELAWARE (WILMINGTON)	100.3	OKLAHOMA (OKLAHOMA CITY)	89.4
WASHINGTON, D.C.	95.4	OREGON (PORTLAND)	101.0
FLORIDA (MIAMI)	89.9	PENNSYLVANIA (PHILADELPHIA)	107.2
GEORGIA (ATLANTA)	89.7	PENNSYLVANIA (PITTSBURGH)	100.6
HAWAII (HONOLULU)	111.1	RHODE ISLAND (PROVIDENCE)	100.8
IDAHO (BOISE)	93.3	SOUTH CAROLINA (CHARLESTON)	80.2
ILLINOIS (CHICAGO)	101.8	SOUTH DAKOTA (SIOUX FALLS)	82.2
INDIANA (INDIANAPOLIS)	97.6	TENNESSEE (MEMPHIS)	87.6
IOWA (DES MOINES)	90.7	TEXAS (DALLAS)	87.8
KANSAS (WICHITA)	86.8	UTAH (SALT LAKE CITY)	91.7
KENTUCKY (LOUISVILLE)	88.3	VERMONT (BURLINGTON)	88.1
LOUISIANA (NEW ORLEANS)	88.6	VIRGINIA (NORFOLK)	83.3
MAINE (PORTLAND)	89.8	WASHINGTON (SEATTLE)	101.6
MARYLAND (BALTIMORE)	96.1	WEST VIRGINIA (CHARLESTON)	97.4
MASSACHUSETTS (BOSTON)	115.6	WISCONSIN (MILWAUKEE)	97.3
MICHIGAN (DETROIT)	106.9	WYOMING (CHEYENNE)	87.4
MINNESOTA (MINNEAPOLIS)	99.4	CANADA (EDMONTON)	100.2
MISSISSIPPI (JACKSON)	81.0	CANADA (MONTREAL)	100.0
MISSOURI (ST. LOUIS)	101.6	CANADA (QUEBEC)	99.0
MONTANA (BILLINGS)	92.1	CANADA (TORONTO)	109.8
NEBRASKA (OMAHA)	88.6	CANADA (VANCOUVER)	105.5
NEVADA (LAS VEGAS)	104.6	CANADA (WINNIPEG)	101.5

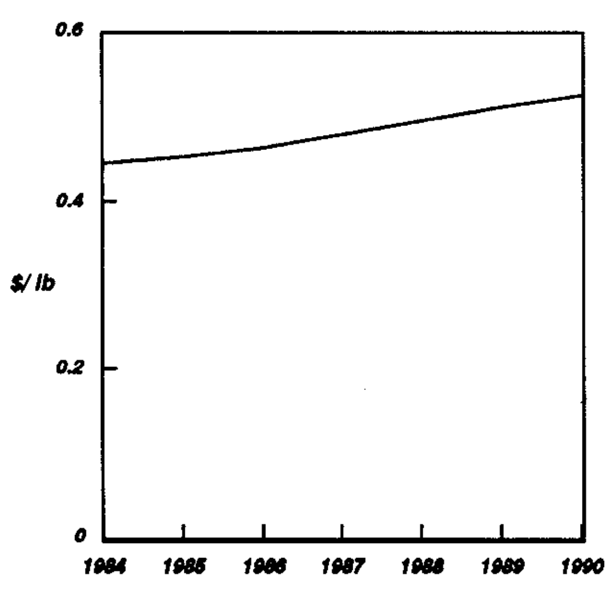


Figure 3 - Cost of Reinforcing Bars in Place

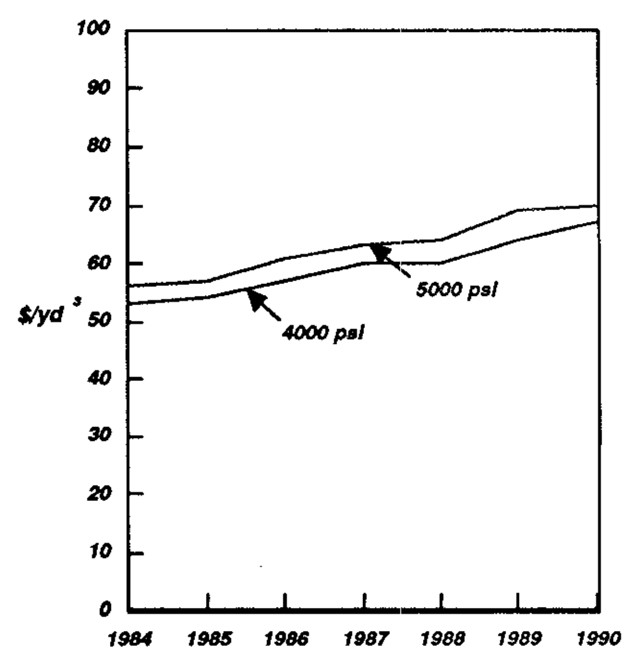


Figure 4 - Cost of Ready-Mixed Concrete

INTRODUCTION

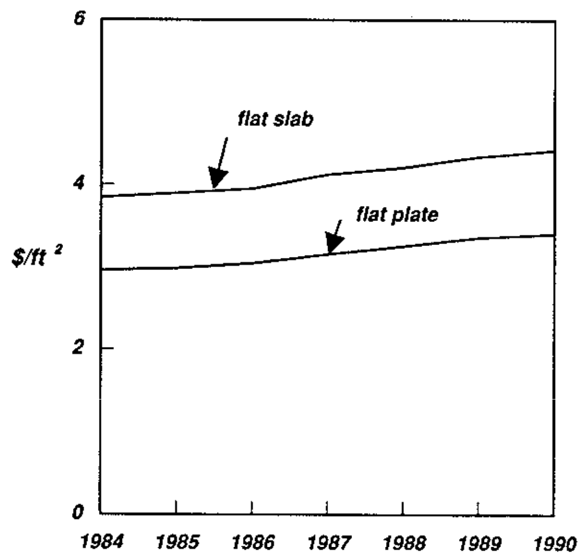


Figure 5 - Cost of Formwork

Presentation of Results

The following pages provide discussion and quantity estimates for the five floor systems. These results were obtained using a five bay by five bay structure. Bay sizes are measured from centerline of column to centerline of column. Floors were designed using ACI 318-89 Building Code Requirements for Reinforced Concrete. Concrete, reinforcing steel and formwork quantities are presented for each of the floor systems. An overview of the floor systems is provided, following the discussion of the floor systems.

Included with each floor system is a discussion of the factors that may affect the estimated quantities. The factors discussed are column dimensions, live loads, and aspect ratios. A cost breakdown is also given in each case. Following the discussion for each individual floor system are several tables and graphs. The graphs show the variation in costs for increased bay size and higher concrete strength. The tables give quantities for various bay sizes.

Fire Resistance of Concrete Floor Systems

Fire resistance rated construction will often be required by the governing building code, or the owner may desire a highly fire resistant structure in order to qualify for the lowest fire insurance rates.

Concrete floor systems offer inherent fire resistance. Therefore, when the floor system is completed, no additional protective measures are necessary in order to achieve code required fire resistance ratings.

On the other hand, for steel floor systems for instance, additional protection must be provided by special acoustical ceilings, or fireproofing sprayed on the underside of the steel deck and/or beams. In addition, when an acoustical ceiling is an integral part of a rated floor/ceiling assembly, special ceiling suspension systems, and special protective devices at penetrations for light fixtures and HVAC diffusers are required.

These additional costs associated with protecting the structural framing members must be added to the cost of the structural frame to produce an accurate cost estimate. If this is not done, the actual cost of the competing floor system is understated, making a valid comparison with a concrete floor system difficult, if not impossible.

Fire resistance rating requirements vary from zero to four hours, with two hours typically being required for high rise buildings. Before selecting the floor system, the designer should determine the fire resistance rating required by the applicable building code. Except for one-way and two-way joist systems, the minimum slab thickness necessary to satisfy structural requirements (usually 5 in.) will normally provide a floor system that has at least a two hour fire resistance rating.

Table 2 shows minimum slab thicknesses necessary to provide fire resistance ratings from one to four hours, for different types of aggregate. If the thickness necessary to satisfy fire resistance requirements exceeds that required for structural purposes, consideration should be given to using a different type of aggregate that provides higher fire resistance for the same thickness. For example, a one-way joist system may require a 3 in. thick slab to satisfy structural requirements. However, if a two hour fire resistance rating is desired, a 5 in. thick slab will be required if siliceous aggregate normal weight concrete is used. By using lightweight aggregate concrete, the slab thickness can be reduced to 3.6 in. This 28% reduction in thickness translates into approximately a 45% reduction in dead load.

Table 2—Minimum Slab Thickness for Fire Resistance Rating

Floor Construction Material	Minimum slab thickness (in.) for fire-resistance rating			
	1 hr	2 hr	3 hr	4 hr
Siliceous Aggregate Concrete	3.5	5.0	6.2	7.0
Carbonate Aggregate Concrete	3.2	4.6	5.7	6.6
Sand-lightweight Concrete	2.7	3.8	4.6	5.4
Lightweight Concrete	2.5	3.6	4.4	5.1

Adequate cover must be provided to keep reinforcing steel temperatures within code prescribed limits. The amount of cover depends on the element considered (i.e., slab, joist or beam), and whether the element is restrained against thermal expansion. All elements of cast-in-place concrete framing systems are considered to be restrained.

For positive moment reinforcement in beams spaced at 4 ft or less on center, and in joists and slabs, regardless of the type of concrete aggregate used, the minimum cover required by ACI 318 is adequate for ratings of up to four hours. For beams spaced at more than 4 ft on center, the cover must not be less than the values given in Table 3.

Table 3—Cover Thickness for Fire Resistance Rating for Beams Spaced More than 4 ft on Center

Beam Width (in.)	Cover thickness (in.) for fire-resistance rating			
	1 hr	2 hr	3 hr	4 hr
5	¾	¾	1	1¼
7	¾	¾	¾	¾
≥ 10	¾	¾	¾	¾

The cover for an individual bar is the minimum cover between the surface of the bar and the fire-exposed surface of the structural member. When more than one bar is used, the cover is assumed to be the average of the minimum cover to each bar, where the cover for corner bars used in the calculation is one-half the actual value. The actual cover for an individual bar must be not less than one-half the value shown in Table 3, nor less than ¾ in. For beam widths between tabulated values, use direct interpolation to determine minimum cover.

The foregoing is intended to give a brief overview of the subject of fire resistance of concrete floor systems. While the information cited is consistent with the three model building codes in use in the United States, the legally adopted building code governing the specific project should be consulted.

OVERVIEW

General Discussion

This section provides overall comparisons of the economics of the various floor systems discussed in this publication. It provides a summary of the factors that may influence the costs of cast-in-place concrete floor systems. These factors include column dimensions, live loads, aspect ratios and proper detailing. A few other aspects that have an influence on economy are also discussed.

Overall Comparisons

Four figures that compare the economics of the different structural floor systems considered are provided at the end of this publication. The figures clearly show that the optimality of the slab system depends on two major factors: the span in the long direction, and the intensity of superimposed dead and live loads. For a given set of loads, the slab system that is optimal for short spans, is not necessarily optimal for longer spans. For a given span, the slab system that is optimal for light superimposed loads, is not necessarily optimal for heavier loads. The four figures should facilitate the section of a structural floor system most appropriate for a certain application.

Column Dimensions

Analysis shows that the height between floors has very little influence on the material quantities for the floor system. Column cross-sectional properties determine the clear span length and the shear capacity of the slab. The column cross-sectional dimensions used in this publication were representative of 10- to 20-story buildings. Increasing or decreasing the column dimensions by 2 in. did not affect the concrete quantities and changed the steel reinforcing quantities by less than 1%.

Live Loads

The material quantities for the floor system are typically controlled by deflections rather then stresses. Increasing the live load from 50 psf to 100 psf only resulted in a 4% to 10% increase in the floor system cost.

Aspect Ratio

Square bays usually represent the most economical floor layout, since deflection control requirements can be exactly met in both directions. A rectangular bay with an aspect ratio of 1.5 ranges between 4% to 10% more in cost than a bay with an aspect ratio of 1.0 and the same floor area. This, however, is not the case for one-way joist systems. This type of floor system should have the joists span in the short direction, and is almost unaffected by aspect ratios of up to 1.5.

Concrete Strengths

Concrete strengths of 4000 psi, 5000 psi, and 6000 psi were used in this publication. Cost analysis shows that for gravity loads, 4000 psi concrete is more economical than higher concrete strengths.

Cost Breakdown

The formwork for the floor systems will absorb from 50% to 58% of the costs. Concrete material, placing and finishing account for 21% to 30%. The material and placing costs of the reinforcing steel amount to between 17% and 25% of the cost.

Repetition

A cost efficient design utilizes repetition. Changes should be minimized from floor to floor. Changing column locations, joist spacing, or the type of floor system increases the cost of formwork, time of construction and the chance of field mistakes, and therefore should be avoided.

Column-Beam Intersections

The beams that frame into columns should be at least as wide as the columns. If the beams are narrower than the columns, the beam forms will require costly field labor to pass the formwork around the columns.

Standard Dimensions

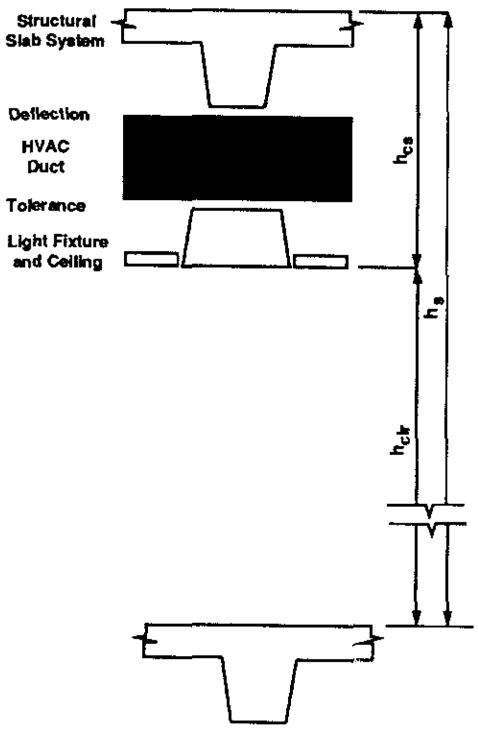
Standard available sizes should be used for structural forming. For instance, joist formwork pans are available in various web depths of 20 in. and from 8 in. to 16 in. in 2 in. increments. Specifying a depth different from these sizes will require the fabrication of costly special formwork. When detailing drop panels or other changes in the floor system depth, actual lumber dimensions should be taken into account.

Depth of the Ceiling Sandwich

This publication has addressed the economy of the structural slab system only. However, the structural engineer usually has to look beyond. The structural slab system is part of the so-called ceiling sandwich which also includes the mechanical system (HVAC ducts), the lighting fixtures, and the ceiling itself.

OVERVIEW

The floor-to-floor height of a building is the total depth of the ceiling sandwich plus the clear floor-to-ceiling height. Any variation in the depth of the ceiling sandwich will have an impact on the total height of: the shearwalls and columns, the mechanical, electrical and plumbing risers, the stairs and interior architectural finishes, and the exterior cladding. It will also have an impact on the total heating, cooling and ventilation volume. To minimize the depth of the ceiling sandwich is very often the goal of the structural engineer. This becomes particularly important in cities like Washington, D.C. that impose a height limit on buildings. Optimization of the ceiling sandwich depth may translate into an extra story or two accommodated within the prescribed height limit.



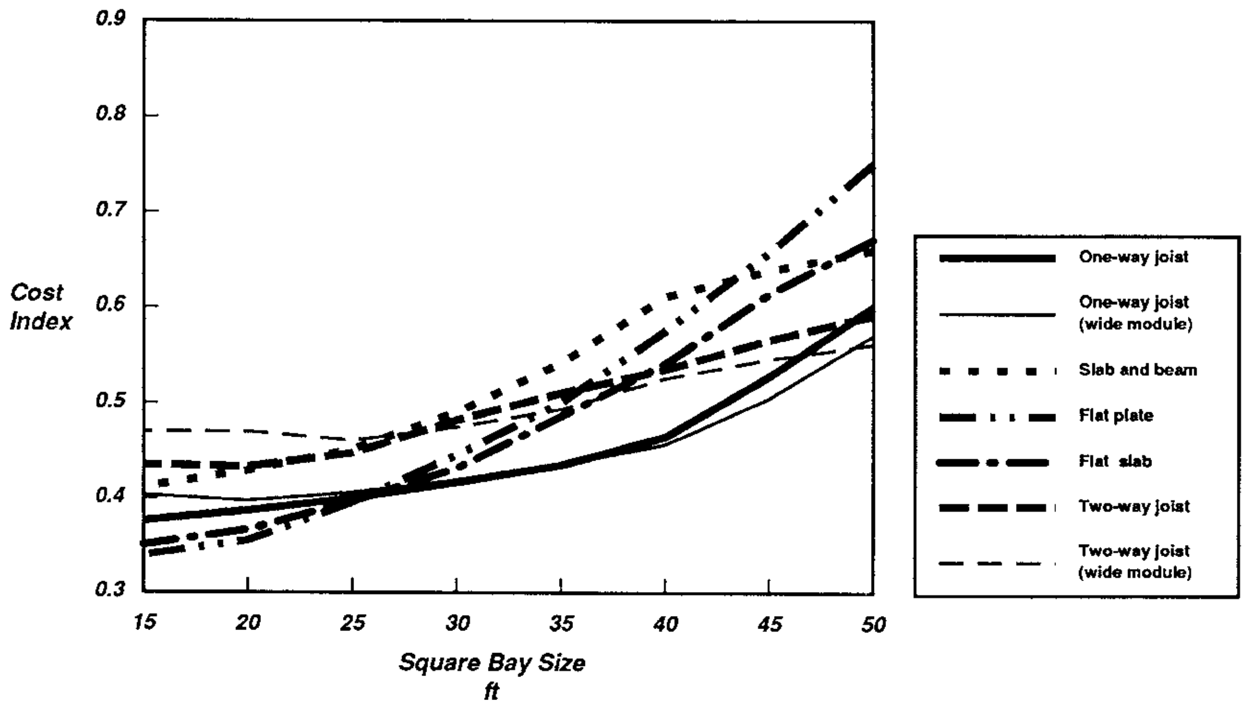
A number of details have been attempted in the past to accomplish a reduced depth of the ceiling sandwich. The HVAC ducts can pass through the webs of joists or beams. This will reduce the floor-to-floor height, but will increase formwork and field labor costs. Another alternative is to cut notches at the bottom of the joist or beam to allow passage of the upper portions of the HVAC ducts. This alternative also requires additional forming costs. Further, special detailing would be needed to meet the structural integrity requirements of the ACI 318-89 Code. More importantly, however, such practices take flexibility away from accommodating future changes in the use of the floor space. Such flexibility is becoming more important in view of the shifting emphasis towards consciously designing buildings for a long service life.

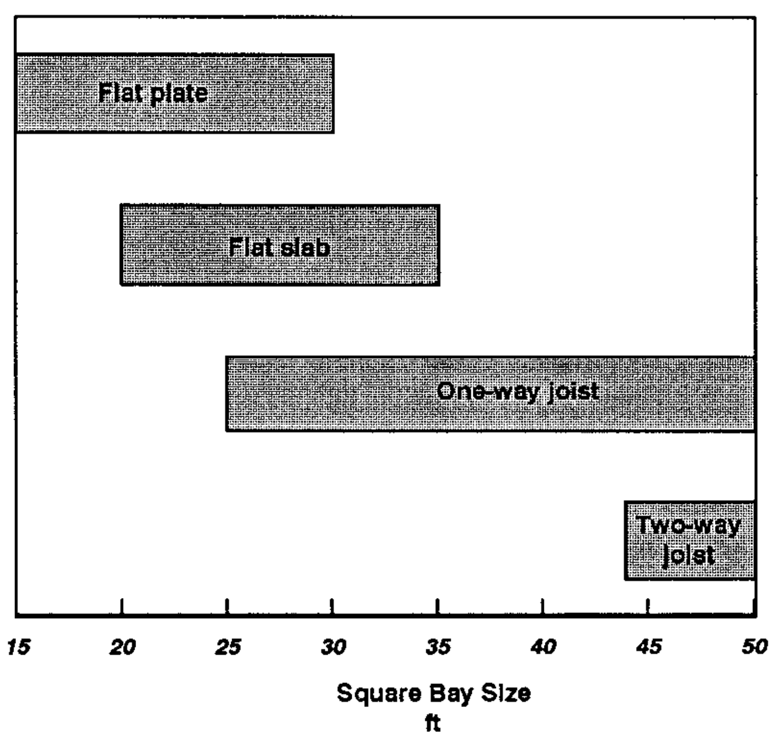
OVERVIEW

Live Load = 50 psf

Superimposed Dead Load = 20 psf

$f_c' = 4000$ psi



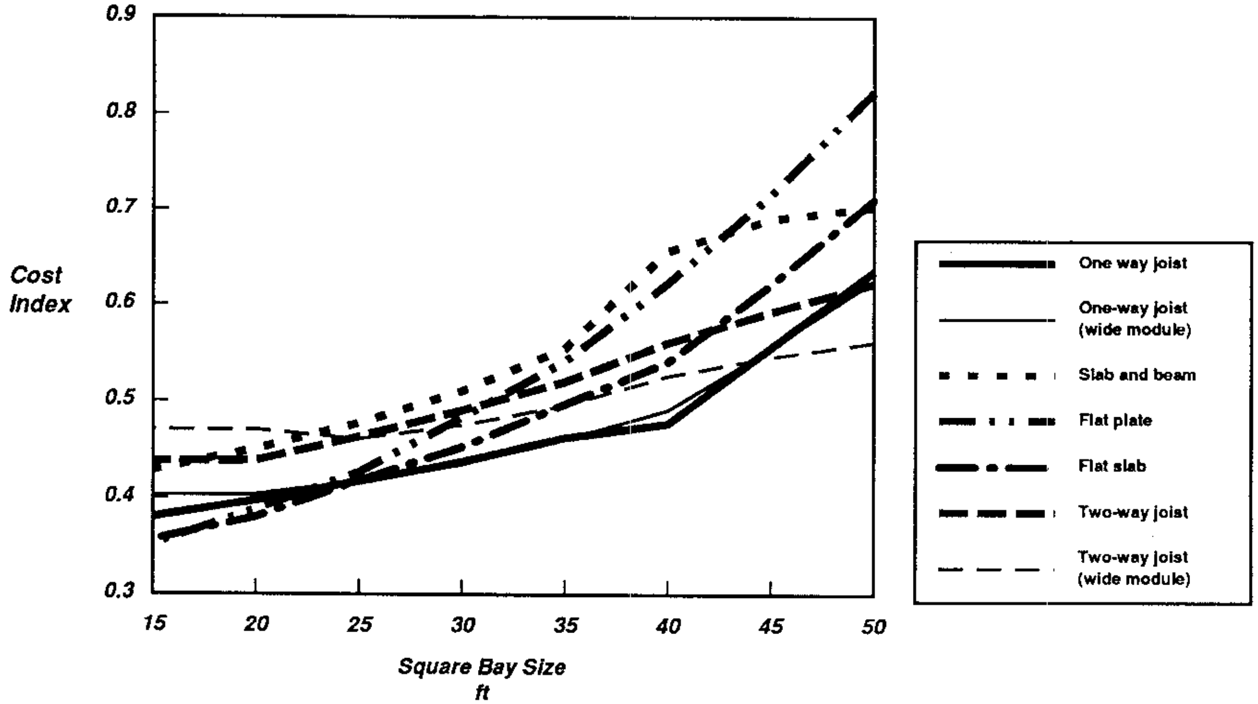


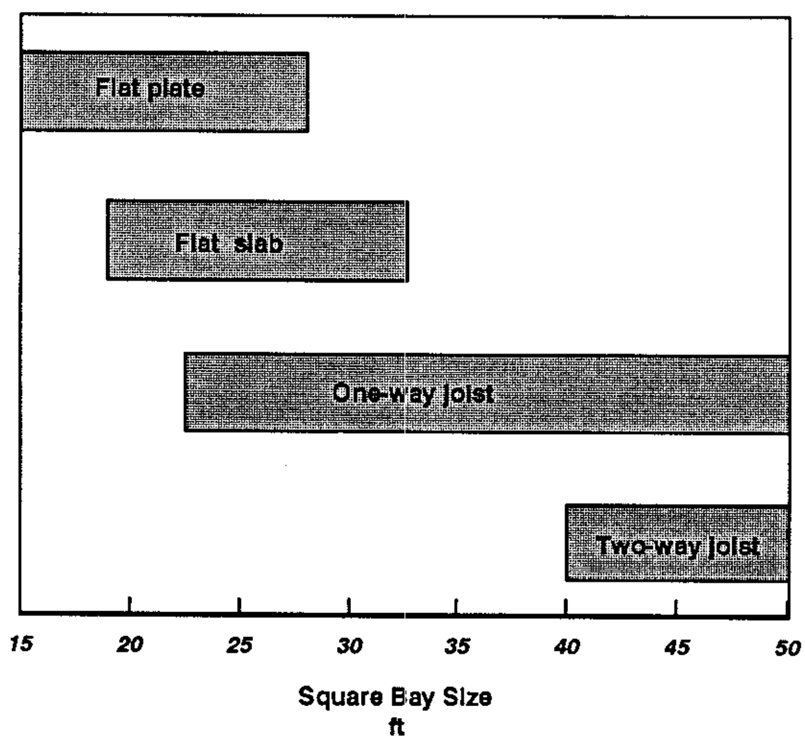
OVERVIEW

Live Load = 100 psf

Superimposed Dead Load = 20 psf

f'_c = 4000 psi





Openings in Concrete Slab Systems
from Notes on ACI 318-99, Portland Cement Association, 1999

11.12.5 Openings in Slabs

The effect of openings (vertical holes through slabs) on the shear strength of slabs must be investigated when the openings are within the column strip areas of slabs or within middle strip areas when the openings are closer than 10 times the slab thickness (10h) from a column. A reduction in shear strength is made by considering as ineffective that portion of the critical section b_o which is enclosed by straight lines projecting from the column centroid to the edges of the opening. Ineffective portions of critical sections b_o are illustrated in Fig. 18-10. For slabs with shear reinforcement, the ineffective portion of the perimeter b_o is one-half of that without shear reinforcement. The one-half factor is interpreted to apply equally to shearhead reinforcement and bar or wire reinforcement.

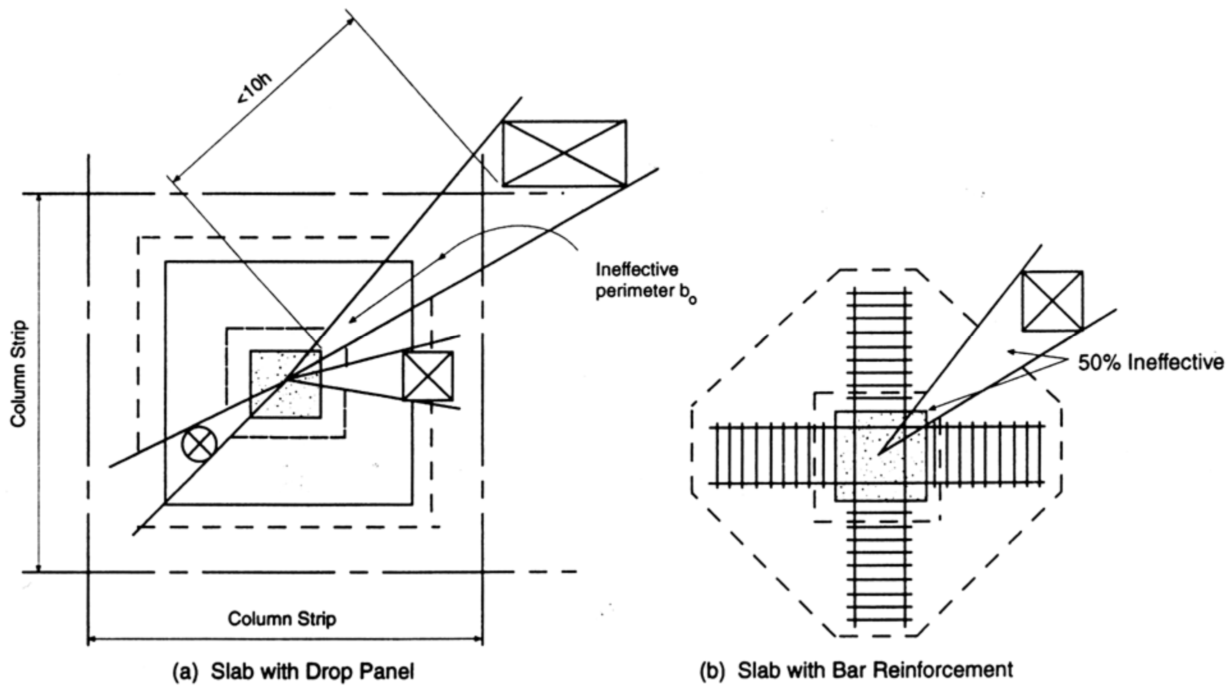


Figure 18-10 Effect of Slab Openings on Shear Strength

13.4 OPENINGS IN SLAB SYSTEMS

Openings of any size are permitted in slab systems without beams if special analysis indicates that both strength and serviceability of the slab system, considering the effects of the opening, are satisfied. Without special analysis, openings up to a certain size are permitted as illustrated in Fig. 18-11. The size of openings located within intersecting middle strip areas is unlimited. Within the area of the slab common to intersecting column strips, size of openings is the most restrictive, due to their effect on slab shear strength or load transfer near slab-column connections. See discussion on effect of slab openings on shear strength (11.12.5) and Fig. 18-10. Without special analysis, size of openings within intersecting column strips is limited to one-sixteenth of the slab span length in either direction $(1/8\ (\ell/2) = \ell/16)$. Within the slab area common to one column and one middle strip, opening size is limited to one-eighth the span length in either direction $(1/4\ (\ell/2) = \ell/8)$.

The total amount of reinforcement required for the panel without openings, in both directions, must be maintained; reinforcement interrupted by any opening must be replaced, one-half on each side of the opening.

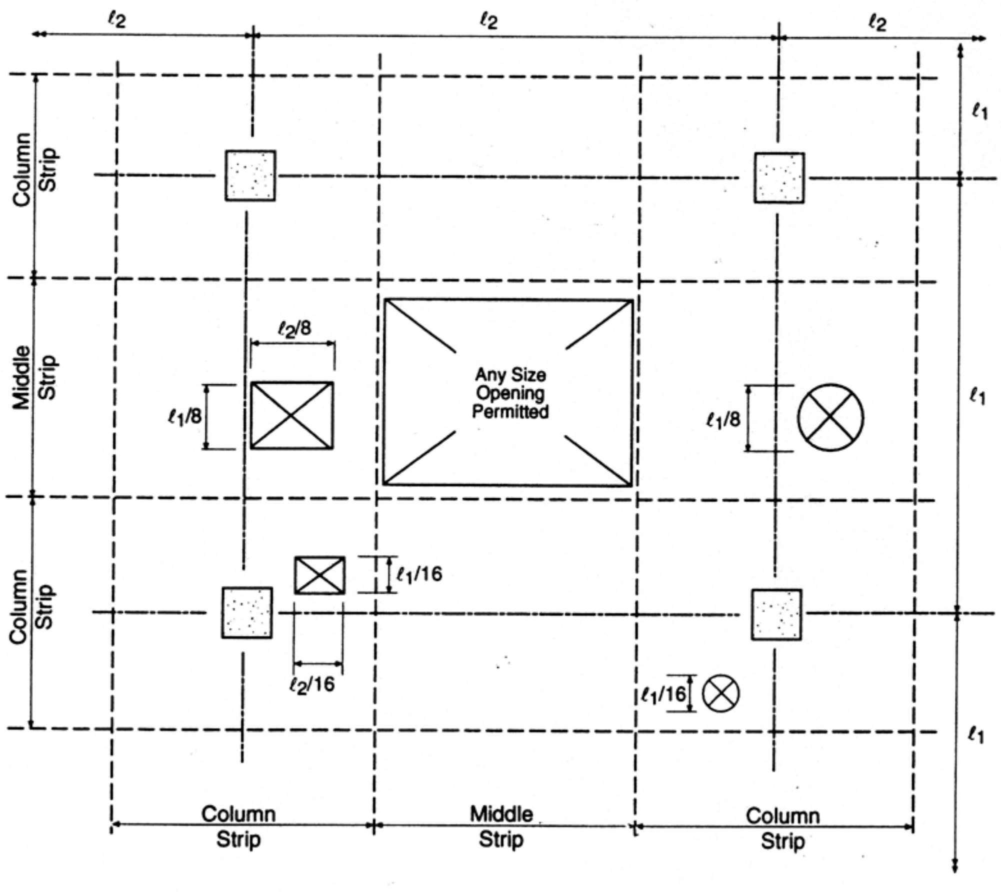
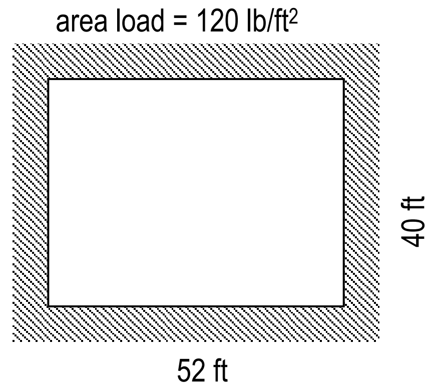


Figure 18-11 Openings in Slab Systems without Beams

Examples:
Plate and Grids

Example 1



What is the maximum positive and negative bending moments developed in a 52 x 40 ft fully fixed plate that carries a load of 120 lb/ft^2?

SOLUTION:

The aspect ratio of the side lengths, *a/b*, must be determined and an appropriate coefficient chart must be found:

a/b = 52/40 = 1.3 (no units, and *a* is always the *bigger* number).

BENDING MOMENTS IN RECTANGULAR PLATES

Aspect ratio $\frac{a}{b}$	Simply supported on all four sides	Fixed on all four sides		Corner slabs fixed on two adjacent sides and free on two sides
1.0	$C_a = +0.0479$	$C_a = +0.0231$,	$C_a = -0.0513$	$C_a = -0.29$
	$C_b = +0.0479$	$C_b = +0.0231$,	$C_b = -0.0513$	$C_b = -0.29$
1.3	$C_a = +0.0298$	$C_a = +0.0131$,	$C_a = -0.0333$	$C_a = -0.35$
	$C_b = +0.0694$	$C_b = +0.0327$,	$C_b = -0.0687$	$C_b = -0.35$
1.5	$C_a = +0.0221$	$C_a = +0.0090$,	$C_a = -0.0253$	$C_a = -0.37$
	$C_b = +0.0812$	$C_b = +0.0368$,	$C_b = -0.0757$	$C_b = -0.37$
2.0	$C_a = +0.0116$	$C_a = +0.0039$,	$C_a = -0.0143$	$C_a = -0.43$
	$C_b = +0.1017$	$C_b = +0.0412$,	$C_b = -0.0829$	$C_b = -0.43$

Note: In all cases,

$$\mathbf{M_a = C_a w a^2}$$

$$\mathbf{M_b = C_b w b^2}$$

The coefficients for moment for the *a* side length and *b* side length for fixed support all sides and a/b = 1.3 are:

$C_a = +0.0131$ and $C_a = -0.0333$ $C_b = +0.0327$ and $C_b = -0.0687$

The maximum moments are calculated with the formula in the table:

$$M_a(positive) = C_a wa^2 = 0.0131(120\ {}^{lb}\!/_{ft^2})(52ft)^2 = 4251\frac{lb-ft}{ft}$$

$$M_a(negative) = C_a wa^2 = -0.0333(120\ {}^{lb}\!/_{ft^2})(52ft)^2 = -10{,}805\frac{lb-ft}{ft}$$

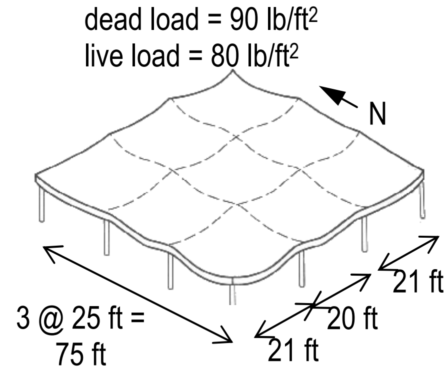
$$M_b(positive) = C_b wb^2 = 0.0327(120\ {}^{lb}\!/_{ft^2})(40ft)^2 = 6278\frac{lb-ft}{ft}$$

$$M_b(negative) = C_b wb^2 = -0.0687(120\ {}^{lb}\!/_{ft^2})(40ft)^2 = -13{,}190\frac{lb-ft}{ft}$$

Example 2

A two-way interior-bay flat (concrete) slab with the dimensions shown supports a live loading of 80 lb/ft^2 and has a dead load of 90 lb/ft^2. The columns can be assumed to be 18 inches square. Determine the design moments based on ACI-318, (ASCE-7) and the Direct Design method.

Also compare design moments for an exterior-interior bay



SOLUTION:

Determine the distributed load combinations:

$w_u = 1.2D + 1.6L$ = 1.2(90 lb/ft^2) + 1.6(80 lb/ft^2) = 236 lb/ft^2

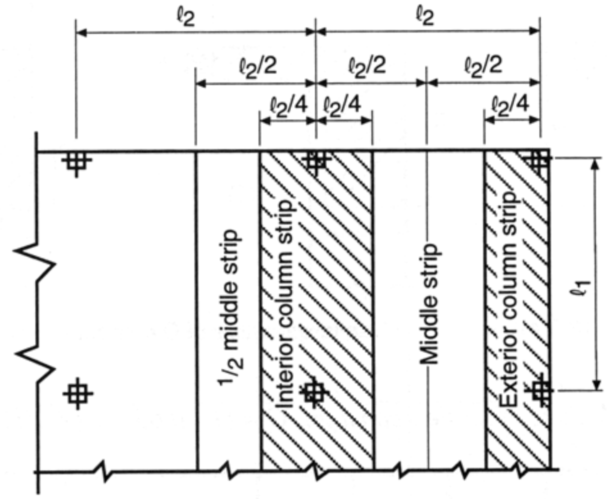
Determine the clear span length for the N-S direction:

$l_n = l_1$ – ½ column width – ½ column width
= 25 ft – ½ (18 in/12 in/ft) – ½ (18 in/12 in/ft) = 23.5 ft

Because l_2 is not the same width on either side of an interior panel, it is taken as the average = (21 ft + 20 ft)/2 = 20.5 ft.

Total moment (to distribute to middle and interior column strip):

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} = \frac{(236\,{}^{lb}\!/_{ft^2})(20.5\,ft)(23.5\,ft)^2}{8} = 333{,}973^{lb-ft}$$



(a) Column strip for $\ell_2 \le \ell_1$

<u>Interior Column Strip</u> ($l_2 \le l_1$):

The column strip width is ¼ the smaller of l_2 either side of the column:

strip width = ¼ (21 ft) + ¼ (20 ft) = 10.25 ft

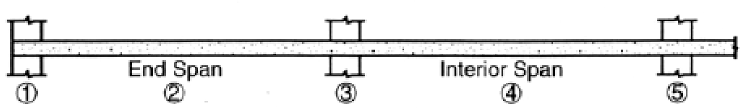
From Table 4.2, the maximum positive moment occurs in an end span:

$M(positive) = 0.31M_o = (0.31)(333{,}973^{lb-ft}) = 103{,}532^{lb-ft}$, distributed over 10.25 ft = 103,532 lb-ft/(10.25 ft)
= 10,101 lb-ft/ft

The positive design moment for an interior span is:

$M(positive) = 0.21M_o = (0.21)(333{,}973^{lb-ft}) = 70{,}134^{lb-ft}$, distributed over 10.25 ft = 70,134 lb-ft/(10.25 ft) =
= 6842 lb-ft/ft

Table 4-2 Flat Plate or Flat Slab Supported Directly on Columns



Slab Moments	End Span			Interior Span	
	1 Exterior Negative	2 Positive	3 First Interior Negative	4 Positive	5 Interior Negative
Total Moment	0.26 M_o	0.52 M_o	0.70 M_o	0.35 M_o	0.65 M_o
Column Strip	0.26 M_o	0.31 M_o	0.53 M_o	0.21 M_o	0.49 M_o
Middle Strip	0	0.21 M_o	0.17 M_o	0.14 M_o	0.16 M_o

Note: All negative moments are at face of support.

From Table 4.2, the maximum negative moment occurs in an end span at the first interior column face:
$M(negative) = 0.53M_o = (0.53)(333{,}973^{lb-ft}) = 177{,}006^{lb-ft}$, distributed over 10.25 ft =177,006 lb-ft/(10.25 ft) =
= 17,269 lb-ft/ft

The negative design moment at the exterior of an end span is:
$M(negative) = 0.26M_o = (0.26)(333{,}973^{lb-ft}) = 86{,}833^{lb-ft}$, distributed over 10.25 ft = 86,833 lb-ft/(10.25 ft) =
= 8472 lb-ft/ft

The negative design moment for an interior span is:
$M(negative) = 0.49M_o = (0.49)(333{,}973^{lb-ft}) = 163{,}647^{lb-ft}$, distributed over 10.25 ft = 163,647 lb-ft/(10.25 ft) =
= 15,966 lb-ft/ft

<u>Middle Strip:</u>

The width is the remaining width of l_2 between column strips:

strip width = 21 ft – ¼ (20 ft) – ¼ (21 ft) = 10.75 ft

From Table 4.2, the maximum positive moment occurs in an end span:
$M(positive) = 0.21M_o = (0.21)(333{,}973^{lb-ft}) = 70{,}134^{lb-ft}$, distributed over 10.75 ft = 70,134 lb-ft/(10.75 ft) =
= 6524 lb-ft/ft

The positive design moment for an interior span is:
$M(positive) = 0.14M_o = (0.14)(333{,}973^{lb-ft}) = 46{,}756^{lb-ft}$, distributed over 10.75 ft = 46,756 lb-ft/(10.75 ft) =
= 4349 lb-ft/ft

From Table 4.2, the maximum negative moment occurs in an end span at the first interior column face:
$M(negative) = 0.17M_o = (0.17)(333{,}973^{lb-ft}) = 56{,}775^{lb-ft}$, distributed over 10.75 ft = 56,775 lb-ft/(10.75 ft) =
= 5281 lb-ft/ft

There is no negative design moment at the exterior of an end span.

The negative design moment for an interior span is:
$M(negative) = 0.16M_o = (0.16)(333{,}973^{lb-ft}) = 53{,}436^{lb-ft}$, distributed over 10.75 ft = 53,436 lb-ft/(10.75 ft) =
= 4971 lb-ft/ft

<u>Exterior Column Strip:</u>

The value to use for l_2 for an edge strip includes the distance to the outside of the columns = 21 ft+ ½ (18 in/12 in/ft) = 21.75 ft

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} = \frac{(236^{lb}/_{ft^2})(21.75\,ft)(23.5\,ft)^2}{8} = 354{,}337^{lb-ft}$$

The width is ¼ l_2 one side of the column plus the distance to the slab edge:

strip width = ¼ (21 ft) + ½ (18 in/12 in/ft) = 6 ft

So a comparison to the interior column strip maximum positive moment occurring in an end span is:

$M(positive) = 0.31M_o = (0.31)(354{,}337^{lb-ft}) = 109{,}844^{lb-ft}$, distributed over 6 ft = 109,844 lb-ft/(6 ft) = 18,307 lb-ft/ft
(as opposed to 10,101 lb-ft/ft)

For the E-W direction:

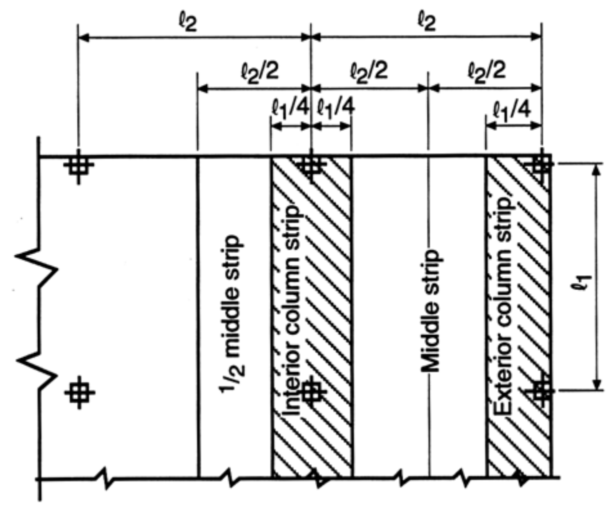
Because the adjacent spans are not the same length, the longer span, which is the END span will be larger:

$l_n = l_1$ – ½ column width – ½ column width
= 21 ft – ½ (18 in/12 in/ft) – ½ (18 in/12 in/ft) =19.5 ft

Because l_2 is 25 ft.

Total moment (to distribute to middle and interior column strip):

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} = \frac{(236 \,^{lb}\!/_{ft^2})(25 ft)(19.5 ft)^2}{8} = 280{,}434^{lb-ft}$$



(b) Column strip for $\ell_2 > \ell_1$

Interior Column Strip END Spans ($l_2 > l_1$):

The column strip width is ¼ the **smaller** of l_1 and l_2 either side of the column:

strip width = ¼ (21 ft) + ¼ (21 ft) = 10.5 ft

From Table 4.2, the maximum positive moment occurs in an end span:
$M(positive) = 0.31M_o = (0.31)(280{,}434^{lb-ft}) = 86{,}935^{lb-ft}$, distributed over 10.5 ft = 86,935 lb-ft/(10.5 ft)
= 8279 lb-ft/ft

From Table 4.2, the maximum negative moment occurs in an end span at the first interior column face:
$M(negative) = 0.53M_o = (0.53)(280{,}434^{lb-ft}) = 148{,}630^{lb-ft}$, distributed over 10.5 ft = 148,630 lb-ft/(10.5 ft) =
= 14,155 lb-ft/ft

The negative design moment at the exterior of an end span is:
$M(negative) = 0.26M_o = (0.26)(280{,}434^{lb-ft}) = 72{,}913^{lb-ft}$, distributed over 10.5 ft = 72,913 lb-ft/(10.5 ft) =
= 6944 lb-ft/ft

Middle Strip END Spans:

The width is the remaining width of l_2 between column strips:

strip width = 25 ft – ¼ (21 ft) – ¼ (21 ft) = 14.5 ft

From Table 4.2, the maximum positive moment occurs in an end span:
$M(positive) = 0.21M_o = (0.21)(280{,}434^{lb-ft}) = 58{,}891^{lb-ft}$, distributed over 14.5 ft = 58,891 lb-ft/(14.5 ft) =
= 4061 lb-ft/ft

From Table 4.2, the maximum negative moment occurs in an end span at the first interior column face:
$M(negative) = 0.17M_o = (0.17)(280{,}434^{lb-ft}) = 47{,}674^{lb-ft}$, distributed over 14.5 ft = 47,674 lb-ft/(14.5 ft) =
= 3288 lb-ft/ft

There is no negative design moment at the exterior of an end span.

Exterior Column Strip END Spans:

The value to use for l_2 for an edge strip includes the distance to the outside of the columns = 25 ft+ ½ (18 in/12 in/ft) = 25.75 ft

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} = \frac{(236\,{}^{lb}\!/\!_{ft^2})(25.75\,ft)(19.5\,ft)^2}{8} = 288{,}847^{lb-ft}$$

The width is ¼ l_1 (because it is smaller than l_2) one side of the column plus the distance to the slab edge:

strip width = ¼ (21 ft) + ½ (18 in/12 in/ft) = 6 ft

So a comparison to the interior column END strip maximum positive moment occurring in an end span is:

$M(\,positive\,) = 0.31M_o = (0.31)(288{,}847^{lb-ft}) = 89{,}543^{lb-ft}$, distributed over 6 ft = 89,543 lb-ft/(6 ft) = 14,923 lb-ft/ft (as opposed to 8279 lb-ft/ft)

TABLE OF DESIGN MOMENTS

slab moments / ft	End Span			Interior Span	
	Exterior Negative	Positive	First Interior Negative	Positive	Interior Negative
NS column strip - interior	8472 lb-ft/ft	10,101 lb-ft/ft	17,269 lb-ft/ft	6842 lb-ft/ft	15,966 lb-ft/ft
NS middle strip	0	6524 lb-ft/ft	5281 lb-ft/ft	4349 lb-ft/ft	4971 lb-ft/ft
NS column strip - edge	15,355 lb-ft/ft	18,307 lb-ft/ft	31,300 lb-ft/ft	12,402 lb-ft/ft	28,937 lb-ft/ft
EW column strip – interior	6944 lb-ft/ft	8279 lb-ft/ft	14,155 lb-ft/ft	5048 lb-ft/ft	11,779 lb-ft/ft
EW middle strip	0	4061 lb-ft/ft	3288 lb-ft/ft	2437 lb-ft/ft	5686 lb-ft/ft
EW column strip – edge	12,517 lb-ft/ft	14,923 lb-ft/ft	25,515 lb-ft/ft	6066 lb-ft/ft	6933 lb-ft/ft

Reinforced Concrete Design

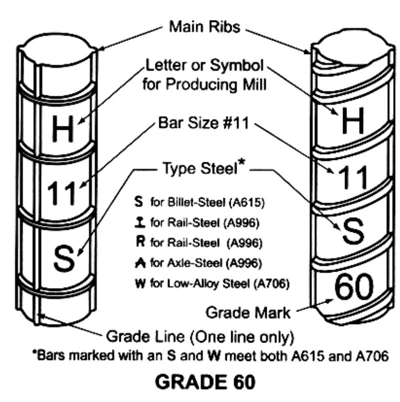
Notation:

a = depth of the effective compression block in a concrete beam
A = name for area
A_g = gross area, equal to the total area ignoring any reinforcement
A_s = area of steel reinforcement in concrete beam design
A'_s = area of steel compression reinforcement in concrete beam design
A_{st} = area of steel reinforcement in concrete column design
A_v = area of concrete shear stirrup reinforcement
ACI = American Concrete Institute
b = width, often cross-sectional
b_E = effective width of the flange of a concrete T beam cross section
b_f = width of the flange
b_w = width of the stem (web) of a concrete T beam cross section
cc = shorthand for clear cover
C = name for centroid
= name for a compression force
C_c = compressive force in the compression steel in a doubly reinforced concrete beam
C_s = compressive force in the concrete of a doubly reinforced concrete beam
d = effective depth from the top of a reinforced concrete beam to the centroid of the tensile steel
d' = effective depth from the top of a reinforced concrete beam to the centroid of the compression steel
d_b = bar diameter of a reinforcing bar
D = shorthand for dead load
DL = shorthand for dead load
E = modulus of elasticity or Young's modulus
= shorthand for earthquake load
E_c = modulus of elasticity of concrete
E_s = modulus of elasticity of steel
f = symbol for stress
f_c = compressive stress
f'_c = concrete design compressive stress
f_s = stress in the steel reinforcement for concrete design
f'_s = compressive stress in the compression reinforcement for concrete beam design
f_y = yield stress or strength
F = shorthand for fluid load
F_y = yield strength
G = relative stiffness of columns to beams in a rigid connection, as is Ψ
h = cross-section depth
H = shorthand for lateral pressure load
h_f = depth of a flange in a T section
$I_{transformed}$ = moment of inertia of a multi-material section transformed to one material
k = effective length factor for columns
ℓ_b = length of beam in rigid joint
ℓ_c = length of column in rigid joint
l_d = development length for reinforcing steel
l_{dh} = development length for hooks
l_n = clear span from face of support to face of support in concrete design
L = name for length or span length, as is l
= shorthand for live load
L_r = shorthand for live roof load
LL = shorthand for live load
M_n = nominal flexure strength with the steel reinforcement at the yield stress and concrete at the concrete design strength for reinforced concrete beam design
M_u = maximum moment from factored loads for LRFD beam design
n = modulus of elasticity transformation coefficient for steel to concrete
$n.a.$ = shorthand for neutral axis (N.A.)
pH = chemical alkalinity
P = name for load or axial force vector

P_o = maximum axial force with no concurrent bending moment in a reinforced concrete column
P_n = nominal column load capacity in concrete design
P_u = factored column load calculated from load factors in concrete design
R = shorthand for rain or ice load
R_n = concrete beam design ratio = M_u/bd^2
s = spacing of stirrups in reinforced concrete beams
S = shorthand for snow load
t = name for thickness
T = name for a tension force
= shorthand for thermal load
U = factored design value
V_c = shear force capacity in concrete
V_s = shear force capacity in steel shear stirrups
V_u = shear at a distance of *d* away from the face of support for reinforced concrete beam design
w_c = unit weight of concrete
w_{DL} = load per unit length on a beam from dead load
w_{LL} = load per unit length on a beam from live load
$w_{self\ wt}$ = name for distributed load from self weight of member
w_u = load per unit length on a beam from load factors
W = shorthand for wind load
x = horizontal distance
= distance from the top to the neutral axis of a concrete beam
y = vertical distance
β_1 = coefficient for determining stress block height, *a*, based on concrete strength, f'_c
Δ = elastic beam deflection
ε = strain
ϕ = resistance factor
ϕ_c = resistance factor for compression
γ = density or unit weight
ρ = radius of curvature in beam deflection relationships
= reinforcement ratio in concrete beam design = A_s/bd
$\rho_{balanced}$ = balanced reinforcement ratio in concrete beam design
υ_c = shear strength in concrete design

Reinforced Concrete Design

Structural design standards for reinforced concrete are established by the *Building Code and Commentary (ACI 318-11)* published by the American Concrete Institute International, and uses ultimate strength design.



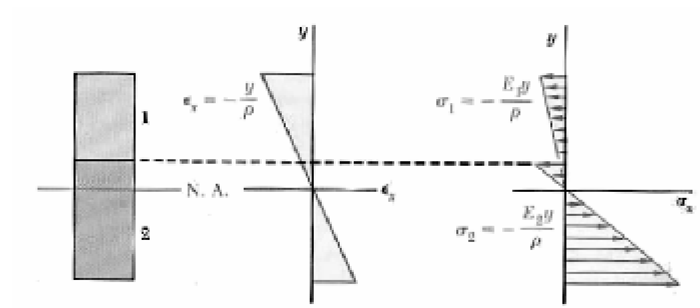
Materials

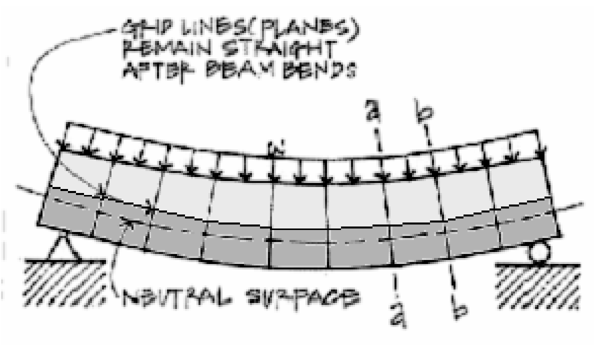
f'_c = concrete compressive design strength at 28 days (units of psi when used in equations)

Deformed reinforcing bars come in grades 40, 60 & 75 (for 40 ksi, 60 ksi and 75 ksi yield strengths). Sizes are given as # of 1/8" up to #8 bars. For #9 and larger, the number is a nominal size (while the actual size is larger).

Reinforced concrete is a composite material, and the average density is considered to be *150 lb/ft³*. It has the properties that it will creep (deformation with long term load) and shrink (a result of hydration) that must be considered.

Plane sections of composite materials can still be assumed to be plane (strain is linear), *but* the stress distribution *is not* the same in both materials because the *modulus of elasticity* is different. (f=E·ε)





$$f_1 = E_1\varepsilon = -\frac{E_1 y}{\rho} \qquad f_2 = E_2\varepsilon = -\frac{E_2 y}{\rho}$$

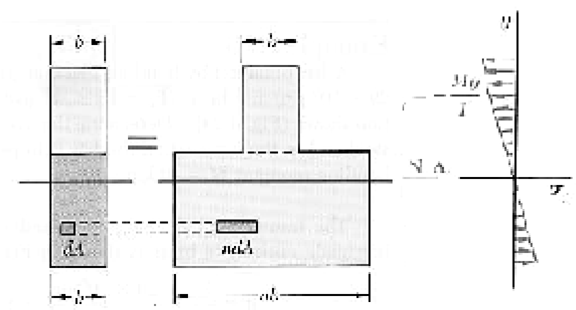
In order to determine the stress, we can define n as the ratio of the elastic moduli:

$$n = \frac{E_2}{E_1}$$

n is used to <u>transform</u> the <u>width</u> of the second material such that it sees the equivalent element stress.

Transformed Section y and I

In order to determine stresses in all types of material in the beam, we transform the materials into a single material, and calculate the location of the neutral axis and modulus of inertia for that material.



ex: When material 1 above is concrete and material 2 is steel:

to transform steel into concrete $n = \frac{E_2}{E_1} = \frac{E_{steel}}{E_{concrete}}$

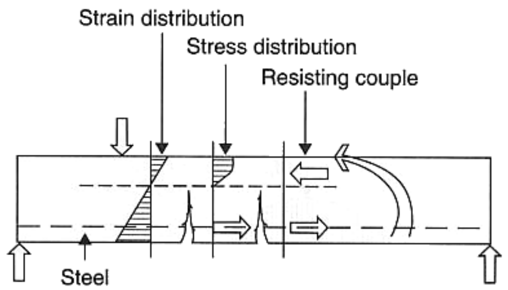
to find the neutral axis of the equivalent concrete member we transform the width of the steel by multiplying by n

to find the moment of inertia of the equivalent concrete member, $I_{transformed}$, use the new geometry resulting from transforming the width of the steel

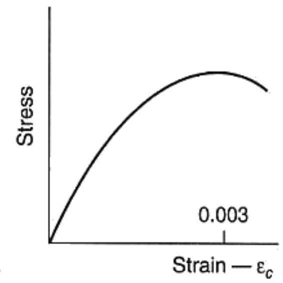
concrete stress: $f_{concrete} = -\frac{My}{I_{transformed}}$

steel stress: $f_{steel} = -\frac{Myn}{I_{transformed}}$

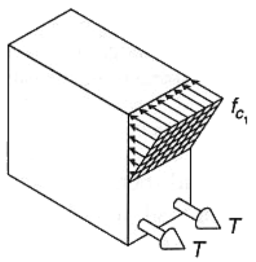
Reinforced Concrete Beam Members



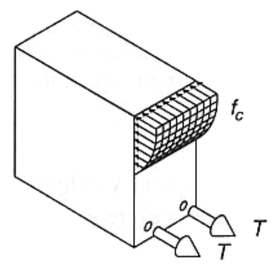
Stresses in the concrete above the neutral axis are compressive and nonlinearly distributed. In the tension zone below the neutral axis, the concrete is assumed to be cracked and the tensile force present to be taken up by reinforcing steel.



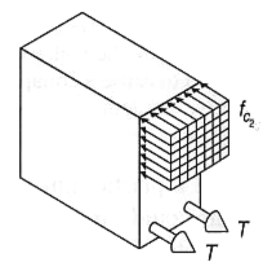
Typical stress-strain curve for concrete.



Working stress analysis. (Concrete stress distribution is assumed to be linear. Service loads are used in calculations.)



Actual stress distribution near ultimate strength (nonlinear).



Ultimate strength analysis. (A rectangular stress block is used to idealize the actual stress distribution. Calculations are based on ultimate loads and failure stresses.)

Ultimate Strength Design for Beams

The ultimate strength design method is similar to LRFD. There is a *nominal* strength that is reduced by a factor ϕ which must exceed the factored design stress. For beams, the concrete only works in compression over a rectangular "stress" block above the n.a. from elastic calculation, and the steel is exposed and reaches the yield stress, F_y

For stress analysis in reinforced concrete beams

- the steel is transformed to concrete
- any concrete in tension is assumed to be cracked and to have no strength
- the steel can be in tension, and is placed in the bottom of a beam that has positive bending moment

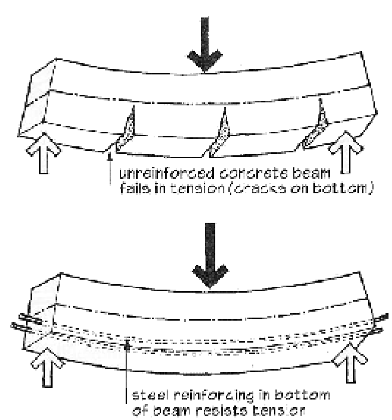
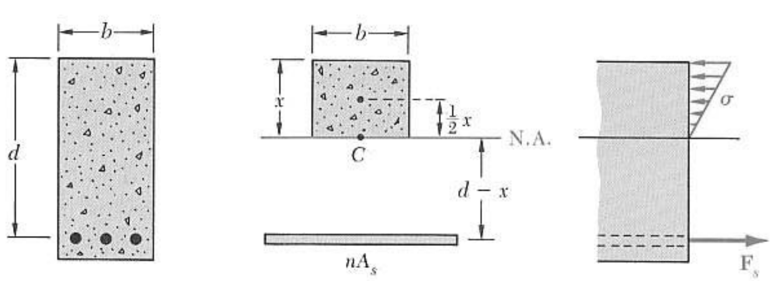


Figure 8.5: Bending in a concrete beam without and with steel reinforcing.



The neutral axis is where there is no stress and no strain. The concrete above the n.a. is in compression. The concrete below the n.a. is considered ineffective. The steel below the n.a. is in tension.

Because the n.a. is defined by the moment areas, we can solve for x knowing that d is the distance from the top of the concrete section to the centroid of the steel:

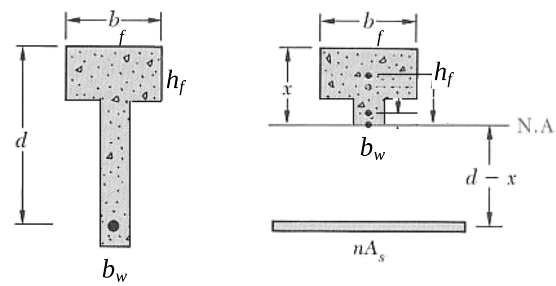
$$bx \cdot \frac{x}{2} - nA_s(d-x) = 0$$

x can be solved for when the equation is rearranged into the generic format with a, b & c in the binomial equation: $ax^2 + bx + c = 0$ by $x = \dfrac{-b \pm \sqrt{b^2 - 4ac}}{2a}$

T-sections

If the n.a. is *above* the bottom of a flange in a T section, x is found as for a rectangular section.

If the n.a. is *below* the bottom of a flange in a T section, x is found by including the flange and the stem of the web (b_w) in the moment area calculation:



$$b_f h_f\left(x - \frac{h_f}{2}\right) + (x - h_f)b_w \frac{(x - h_f)}{2} - nA_s(d - x) = 0$$

Load Combinations - (Alternative values allowed)

1.4*D*

1.2*D* + 1.6*L* + 0.5(L_r or *S* or *R*)

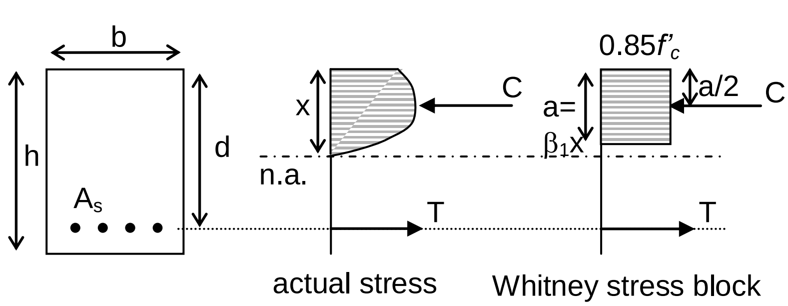
1.2*D* + 1.6(L_r or *S* or *R*) + (1.0*L* or 0.5*W*)

1.2*D* + 1.0*W* + 1.0*L* + 0.5(L_r or *S* or *R*)

1.2*D* + 1.0*E* + 1.0*L* + 0.2*S*

0.9*D* + 1.0*W*

0.9*D* + 1.0*E*



Internal Equilibrium

ASTM STANDARD REINFORCING BARS

Bar size, no.	Nominal diameter, in.	Nominal area, in.^{2}	Nominal weight, lb/ft
3	0.375	0.11	0.376
4	0.500	0.20	0.668
5	0.625	0.31	1.043
6	0.750	0.44	1.502
7	0.875	0.60	2.044
8	1.000	0.79	2.670
9	1.128	1.00	3.400
10	1.270	1.27	4.303
11	1.410	1.56	5.313
14	1.693	2.25	7.650
18	2.257	4.00	13.600

C = compression in concrete = stress x area = $0.85 f'_c ba$
T = tension in steel = stress x area = $A_s f_y$

$C = T$ and $M_n = T(d-a/2)$

where
f'_c = concrete compression strength
a = height of stress block
b = width of stress block
f_y = steel yield strength
A_s = area of steel reinforcement
d = effective depth of section
(depth to n.a. of reinforcement)

With C=T, $A_s f_y = 0.85 f'_c ba$ so a can be determined with $a = \dfrac{A_s f_y}{0.85 f'_c b}$

Criteria for Beam Design

For flexure design:

$M_u \leq \phi M_n \quad \phi = 0.9$ *for flexure*
so, M_u *can be set* $= \phi M_n = \phi T(d-a/2) = \phi A_s f_y (d-a/2)$

Reinforcement Ratio

The amount of steel reinforcement is *limited.* Too much reinforcement, or *over-reinforced* will not allow the steel to yield before the concrete crushes and there is a sudden failure. A beam with the proper amount of steel to allow it to yield at failure is said to be *under reinforced*.

The reinforcement ratio is just a fraction: $\rho = \dfrac{A_s}{bd}$ (or p) and must be less than a value determined with a concrete strain of 0.003 and tensile strain of 0.004 (minimum). The practical value for the strain in the reinforcement is a value of 0.005. Previous codes limited the amount to $0.75\rho_{balanced}$ where $\rho_{balanced}$ was determined from the amount of steel that would make the concrete start to crush at the exact same time that the steel would yield based on strain.

Flexure Design of Reinforcement

One method is to "wisely" estimate a height of the stress block, *a*, and solve for A_s, and calculate a new value for *a* using M_u.

1. guess *a* (less than n.a.)
2. $A_s = \dfrac{0.85 f'_c ba}{f_y}$
3. solve for *a* from

   $M_u = \phi A_s f_y (d-a/2)$:

   $$a = 2\left(d - \frac{M_u}{\phi A_s f_y}\right)$$
4. repeat from 2. until *a* found from step 3 matches *a* used in step 2.

Maximum Reinforcement Ratio ρ for Singly Reinforced Rectangular Beams (tensile strain = 0.005)

f_y	$f'_c = 3000$ psi $\beta_1 = 0.85$	$f'_c = 3500$ psi $\beta_1 = 0.85$	$f'_c = 4000$ psi $\beta_1 = 0.85$	$f'_c = 5000$ psi $\beta_1 = 0.80$	$f'_c = 6000$ psi $\beta_1 = 0.75$
40,000 psi	0.0203	0.0237	0.0271	0.0319	0.0359
50,000 psi	0.0163	0.0190	0.0217	0.0255	0.0287
60,000 psi	0.0135	0.0158	0.0181	0.0213	0.0239
f_y	$f'_c = 20$ MPa $\beta_1 = 0.85$	$f'_c = 25$ MPa $\beta_1 = 0.85$	$f'_c = 30$ MPa $\beta_1 = 0.85$	$f'_c = 35$ MPa $\beta_1 = 0.81$	$f'_c = 40$ MPa $\beta_1 = 0.77$
300 MPa	0.0181	0.0226	0.0271	0.0301	0.0327
350 MPa	0.0155	0.0194	0.0232	0.0258	0.0281
400 MPa	0.0135	0.0169	0.0203	0.0226	0.0245
500 MPa	0.0108	0.0135	0.0163	0.0181	0.0196

Design Chart Method:

1. calculate $R_n = \dfrac{M_n}{bd^2}$
2. find curve for f'_c and f_y to get ρ
3. calculate A_s and *a*

   $A_s = \rho bd$ *and* $a = \dfrac{A_s f_y}{0.85 f'_c b}$

Any method can simplify the size of d using h = 1.1d

from Reinforced Concrete, 7th, Wang, Salmon, Pincheira, Wiley & Sons, 2007

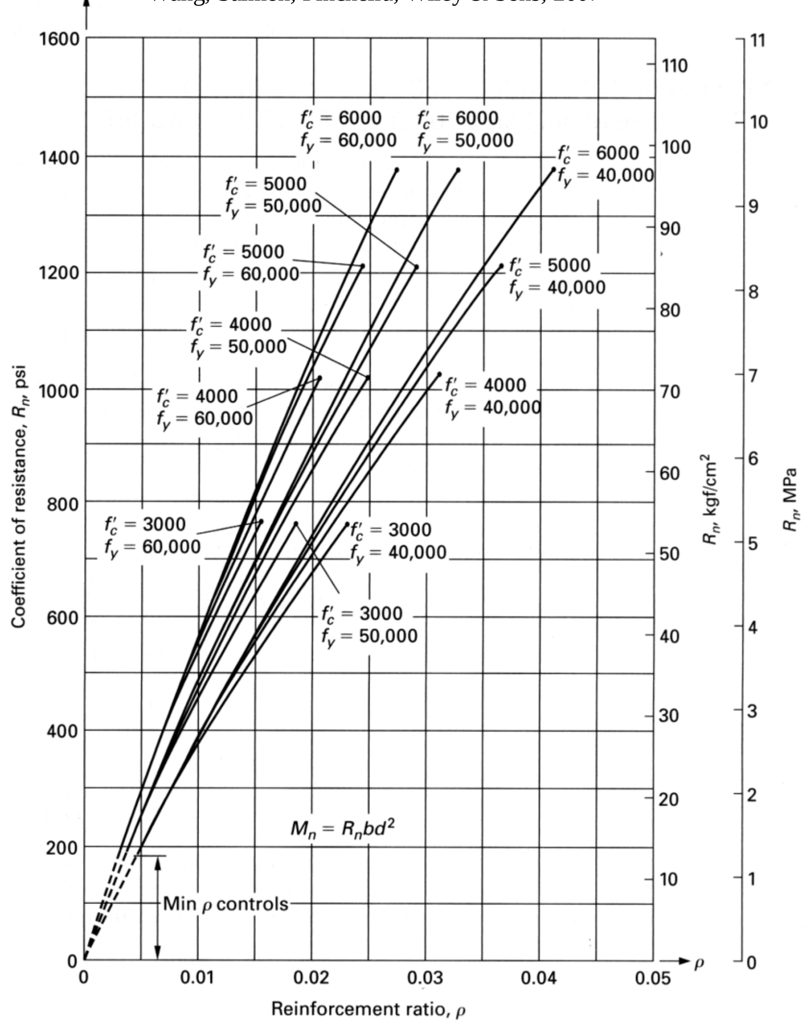


Figure 3.8.1 Strength curves (R_n vs ρ) for singly reinforced rectangular sections. Upper limit of curves is at ρ_{max}. (tensile strain of 0.004)

Maximum Reinforcement

Based on the limiting strain of 0.005 in the steel, *x(or c) = 0.375d* so

$a = \beta_1(0.375d)$ to find $A_{s\text{-max}}$

(β_1 is shown in the table above)

Minimum Reinforcement

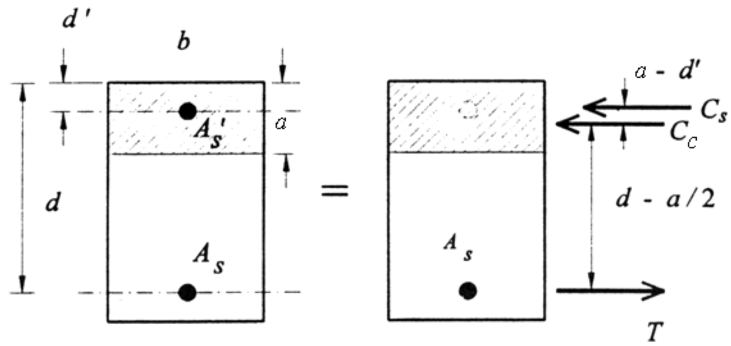
Minimum reinforcement is provided even if the concrete can resist the tension. This is a means to control cracking.

Minimum required: $A_s = \dfrac{3\sqrt{f'_c}}{f_y}(b_w d)$

but not less than: $A_s = \dfrac{200}{f_y}(b_w d)$

where f'_c is in psi. This can be translated to $\rho_{min} = \dfrac{3\sqrt{f'_c}}{f_y}$ but not less than $\dfrac{200}{f_y}$

Compression Reinforcement



If a section is *doubly reinforced*, it means there is steel in the beam seeing compression. The force in the compression steel at yield is equal to stress x area, $C_s = A_c \cdot F_y$. The total compression that balances the tension is now: $T = C_c + C_s$. And the moment taken about the centroid of the compression stress is $M_n = T(d-a/2)+C_s(a-d')$

where A_s' is the area of compression reinforcement, and d' is the effective depth to the centroid of the compression reinforcement

T-sections (pan joists)

T beams have an effective width, b_E, that sees compression stress in a wide flange beam or joist in a slab system.

For *interior* T-sections, b_E is the smallest of

L/4, b_w + 16t, or center to center of beams

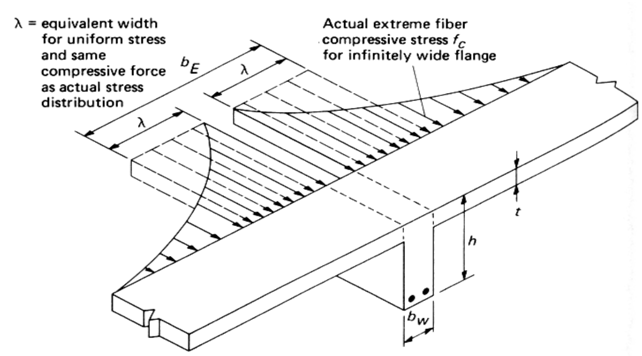


Figure 9.3.1 Actual and equivalent stress distribution over flange width.

For *exterior* T-sections, b_E is the smallest of

b_w + L/12, b_w + 6t, or b_w + ½(clear distance to next beam)

When the **web** is in tension the minimum reinforcement required is the same as for rectangular sections with the web width (b_w) in place of *b*.

When the **flange** is in tension (negative bending), the minimum reinforcement required is the greater value of $A_s = \frac{6\sqrt{f'_c}}{f_y}(b_w d)$ or $A_s = \frac{3\sqrt{f'_c}}{f_y}(b_f d)$

where f'_c is in psi, b_w is the beam width, and b_f is the effective flange width

Cover for Reinforcement

Cover of concrete over/under the reinforcement must be provided to protect the steel from corrosion. For indoor exposure, 3/4 inch is required for slabs, 1.5 inch is typical for beams, and for concrete cast against soil, 3 inches is typical.

Bar Spacing

Minimum bar spacings are specified to allow proper consolidation of concrete around the reinforcement.

Slabs

One way slabs can be designed as "one unit"-wide beams. Because they are thin, control of deflections is important, and minimum depths are specified, as is minimum reinforcement for shrinkage and crack control when not in flexure. Reinforcement is commonly small diameter bars and welded wire fabric. Minimum spacing between bars is also specified for shrinkage and crack control as five times the slab thickness not exceeding 18". For required flexure reinforcement spacing the limit is three times the slab thickness not exceeding 18".

TABLE 9.5(a)—MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE COMPUTED

	Minimum thickness, *h*			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections.			
Solid one-way slabs	$\ell/20$	$\ell/24$	$\ell/28$	$\ell/10$
Beams or ribbed one-way slabs	$\ell/16$	$\ell/18.5$	$\ell/21$	$\ell/8$

Notes:
Values given shall be used directly for members with normalweight concrete and Grade 60 reinforcement. For other conditions, the values shall be modified as follows:
a) For lightweight concrete having equilibrium density, w_c, in the range of 90 to 115 lb/ft^3, the values shall be multiplied by $(1.65 - 0.005w_c)$ but not less than 1.09.
b) For f_y other than 60,000 psi, the values shall be multiplied by $(0.4 + f_y/100{,}000)$.

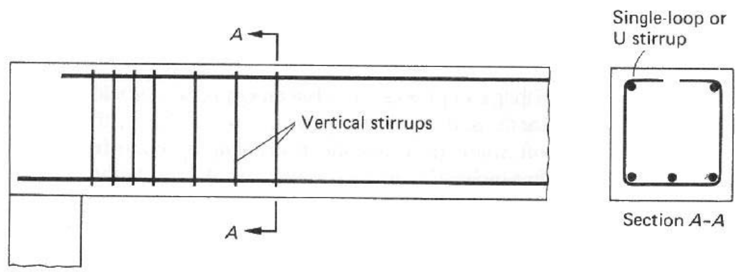
<u>Shrinkage and temperature reinforcement (and minimum for flexure reinforcement):</u>

Minimum for slabs with grade 40 or 50 bars: $\rho = \frac{A_s}{bt} = 0.002$ *or* $A_{s\text{-}min} = 0.002bt$

Minimum for slabs with grade 60 bars: $\rho = \frac{A_s}{bt} = 0.0018$ *or* $A_{s\text{-}min} = 0.0018bt$

Shear Behavior

Horizontal shear stresses occur along with bending stresses to cause tensile stresses where the concrete cracks. Vertical reinforcement is required to bridge the cracks which are called *shear stirrups*.



The maximum shear for design, V_u is the value at a distance of d from the face of the support.

Nominal Shear Strength

The shear force that can be resisted is the shear stress × cross section area: $V_c = v_c \times b_w d$

The shear stress for beams (one way) $v_c = 2\sqrt{f'_c}$ so $\phi V_c = \phi 2\sqrt{f'_c}\, b_w d$

where b_w = the beam width or the minimum width of the stem.

One-way joists are allowed an increase of 10% V_c if the joists are closely spaced.

Stirrups are necessary for strength (as well as crack control): $V_s = \frac{A_v f_y d}{s}$

where A_v = area of all vertical legs of stirrup
s = spacing of stirrups
d = effective depth

For shear design:

$$V_U \le \phi V_C + \phi V_S \quad \phi = 0.75 \text{ for shear}$$

Spacing Requirements

Stirrups are required when V_u is greater than $\dfrac{\phi V_c}{2}$

Table 3-8 ACI Provisions for Shear Design*

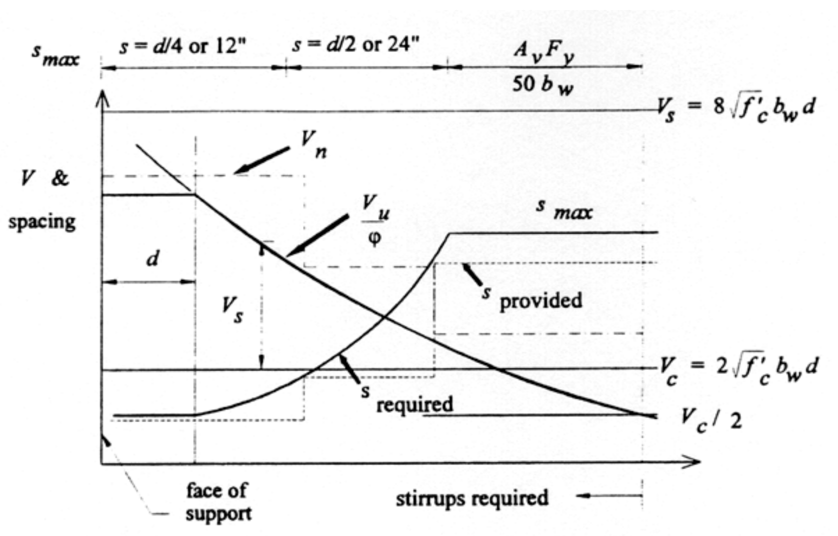
		$V_u \le \dfrac{\phi V_c}{2}$	$\phi V_c \ge V_u > \dfrac{\phi V_c}{2}$	$V_u > \phi V_c$
Required area of stirrups, A_v**		none	$\dfrac{50 b_w s}{f_y}$	$\dfrac{(V_u - \phi V_c)s}{\phi f_y d}$
Stirrup spacing, s	Required	—	$\dfrac{A_v f_y}{50 b_w}$	$\dfrac{\phi A_v f_y d}{V_u - \phi V_c}$
	Recommended Minimum†	—	—	4 in.
	Maximum†† (ACI 11.5.4)	—	$\dfrac{d}{2}$ or 24 in.	$\dfrac{d}{2}$ or 24 in. for $\left(V_u - \phi V_c\right) \le \phi 4\sqrt{f'_c}\, b_w d$
				$\dfrac{d}{4}$ or 12 in. for $\left(V_u - \phi V_c\right) > \phi 4\sqrt{f'_c}\, b_w d$

*Members subjected to shear and flexure only; $\phi V_c = \phi 2\sqrt{f'_c}\, b_w d$, $\phi = 0.75$ (ACI 11.3.1.1)
**$A_v = 2 \times A_b$ for U stirrups; $f_y \le 60$ ksi (ACI 11.5.2)
†A practical limit for minimum spacing is d/4
††Maximum spacing based on minimum shear reinforcement (= $A_v f_y/50b_w$) must also be considered (ACI 11.5.5.3).

Economical spacing of stirrups is considered to be greater than d/4. Common spacings of d/4, d/3 and d/2 are used to determine the values of ϕV_s at which the spacings can be increased.

$$\phi V_s = \frac{\phi A_v f_y d}{s}$$

This figure shows the size of V_n provided by $V_c + V_s$ (long dashes) exceeds V_u/ϕ in a step-wise function, while the spacing provided (short dashes) is at or less than the required s (limited by the maximum allowed). *(Note that the maximum shear permitted from the stirrups is* $8\sqrt{f'_c}\, b_w d$



The minimum recommended spacing for the first stirrup is 2 inches from the face of the support.

Torsional Shear Reinforcement

On occasion beam members will see twist along the access caused by an eccentric shape supporting a load, like on an L-shaped spandrel (edge) beam. The torsion results in shearing stresses, and closed stirrups may be needed to resist the stress that the concrete cannot resist.

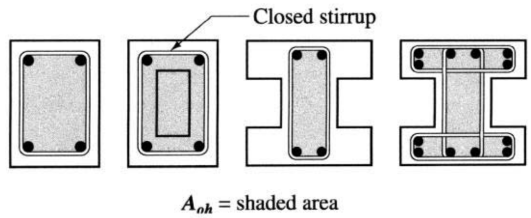


Fig. R11.6.3.6(b)—Definition of $\mathbf{A_{oh}}$

Development Length for Reinforcement

Because the design is based on the reinforcement attaining the yield stress, the reinforcement needs to be properly bonded to the concrete for a finite length (*both sides*) so it won't slip. This is referred to as the development length, l_d. Providing sufficient length to anchor bars that need to reach the yield stress near the end of connections are also specified by hook lengths. *Detailing reinforcement is a tedious job.* Splices are also necessary to extend the length of reinforcement that come in standard lengths. The equations are not provided here.

Development Length in Tension

With the proper bar to bar spacing and cover, the common development length equations are:

#6 bars and smaller: $l_d = \dfrac{d_b F_y}{25\sqrt{f'_c}}$ or 12 in. minimum

#7 bars and larger: $l_d = \dfrac{d_b F_y}{20\sqrt{f'_c}}$ or 12 in. minimum

Development Length in Compression

$$l_d = \frac{0.02 d_b F_y}{\sqrt{f'_c}} \le 0.0003 d_b F_y$$

Hook Bends and Extensions

The minimum hook length is $l_{dh} = \dfrac{1200 d_b}{\sqrt{f'_c}}$

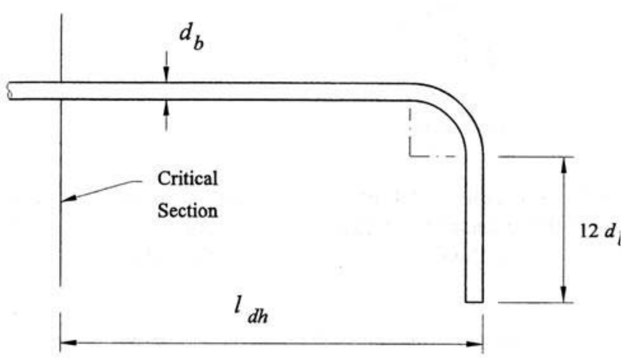


Figure 9-17: Minimum requirements for 90°bar hooks.

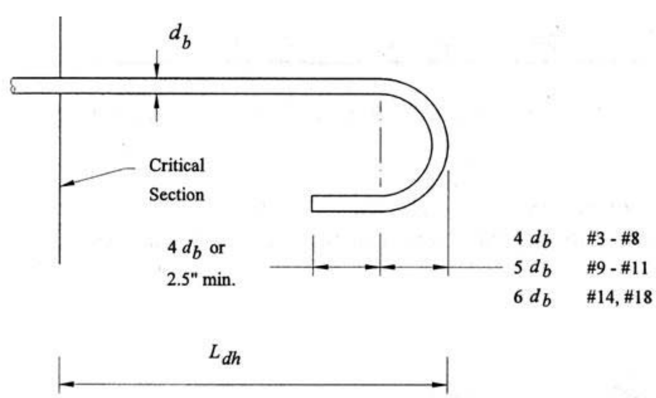


Figure 9-18: Minimum requirements for 180° bar hooks.

Modulus of Elasticity & Deflection

E_c for deflection calculations can be used with the transformed section modulus in the elastic range. After that, the cracked section modulus is calculated and E_c is adjusted.

Code values:

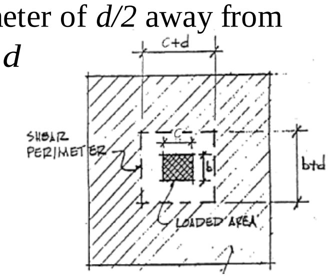
$$E_c = 57{,}000\sqrt{f'_c}\ \textit{(normal weight)} \qquad E_c = w_c^{1.5}33\sqrt{f'_c},\ w_c = 90\ lb/ft^3\ -\ 160\ lb/ft^3$$

Deflections of beams and one-way slabs need not be computed if the overall member thickness meets the minimum specified by the code, and are shown in Table 9.5(a) (see *Slabs*).

Criteria for Flat Slab & Plate System Design

Systems with slabs and supporting beams, joists or columns typically have multiple bays. The horizontal elements can act as one-way or two-way systems. Most often the flexure resisting elements are continuous, having positive and negative bending moments. These moment and shear values can be found using beam tables, or from code specified approximate design factors. Flat slab two-way systems have drop panels (for shear), while flat plates do not.

Two way shear at columns is resisted by the thickness of the slab at a perimeter of *d/2* away from the face of the support by the shear stress × cross section area: $V_c = v_c \times b_o d$

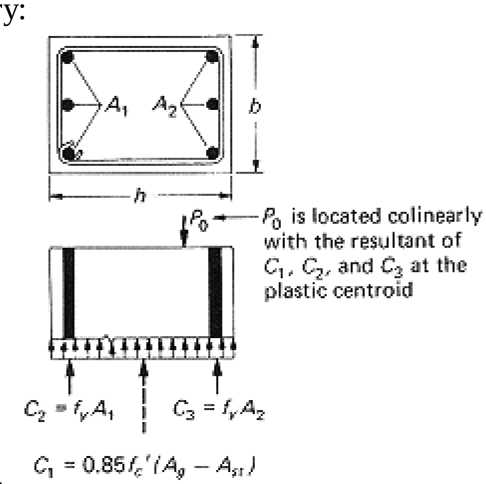


The shear stress (two way) $v_c = 4\sqrt{f'_c}$ so $\phi V_c = \phi 4\sqrt{f'_c}\ b_o d$

where b_o = perimeter length.

Criteria for Column Design

(American Concrete Institute) ACI 318-11 Code and Commentary:



$P_u \le \phi_c P_n$ where

P_u is a factored load

ϕ is a resistance factor

P_n is the nominal load capacity (strength)

Load combinations, ex:

1.4D (D is dead load)

1.2D + 1.6L (L is live load)

1.2D + 1.6L + 0.5W (W is wind load)

0.90D + 1.0W

For compression, ϕ_c = 0.75 and $P_n = 0.85P_o$ for spirally reinforced, ϕ_c = 0.65 $P_n = 0.8P_o$ for tied columns where $P_o = 0.85f'_c(A_g - A_{st}) + f_y A_{st}$ and P_o is the name of the maximum axial force with no concurrent bending moment.

Columns which have reinforcement ratios, $\rho_g = {}^{A_{st}}\!/\!_{A_g}$, in the range of 1% to 2% will usually be the most economical, with 1% as a minimum and 8% as a maximum by code..

Bars are symmetrically placed, typically.

Columns with Bending (Beam-Columns)

Concrete columns rarely see only axial force and must be designed for the combined effects of axial load and bending moment. The *interaction* diagram shows the reduction in axial load a column can carry with a bending moment.

Design aids commonly present the interaction diagrams in the form of load vs. equivalent eccentricity for standard column sizes and bars used.

Eccentric Design

The strength interaction diagram is dependant upon the strain developed in the steel reinforcement.

If the strain in the steel is less than the yield stress, the section is said to be *compression controlled.*

Below the *transition zone*, where the steel starts to yield, and when the net tensile strain in the reinforcement exceeds 0.005 the section is said to be *tension controlled.* This is a ductile condition and is preferred.

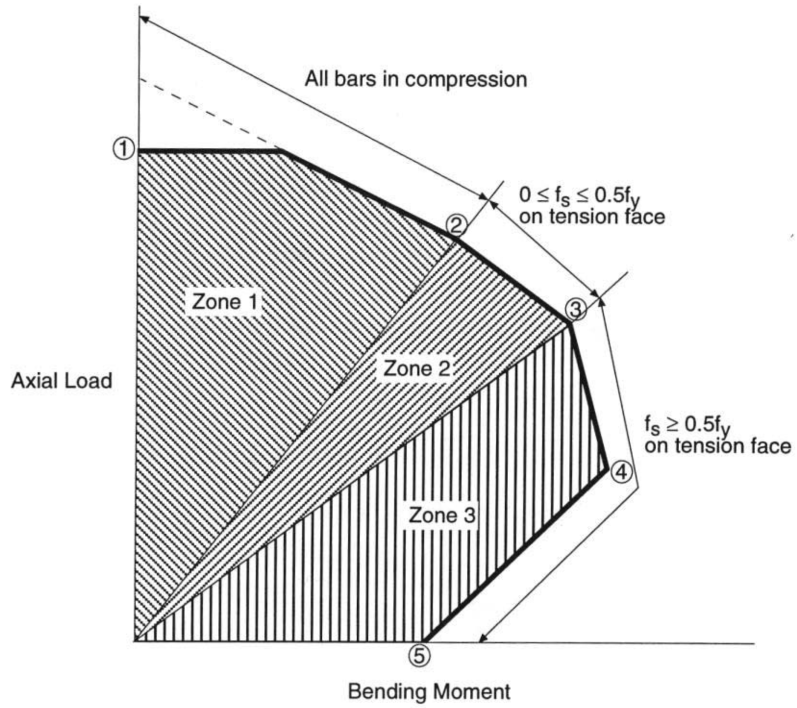


Figure 5-3 Transition Stages on Interaction Diagram

Rigid Frames

Monolithically cast frames with beams and column elements will have members with shear, bending and axial loads. Because the joints can rotate, the effective length must be determined from methods like that presented in the handout on Rigid Frames. The charts for evaluating k for non-sway and sway frames can be found in the ACI code.

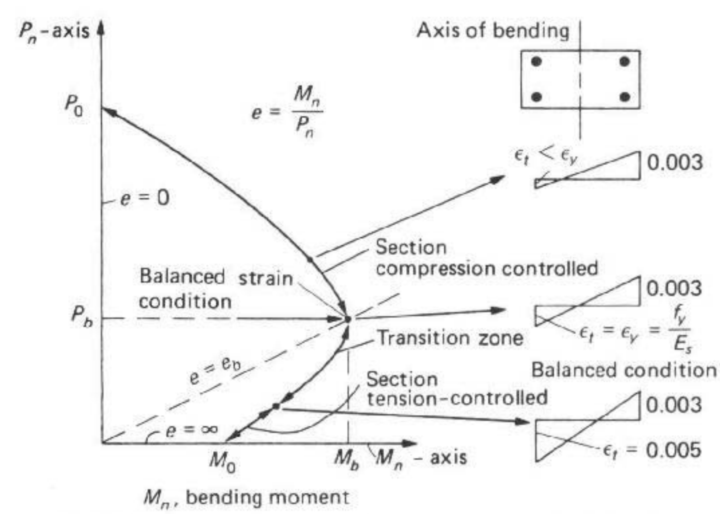
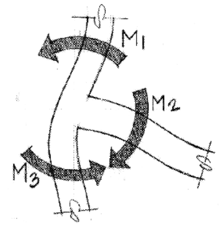


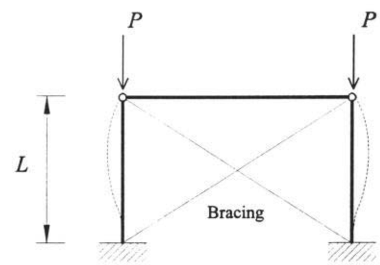
Figure 13.6.1 Typical strength interaction diagram for axial compression and bending moment about one axis. Transition zone is where $\epsilon_y \le \epsilon_t \le 0.005$.

<u>Frame Columns</u>

Because joints can rotate in frames, the effective length of the column in a frame is harder to determine. The stiffness (EI/L) of each member in a joint determines how rigid or flexible it is. To find k, the relative stiffness, G or Ψ, must be found for both ends, plotted on the alignment charts, and connected by a line for braced and unbraced fames.

$$G = \Psi = \frac{\Sigma EI/l_c}{\Sigma EI/l_b}$$



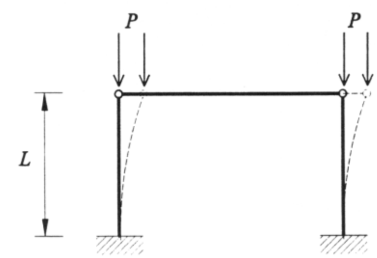


Braced – non-sway frame

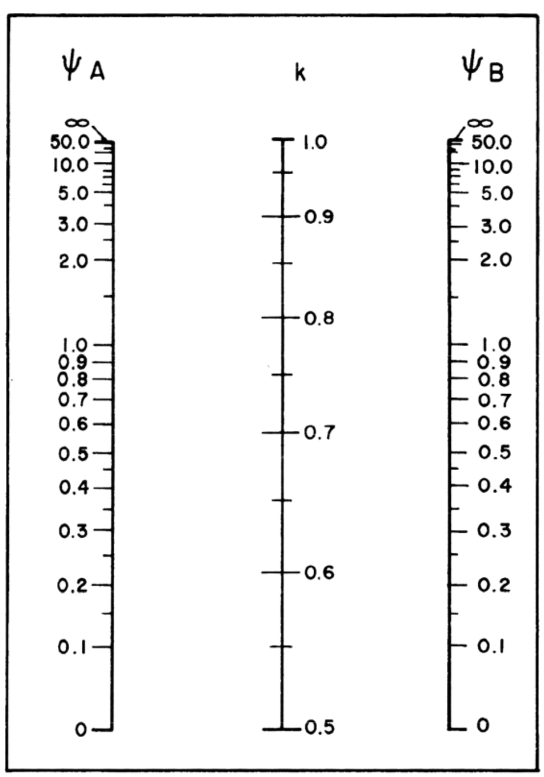
where

E = modulus of elasticity for a member
I = moment of inertia of for a member
l_c = length of the column from center to center
l_b = length of the beam from center to center

- For pinned connections we typically use a value of 10 for Ψ.
- For fixed connections we typically use a value of 1 for Ψ.

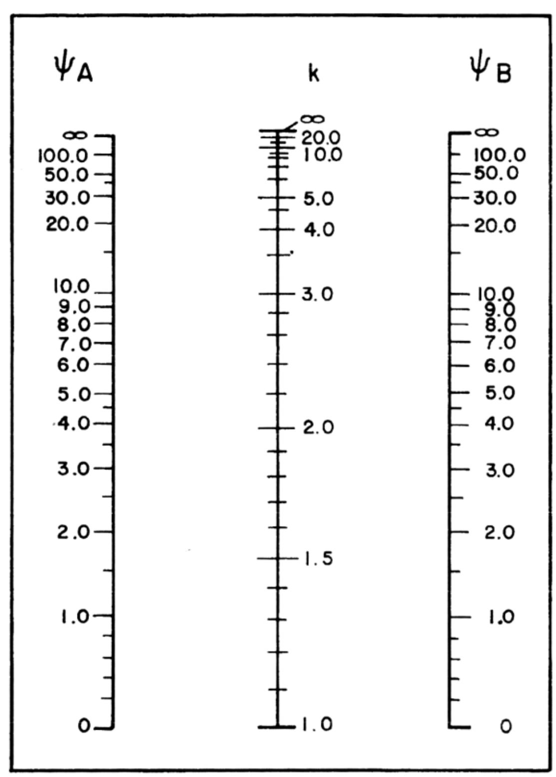


Unbraced – sway frame



(a)

Nonsway Frames



(b)

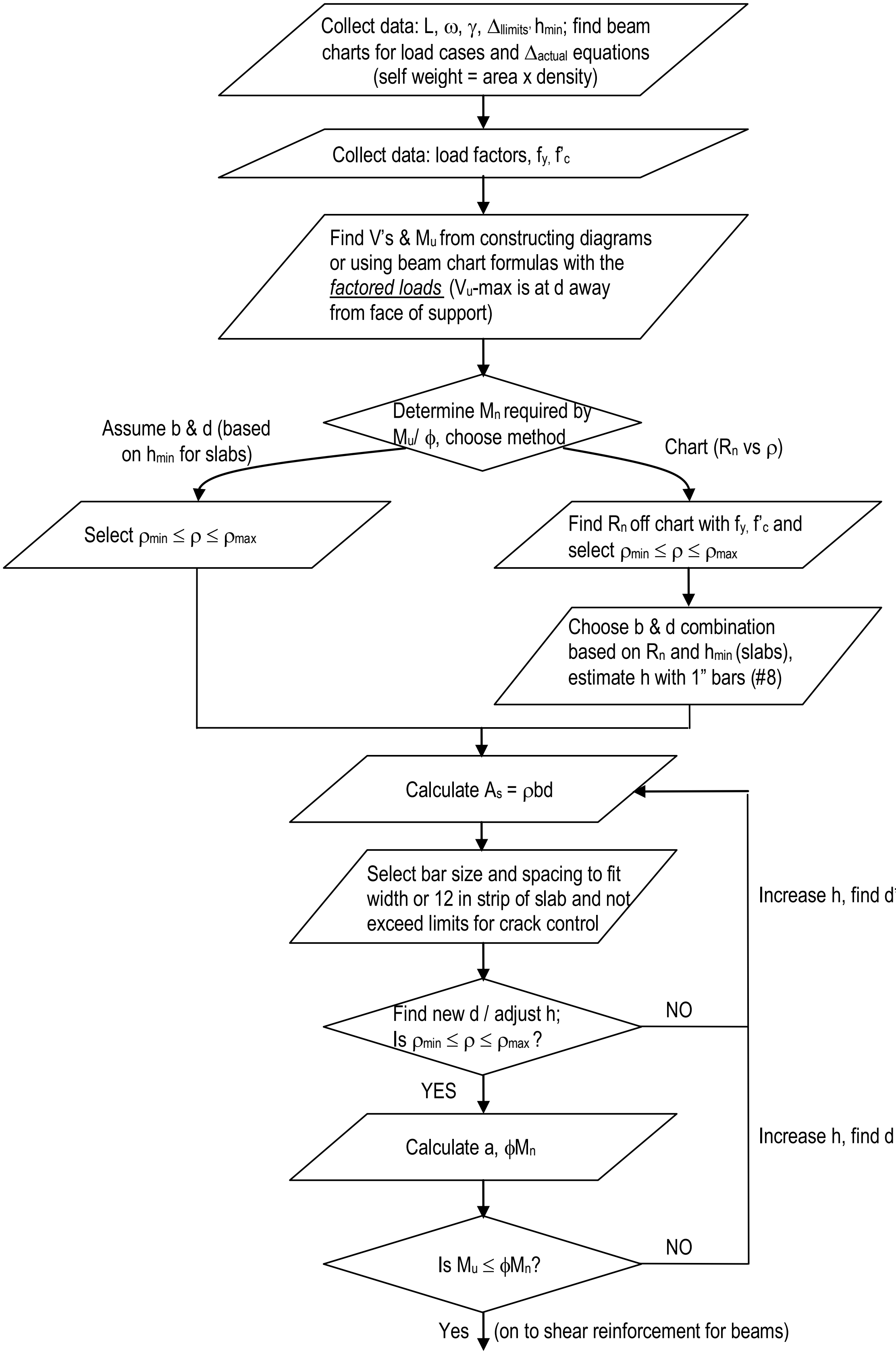
Sway Frames

Factored Moment Resistance of Concrete Beams, ϕM_n (k-ft) with f'_c = 4 ksi, f_y = 60 ksi[a]

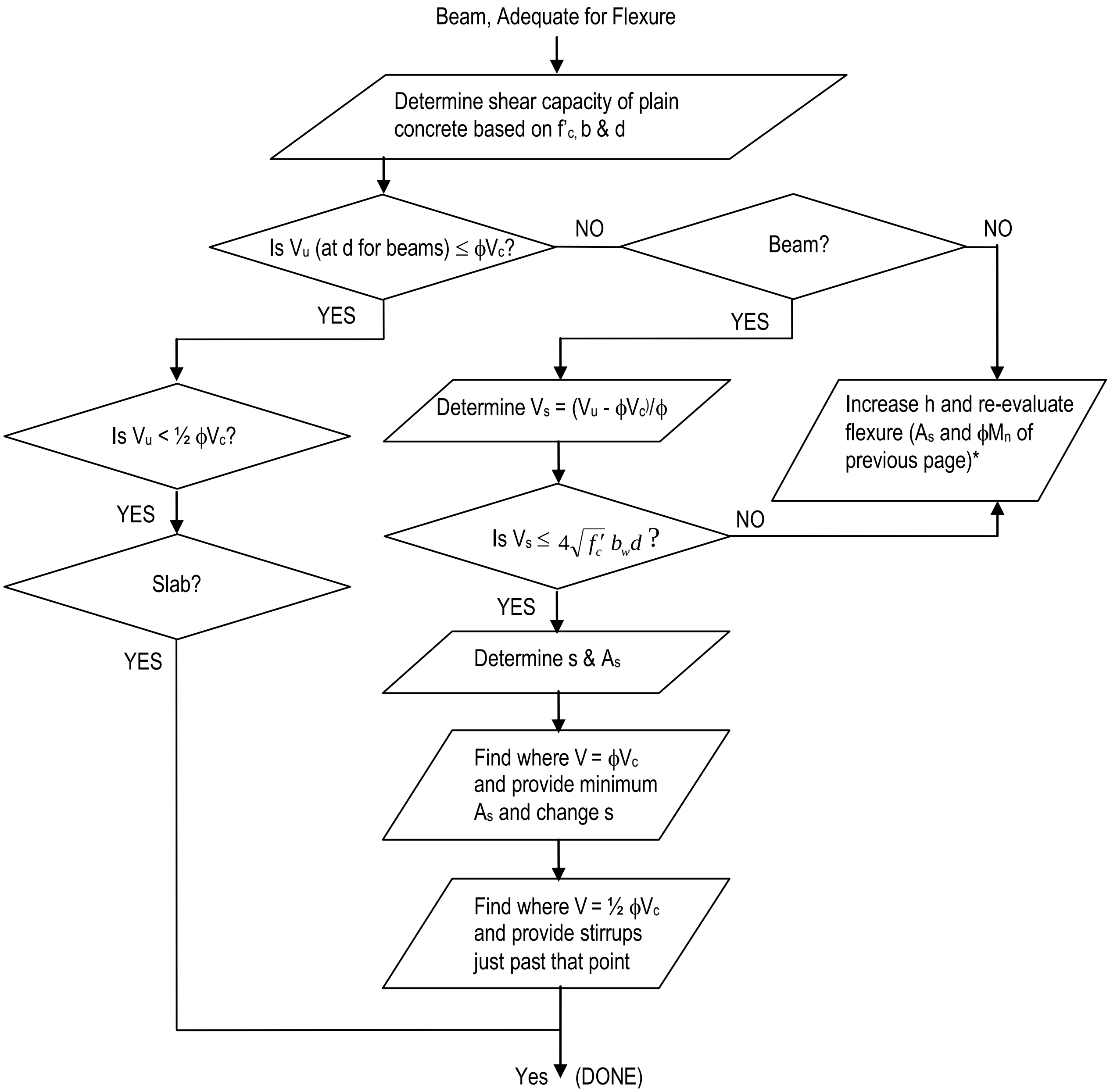
	Approximate Values for a/d		
	0.1	0.2	0.3
	Approximate Values for ρ		
b x d (in)	0.0057	0.01133	0.017
10 x 14	2 #6	2 #8	3 #8
	53	90	127
10 x 18	3 #5	2 #9	3 #9
	72	146	207
10 x 22	2 #7	3 #8	(3 #10)
	113	211	321
12 x 16	2 #7	3 #8	4 #8
	82	154	193
12 x 20	2 #8	3 #9	4 #9
	135	243	306
12 x 24	2 #8	3 #9	(4 #10)
	162	292	466
15 x 20	3 #7	4 #8	5 #9
	154	256	383
15 x 25	3 #8	4 #9	4 #11
	253	405	597
15 x 30	3 #8	5 #9	(5 #11)
	304	608	895
18 x 24	3 #8	5 #9	6 #10
	243	486	700
18 x 30	3 #9	6 #9	(6 #11)
	385	729	1074
18 x 36	3 #10	6 #10	(7 #11)
	586	1111	1504
20 x 30	3 # 10	7 # 9	6 # 11
	489	851	1074
20 x 35	4 #9	5 #11	(7 #11)
	599	1106	1462
20 x 40	6 #8	6 #11	(9 #11)
	811	1516	2148
24 x 32	6 #8	7 #10	(8 #11)
	648	1152	1528
24 x 40	6 #9	7 #11	(10 #11)
	1026	1769	2387
24 x 48	5 #10	(8 #11)	(13 #11)
	1303	2426	3723

[a]Table yields values of factored moment resistance in kip-ft with reinforcement indicated. Reinforcement choices shown in parentheses require greater width of beam or use of two stack layers of bars. (*Adapted and corrected from Simplified Engineering for Architects and Builders, 11th ed, Ambrose and Tripeny, 2010.*

Beam / One-Way Slab Design Flow Chart



Beam / One-Way Slab Design Flow Chart - continued



ONE-WAY CONCRETE JOIST CONSTRUCTION: STEEL LAP PAN FORMING SYSTEM

CRSI

ENGINEERING DATA REPORT NUMBER 43

933 N. Plum Grove Rd., Schaumburg, Illinois 60173-4758

INTRODUCTION

This type of construction has frequently utilized ready-made steel pan forms of standard sizes. Depths and spacings for one-way concrete joist construction were standardized in 1932 by the U.S. Department of Commerce on the basis of an industry study by a committee of Architects/Engineers, Contractors and Steel Form Suppliers. That early standard has since been replaced by ANSI/CRSI A48.1–1986 “Forms for One-Way Joist Construction,” which establishes standard dimensions for one-way joist forms. Standard form widths are 20, 30, 40*, 53* and 66* inches, corresponding to structural modules ranging from 2 to 6 feet. Standard depths are 8, 10, 12, 14, 16, 18, 20, 22* and 24* inches. Not all depths are manufactured in each form width. Filler forms and tapered endforms are usually available locally to fit varying floor layouts and sizes. This type of construction is well established with a long record of successful use.

Joist construction was developed to reduce dead weight and reinforcement. As desired spans increase, the efficiency of solid slab construction is rapidly offset by the increase in the dead load. Joist construction enables the Architect/Engineer to provide the depth required for adequate stiffness and efficient utilization of the reinforcement without excessively high dead load/live load ratios. Standard size reusable forms make it possible to eliminate unnecessary dead weight with overall economy. Longer spans or relatively heavy loads can be accommodated by using tapered end forms which permit widening of the ribs in areas of high shear.

DESCRIPTION

One-way concrete joist construction provides a monolithic combination of regularly-spaced joists (ribs) and a thin slab of concrete cast in place to form an integral unit with the supporting beams, columns and walls. In one-way concrete joist construction, the joists are arranged in one direction between parallel supports. Joist rib widths vary from 4 to 6 inches. Standard endforms consist of square endforms. A tapered endform for a 2-foot module tapers from 20 to 16 inches wide in a distance of 36 inches. A tapered endform for a 3-foot module tapers from 30 to 25 inches (in some systems 26 inches) wide in a distance of 36 inches. See Figure 1.

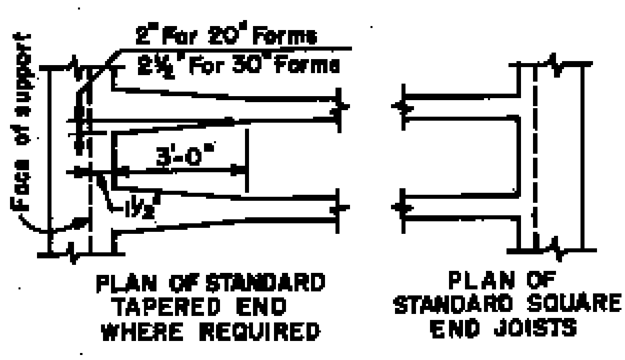


Figure 1 Tapered Endforms

Wide-module joist systems (also referred to as “skip-joist” systems) are defined as joist systems with a clear spacing between the ribs of more than 30 inches. Since this module, for application of the ACI Building Code, exceeds the rib spacing limit for standard joist construction (Section 8.11.3), wide-module joists become repetitive “T” beams and are subject to design requirements for such members.

Joist widths for wide-module joist systems vary from 6 to 8 inches. Standard forms for void spaces between ribs are 40, 53 or 66 inches wide and 12, 14, 16, 18, 20, 22 or 24 inches deep. Standard endforms consist of square endforms. Use of square end joist forms simplifies forming. Tapered endforms are generally not available for wide-module systems. See Table 1.

FORMWORK SELECTION CONSIDERATIONS

Maximum overall economy in concrete joist construction, as in any cast-in-place reinforced concrete design, is achieved by considering the relatively high cost of formwork and construction time versus material costs. Almost invariably overall economy is achieved by the maximum reuse of the same forms throughout the project, not only throughout each floor but also the same layout and

* For wide-module joists only.
1 inch = 25.4 millimeters

TABLE 1 Standard Dimensions of Forms for One-way Joist Construction[1]

System	Standard Forms		Special Filler Forms[4]	
	Width[2]	Depth[3]	Width[2]	Depth[3]
2N-00	20	8,10,12	10,15	8,10,12
3N-00[5]	30	8,10,12,14,16,20	10,15,20	8,10,12,14,16,20
4N-00[6]	40	12,14,16,18,20,22,24	20,30	12,14,16,18,20,22,24
5N-00	53	16,20	—	—
6N-00	66	14,16,20	—	—

NOTES

1. All dimensions are in inches, except the module designations.
2. Width is the horizontal clear distance, between two consecutive joists, measured at the bottom of the joists.
3. Depth is the vertical distance, measured between two consecutive joists, from the underside of the concrete slab to the bottom of the joists.
4. Special filler forms may be available only in limited quantities. Availability should be investigated before specifying these forms.
5. Tapered endforms are available for the one-way 3N-00 module. These forms are 30 inches wide at one end and 25 inches wide at the other end, and they are 36 inches long. Standard depths of these forms are 8, 10, 12, 14, 16, and 20 inches.
6. Tapered endforms are available for the one-way 4N-00 module. These forms are 40 inches wide at one end and 34 inches wide at the other end, and they are 36 inches long. Standard depths of these forms are 12, 14, 16, 18, 20, 22, and 24 inches. These forms are generally available only on the West Coast.

size of forms for all levels of the structure.

The use of the lap-type steel one-way pan system is probably one of the most efficient methods of reinforced concrete construction ever devised in terms of spans and applied loads versus volume of concrete and weight of reinforcing steel. A steel lap pan system has one major drawback: typically it can produce no better than a Class 'C' finish.

Sectional steel pan forms can adjust to varying site conditions without extensive detailing and fabricating of special shapes. The Architect/Engineer is allowed great freedom in varying joist widths for accommodating concentrated loads by slightly adjusting the center-to-center spacing of the ribs. Clearing blockouts, drops and other interferences is accomplished by workers simply starting and stopping pan runs as required. Steel pan forms are a proper forming system to consider when evaluating design choices because they provide inherently stiff floor systems for the volume of concrete and reinforcing steel, and the forms are economical to obtain and erect when concrete esthetics are not a concern.

Project specifications are often vague with reference to laps and single one-piece voids. The Architect/Engineer's expectations are generally different from those of the Contractor. The Contractor should be very sensitive to the Class of finish for which the pan forms are intended. Lap pans are generally inappropriate for exposed work. The Architect/Engineer's attention should be focused on the end product results during pre-construction meetings as to the finish that these forms are and are not capable of producing. For instance, when pans are lapped, both the joist width and slab thickness vary slightly. ACI 117 tolerances for joists and slabs are +3/8, –1/4 inch in width and thickness (Section 4.4.1). The Contractor needs to ensure that the erection of the formwork is performed with a reasonable degree of accuracy. Finally, the Architect/Engineer may want to recognize the challenges with this type of forming and specify a joist width one inch larger than required by design. While it is usually better to cast an onsite mockup section, it may be more practical and prudent to have the Architect/Engineer and Owner participate in a site visit to a structure of similar construction and application to measure both esthetics and performance.

FABRICATION AND ERECTION

The typical lap pan is a 16-gauge or a 14-gauge piece of sheet metal, 3 feet long, bent into one of three traditional shapes (see Figure 2) with varying flange widths dependent on style and Supplier. Both ends are open. A chalkline on the deck or soffit form should be used to align the pans. End caps are placed first and work proceeds toward the center of the member from both ends, overlapping the pans until proper closure is achieved. Flanged pans are nailed into position. After the pans are tightly in place, they should be oiled before other trades proceed with their work. See Figure 3.

The normal procedure for setting pans is to set the end caps first, nailed to the deck form on the line where the coffer begins. A long section of pan is first placed over the end cap. Then, through pre-punched matching holes in the top flange of the end cap and the top surface of the pan section, nails are dropped in to form a bond between the form sections. It is not uncommon to see small machine screws or center pin rivets used. However, form stripping procedures need to be considered with these types of fasteners. This connecting procedure also assists in preventing the end cap from collapsing inward under the pressure of concrete placing. The pan section is then nailed in place and a free standing steel or wood diaphragm (internal brace) is inserted into the form (suggested spacing is 18 inches on center under normal concrete placing conditions and should include the lap point between pans) and nailed in place. The next pan section is then installed, reasonably lapping (1 to 5 inches) the previous section and the previous procedure is repeated until the coffer is completely formed. It should be noted that all pans may require diaphragms to resist lateral pressures. However, 14-gauge pans with a depth of 16 inches or greater should always be installed with internal bracing. The soffits of all steel pans should be strengthened with some type of permanently attached internal brace, the most common of which is a welded sheet metal angle at least of the same gauge as the body of the pan. Because the steel lap pan system is characterized by offsets, fins and

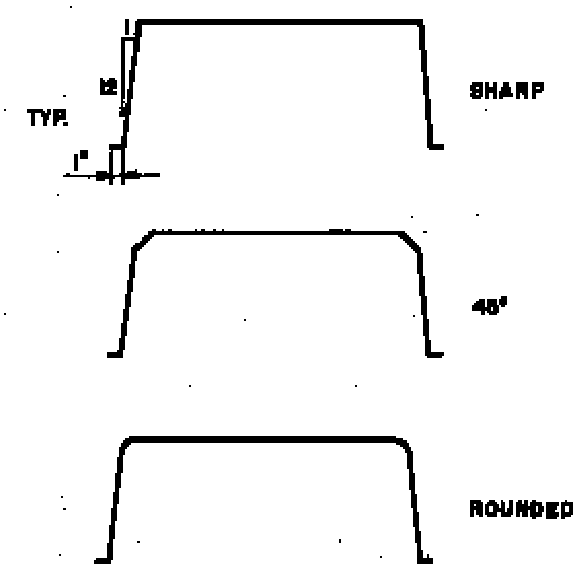


Figure 2 Lap Pan Shapes

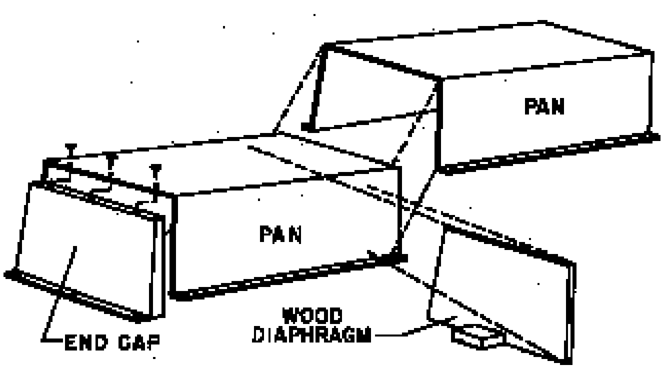


Figure 3 Setting Lap Pans

protrusions as well as chips and dings that result from the removal of the pan sections, the contract documents should include guidance and information on acceptable tolerances for formed surfaces.

It should be pointed out that care must be taken with the installation of any embedded items or mechanical inserts or fixtures. If the attachment of these items is not considered in relationship to the stripping of these forms they may act as anchors preventing the removal of the pan forms. Therefore, it is recommended that only center pin soft rivets be used. The center pin of the rivet will remain exposed on the underside of the pan form and can be removed prior to stripping. This will allow the body of the rivet to close as the pan is stripped, permitting the easy removal of the forms.

TOLERANCES

Tolerance guidance can be found in several ACI standards and reports. ACI 117, "Standard Tolerances for Concrete Construction and Materials"; ACI 301, "Specifications for Structural Concrete"; and ACI 347R, "Guide to Formwork for Concrete," provide information on finished surfaces, but do not address pan joist surfaces specifically. ACI 117 and ACI 347R limit offsets and other irregularities based on "Class" of surface finish. See Table 2. The ACI 117 standard might be regarded as the most authoritative. The mandatory specification checklist in ACI 117 requires the Architect/Engineer to designate the intended Class of surface finish and thereby establish the tolerance for form offsets. ACI 301 addresses the finishing of formed surfaces in Chapter 2 and differentiates between rough form finishes (those not exposed to public view) and smooth finishes (exposed to public view). ACI 301 requires: "Patch tie holes and defects. Remove all fins completely." for smooth formed finishes, but permits up to 1/4 inch fins for rough finishes. As a default, Article 5.3.3.5 of ACI 301 calls for the finish to be based on exposure to public view where

Type of Irregularity	Class of Surface Finish			
	A	B	C	D
Gradual (ACI 347R)	1/8	1/4	1/2	1
Abrupt (ACI 117)	1/8	1/4	1/2	1
(ACI 347R)	1/8	1/4	1/4	1

Class A: For surfaces prominently exposed to public view where appearance is of special importance.

Class B: Coarse-textured concrete-formed surfaces intended to receive plaster, stucco, or wainscoting.

Class C: General standard for permanently exposed surfaces where other finishes are not specified.

Class D: Minimum-quality surface where roughness is not objectionable, usually applied where surfaces will be concealed.

surface finish is not designated in the contract documents.

TABLE 2 Surface Finish Class

ACI Committee 347 notes that revisions of the 347R report are in progress to change the limit for abrupt offsets within Class C finish to 1/2 inch, consistent with ACI 117. Although ACI 347R cautions against using pry bars directly against concrete to remove formwork, this is common practice in many areas of the country.

CONCLUSION

Surface irregularities should be expected in pan joist construction. It is difficult to patch surface spalls successfully. The patch may be more noticeable

ACKNOWLEDGMENT

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than the spall. The continuing use of the steel lap pan joist form system remains a proven and excellent method of forming site cast reinforced concrete floor systems in non-critically exposed applications.

REFERENCES

1. *Formwork For Concrete*, SP-4, 5th Edition, American Concrete Institute, Farmington Hills, Michigan, 1995.

2. “Standard Tolerances for Concrete Construction and Materials (ACI 117-90)”, American Concrete Institute.

3. “Specifications for Structural Concrete (ACI 301-96)”, American Concrete Institute.

4. “Guide to Formwork for Concrete (ACI 347R-94)”, American Concrete Institute.

5. “Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95)”, American Concrete Institute.

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ENGINEERING
DATA REPORT
NUMBER 46

CRSI

WIDE-MODULE JOIST SYSTEMS — REVISITED

A SERVICE OF THE CONCRETE REINFORCING STEEL INSTITUTE
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INTRODUCTION

Standard joist construction, as defined in ACI 8.11*, includes a limit of 30 in. on the maximum clear spacing between ribs. According to the accompanying Commentary for ACI 8.11, the rationale for the limit on rib spacing is that ACI 8.11 includes special provisions for higher design shear strengths of the concrete and less concrete cover over the reinforcement. Dimensions of removable standard size forms for modules of 2′-0″ and 3′-0″ are given in Table 1.

An increasingly popular type of joist construction is the wide-module joist system. Wide-module joist systems may be defined as joist systems with clear spacings of ribs exceeding 30 in. Since the rib spacings for wide-modules are greater than the ACI 318 Code limit for standard joist construction, wide-module joists must be designed as T-beams. In other words, the design shear strength of concrete and minimum concrete cover requirements for beams are applicable to the T-beams of wide-module joist systems. Economy in construction is achieved through the use of combinations of commonly available, re-usable standard joist forms. Standard form dimensions for modules of 4′-0″, 5′-0″ and 6′-0″ are given in Table 1.

* References in this report to "Building Code Requirements for Structural Concrete (ACI 318-99)" are given as "ACI" followed by the appropriate section number.

Table 1 Dimensions of Forms for One-Way Joist Construction (1)

Module	Standard Forms		Special Filler Forms (4)	
	Width (2)	Depth (3)	Width (2)	Depth (3)
Standard Joist Construction				
2′-0″	20	8, 10, 12	10, 15	8, 10, 12
3′-0″ (5)	30	8, 10, 12, 14, 16, 20	10, 15, 20	8, 10, 12, 14, 16, 20
Wide-Module Joist Construction				
4′-0″ (6)	40	12, 14, 16, 18, 20, 22, 24	20, 30	12, 14, 16, 18, 20, 22, 24
5′-0″	53	16, 20, 24	–	–
6′-0″	66	14, 16, 20, 24	–	–

NOTES

1. All dimensions are in inches, except the module designations.
2. Width is the horizontal clear distance, between two consecutive joists, measured at the bottom of the joists.
3. Depth is the vertical distance, measured between two consecutive joists, from the underside of the concrete slab to the bottom of the joists.
4. Special filler forms may be available only in limited quantities. Availability should be investigated before specifying these forms.
5. Tapered endforms are available for the one-way 3′-0″ module. These forms are 30 in. wide at one end and 25 in. wide at the other end, and they are 36 in. long. Standard depths of these forms are 8, 10, 12, 14, 16, and 20 in.
6. Tapered endforms are available for the one-way 4′-0″ module. These forms are 40 in. wide at one end and 34 inches wide at the other end, and they are 36 in. long. Standard depths of these forms are 12, 14, 16, 18, 20, 22, and 24 in. These forms are generally available only on the West Coast.

TYPICAL WIDE-MODULE JOIST DIMENSIONS

Figure 1 shows typical cross-sectional dimensions for 5′-0″ and 6′-0″ wide modules. As noted in the figure, the modules are formed with single size forms in widths of 53-in. and 66-in.

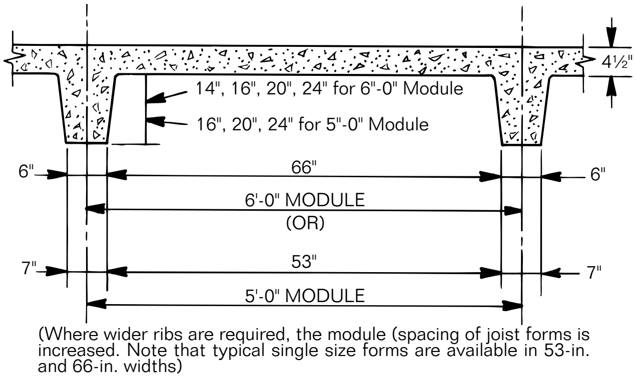

Figure 1 – Typical Wide-Module Joist Dimensions

The wide-module joist system can easily adapt to wider modules where required for architectural purposes, or to provide wider ribs where structural considerations require the use of larger reinforcing bars and higher shear capacity. Where single full-width forms are not readily available, combinations of smaller standard forms may be used with covers over the omitted ribs. See Figure 2 for an example of a 66-in. clear spacing of ribs resulting from using 30-in. standard joist forms.

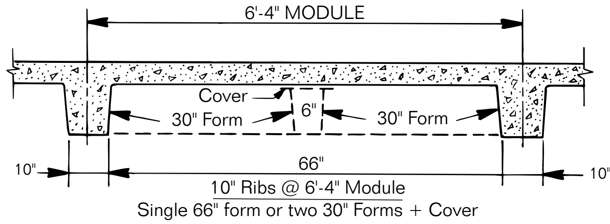

Figure 2 – 66-in. Module Using 30-in. Standard Forms

TWO-WAY JOIST CONSTRUCTION

Two-way joist construction, meeting the requirements of ACI 8.11, is commonly called waffle slab construction. Waffle slabs are designed as two-way flat slab systems under Chapter 13 of the ACI 318 Building Code. The dimensions of forms for standard two-way joist construction, i.e., waffle slabs, with modules of 2′-0″, 2′-6″ and 3′-0″, are given in Table 2.

Table 2 also includes the standard dimensions of forms for two-way joist construction with 4′-0″ and 5′-0″ modules.

GENERAL STRUCTURAL CONSIDERATIONS

Top Slab. To meet the fire ratings of the statutory building codes, the required thickness of the top slab is usually about 4½-in. In standard joist construction (ACI 8.11), which limits the maximum clear spacing of the ribs to 30-in., the flexural capacity of a 4½-in. top slab is underutilized. In contrast, the wide-module joist system takes advantage of the structural value of the slab thickness. A 4½-in. thick top slab is utilized more fully as a structural element.

Table 2 Dimensions of Forms for Two-Way Joist Construction (1)

System	Standard Forms		Special Filler Forms (4)	
	Width (2)	Depth (3)	Width (2)	Depth (3)
2′-0″ Module 19″ x 19″ Square with 2½″ Flanges	19 x 19	8, 10, 12, 14, 16	—	—
2′-6″ Module 24″ x 24″ Square with 3″ Flanges	24 x 24	8, 10, 12, 14, 16, 20	—	—
3′-0″ Module 30″ x 30″ Square with 3″ Flanges	30 x 30	8, 10, 12, 14, 16, 20	20 x 20 20 x 30	8, 10, 12, 14, 16, 20 8, 10, 12, 14, 16, 20
4′-0″ Module 41″ x 41″ Square with 3½″ Flanges	41 x 41	12, 14, 16, 18, 20, 24	—	—
5′-0″ Module 52″ x 52″ Square with 4″ Flanges	52 x 52	14, 16, 20, 24	40 x 40	14, 16, 20, 24

Notes 1 through 4 under Table 1 are also applicable to this Table 2.

Ribs. Since the wide-module "joists" are technically classified as beams, their design must conform to the requirements for T-beams (ACI 8.10). Principal design requirements are:

1. Minimum concrete cover. 1½ in. to stirrups and main flexural bars (top, bottom, and sides) instead of ¾ in. (ACI 7.7.1).

2. Design shear strength of concrete. $\phi V_c = \phi 2\sqrt{f'_c}\, b_w d$ instead of $\phi V_c = \phi 2.2\, b_w d$ (ACI 11.3.1.1 and 8.11.8).

3. Minimum area of shear reinforcement. $A_v = 50(b_w s)/f_y$ where factored shear $V_u > 0.5\,\phi V_c$ (ACI 11.5.5.3 and 11.5.5.1).

4. Reinforcing steel requirements and recommended details. Alternative arrangements to provide required shear reinforcement include the common open U-stirrup. With minimum rib widths, the maximum size of the main tensile reinforcement becomes limited by concrete cover requirements. And with minimum rib widths, fabricating constraints may require wider U-stirrups. These conditions may require that the U-stirrups be angled to fit. For minimum rib widths, the use of single leg stirrups simplifies placing. A special note should be included on the design drawings and placing drawings to require alternating of the stirrup positions. See Figure 3.

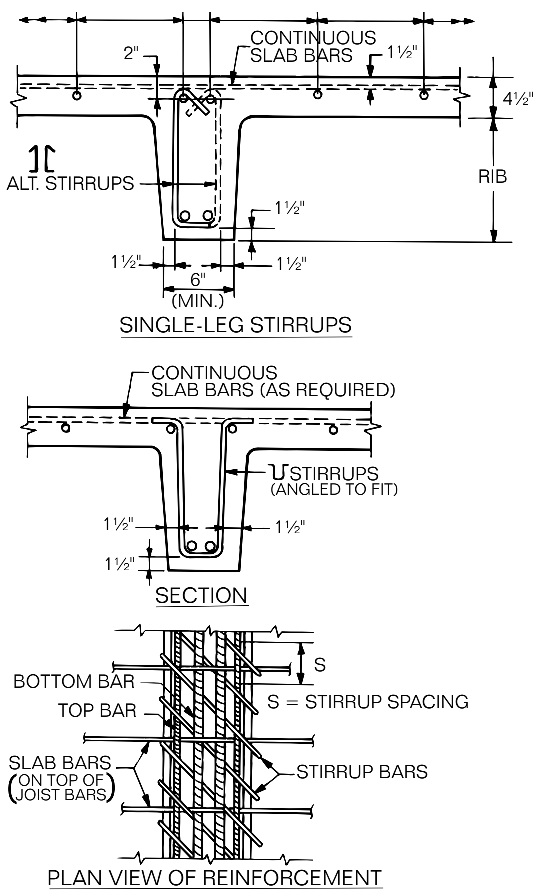

Figure 3 – Details for Single Leg and Common U-Stirrup Applictions for Shear Reinforcement in Wide-Module Joist

Welded wire fabric (plain or deformed) can also be used as shear reinforcement. The vertical wires are developed by two horizontal wires spaced at 2 in. maximum at the top and at the bottom (Figure 4).

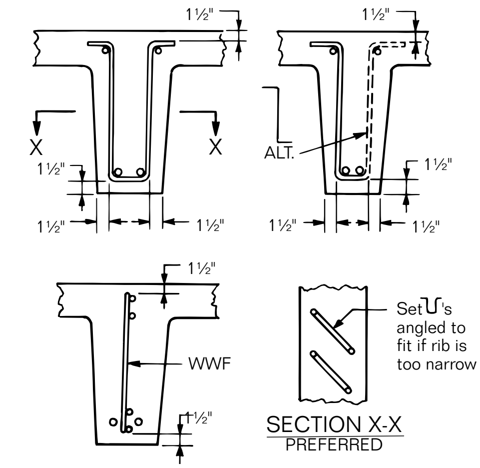

Figure 4 – Alternative Stirrup Arrangements

Live-load reduction. A typical wide-module joist system can often be laid out to take maximum live-load reductions as permitted by most national and local building codes. For rectangular bays, where the joists' capacity permits the layout of joists parallel to the longer span, the longer span wide module width provides a larger area to qualify for the reduction.

Formwork economy. If conditions permit, further formwork economy will result from the use of a uniform depth. See Section A-A in Figure 5. The entire procedure of formwork utilizes the same height shores and provides a solid level work platform and the simplest formwork. Placing of reinforcement is facilitated and minimum time is achieved for completion of each floor level. The supporting beam will usually be wide enough to reduce shear reinforcement, often with the use of the higher two-way design shear strength immediately around the column. See layout of typical wide-module joist system in Figure 5.

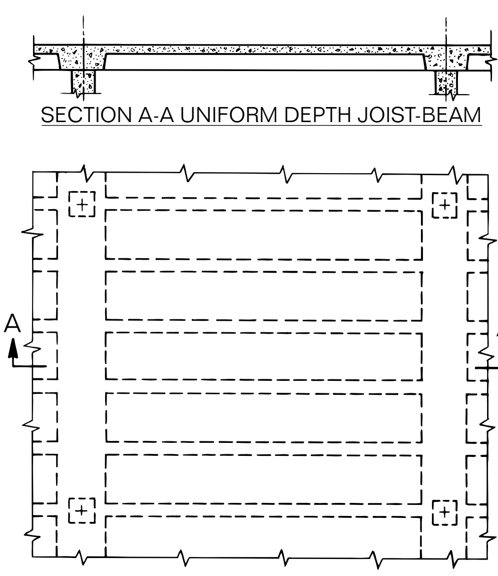

Figure 5 – Typical Wide-Module Joist Layout

SUPPORTING REINFORCEMENT

Chapter 3 in the CRSI *Manual of Standard Practice* contains information on the various types of bar supports used in reinforced concrete construction. Industry practices for the placing of bar supports are presented in the chapter. Recommendations for supporting reinforcing bars in standard one-way and two-way (waffle slabs) joist construction are also included in Chapter 3.

Recently, the CRSI technical committees have prepared recommendations for supporting the shrinkage and temperature reinforcement in the top slabs of wide-module joist construction:

> For wide-module joist systems, it is recommended that the shrinkage and temperature reinforcement be supported by placing rows of slab bolsters at right angles to the shrinkage and temperature bars and spaced at 4′-0″ on center maximum, unless otherwise shown in the Contract Documents.
>
> Placing practices in certain geographical areas of the country may prefer to substitute individual bar supports (steel wire, all-plastic, or precast concrete) in lieu of continuous bar supports. If individual bar supports are used, they should be placed at a maximum spacing of 4′-0″ on center each way.

DESIGN AND DETAILING AIDS

The following publications provide guidance in designing and detailing reinforced concrete standard joist and wide-module joist systems.

1. *CRSI Design Handbook*, Concrete Reinforcing Steel Institute, 8th Edition, 1996.
2. *Reinforcing Bar Detailing*, Concrete Reinforcing Steel Institute, 4th Edition, 2000.
3. *ACI Detailing Manual*, American Concrete Institute, SP-66, 1994.
4. "HB1JOIST and HB2JOIST, Handbook Computer Programs", Concrete Reinforcing Steel Institute, 1997.
5. "Effective Width of One-Way Monolithic Joist Construction as a Two-Way System", Structural Bulletin No. 8, Concrete Reinforcing Steel Institute, 1983.
6. *Workbook for Evaluating Concrete Building Designs*, Concrete Reinforcing Steel Institute, 2nd Edition, 1997.

CLOSING COMMENTS

Potential savings in both materials and construction with the use of wide-module joist systems include:

- Utilization of the top slab required for fire rating,
- Elimination of 50 % of the ribs,
- Uniform height of the deck form with the wide beam,
- Easy adjustments to fit the common range of modular column layouts,
- Less field labor time for construction.

The five preceding items are *direct* potential savings. *Indirect* benefits are:

- Elimination of half the ribs reduces dead load and reinforcement, and
- The wider rib spacing creates a larger supported area per rib, thereby increasing the allowable live load reductions and further reducing reinforcement.

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Examples:
Reinforced Concrete

Example 1

A simply supported beam 20 ft long carries a service dead load of 300 lb/ft and a live load of 500 lb/ft. Design an appropriate beam (for flexure only). Use grade 40 steel and concrete strength of 5000 psi.

SOLUTION:
Find the design moment, M_u, from the factored load combination of 1.2D + 1.6L. It is good practice to guess a beam size to include self weight in the dead load, because “service” means dead load of everything except the beam itself.

Guess a size of 10 in x 12 in. Self weight for normal weight concrete is the density of 150 lb/ft^3 multiplied by the cross section area: self weight = $150\,{}^{lb}\!/\!_{ft^3}(10in)(12in)\cdot(\frac{1\,ft}{12in})^2$ = 125 lb/ft

w_u = 1.2(300 lb/ft + 125 lb/ft) + 1.6(500 lb/ft) = 1310 lb/ft^2

The maximum moment for a simply supported beam is $\frac{wl^2}{8}$:

M_u = $\frac{w_u l^2}{8} = \frac{1310\,{}^{lb}\!/\!_{ft}(20\,ft)^2}{8}$ 65,500 lb-ft

M_n required = M_u/ϕ = $\frac{65{,}500^{lb-ft}}{0.9}$ = 72,778 lb-ft

To use the design chart aid, find $R_n = \frac{M_n}{bd^2}$, estimating that d is about 1.75 inches less than h:

d = 12in – 1.75 in = 10.25 in

Table 3.7.1
Total Areas for Various Numbers of Reinforcing Bars

Bar Size	Nominal Diameter (in.)	Weight (lb/ft)	Number of Bars 1	2	3	4	5	6	7	8	9	10
#3	0.375	0.376	0.11	0.22	0.33	0.44	0.55	0.66	0.77	0.88	0.99	1.10
#4	0.500	0.668	0.20	0.40	0.60	0.80	1.00	1.20	1.40	1.60	1.80	2.00
#5	0.625	1.043	0.31	0.62	0.93	1.24	1.55	1.86	2.17	2.48	2.79	3.10
#6	0.750	1.502	0.44	0.88	1.32	1.76	2.20	2.64	3.08	3.52	3.96	4.40
#7	0.875	2.044	0.60	1.20	1.80	2.40	3.00	3.60	4.20	4.80	5.40	6.00
#8	1.000	2.670	0.79	1.58	2.37	3.16	3.95	4.74	5.53	6.32	7.11	7.90
#9	1.128	3.400	1.00	2.00	3.00	4.00	5.00	6.00	7.00	8.00	9.00	10.00
#10	1.270	4.303	1.27	2.54	3.81	5.08	6.35	7.62	8.89	10.16	11.43	12.70
#11	1.410	5.313	1.56	3.12	4.68	6.24	7.80	9.36	10.92	12.48	14.04	15.60
#14[a]	1.693	7.65	2.25	4.50	6.75	9.00	11.25	13.50	15.75	18.00	20.25	22.50
#18[a]	2.257	13.60	4.00	8.00	12.00	16.00	20.00	24.00	28.00	32.00	36.00	40.00

[a] #14 and #18 bars are used primarily as column reinforcement and are rarely used in beams.

R_n = $\frac{72{,}778^{lb-ft}}{(10in)(10.25in)^2}\cdot(12\,{}^{in}\!/\!_{ft})$ = 831 psi

ρ corresponds to approximately 0.023, so the estimated area required, A_s, can be found:

$A_s = \rho bd$ = (0.023)(10in)(10.25in) = 2.36 in^2

The number of bars for this area can be found from handy charts.

(Whether the number of bars actually fit for the width with cover and space between bars must also be considered.)

Try A_s = 2.37 in^2 from 3#8 bars

d = 12 in – 1.5 in (cover) – ½ (8/8in diameter bar) = 10 in

Find the moment capacity of the beam as designed, ϕM_n

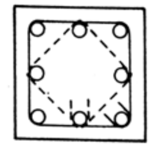
a = $A_sf_y/0.85f'_cb$ = 2.37 in^2 (40 ksi)/[0.85(5 ksi)10 in] = 2.23 in

$\phi M_n = \phi A_sf_y(d-a/2)$ = $0.9(2.37in^2)(40ksi)(10in-\frac{2.23in}{2})\cdot(\frac{1}{12\,{}^{in}\!/\!_{ft}})$ = 63.2 k-ft $\ngtr$ 64 k-ft needed (not OK)

So, we can increase d to 13 in, and ϕM_n = 70.3 k-ft (OK). Or increase A_s to 2 # 10’s (2.54 in^2), for a = 2.39 in and ϕM_n of 67.1 k-ft (OK).

Example 2

Determine the capacity of a 16" x 16" column with 8- #10 bars, tied. Grade 40 steel and 4000 psi concrete.



SOLUTION:

Find ϕP_n, with ϕ=0.65 and $P_n = 0.80P_o$ for tied columns and

$$P_o = 0.85 f_c'(A_g - A_{st}) + f_y A_{st}$$

Steel area (found from reinforcing bar table for the bar size):

A_{st} = 8 bars × (1.27 in^2) = 10.16 in^2

ASTM STANDARD REINFORCING BARS

Bar size, no.	Nominal diameter, in.	Nominal area, in.^{2}	Nominal weight, lb/ft
3	0.375	0.11	0.376
4	0.500	0.20	0.668
5	0.625	0.31	1.043
6	0.750	0.44	1.502
7	0.875	0.60	2.044
8	1.000	0.79	2.670
9	1.128	1.00	3.400
10	1.270	1.27	4.303
11	1.410	1.56	5.313
14	1.693	2.25	7.650
18	2.257	4.00	13.600

Concrete area (gross):

A_g = 16 in × 16 in = 256 in^2

Grade 40 reinforcement has f_y = 40,000 psi and f_c' = 4000psi

ϕP_n = (0.65)(0.80)[0.85(4000 psi)(256 in^2 – 10.16 in^2) + (40,000 psi)(10.16 in^2)] = 646,026 lb = 646 kips

Case Study in Reinforced Concrete

adapted from Simplified Design of Concrete Structures, James Ambrose, 7th ed.

Building description

The building is a three-story office building intended for speculative rental. Figure 17.37 presents a full-building section and a plan of the upper floor. The exterior walls are permanent. The design is a rigid perimeter frame to resist lateral loads.

Loads (UBC 1994)

Live Loads:
- Roof:
  - 20 lb/ft^2
- Floors:
  - Office areas: 50 lb/ft^2 (2.39 kPa)
  - Corridor and lobby: 100 lb/ft^2 (4.79 kPa)
  - Partitions: 20 lb/ft^2 (0.96 kPa)

Wind: map speed of 80 mph (190 km/h); exposure B

Assumed Construction Loads:
- Floor finish: 5 lb/ft^2 (0.24 kPa)
- Ceilings, lights, ducts: 15 lb/ft^2 (0.72 kPa)
- Walls (average surface weight):
  - Interior, permanent: 10 lb/ft^2 (0.48 kPa)
  - Exterior curtain wall: 15 lb/ft^2 (0.72 kPa)

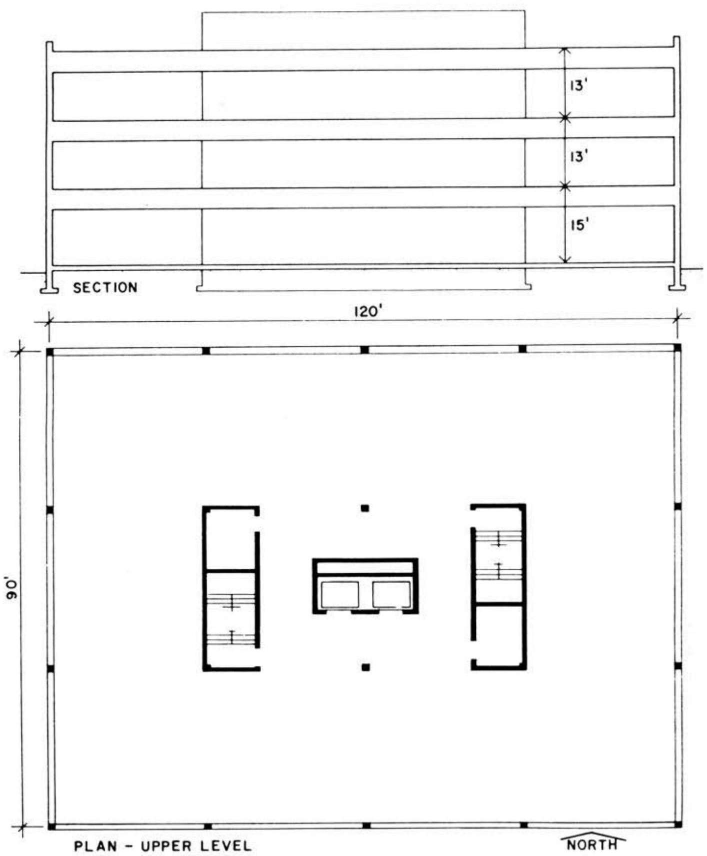


FIGURE 17.37 Building Five: General form.

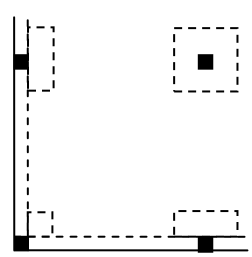
Materials

Use f'_c =3000 psi (20.7 MPa) and grade 60 reinforcement (f_y = 60 ksi or 414 MPa).

Structural Elements/Plan

Case 1 is shown in Figure 17.44 and consists of a flat plate supported on interior beams, which in turn, are supported on girders supported by columns. We will examine the slab, and a four-span interior beam.

Case 2 will consider the bays with flat slabs, no interior beams with drop panels at the columns and an exterior rigid frame with spandrel (edge) beams. An example of an edge bay is shown to the right. We will examine the slab and the drop panels.



For both cases, we will examine the exterior frames in the 3-bay direction.

Case 1:

Slab:

	Minimum thickness, ***h***			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections.			
Solid one-way slabs	**ℓ/20**	**ℓ/24**	**ℓ/28**	**ℓ/10**
Beams or ribbed one-way slabs	**ℓ/16**	**ℓ/18.5**	**ℓ/21**	**ℓ/8**

The slabs are effectively 10 ft x 30 ft, with an aspect ratio of 3, making them one-way slabs. Minimum depths (by ACI) are a function of the span. Using Table 3-1 for one way slabs the minimum is $\frac{l_n}{24}$ with 5 inches minimum for fire rating. We'll presume the interior beams are 12" wide, so

l_n = 10 ft -1 ft = 9 ft

minimum t (or h) $= \dfrac{9^{ft} \cdot 12^{in/ft}}{24} = 4.5in$

Use 5 in.

dead load from slab = $\dfrac{150^{lb/ft^3} \cdot 5^{in}}{12^{in/ft}}$ = 62.5 lb/ft^2

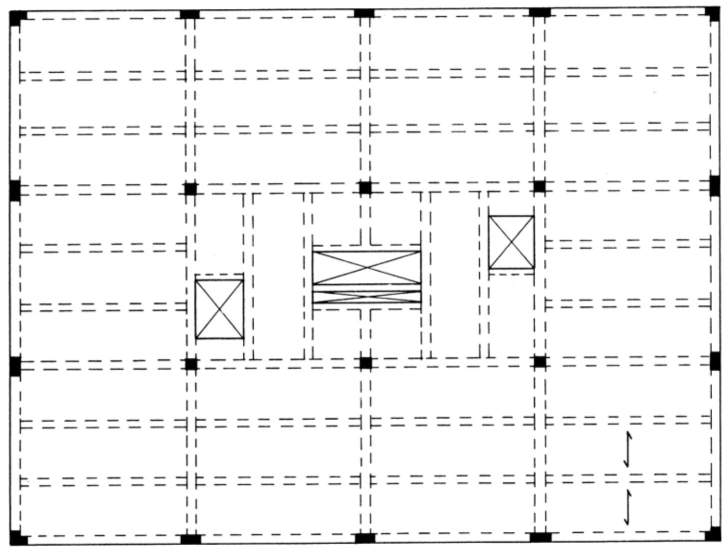


FIGURE 17.44 Building Five: Framing plan for the concrete structure for the upper floor.

total dead load = (5 + 15 + 62.5) lb/ft^2 + 2" of lightweight concrete topping with weight of 18 lb/ft^2 (0.68 KPa) (presuming interior wall weight is over beams & girders)

dead load = 100.5 lb/ft^2

live load (worst case in corridor) = 100 lb/ft^2

total *factored* distributed load (ASCE-7) of 1.2D+1.6L:

w_u'= 1.2(100.5 lb/ft^2) + 1.6(100 lb/ft^2) = 280.6 lb/ft^2

Maximum Positive Moments from Figure 2-3, end span (integral with support) for a **1 ft wide strip:**

$$M_u \text{ (positive)} = \frac{w_u \ell^2{}_n}{14} = \frac{w'_u \cdot 1ft \cdot \ell^2{}_n}{14} = \frac{(280.6^{lb/ft^2})(1ft)(9^{ft})^2}{14} \cdot \frac{1k}{1000lb} = 1.62 \text{ k-ft}$$

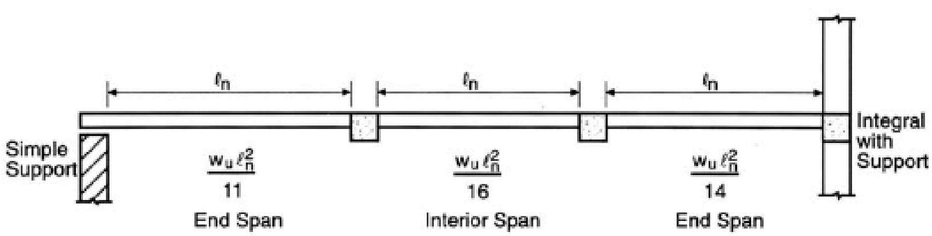


Figure 2-3 Positive Moments—All Cases

Maximum Negative Moments from Figure 2-5, end span (integral with support) for a **1 ft wide strip:**

$$M_{u\text{-}}\text{(negative)} = \frac{w_u \ell^2{}_n}{12} = \frac{w_u \cdot 1ft \cdot \ell^2{}_n}{12} = \frac{(280.6^{lb/ft^2})(1ft)(9^{ft})^2}{12} \cdot \frac{1k}{1000lb} = 1.89 \text{ k-ft}$$

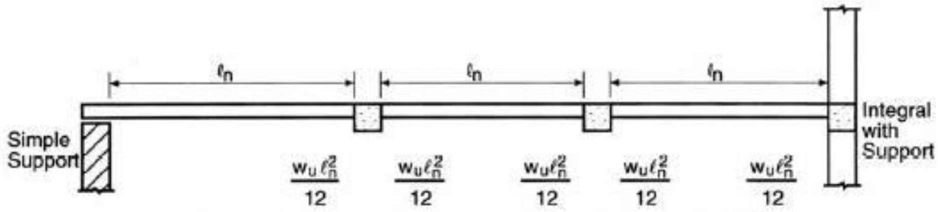


Figure 2-5 Negative Moments—Slabs with spans ≤ 10 ft

The design aid (Figure 3.8.1) can be used to find the reinforcement ratio, ρ, knowing $R_n = M_n/bd^2$ with $M_n = M_u/\phi_f$, where $\phi_f = 0.9$. We can presume the effective depth to the centroid of the reinforcement, d, is 1.25" less than the slab thickness (with ¾" cover and ½ of a bar diameter for a #8 (1") bar) = 3.75".

$$R_n = \frac{1.89^{k-ft}}{(0.9)(12^{in})(3.75^{in})^2} \cdot 12^{in/ft} \cdot 1000^{lb/k} = 149.3 \text{ psi}$$

so ρ for f'_c = 3000 psi and f_y = 60,000 psi is the minimum. For slabs, A_s minimum is 0.0018bt for grade 60 steel.

$$A_s = 0.0018(12in)(5in) = 0.108 \text{ in}^2/\text{ft}$$

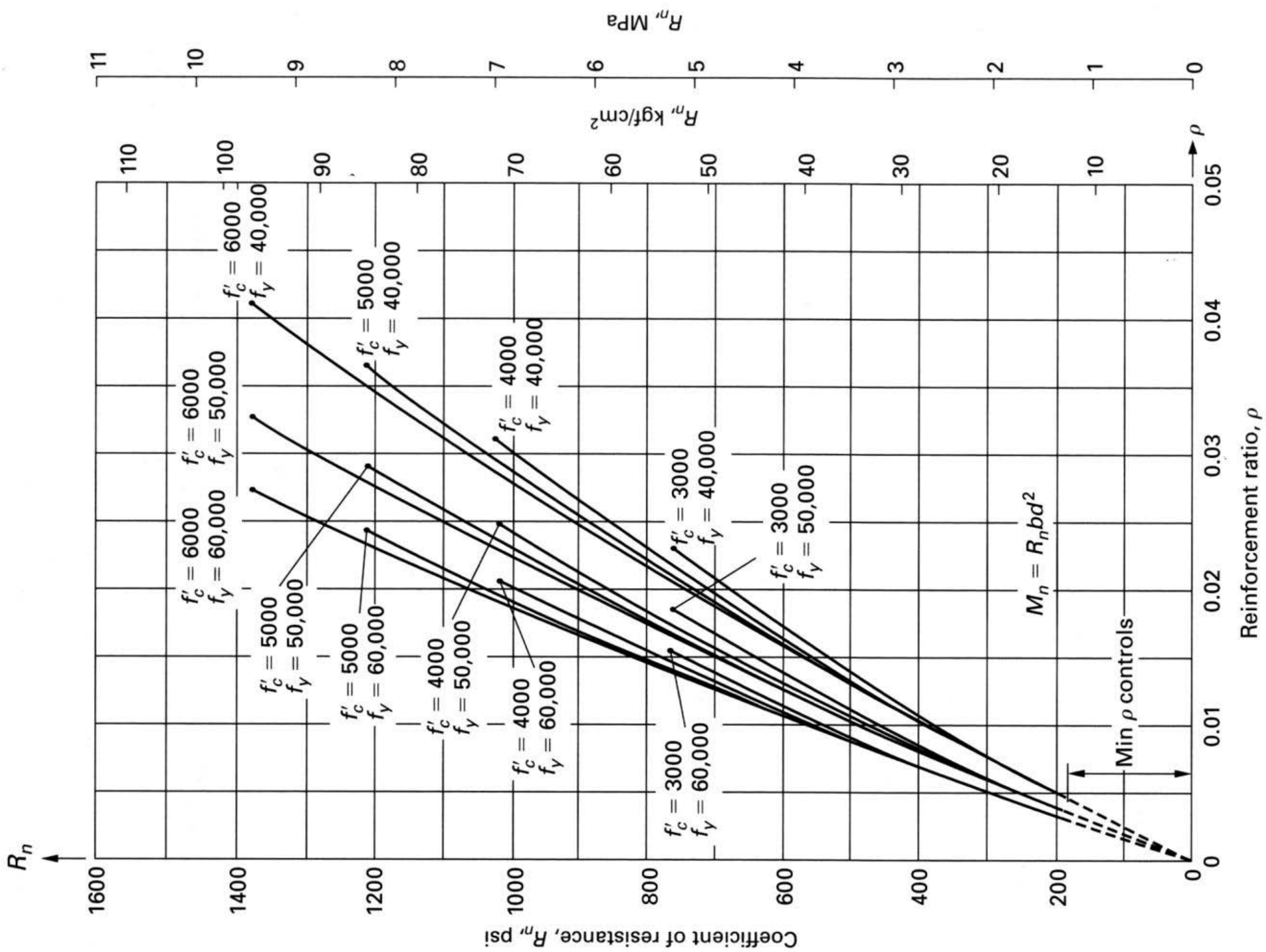


Figure 3.8.1 Strength curves (R_n vs ρ) for singly reinforced rectangular sections. Upper limit of curves is at ρ_{max}.

Pick bars and spacing off Table 3-7. Use #3 bars @ 12 in (A_s = 0.11 in^2).

Table 3-7 Areas of Bars per Foot Width of Slab—A_s (in.^{2}/ft)

Bar size	Bar spacing (in.) 6	7	8	9	10	11	12	13	14	15	16	17	18
#3	0.22	0.19	0.17	0.15	0.13	0.12	0.11	0.10	0.09	0.09	0.08	0.08	0.07
#4	0.40	0.34	0.30	0.27	0.24	0.22	0.20	0.18	0.17	0.16	0.15	0.14	0.13
#5	0.62	0.53	0.46	0.41	0.37	0.34	0.31	0.29	0.27	0.25	0.23	0.22	0.21
#6	0.88	0.75	0.66	0.59	0.53	0.48	0.44	0.41	0.38	0.35	0.33	0.31	0.29
#7	1.20	1.03	0.90	0.80	0.72	0.65	0.60	0.55	0.51	0.48	0.45	0.42	0.40
#8	1.58	1.35	1.18	1.05	0.95	0.86	0.79	0.73	0.68	0.63	0.59	0.56	0.53
#9	2.00	1.71	1.50	1.33	1.20	1.09	1.00	0.92	0.86	0.80	0.75	0.71	0.67
#10	2.54	2.18	1.91	1.69	1.52	1.39	1.27	1.17	1.09	1.02	0.95	0.90	0.85
#11	3.12	2.67	2.34	2.08	1.87	1.70	1.56	1.44	1.34	1.25	1.17	1.10	1.04

Check the moment capacity. d is actually 5 in – 0.75in (cover) – ½ (3/8in bar diameter) = 4.06 in

$a = A_s f_y / 0.85 f'_c b$ = 0.11 in^2 (60 ksi)/[0.85(3 ksi)12 in] = 0.22 in

$\phi M_n = \phi A_s f_y (d - a/2) = 0.9(0.11in^2)(60ksi)(4.06in - \frac{0.22in}{2}) \cdot (\frac{1}{12^{in}/_{ft}}) =$ 1.96 k-ft > 1.89 k-ft needed

(OK)

Maximum Shear: Figure 2-7 shows end shear that is $w_u l_n/2$ except at the end span on the interior column which sees a little more and you must design for 15% increase:

$V_{u\text{-}max} = 1.15 w_u l_n/2 = \frac{1.15(280.6^{lb/ft^2})(\cdot 1^{ft})(9^{ft})}{2}$ = 1452 lb (for a 1 ft strip)

V_u at d away from the support = $V_{u\text{-}max} - w(d)$ = 1452 lb – $\frac{(280.6^{lb/ft^2})(\cdot 1^{ft})(4.06in)}{12^{in/ft}}$ = 1357 lb

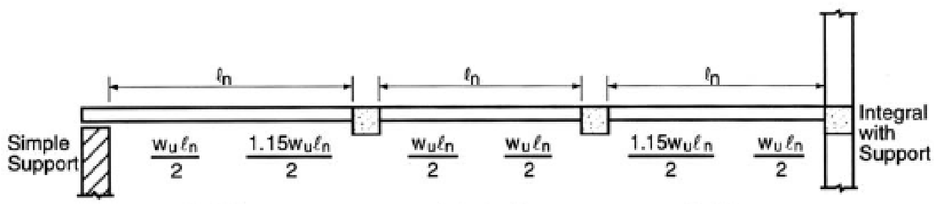


Figure 2-7 End Shears—All Cases

Check the one way shear capacity: $\phi_v V_c = \phi_v 2\sqrt{f'_c}\,bd$ (ϕ_v = 0.75):

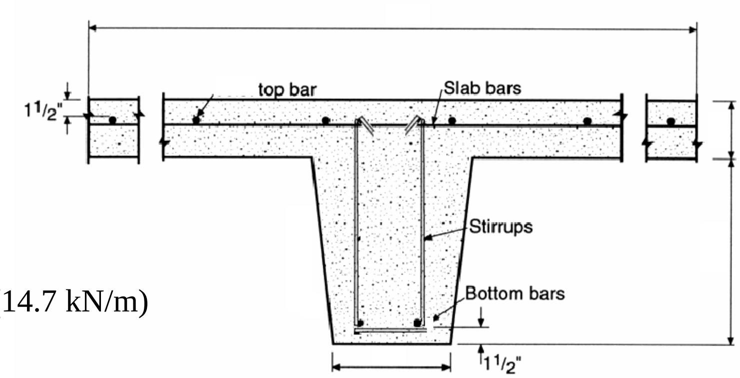
$\phi_v V_c = 0.75(2)\sqrt{3000}psi(12^{in})(4.06^{in})$ = 4003 lb

Is V_u (needed) < $\phi_v V_c$ (capacity)? YES: 1357 lb ≤ 4003 lb, so we don't need to make the slab thicker.

Interior Beam (effectively a T-beam):

Tributary width = 10 ft for an interior beam.

dead load = (100.5 lb/ft^2)(10ft) = 1005 lb/ft (14.7 kN/m)



Reduction of live load is allowed, with an influence area, A_I, of 2 panels beside an interior beam, assuming the girder is 12" wide. The live load is 100 lb/ft^2:

$$L = L_o(0.25+\frac{15}{\sqrt{A_I}}) = 100\,{}^{lb}\!/\!_{ft^2}(0.25+\frac{15}{\sqrt{(30\,ft-1\,ft)(2\times 10\,ft)}}) = 87.3 \text{ lb/ft}^2$$

(Reduction Multiplier = 0.873)

live load = 87.3 lb/ft^2(10ft) = 873 lb/ft (12.7 kN/m)

Estimating a 12" wide x 24" deep beam means the additional dead load from self weight ($w = \gamma \cdot A$ in units of load/length) can be included. The top 5 inches of slab has already been included in the dead load:

$$\text{dead load from self weight} = 150\,{}^{lb}\!/\!_{ft^3}(12in\ wide)(24-5in\ deep)\cdot\left(\frac{1ft}{12in}\right)^2 = 237.5 \text{ lb/ft (3.46 kN/m)}$$

w_u = 1.2(1005 lb/ft+237.5 lb/ft) + 1.6(873 lb/ft) = 2888 lb/ft (4.30 kN/m)

The effective width, b_E, of the T part is the smaller of $\frac{\ell_n}{4}$, $b_w + 16t$, or center-center spacing

b_E = minimum{29 ft/4 = 7.25 ft = 87 in, 12 in+16x5 in = 92 in, 10 ft = 120 in} = 87 in

The clear span for the beam is

l_n = 30 ft – 1 ft = 29 ft

Maximum Positive Moments from Figure 2-3, end span (integral with support)**:**

$$M_u \text{ (positive)} = \frac{w_u \ell^2{}_n}{14} = \frac{2888^{lb/ft}(29^{ft})^2}{14}\cdot\frac{1k}{1000lb} = 173.5 \text{ k-ft}$$

Maximum Negative Moments from Figure 2-4, end span (integral with support)**:**

$$M_{u-}\text{(negative)} = \frac{w_u \ell^2{}_n}{10} = \frac{2888^{lb/ft}(29^{ft})^2}{10}\cdot\frac{1k}{1000lb} = 242.9 \text{ k-ft}$$

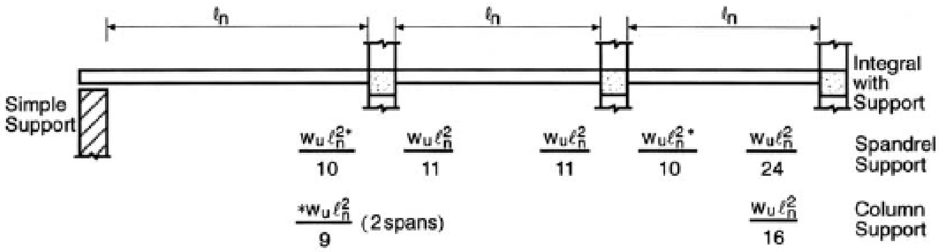


Figure 2-4 Negative Moments—Beams and Slabs

Figure 3.8.1 can be used to find an approximate ρ for top reinforcement if $R_n = M_n/bd^2$ and we set $M_n = M_u/\phi_f$. We can presume the effective depth is 2.5" less than the 24" depth (for 1.5" cover and ½ bar diameter for a #10 (10/8)" bar + #3 stirrups (3/8" more)), so d = 21.5".

$$R_n = \frac{1000^{lb/k} \cdot 242.9^{k-ft}}{0.9 \cdot (12^{in})(21.5^{in})^2} \cdot 12^{in/ft} = 584 \text{ psi}$$

so ρ for f'_c = 3000 psi and f_y = 60,000 psi is about 0.011

Then we pick bars and spacing off Table 3-7 to fit in the effective flange width in the slab.

For bottom reinforcement (positive moment) the effective flange is so wide at 87 in, that it resists a lot of compression, and needs very little steel to stay under-reinforced (*a* is between 0.6" and 0.5"). We'd put in bottom bars at the minimum reinforcement allowed and for tying the stirrups to.

Maximum Shear: $V_{max} = w_u l/2$ normally, but the end span sees a little more and you must design for 15% increase. But for beams, we can use the lower value of V that is a distance of d from the face of the support

$$V_{u\text{-design}} = 1.15 w_u l_n/2 - w_u d = \frac{1.15(2888^{lb/ft})(29^{ft})}{2} - \frac{2888^{lb/ft}(21.5^{in})}{12^{in/ft}} = 42{,}983 \text{ lb} = 43.0 \text{ k}$$

Check the one way shear capacity = $\phi_v V_c = \phi_v 2\sqrt{f'_c}\,bd$, where $\phi_v = 0.75$

$$\phi_v V_c = 0.75(2)\sqrt{3000}\,psi(12^{in})(21.5^{in}) = 21{,}197 \text{ lb} = 21.2 \text{ k}$$

Is V_u (needed) < $\phi_v V_c$ (capacity)?

NO: 43.0 k is greater than 21.2 k, so stirrups are needed

$\phi_v V_s = V_u - \phi_v V_c$ = 43.0 k – 21.2 k = 21.8 k (max needed)

Using #3 bars (typical) with two legs means $A_v = 2(0.11 in^2) = 0.22 in^2$.

To determine required spacing, use Table 3-8. For d = 21.5" and $\phi_v V_s \le \phi_v 4\sqrt{f'_c}\,bd$ (where $\phi_v 4\sqrt{f'_c}\,bd = 2\phi V_c$ = 2(21.2 k) = 42.4 k), the maximum spacing is d/2 = 10.75 in. or 24".

$$s_{required} = \frac{\phi A_v f_y d}{V_u - \phi V_c} = \frac{\phi A_v f_y d}{\phi V_s} = \frac{0.75 \cdot 0.22^{in} \cdot 60^{ksi} \cdot 21.5^{in}}{21.8^k} = 9.75 \text{ in, so use 9 in.}$$

We would try to increase the spacing as the shear decreases, but it is a tedious job. We need stirrups anywhere that $V_u > \phi_v V_c/2$. One recommended intermediate spacing is d/3.

Table 3-8 ACI Provisions for Shear Design*

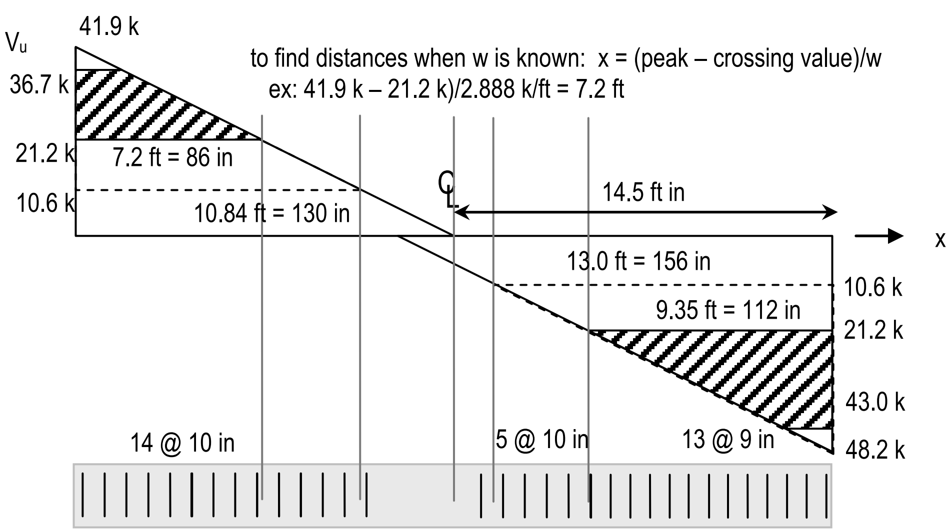
		$V_u \le \frac{\phi V_c}{2}$	$\phi V_c \ge V_u > \frac{\phi V_c}{2}$	$V_u > \phi V_c$
Required area of stirrups, A_v**		none	$\frac{50 b_w s}{f_y}$	$\frac{(V_u - \phi V_c)s}{\phi f_y d}$
Stirrup spacing, s	Required	—	$\frac{A_v f_y}{50 b_w}$	$\frac{\phi A_v f_y d}{V_u - \phi V_c}$
	Recommended Minimum†	—	—	4 in.
	Maximum†† (ACI 11.5.4)	—	$\frac{d}{2}$ or 24 in.	$\frac{d}{2}$ or 24 in. for $(V_u - \phi V_c) \le \phi 4\sqrt{f'_c}\, b_w d$
				$\frac{d}{4}$ or 12 in. for $(V_u - \phi V_c) > \phi 4\sqrt{f'_c}\, b_w d$

*Members subjected to shear and flexure only; $\phi V_c = \phi 2\sqrt{f'_c}\, b_w d$, $\phi = 0.75$ (ACI 11.3.1.1)
**$A_v = 2 \times A_b$ for U stirrups; $f_y \le 60$ ksi (ACI 11.5.2)
†A practical limit for minimum spacing is d/4
††Maximum spacing based on minimum shear reinforcement ($= A_v f_y / 50 b_w$) must also be considered (ACI 11.5.5.3).

The required spacing where stirrups are needed for crack control ($\phi_v V_c \ge V_u > \frac{1}{2}\phi_v V_c$) is

$s_{required} = \frac{A_v f_y}{50 b_w} = \frac{0.22\text{in}^2(60{,}000\text{psi})}{50(12\text{ in})}$ =22 in and the maximum spacing is d/2 = 10.75 in. or 24". Use 10 in.

A recommended minimum spacing for the first stirrup is 2 in. from the face of the support. A distance of one half the spacing near the support is often used.



END SPAN STIRRUP LAYOUT

Spandrel Girders:

Because there is a concentrated load on the girder, the approximate analysis can't technically be used. If we converted the maximum moment (at midspan) to an equivalent distributed load by setting it equal to $w_u l^2/8$ we would then use:

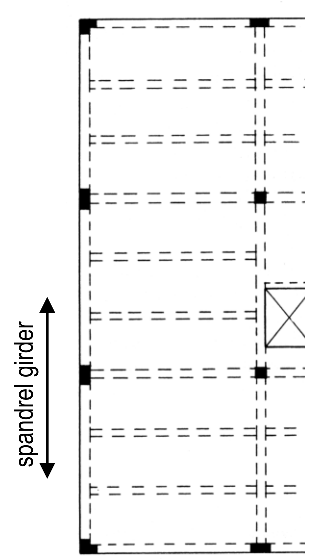


FIGURE 17.44 Building F floor

Maximum Positive Moments from Figure 2-3, end span (integral with support):

$$M_{u+} = \frac{w_u \ell^2{}_n}{14}$$

Maximum Negative Moments from Figure 2-4, end span (column support):

$$M_{u-} = \frac{w_u \ell^2{}_n}{10} \text{ (with } \frac{w_u \ell^2{}_n}{16} \text{ at end)}$$

<u>Column:</u>

An exterior or corner column will see axial load and bending moment. We'd use interaction charts for P_u and M_u for standard sizes to determine the required area of steel. An interior column sees very little bending. The axial loads come from gravity. The factored load combination is 1.2D+1.6L + 0.5L_r.

The girder weight, presuming 1' x 4' girder at 150 lb/ft^3 = 600 lb/ft

Top story: presuming 20 lb/ft^2 roof live load, the total load for an interior column (tributary area of 30'x30') is:

DL_{roof^*}:	1.2 x 100.5 lb/ft^2 x 30 ft x 30 ft	= 108.5 k
* assuming the same live load and materials as the floors		
DL_{beam}	1.2 x 237.5 lb/ft x 30 ft x 3 beams	= 25.6 k
DL_{girder}	1.2 x 600 lb/ft x 30 ft	= 21.6 k
LL_r:	**0.5** x 20 lb/ft^2 x 30 ft x 30 ft	= 9.0 k
Total		= 164.7 k

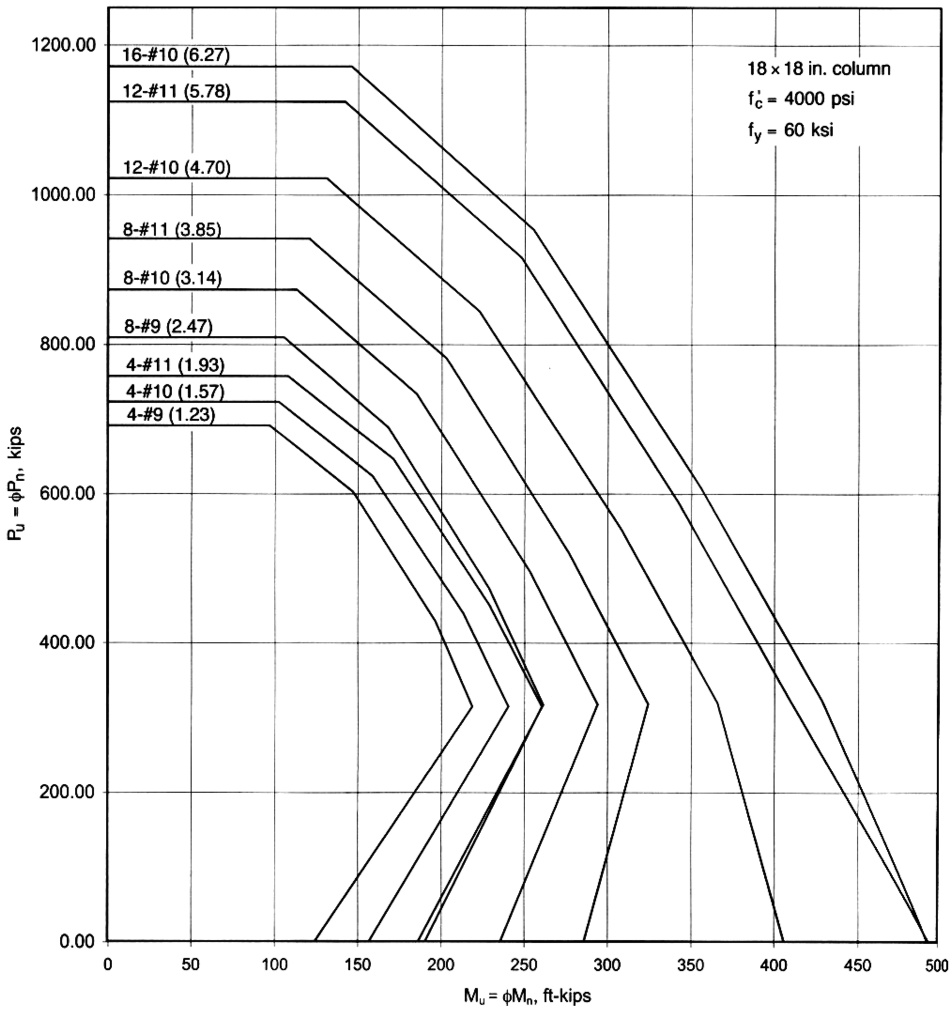
Lower stories:

DL_{floor}:	1.2 x 100.5 lb/ft^2 x 30 ft x 30 ft	= 108.5 k	
DL_{beam}	1.2 x 237.5 lb/ft x 30 ft x 3 beams	= 25.6 k	
DL_{girder}	1.2 x 600 lb/ft x 30 ft	= 21.6 k	
LL_{floor}:	1.6 x (0.873)x100 lb/ft^2 x 30 ft x 30 ft	= 125.7 k	(includes reduction)
Total		= 281.4 k	

2nd floor column sees P_u = 164.7+281.4 = 446.1 k
1st floor column sees P_u = 446.1+281.4 = 727.5 k

Look at the example interaction diagram for an 18" x 18" column (Figure 5-20 – ACI 318-02) using <u>f'_c = 4000 psi</u> and f_y = 60,000 psi for the first floor having P_u = 727.5 k, and M_u to the column being approximately 10% of the beam negative moment = 0.1*242.9 k-ft = 24.3 k-ft: (See maximum negative moment calculation for an interior beam.) The chart indicates the capacity for the reinforcement amounts shown by the solid lines.

For P_u = 727.5 k and Mu = 24.3 k-ft, the point plots below the line marked 4-#10 (1.57% area of steel to an 18 in x 18 in area).



Lateral Force Design:

The wind loads from the wind speed, elevation, and exposure we'll accept as shown in Figure 17.42 given on the left in psf. The wind is acting on the long side of the building. The perimeter frame resists the lateral loads, so there are two with a tributary width of ½ [(30ft)x(4 bays) + 2ft for beam widths and cladding] = 122ft/2 = 61ft

The factored combinations with dead and wind load are:

$1.2D + 1.6L_r + 0.5W$

$1.2D + 1.0W + L + 0.5L_r$

The tributary height for each floor is half the distance to the next floor (top and bottom):

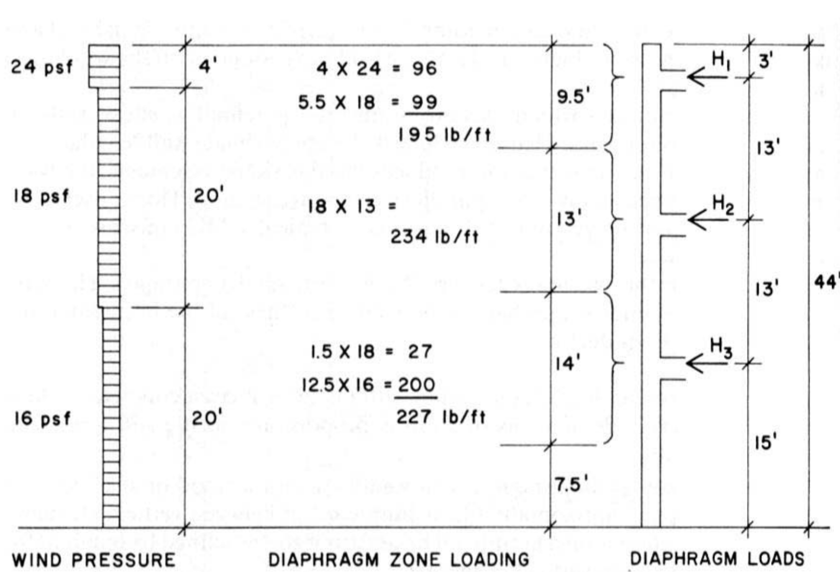


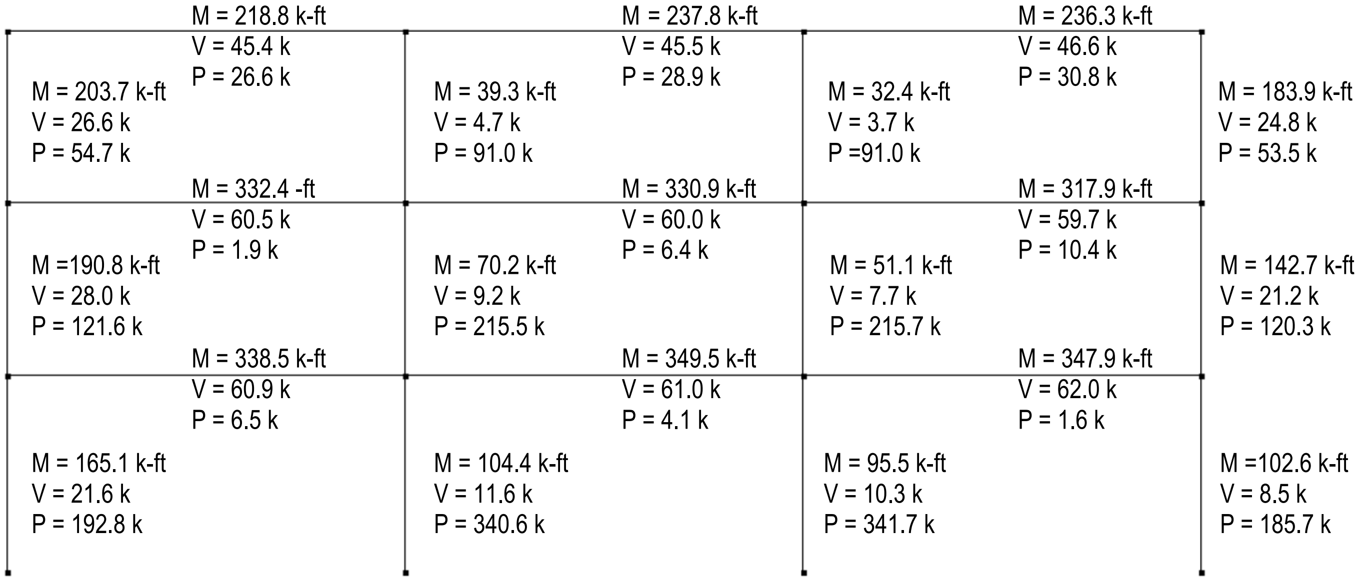
FIGURE 17.42 Building Five: How wind loads affect the lateral bracing system.

Exterior frame (bent) loads:

$H_1 = 195^{lb/ft}(61^{ft}) = 11{,}895 \text{ lb} = 11.9 \text{ k/bent}$

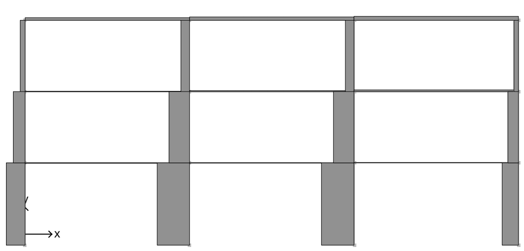
$H_2 = \dfrac{234^{lb/ft}(61^{ft})}{1000^{lb/k}} = 14.3 \text{ k/bent}$ $\quad$ $H_3 = \dfrac{227^{lb/ft}(61^{ft})}{1000^{lb/k}} = 13.8 \text{ k/bent}$

Using Multiframe4D, the axial force, shear and bending moment diagrams can be determined using the load combinations, and the largest moments, shear and axial forces for each member determined.

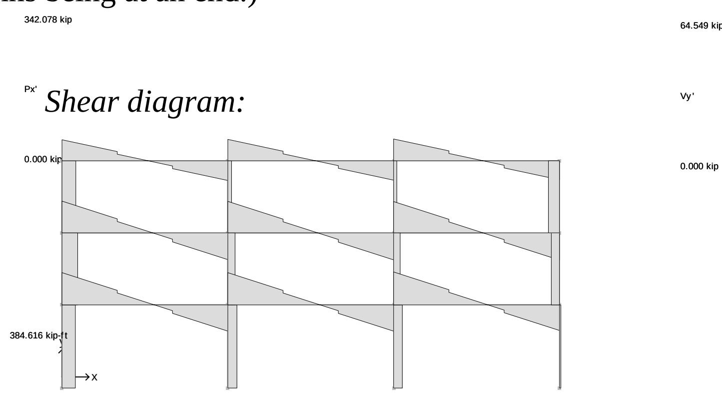


(This is the summary diagram of force, shear and moment magnitudes refer to the maximum values in the column or beams, with the maximum moment in the beams being negative over the supports, and the maximum moment in the columns being at an end.)

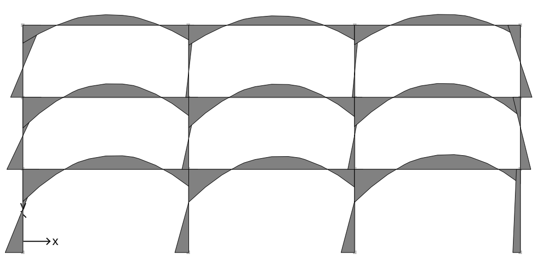
Axial force diagram:



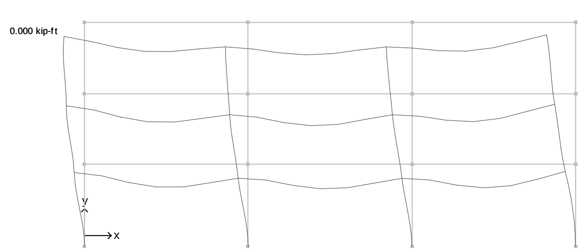
Shear diagram:



Bending moment diagram:



Displacement:



Beam-Column loads for design:

The bottom exterior columns see the largest bending moment on the lee-ward side (left):

P_u = 192.8 k and M_u = 165.1 k-ft *(with large axial load)*

The interior columns see the largest axial forces:

P_u = 341.7 k and M_u = 95.5 k-ft and P_u = 340.6 k and M_u = 104.4 k-ft

Refer to an interaction diagram for column reinforcement and sizing.

Case 2

Slab:

The slabs are effectively 30 ft x 30 ft, making them two-way slabs. Minimum thicknesses (by ACI) are a function of the span. Using Table 4-1 for two way slabs, the minimum is the larger of l_n/36 or 4 inches. Presuming the columns are 18" wide, l_n = 30 ft –(18 in)/(12 ft/in) = 28.5 ft,

$h = l_n/36 = (28.5\text{x}12)/36 = 9.5$ in

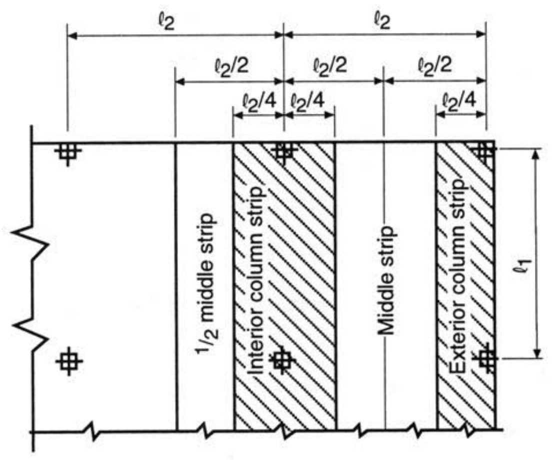
Table 4-1 Minimum Thickness for Two-Way Slab Systems

Two-Way Slab System		α_m	β	Minimum h
Flat Plate		—	≤ 2	$\ell_n/30$
Flat Plate with Spandrel Beams [1]	[Min. h = 5 in.]	—	≤ 2	$\ell_n/33$
Flat Slab [2]		—	≤ 2	$\ell_n/33$
Flat Slab [2] with Spandrel Beams[1]	[Min. h = 4 in.]	—	≤ 2	$\ell_n/36$
Two-Way Beam-Supported Slab[3]		≤ 0.2	≤ 2	$\ell_n/30$
		1.0	1	$\ell_n/33$
			2	$\ell_n/36$
		≥ 2.0	1	$\ell_n/37$
			2	$\ell_n/44$
Two-Way Beam-Supported Slab[1,3]		≤ 0.2	≤ 2	$\ell_n/33$
		1.0	1	$\ell_n/36$
			2	$\ell_n/40$
		≥ 2.0	1	$\ell_n/41$
			2	$\ell_n/49$

[1]Spandrel beam-to-slab stiffness ratio $\alpha \geq 0.8$ (ACI 9.5.3.3)

[2]Drop panel length ≥ $\ell/3$, depth ≥ 1.25h (ACI 13.4.7)

[3]Min. h = 5 in. for $\alpha_m \leq 2.0$; min. h = 3.5 in. for $\alpha_m > 2.0$ (ACI 9.5.3.3)



(a) Column strip for $\ell_2 \leq \ell_1$

The table also says the drop panel needs to be l/3 long = 28.5 ft/3 = 9.5 ft, and that the minimum depth must be 1.25h = 1.25(9.5 in) = 12 in.

For the strips, l_2 = 30 ft, so the interior column strip will be 30 ft/4+30ft/4 = 15 ft, and the middle strip will be the remaining 15 ft.

dead load from slab = $\dfrac{150^{lb/ft^3} \cdot 9.5^{in}}{12^{in/ft}}$ = 118.75 lb/ft^2

total dead load = 5 + 15 + 118.75 lb/ft^2 + 2" of lightweight concrete topping @ 18 lb/ft^2 (0.68 KPa) (presuming interior wall weight is over beams & girders)

total dead load = 156.75 lb/ft^2

live load with reduction, where the influence area, A_I, for two way slabs is one panel:

$$L = L_o(0.25+\frac{15}{\sqrt{A_I}}) = 100\,{}^{lb}\!/\!_{ft^2}(0.25+\frac{15}{\sqrt{(30\,ft)(30\,ft)}}) = 75 \text{ lb/ft}^2$$

total *factored* distributed load:

$$w_u = 1.2(156.75 \text{ lb/ft}^2) + 1.6(75 \text{ lb/ft}^2) = 308.1 \text{ lb/ft}^2$$

total panel moment to distribute:

$$M_o = \frac{w_u l_2 l_n^2}{8} = \frac{308.1^{lb/ft^2}(30^{ft})(28.5^{ft})^2}{8}\cdot\frac{1k}{1000lb} = 938.4 \text{ k-ft}$$

Column strip, end span:

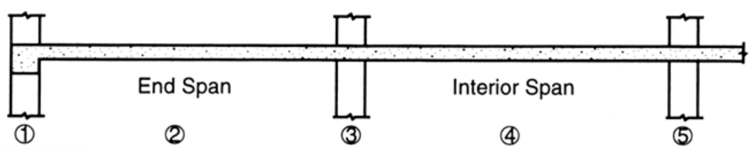
Maximum Positive Moments from Table 4-3, (flat slab with spandrel beams)**:**

$M_{u+} = 0.30M_o = 0.30\cdot(938.4 \text{ k-ft}) = 281.5$ k-ft

Maximum Negative Moments from Table 4-3, (flat slab with spandrel beams)**:**

$M_{u-} = 0.53M_o = 0.53\cdot(938.4 \text{ k-ft}) = 497.4$ k-ft

Table 4-3 Flat Plate or Flat Slab with Spandrel Beams



Slab Moments	End Span: 1 Exterior Negative	2 Positive	3 First Interior Negative	Interior Span: 4 Positive	5 Interior Negative
Total Moment	0.30 M_o	0.50 M_o	0.70 M_o	0.35 M_o	0.65 M_o
Column Strip	0.23 M_o	0.30 M_o	0.53 M_o	0.21 M_o	0.49 M_o
Middle Strip	0.07 M_o	0.20 M_o	0.17 M_o	0.14 M_o	0.16 M_o

Notes: (1) All negative moments are at face of support.

(2) Torsional stiffness of spandrel beams $\beta_t \geq 2.5$. For values of β_t less than 2.5, exterior negative column strip moment increases to $(0.30 - 0.03\beta_t)$ M_o.

Middle strip, end span:

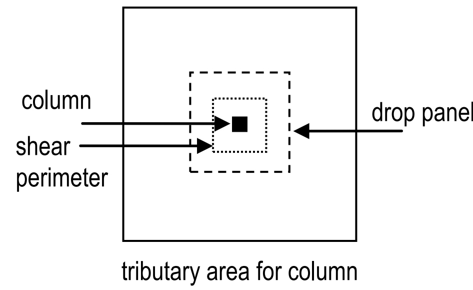
Maximum Positive Moments from Table 4-3, (flat slab with spandrel beams)**:**

$M_{u+} = 0.20M_o = 0.20\cdot(938.4 \text{ k-ft}) = 187.9$ k-ft

Maximum Negative Moments from Table 4-3, (flat slab with spandrel beams)**:**

$M_{u-} = 0.17M_o = 0.17\cdot(938.4 \text{ k-ft}) = 159.5$ k-ft

Design as for the slab in Case 1, but provide steel *in both directions* distributing the reinforcing needed by strips.



Shear around columns: The shear is critical at a distance d/2 away from the column face. If the drop panel depth is 12 inches, the minimum d with two layers of 1" diameter bars would be 12" – ¾" (cover) – (1") – ½(1") =about 9.75 in (to the top steel).

The shear resistance is $\phi_v V_c = \phi_v 4\sqrt{f'_c}\, b_o d$, $\phi_v = 0.75$ where b_o, is the perimeter *length*.

The design shear value is the distributed load over the tributary area *outside* the shear perimeter, $V_u = w_u$ (tributary area - b_1 x b_2) where b's are the column width plus d/2 each side.

$b_1 = b_2 = 18" + 9.75"/2 + 9.75"/2 = 27.75$ in

$b_1 \text{ x } b_2 = (27.75in)^2 \cdot (\frac{1ft}{12in})^2 = 5.35 \text{ ft}^2$

$V_u = (284.7^{lb/ft^2})(30^{ft} \cdot 30^{ft} - 4.97^{ft^2}) \cdot \frac{1k}{1000lb} = 254.7 \text{ k}$

Shear capacity:

$b_o = 2(b_1) + 2(b_2) = 4(27.75 \text{ in}) = 111 \text{ in}$

$\phi_v V_c = 0.75 \cdot 4 \cdot \sqrt{3000} psi \cdot 111^{in} \cdot 9.75^{in} = 177{,}832 \text{ lb} = 177.8 \text{ ksi} < V_u!$

The shear capacity is not large enough. The options are to provide shear heads or a deeper drop panel, or change concrete strength, or even a different system selection...

There also is some transfer by the moment across the column into shear.

Deflections:

Elastic calculations for deflections require that the steel be turned into an equivalent concrete material using $n = \frac{E_s}{E_c}$. E_c can be measured or calculated with respect to concrete strength.

For normal weight concrete (150 lb/ft^3): $E_c = 57{,}000\sqrt{f'_c}$

$E_c = 57{,}000\sqrt{3000} psi = 3{,}122{,}019 \text{ psi} = 3122 \text{ ksi}$

$n = 29{,}000 \text{ psi}/3122 \text{ ksi} = 9.3$

Deflection limits are given in Table 9.5(b)

TABLE 9.5(b) — MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to live load ***L***	$\ell/180$*
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load ***L***	$\ell/360$
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)†	$\ell/480$‡
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$\ell/240$§

* Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

† Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.2, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

‡ Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.

§ Limit shall not be greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

Materials for Membrane Structures

R. Houtman, M. Orpana

The most prominent material for stressed membrane structures is obviously the fabric. It is prominently present, attracts much attention and looks very simple. To obtain this pleasant charisma there has been done a lot of research. The material is analysed and specific properties are defined and adapted. Properties like transparency, durability, fire retardance but also elasticity, strength.

In this paper the fabric is discussed to get a better understanding of those properties. The composition of the fabric is explained, followed by a discussion of the most common fabrics available today. Finally the structural behaviour of the fabric is discussed.

Threads

A thread is built up out of fibres. There are natural fibres and chemical fibres. Natural fibres have a restricted length and are bound up in strands. These are the so-called spin fibres. Chemical fibres theoretically have an endless length and are called filaments. The cross-section of natural fibres is smaller than 0.1mm, where chemical fibres can have larger cross-sections. The shape of the cross-section is round for natural fibres but can have any shape in chemical fibres. For membrane structures it is best to have a yarn with a circular cross-section.

The mechanical properties of materials in the building industry are normally specified in N/mm^2. In technical textiles this is not common because it is not easy to determine the cross-section of a very small fibre. Therefore it is usual to determine the weight of a fibre with a certain length. When the specific mass is known from the fibre, it is possible to determine an average cross-section of the material. This mass-per-length unit is indicated with Titer with the symbol Tex: 1 Tex weight in grams per 1 000m length. In synthetic fibres it is common to use decitex: 1 dtex = weight in grams per 10 000m length [7].

A Polyester fibre for example with a Titer of 8.35 dtex has a weight of 8.35 grams at a length of 10000 m. When the product is that small, it is very difficult to use it in industrial processes. Therefore it is spun into threads. One thread possibly consists out of hundreds of fibres. When a thread only has one fibre, it is called monofil. Spin fibres need to be stabilised by twisting around the centre of the thread. Filaments do not need it, but it facilitates the handling. The twisting influences the stress-strain behaviour of the threads. The more the thread is twisted the more the elasticity decreases compared to the elasticity of the fibre. With the adjustment of the twisting the mechanical properties of the thread can be determined precisely.

The characterisation of a filament thread is according to the System Tex, where the number of fibres and twists are added. A thread for example which is called 2200 dtex f 200 z 60 has a total Titer of 2200 dtex, made out of 200 fibres, the thread is twisted 60 times per meter in z direction [7].

There are several fibres that can be applied in membrane structures. For each project it is necessary to consider which type of fabric can be used. Several fibres do have the potential to be applied, however the high costs of it prevent a wide utilisation.

Cotton fibre

This type of fibre is the only organic fibre, which is being used in membrane structures. Frei Otto used it for his early garden show structures and nowadays it still is applied in some rental tents. As of its organic properties the material is subject to fungi and moisture. When used permanently it has an expected lifetime of about 4 years.

Polyamide 6.6 (Nylon)

The nylon fibre has a bad resistance against UV light, swells in length direction when it gets wet and is herewith of little importance for textile architecture. It is frequently applied in the sailing industry because of the little weight and high strength.

Polyester

Polyester fibre together with fibreglass is the most common fibre in textile architecture and regarded as a standard product. The fibre has a good tensile strength and elasticity. Because of its considerable elongation before yield, the material is "forgiving". It enables to make small corrections during installation. The mechanical properties of the material decrease by sunlight and there is ageing.

Material	Density (g/cm^3)	Tensile strength (N/mm^2)	Tensile strain (%)	Elasticity (N/mM2)	Remarks
Cotton	1.5-1.54	350-700	6-15	4500 - 9000	Only for temporary use of interest
Polyamid 6.6 (Nylon)	1.14	Until 1 000	15-20	5000 - 6000	- When exposed to light only average resistance to ageing - Swelling when exposed to moisture - Only of little importance in textile architecture
Polyester fibre (Trevira, Teryiene, Dacron, Diolen)	1.38-1.41	1 000-1 300	10-18	10000 – 15000	- Widely spread, together with fibreglass a standard product in textile architecture
Fibreglass	2.55	Until 3500	2.0-3.5	70000 - 90000	- When exposed to moisture, reduction of breaking strength - Brittle fibres, therefore is spun into filaments of 3 µm diameter - Together with Polyester a standard product in textile architecture
Aramid fibre (Kevlar, Arenka Twaron)	1.45	Until 2700	2-4	130000 - 150000	- Special fibre for high-tech products
Polytetrafluor-ethylen (Teflon, Hostaflon Polyflon, Toyoflon etc.)	2.1-2.3	160-380	13-32	700 - 4000	- High moisture resistance - Remarkable anti adhesive - In air non-combustible - Chemical inert
Carbon fibres (Celion, Carbolon, Sigrafil, Thornel)	1.7-2.0	2000-3000	< 1	200000 - 500000	- Special fibres for high-tech products - Very low expansion coefficient - Non-combustible

Table 1: Material properties of the base material of fabrics [7]

Fibreglass

The material where fibreglass is made of is of course glass, where threads are spun from, which have a certain bending capacity. The fibreglass has a high tensile strength, but remains brittle and has low elastic strain. Because of the brittleness the material needs to be handled carefully and needs very accurate manufacturing. Ageing exerts little influence on the material what has a tremendous impact on the expected lifetime of the structure. But the tensile strength of the material decreases when it is subjected to moisture.

Aramid fibre

This is a relative new type of fibre, discovered simultaneously by Akzo (Twaron fibre) and DuPont (Keviar fibre). The material has a high tensile strength and is chemically resistant. A drawback is the low elastic strain and the bad resistance against high temperature and UV-light

Composition of the base material

Fabric that is used normally for membrane structures is built up out of a woven structural base material, which has a covering on both sides to protect it from water and pollutants, the so-called coating. There are several ways to establish a coherent woven cloth. The basic method of weaving is called basket bond, where the weft threads pass the warp threads alternating above and underneath. There are a lot of varieties possible, like passing three warp threads underneath and one above.

Fig. 1 Basket bond (left) and Panama bond

Doing this, all kinds of patterns occur like is done in the carpet industry. But for structural use this it is not very sufficient and therefore only the basket bond and panama bond is used for membrane structures. Panama bond indicates that the weave operation is done with more than one thread at a time. 12*12 panama means that one cm of fabric contains 12 warp and 12 weft threads. At the other hand it is also usual to say 2-2 panama or 3-3 panama which means that the weaving operation is done with two, respectively 3 threads at a time. Panama bond has a better mechanical behaviour than basket weave because of the multiple yarns that are used.

Coatings

In the table above the fibres are described from which the fabric is woven. To create durable and water tight cloths most of the fibres need a coating on both sides. There are several coatings available. The most common ones are PVC coatings, Teflon coatings and silicone coatings. Sometimes not a coating is applied, but a foil is laminated upon the fabric.

The coating often is used to weld the different parts of the membrane together. The adhesion of the coating to the fabric is an indication for the strength of seams. The adhesion of a lamination to the fabric is much lower and therefore requires other connection methods for the seams.

PVC coating on Polyester cloth

This type of coating is used mostly on Polyester fabric. It is either coated or laminated upon the cloth. Dozens of different manufacturers provide such a material, which range from laminated fabrics for party rental tents to heavy coated fabrics for permanent (15-20 year replacement cycle) architectural installations. The fabric comes in numerous colours, has three different top coatings (PVDF, PVF, Acrylic) and is considered a fire-resistive material (see figure 2a).

PVC coating on Aramid weave

Another interesting lightweight building material is Aramid fibre used for air tubes. These high-pressure air tubes can take on the support function of a beam, an arch or a grid becoming a type of frame structure. The Aramid fibres are braided into curved forms and bonded to an inner urethane membrane to create seamless inflatable arches of approximate 30 psi. The Aramid fabric is enclosed with a PVC cover to protect the fibres from UV-degradation [1].

PTFE coating on fibreglass weave

Teflon coated fibreglass fabric is the most permanent of the coated architectural fabrics. First employed for a roof in 1973 for the La Verne College Student Centre in California (figure 2b) it has a lifetime of over 30 years. It can be used only for permanent applications and is not relocatable. The fabric is considered non-combustible and as such meets the most stringent building codes worldwide. Off the role it has an oatmeal appearance, which bleaches out to white in the sun after a couple of months. With translucency's up to 25 % it has been used in such projects as the Georgia dome, Denver Airport and currently used on the Millennium Dome.

Silicone coatings on fibreglass weave

Silicone coated fibreglass, which dates from 1981, has been used for Callaway Gardens in Georgia and the tensegrity domes for the Seoul Olympics. Silicone rubber is more flexible than Teflon, and fibreglass coated with it is less likely to be damaged during shipment and erection than fibreglass coated with Teflon. The greatest advantage, however, is that the fabric can be made very translucent, which is claimed to be as much as 25% translucency for the architectural membrane and 90 %

Fig. 2a PVC coated Polyester structure in the Netherlands After local fire a hole occurs in the membrane, but the fabric itself is not destroyed.

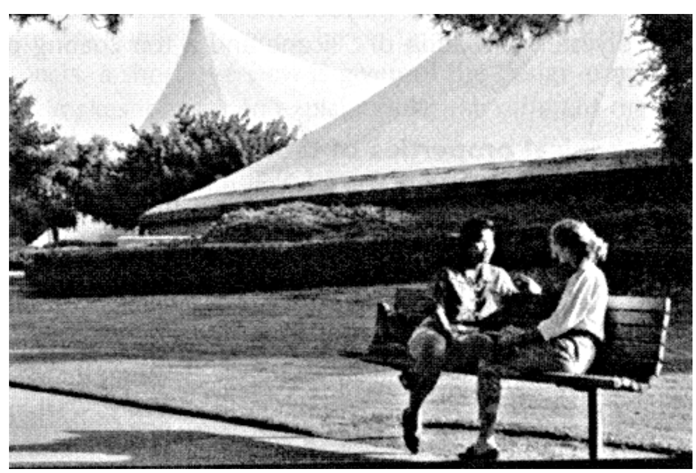


Fig. 2b Oldest commercial PTFE/Fibreglass roof – The LA Verne College Student Centre

	Polyester fabric			Fibreglass fabric	
Coating	PVC	PVC	PVC	PTFE	Si
Top coating	Acrylic	PVF-lamination	PVDF-merging		
Expected lifetime	8-1 0 years	12-15 years	12-15 years	>30 years	>30
Ageing Resistance	Average	Good	Good	Very good	Very good
Self-cleaning	Average	Good	Good	Very good	Average
Transparency	Good	Good	Good	Good	Very good
Fire-retardant	Good	Average	Good	Very good	Very good
Foldable	Very good	Average	Good	Bad	Average

Table 2 Properties of fabrics [4]

translucency for the thin liner material. With multiple layers of translucent membrane and glass fibre there can be both daylight illumination and very high heat retention. Silicone (Si) is one of the most abundant of the earth's elements, and forms the basis both of the fibreglass threads of the fabric and the silicone rubber of the coating. This similarity in chemical structure allows the design of highly translucent fabrics, while the water protection provided by the silicone coating assures long life span for the fibreglass. With regard to cost and handling, silicone coated fibreglass can be positioned somewhere between Teflon coated fibreglass and PVC coated Polyester.
Recent advantages have partly or wholly resolved early concerns about building with silicone coated fibreglass. Normally the seams are glued, which needs to be done under controlled circumstances. It is said that seams can now be chemically bonded (heat accelerated) to be stronger than the material itself, as with Teflon-coated fibreglass. Some engineers still question whether this process can be adequately applied with patch kits, used on site. The self-cleaning properties have been improved and are said to be equal to Teflon's, yet a once-a-year cleaning is recommended.

Silicone coating on Polyester weave

An ideal fabric would combine the low cost, easy handling and excellent structural behaviour of PVC-coated Polyester with the translucency and long life of Silicone- coated fibreglass, and the high reflectivity and resistance to dirt of Teflon. Is that maybe a membrane with a fabric of Polyester, a coating of Silicone and a top coating of ETFE?

Mechanical properties of fabrics [8]

The fabric behaves in a special way due to the weaving process. Conventional building materials are characterised by their linear elastic and isotropic behaviour. Only when the elastic limit is reached and yield area starts, different rules need to be applied. Materials used in textile architecture have a completely different behaviour and act as following:

- Non-linear, that means that the stress-strain behaviour of the material can not be modelled with a linearization of the curve
- Anisotropy, that means that the material itself has two dominant head directions, which makes all the important mechanical properties direction-dependent.
- Non-elastic, that means that the behaviour of the material is dependent on the added loading.

Non-linearity

At first the non-linearity will be explained. A fabric sample is tested in an uni-axial testing machine. In figure 3 a typical result is displayed from such a test. The stress and strain are displayed.

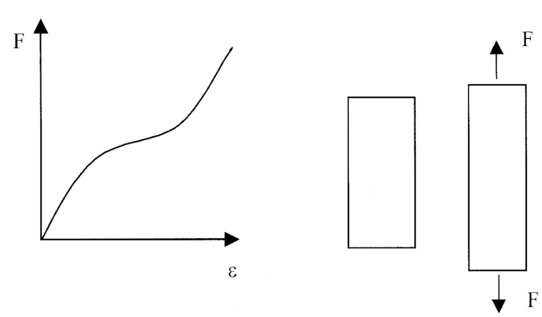


Fig. 3 Typical stress-strain curve uni-axial loaded [8]

It is clear that there is no linear relation between the stress and strain.
Only with a lot of creativity it is possible to draw a straight line along the curve.
Next is the anisotropy to be explained. Therefore several strips are cut out of the fabric, but a different orientation of the fibres is regarded (see figure 4).

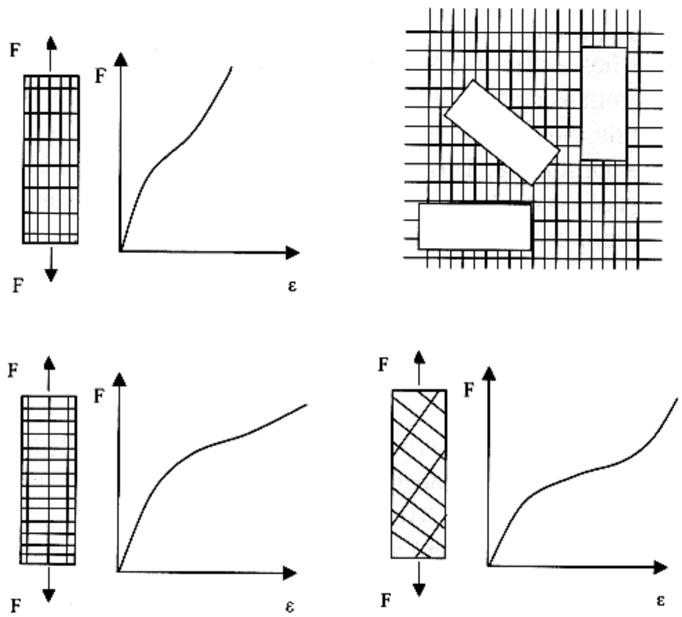


Fig. 4 Anisotropy shown in different fibre orientations [8]

It is obvious that in the different fibre directions there is a distinctive behaviour. This behaviour is caused by the presence of the woven base material in the fabric. During the weaving process, the warp threads are tensioned in the weaving machine and therefore initially straight. The weft threads sneak around them in alternate patterns, as a result of a weaving process in which alternate warp threads are pulled upwards or downwards and a weft thread is shuttled in between them. In the resulting long rolls of fabric, the weft threads running side to side, are kinked around the straight warp threads, which run the full length. In most coating processes this configuration is maintained. One fabricator of Polyester fabric, Ferrari, stretches the weft threads before coating.

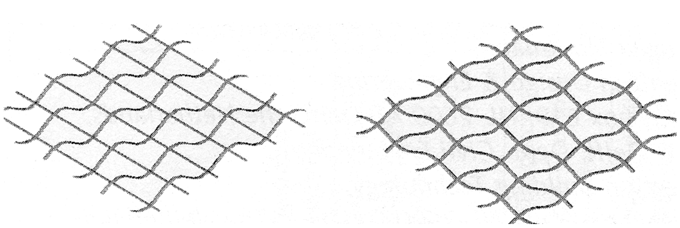


Fig. 5 Left: warp and weft configuration before stressing; right: warp and weft configuration after stressing

The effect of this configuration on the mechanical properties is that the strain is not the same in warp and weft direction. When the warp direction is tensioned, there will be little deformation because the fibres are straight already. When the weft fibres are tensioned, they are kinky, but become straight and therefore have a large deformation compared to the warp direction. In figure 5 the configuration is shown before tensioning and after tensioning.
The last aspect, the non-elasticity is explained by means of the same test examples but then carried out more than once on the same sample (see figure 6).

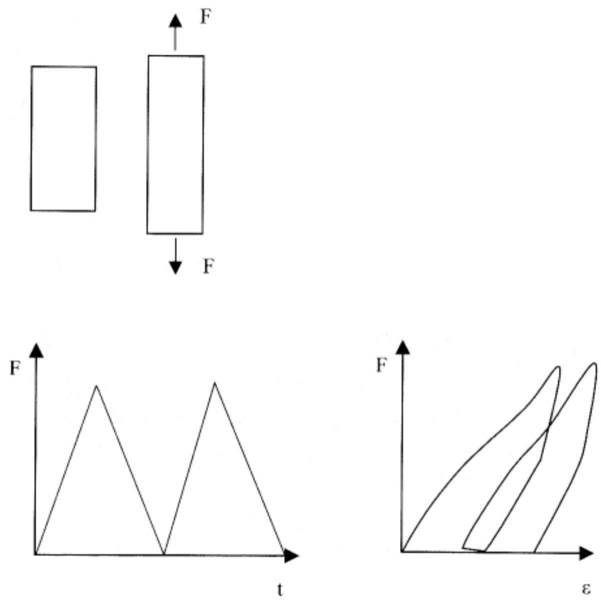


Fig. 6 Non-elastic behaviour of woven material [8]

It can be seen that the loading curve is different from the unloading curve. When the second loading cycle starts, it differs from the first one, as well as the second unloading curve differs from the first one. When the loading cycles are repeated, each loading and unloading cycle is different, although the differences are getting smaller. The difference remains between loading and unloading, which results in a permanent elongation of the fabric. The size of the elongation depends on the previous applied loads. All these aspects act simultaneously. Therefore it is very difficult to describe the mechanical behaviour of fabric with one model. To get a better understanding of those aspects, a short overview is given of the design process. This makes it easier to explain when the different material aspects need to be regarded.

Design process

The design of a membrane structure starts with the formfinding. Since there is a double opposite curvature, there need to be found equilibrium between the pretension in the membrane and the boundary conditions. This is normally done by means of computer software. Modelling the membrane as a two-way net is a very representative basis for computer analysis. One direction of the, mesh can be seen as the warp threads, the other direction of the mesh can be seen as the weft threads. When the boundary conditions are set, a first shape is obtained. This can serve as an image to explain the customer what the shape looks like and if it fulfils the needs. When is decided to go on with the structure, it is necessary to think about the patterning layout. The membrane is built up out of small strips because the fabric comes with rolls of a certain width. The strips are welded together and form the membrane. Because of the anisotropy of the material, it is necessary to orient the warp and weft threads in the head directions of the curvature. The load bearing behaviour is influenced considerably when the head direction of the fabric does not correspond with the head direction of the curvature (see figure 7). There is much more deflection possible as the mesh does not have shear stiffness. So the stiffness of the shape is depending on the adhesion from the coating to the fabric.

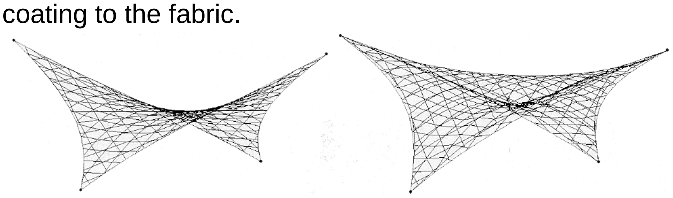


Fig. 7 Two ways of mesh orientation: These result in the same shape but with different load bearing behaviour (1 kN/m^2 upward load)

When the main direction of the anisotropy is known, the points of departure for the stiffness of the structure can be determined. With these values a statical analysis is made, which results in forces in the primary structure and the membrane. The results of the statical analysis on stresses and deformation are used to check the loading limits and failure modes. For membranes the following failure modes are critical:

- Failure of the bi-axial loaded membrane within the assumed lifetime of the structure
- Failure of a seam or connection of membrane to primary structure
- Tear failure during installation or because of vandalism.

The first failure mode is depending on the safety factors used upon the ultimate strength of the material. The difficulty of the non-elastic material property is dealt with in a very simple way. Just a small amount of the strip tensile capacity is used. Depending on the used fabric, there is the risk of brittle failure (fibreglass) or large plastic deformation (Polyester). So for permanent loading sometime a ratio f 1/8 is used, for windloads 1/4 is used and for snowload 1/5 is used because it can last for several weeks and therefore is a semi-permanent loading. According to the DIN, the design load cannot be larger than 0.85/3.1 *strip tensile strength. Another approach is to stay under the tear strength of the material to prevent tear failure. This results in a ratio of 1 /5-1 /6.

Fabric/ Coating	Weight [g/m^2]	Fire retardant	Tensile strength Warp/weft [N/50mm]	Tensile strain Warp/weft [%]	Tear strength [N]	Bending capacity	Seam strength [N/50mm]
Polyester/PVC		B1				Very good	
Type 1	800		3000/3000	15/20	350		2400 (30mm, 70'C)
Type 2	900		4400/3950	15/20	580		2850 (60mm, 70 'C)
Type 3	1050		5750/5100	15/25	950		3350 (60mm, 70 'C)
Type 4	1300		7450/6400	15/30	1400		4600 (60mm, 70 'C)
Type 5	1450		9800/8300	20/30	1800		4600 (60mm, 70 'C)
Fibreglass/PTFE	800 1270	A2 A2	3500/3000 6600/6000	7/10 7/10	300 570	Sufficient	6000 (60mm, 70 'C)
Fibreglass/Si	800 1270	A2 A2	3500/3000 6600/6000	7/10 7/10	300 570	Good	
Aramid/PVC	900 2020	B1 B1	7000/9000 24500/24500	5/6 5/6	700 4450	Good	4800 (30mm, 70 'C)
PTFE/-	520	Non com-bustible	2000/2000	40/30	500	Very good	
Cotton- Polyester/ -	350 520	B2 B2	1700/1000 2500/2000	35/18 38/20	60 80	Very good	

Table 3 Mechanical properties of common fabrics [7]

So there are several ways the admissible tensile load is determined.

The second failure mode, failure of a seam, should be avoided by testing which seam width is needed at which temperature. When the temperature rises, the seams get weaker. Above 70* the strength of the seam gets considerably lower.

Tear failure (the third mode) often occurs during installation. It starts at an open edge or at a hole in the fabric. It is critical, therefore, that the fabric panels are contained all around the edges, with a continuity that is meticulously maintained. Most commonly, edge ropes in continuous sleeves, which are connected cables or other structural members, achieve this. Another cause for tear failure is the acting of tangential forces in the membrane. When no proper take-up of these forces is provided, the fabric can tear under heavy loading. When the quality of the membrane is determined, the cutting patterns can be made from the final shape. The shape has a certain pretension, and the patterns need to be compensated for that. The needed compensation is depending on the strain of the fabric under the prestress in the membrane. This strain needs to be investigated by means of biaxial tests on the fabric under similar prestress conditions as present in the membrane.

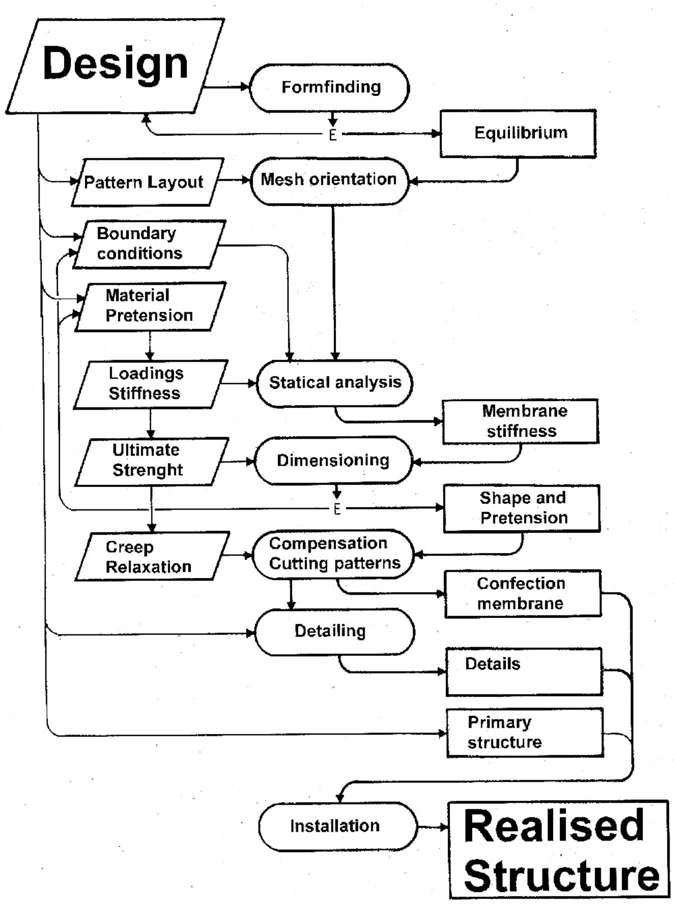


Fig. 8 Possible design scheme

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[2] Michael Haist, Christoph Niklasch, Yahya Bayraktarli: "Vorgespannte Membrantragwerke", Seminar Leichte Flächentragwerke, TU Berlin 1998/99.

[3] Horst Berger.- "Light structures, structures of light" Birk- hduser Verlag Berlin, 1996.

[4] Rogier Houtman: "From computer model to realised structure", TU Delft, 1996.

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[7] Wemer Sobek, Martin Speth: "Von der Faser zum Gewebe" page 74-81 DB nr 9 Sept. 1993.

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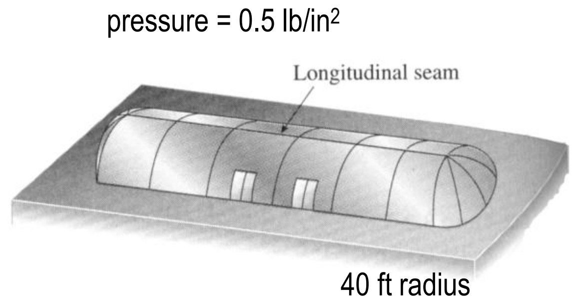
Examples:
Membranes, Nets & Shells

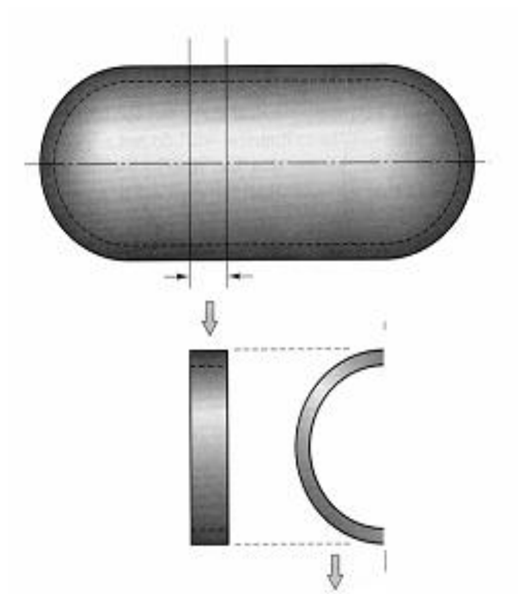
Example 1

An inflatable structure used by a traveling circus has the shape of a half-circular cylinder with closed ends. The fabric and plastic structure is inflated by a small blower and has a radius of 40 ft when fully inflated. A longitudinal seam runs the entire length of the "ridge" of the structure.



If the seam tears open when it is subjected to a tensile load of 540 pounds per inch of seam, what is the factor of safety against tearing when the internal pressure is 0.5 psi and the structure is fully inflated?

What is the force on the seam at the intersection with the quarter spheres? If the thickness of the membrane is 0.025 in, what is the stress?



SOLUTION:
Find the tensile load for a one inch section of the membrane structure. A free body diagram is helpful to show the pressure:

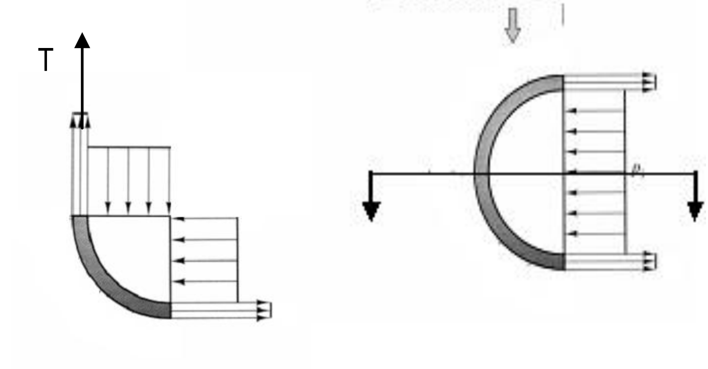
T for a circular membrane for a unit width carrying an internal pressure p_r is: T = p_rR

It doesn't matter where we cut a section, the force will still be T.

$$T = 0.5\,{}^{lb}\!/\!_{in^2}\,(40\,ft)\cdot\left(\frac{12in}{1ft}\right) = 240\ \text{lb/in}$$

The factor of safety is the ratio of limit load to actual load:

$$F.S. = \frac{540\,{}^{lb}\!/\!_{in}}{240\,{}^{lb}\!/\!_{in}} = 2.25$$



The ends are spherical, so the equation for force is T= p_rR/2.
The force will be ½ (240 lb/in) = 120 lb/in

The stress is equal to the force per length divided by the thickness, f = T/t

f = T/t = (120 lb/in)/(0.025 in) = 4,800 lb/in^2

Example 2

Investigate with computer modeling the stresses and behavior of a hyperbolic paraboloid under uniform roof loading with column supports away from the edges, as actually built for a residence with glazing between columns. (Ref. Architectural Structures, Wayne Place, 2007, Wiley, NJ.)

Figure 8.260 Residence with hyperbolic paraboloid roof, designed by architect Eduardo Catalano, in Raleigh, North Carolina.

Figure 8.261 Residence with hyperbolic paraboloid roof, showing the ample overhang of the roof, the boundary members, and the mullions in the glass walls.

SOLUTION:

The axial force diagram (b) shows that the axial forces appear to be uniform, as the discussion in the text indicates, but that the edge members have higher axial forces.

The deflection diagram (b) indicates negative bending over the columns, which indicates there are probably significant bending moments (which should be minimal in a shell), verified by the bending moment diagram (d).

This house had significant problems.

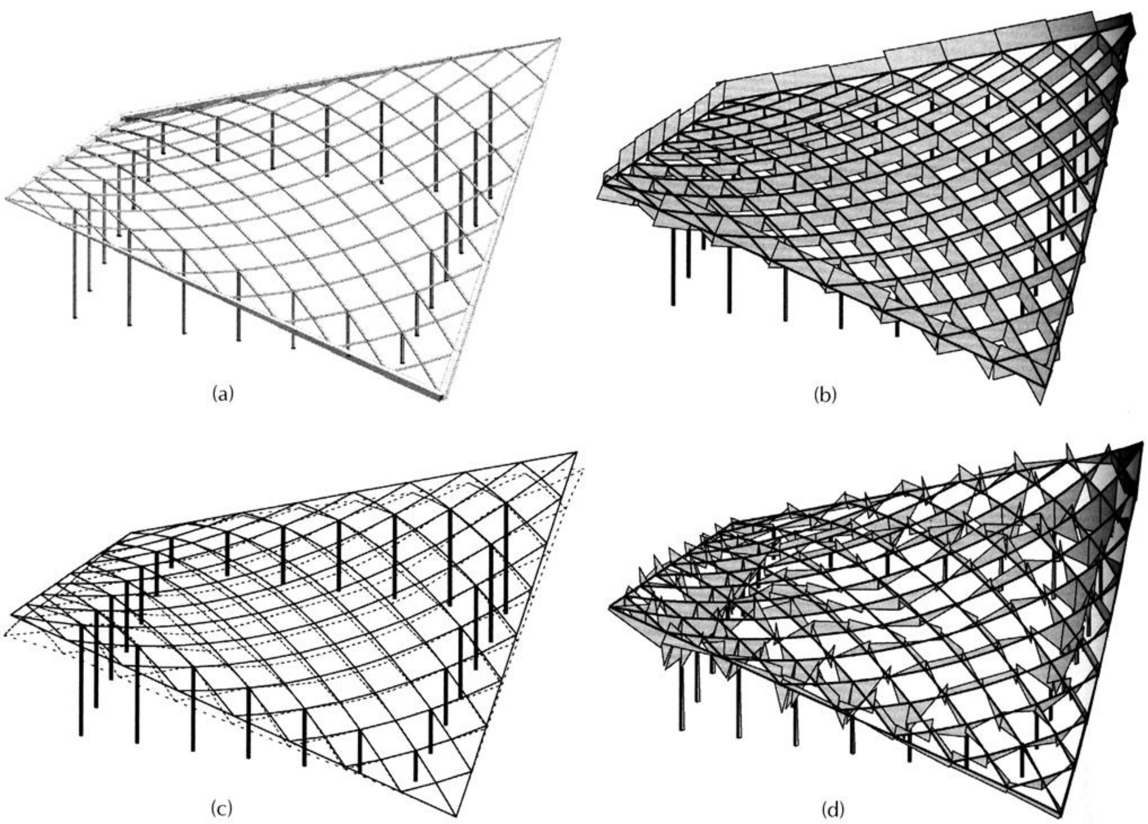


Figure 8.262 Hyperbolic paraboloid under uniform load, showing stabilizing columns (a), axial forces (b), deflection (c), and bending stress (d).

Areas Where Teaching Structures Should Be Strengthened

William L. Thoen
Retired principal, LeMessurier Consultants, Cambridge, MA

For nearly fifty years I have been pleased to provide structural consulting to architects on building projects throughout the United States and the Mideast, and in size from smaller than a house to as large as a city[b]

Most of the preliminary designs an architect brings to me for structural services are pretty well thought out in terms of appropriate column spacing and allowance for beam depths, and have suitable locations to accommodate the structural frame. In subsequent discussions an appropriate framing scheme usually develops without a great deal of conflict. Sometimes, knowing what the architect is trying to achieve, a unique structural arrangement becomes obvious, and if the architect can incorporate that in his plans, a strikingly new form evolves[c].

Having said that, there are some common planning weaknesses that occur frequently. They are: 1) Building stability and lateral bracing, 2) Structural frame vertical organization, 3) Tolerances between the structural frame and the architectural finish, 4) Site considerations, and 5) Floor vibration and comfort performance.

Lateral Bracing

If he has thought about lateral forces at all, the architect will often say, "Well, I will allow you bracing in the core," as if that were the end of the matter.

Connector / SPRING 2007 VOLUME XV

Core bracing alone makes the width or depth of the core become *the structural depth* of the building, regardless as to how wide or how long the building is. Accordingly, the core becomes a flagpole, or mast, that braces the entire building, and which may be too slender for acceptable sway performance in taller structures. In addition, lateral forces eccentric from the core may twist the building back and forth uncomfortably because the core alone cannot provide sufficient torsional stiffness. Even though the building may have sufficient *strength*, the inability of the core alone to provide sufficient *stiffness* can result in undesirable building motion, slapping of elevator cables against sidewalls, sloshing of water in toilet bowls, swinging doors, binding windows, squeaks, groans and mal-de-mer.

Another popular, but ineffective, location for lateral bracing is the exterior wall corner bays of the building, which are the worst exterior wall locations because the corner columns are the most lightly loaded and therefore have the least gravity weight to offset overturning uplift.

Vertical Alignment

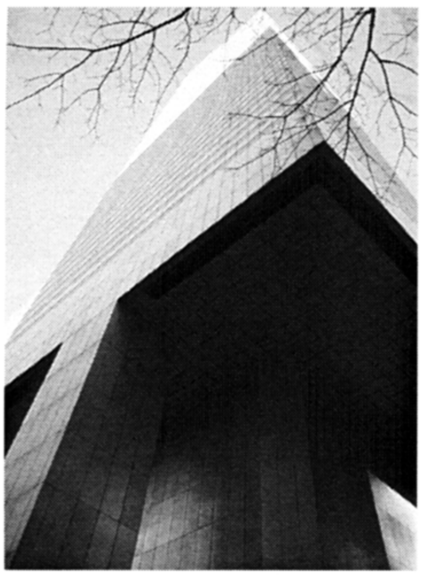
Another common planning weakness is structural frame discontinuity in the vertical direction. Think of a building with

Continued on page 4

Continued from page 3

upper level apartments above lower level office spaces, all over ground floor commercial spaces with basement parking underneath. Each occupancy has its own optimum structural module, which, if rigorously applied, results in massive transfer girders or story-deep trusses and each change of occupancy.

Teaching should include planning of an efficient structural module that can be threaded through the differing occupancy levels.



Tolerance

The need for tolerance between the structure and the architectural finish is often not considered. The actual depth of a steel column may be as much as two inches larger than the nominal depth. Splice plates, connections and bolts can make the structural cross-section even deeper, and fireproofing, where required, adds to that. Base plates will be larger than the column they support for welding and area requirements, and commonly sit atop a bed of grout.

Remember, too, that concrete has a way of hardening up and a slightly misplaced wall or anchor bolts have to be accommodated.

Teaching should include the necessity for providing "float" between the structure and the architectural finish when preparing preliminary sketches, especially where concrete work joins the superstructure.

Site Considerations

Site constraints may influence the choice of structural module or type. Most architects practicing within a region are aware of its special requirements. Hurricanes (Southeast), earthquakes (West coast), tornadoes (Midwest), expansive clays (Texas), permafrost (Alaska), and extreme temperature or humidity variation (Mideast) represent localities with special requirements. Sometimes availability of materials or lack of skilled labor will govern the design vernacular.

As important, site constraints or subgrade conditions may strongly affect the structural system and even the architectural form.

On good soils, the structure can be founded on simple footings. Where the building location is underlain with organic material, soft clays and the like, special foundation systems are required, differential settlements and control of groundwater considered, and these may influence the choice of the superstructure system, including column spacing, to achieve an optimal system.

Floor Vibration and serviceability

Today's buildings are lighter and more gossamer than their ancestors. With today's high-strength materials, composite construction, and lightweight concretes, floor spans can be made longer, and *stiffness*, rather than *strength*, often governs the structural depth. As a consequence, floor vibration, cambering, and careful deflection control become important factors for occupancy comfort, especially in large, column-free spaces without damping partitions. Often floor vibrations are not sensible to walkers, but become intolerable to a person sitting, as in an office.

All too often thin, long span floors are envisioned, but which must be deepened (stiffened), or otherwise damped against excessive vibration.

How these can be taught

A good way to teach these areas of structural planning (and structures in general), I think, is to choose a building of interest that has been built and study its structure. How has the architect and engineer collaborated to make it successful? What constitutes the lateral bracing system? What were the site constraints, if any, and how did they influence the design? What is the column grid module and what is the floor system depth for its spans?

If a student studies two or three built projects a semester, each illustrating a type of building the student is likely to encounter in practice, and if the student is made to keep a notebook of sketches and notes relative to each type of building, he will have then studied structural solutions in context with the architectural problem, and will also have a useful future reference upon graduation.

Endnotes:

a. King Kahlid Military City, Saudi Arabia
b. Citicorp, NYC; Gymnasium, Philips Exeter Academy, Exeter, NH; Fiduciary Trust Bldg, Boston, MA

<u>Buildings at Risk: Wind Design Basics for Practicing Architects</u>, AIA, 1998

3

Wind Impacts on Buildings

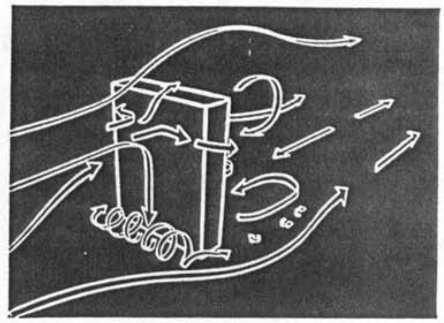


Figure 3.1: Flow of air around a high-rise building.

3.1 WIND FORCES

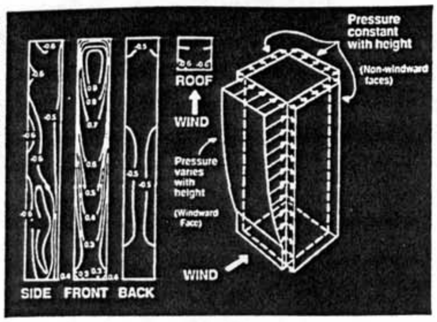
Buildings are continually subjected to wind forces. Generally, these wind forces are at levels that the structure is capable of resisting, whether that capability is based on an engineered design using building code-specified wind loads, or, as is the case with most residential construction, it is based on standard construction practices that have developed over time. Periodically, structures are subjected to wind forces that cause damage. In some instances, the damage is due to wind loads exceeding design criteria. In most cases, the damage results from a weakness in the building itself.[1]

Damaging Winds

Damaging wind forces usually are associated with extreme weather phenomena, such as tornadoes, hurricanes, or thunderstorms. Maps indicating wind speeds for 50-year mean return periods have been used in building codes to establish wind loads for building design. The maps and other factors in design standards take into account the varying wind loads experienced in different environments, i.e. near the coast, inland, open terrain and urban environment. Building codes and standards generally use gust and other factors that are applied to the basic wind speed to account for the dynamic effects of wind.

In practice, the actual wind loads on a building rarely exceed the design wind load. Even in cases where design-level winds are somewhat exceeded, a well-designed and constructed building should sustain relatively little damage to the structural frame.[2] The building envelope (roof, walls, and openings) is another story. Breaches to the envelope have been observed to be the major cause of damage in high wind events, and envelope systems have sustained considerable damage even at wind speeds below design levels.

Many buildings would suffer severe damage if struck directly by a moderate to strong tornado. This damage results not only from the extreme



Cermack, Chiu, Perry

Figure 3.2: Wind tunnel analysis of the World Trade Center buildings and code approach.

wind speeds, but from the dynamically changing wind directions and the impact of wind-borne debris. Similarly, structures along the coast in the path of a hurricane may be simultaneously subjected to the severe forces of both wind and water, the greatest magnitude of each occurring at approximately the same time. The wind velocoties in a hurricane may exceed design levels and may subject the building to high winds first from one direction and then the other.[3]

Wind Loads

Wind loads on buildings can be calculated using the formula contained in the *American Society of Civil Engineers (ASCE 7-95) Standard for Minimum Design Loads for Buildings and Other Structures*. The wind load is an expression of the formula:

$$p = qGC$$
$$q = 0.00256K_zK_{zt}V^2I$$

where:

p = design pressure in psf
q = velocity pressure in psf
0.00256 = constant for mass density of air and appropriate conversion constants so that V may be given in mph
K_z = velocity pressure exposure coefficient
K_{zt} = topographic factor
V = basic 3-second peak gust wind speed in mph
I = importance factor, defines the level of risk depending on occupancy
G = gust effect factor, which considers spatial size of gust relative to the size of buildings, gust frequency relative to natural frequency and damping of structure, basic reference design speed, and terrain exposure
C = mean pressure coefficient (combining internal and external coefficients)

Use of this formula by an architect is relatively rare, as most wind load analysis is conducted by engineers and specialists. However, it is important for architects to be familiar with the formula so that they understand the impact of wind on the building's design and can discuss it with the engineer. Regardless of who performs the wind load calculations, it is imperative that loads be determined for the building envelope as well as the structure.

Vibration

Wind-induced structural vibration can be a concern in specialty structures such as tensile roofs, bridges, and other unusual configurations.

Buffeting vibration is produced by the unsteady loading of a building due to turbulence (velocity fluctuations in magnitude and direction) in the approaching free flow wind field. If the turbulence is generated by an upwind neighboring structure or obstacle, the unsteady loading is called wake buffeting or interference. The World Trade Center Twin Towers in New York City (Figure 3.2) and the John Hancock building in Boston are examples of buildings that experience the latter type.

Most building codes (e.g. ASCE 7) treat the along-wind vibration but do not address across-wind or torsional buffeting vibration.

The flow behind a long cylinder held perpendicular to wind is characterized by the periodic shedding of vortices (whirling air flows). Vortex shedding creates periodic lateral forces that can cause vibration of slender structures such as towers and tall buildings. Although vortex shedding is most noticeable for cylindrical buildings, it also happens to a lesser degree to tall buildings of other shapes.[4]

Figure 3.3: Vortex shedding.

Vortex-shedding vibration takes place when the wind speed is such that the shedding frequency becomes approximately equal to the natural frequency of the cylinder—a condition that causes resonance. When resonance takes place, further increase in wind speed by a few percent will not alter the shedding frequency. This phenomenon is called "lock-in." Because the structure vibrates excessively only in the lock-in range, having a wind speed either below or above the lock-in range will not cause serious vibration. If the shedding frequency is the same as the natural period of the building, it can have a load impact on the structure, pulling the building back and forth in an across-wind direction.[5] (Figures 3.3 and 3.4)

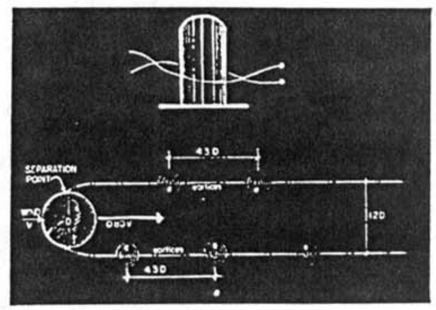


Figure 3.4: The Karman Vortex Phenomenon.

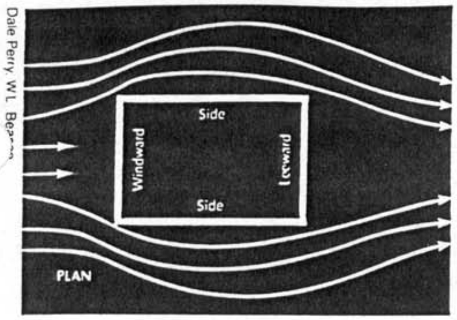
Classical flutter (or simply flutter) is a two-degrees-of-freedom vibration involving simultaneous lateral (across-wind translational) and torsional (rotational) vibrations. It occurs in structures that have approximately the same magnitude of natural frequencies for both the translational and the rotational modes. Similar to galloping and torsional divergence, flutter is produced by aerodynamic instability completely unrelated to vortex shedding.[6]

Damage Mechanisms

The four primary damage mechanisms associated with severe windstorms involve:

(1) aerodynamic pressures created by flow of air around a structure;
(2) induced internal pressure fluctuations due to a breach in the building envelope;
(3) impact forces created by wind-borne debris; and
(4) pressures created by rapid atmospheric pressure fluctuations (associated primarily with tornadoes).

Examinations of building damage caused by various types of windstorms suggest that most winds produce damage due to a combination of aerodynamic pressures and internal pressure fluctuations and, for hurricanes and tornadoes, debris impacts. Atmospheric pressure fluctuations have little or no effect on the performance of ordinary structures because most ordinary structures have sufficient building envelope permeability (or venting) to allow equalization of pressures induced by atmospheric pressure changes. In airtight structures such as nuclear containment vessels, atmospheric pressure changes can impose significant loading to the building envelope.[7]



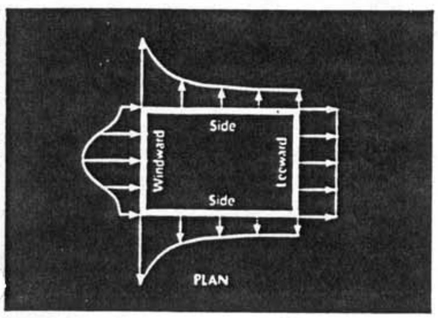
Dale Perry, W.L. Beason

Figure 3.5: Building in wind flow.

Wind pressures acting on buildings are distributed loads that are assumed to act normal to the building surface. Positive wind pressures act toward the surface of the building element and negative pressures (suction) act away from the building surface. The fundamental characteristics of wind pressures are described below based on the building component affected and the orientation of the building in the wind environment.[8]

As winds increase, pressure against objects is added at a non-linear rate. Pressure force against a wall mounts with the square of the wind speed so that a three-fold increase in wind speed, for example, results in a nine-fold increase in pressure. A 25 mph wind causes about 1.6 pounds of pressure per square foot. Therefore a 4x8 sheet of plywood will be pushed by a force of about 50 pounds. In 75 mph winds, that force becomes 450 pounds, and at 125 mph, it becomes 1,250 pounds.[9]

3.2 AERODYNAMIC PRESSURE IMPACTS



Dale Perry, W.L. Beason

Figure 3.6: Relative wind pressure on walls.

Impacts on Walls

Figure 3.5 presents a plan view of a simple rectangular building that is submerged in a wind flow as shown. Each wall of the structure is identified as a windward, side, or leeward wall depending upon its location with respect to the direction of wind flow. The windward wall is the wall facing the wind; the leeward wall is on the side opposite to the windward wall; and the side walls are parallel to the wind flow.[10]

Because the windward wall is perpendicular to the wind flow, the wind impinges directly on the windward wall producing positive pressures (Figure 3.6). As the wind flows around the windward corners, the local wind speed increases and the flow lines have a tendency to separate from the corner of the building. This causes the side walls to be subjected to negative pressures as shown. In addition, the turbulence and flow separations that occur at the windward corners of the building induce high negative pressures for short distances along the side walls. The leeward wall is also subjected to negative wind pressures that tend to be relatively uniformly distributed.[11]

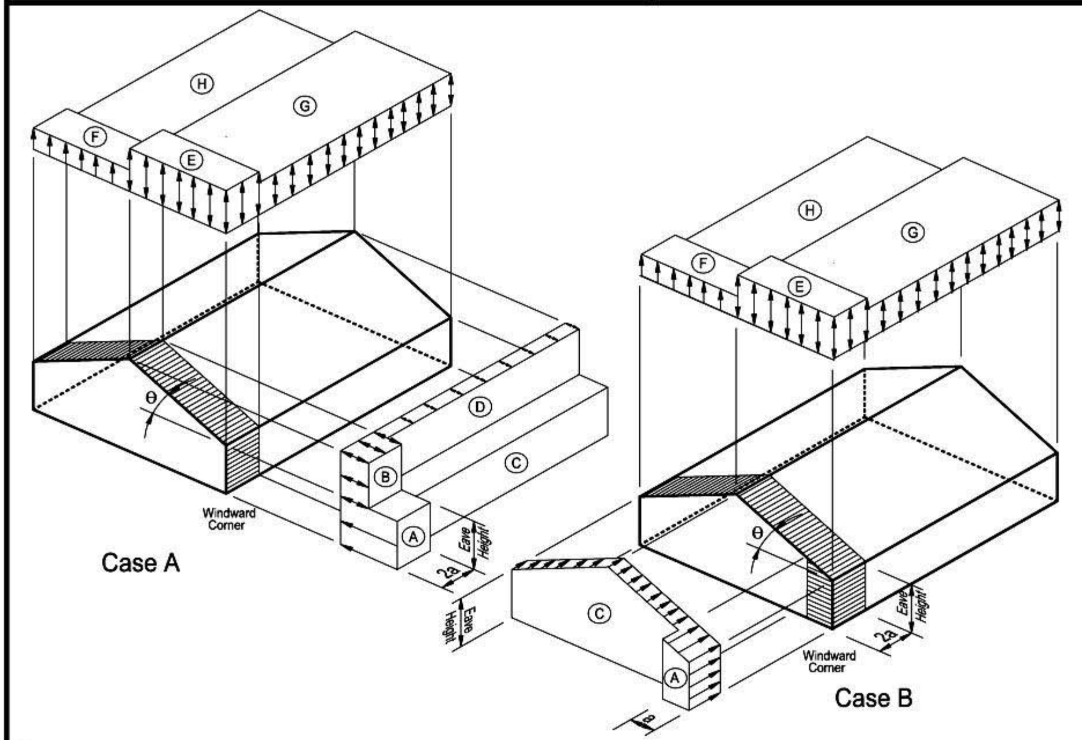
Impacts on Roofs

Wind creates a greater load on the roof covering than on any other element of a building. When a FEMA team investigated wind damage to buildings in Florida in the wake of Hurricane Andrew, their field observations concluded that the loss of roof covering was the most pervasive type of damage to buildings in southern Dade County. To varying degrees, all of the different roof types observed suffered damage due to the failure of the method of attachment and/or material, inadequate design, inadequate workmanship, or debris impact. Similar damage has been observed in the aftermath of other windstorms (Figure 3.7).

Design Wind Pressures – Envelope Procedure
SEI/ASCE 7-10:

Velocity pressure, *p*, irrespective of terrain and height above ground or recurrence probability is related to the wind speed, *V*, by $p = 0.00256V^2$. Wind codes also consider the effect of the geometry of the building and location on the surface, wind gusts or turbulence, the local terrain, and annual probability of exceeding the design wind speed.

Main Wind Force Resisting System – Method 2		$h \le 60$ ft.
Figure 28.6-1	**Design Wind Pressures**	**Walls & Roofs**
Enclosed Buildings		



Notes:

1. Pressures shown are applied to the horizontal and vertical projections, for exposure B, at h=30 ft (9.1m). Adjust to other exposures and heights with adjustment factor λ.
2. The load patterns shown shall be applied to each corner of the building in turn as the reference corner. (See Figure 28.4-1)
3. For Case B use $\theta = 0°$.
4. Load cases 1 and 2 must be checked for $25° < \theta \le 45°$. Load case 2 at 25° is provided only for interpolation between 25° and 30°.
5. Plus and minus signs signify pressures acting toward and away from the projected surfaces, respectively.
6. For roof slopes other than those shown, linear interpolation is permitted.
7. The total horizontal load shall not be less than that determined by assuming $p_S = 0$ in zones B & D.
8. Where zone E or G falls on a roof overhang on the windward side of the building, use E_{OH} and G_{OH} for the pressure on the horizontal projection of the overhang. Overhangs on the leeward and side edges shall have the basic zone pressure applied.
9. Notation:
   a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).
   h: Mean roof height, in feet (meters), except that eave height shall be used for roof angles <10°.
   θ: Angle of plane of roof from horizontal, in degrees.

Main Wind Force Resisting System – Method 2		h ≤ 60 ft.
Figure 28.6-1 (cont'd)	Design Wind Pressures	Walls & Roofs
Enclosed Buildings		

Simplified Design Wind Pressure , p_{S30} (psf) *(Exposure B at h = 30 ft. with I = 1.0)*

Basic Wind Speed (mph)	Roof Angle (degrees)	Load Case	Zones									
			Horizontal Pressures				Vertical Pressures				Overhangs	
			A	B	C	D	E	F	G	H	E_{OH}	G_{OH}
110	0 to 5°	1	19.2	-10.0	12.7	-5.9	-23.1	-13.1	-16.0	-10.1	-32.3	-25.3
	10°	1	21.6	-9.0	14.4	-5.2	-23.1	-14.1	-16.0	-10.8	-32.3	-25.3
	15°	1	24.1	-8.0	16.0	-4.6	-23.1	-15.1	-16.0	-11.5	-32.3	-25.3
	20°	1	26.6	-7.0	17.7	-3.9	-23.1	-16.0	-16.0	-12.2	-32.3	-25.3
	25°	1	24.1	3.9	17.4	4.0	-10.7	-14.6	-7.7	-11.7	-19.9	-17.0
		2	-------	-------	-------	-------	-4.1	-7.9	-1.1	-5.1	-------	-------
	30 to 45	1	21.6	14.8	17.2	11.8	1.7	-13.1	0.6	-11.3	-7.6	-8.7
		2	21.6	14.8	17.2	11.8	8.3	-6.5	7.2	-4.6	-7.6	-8.7
115	0 to 5°	1	21.0	-10.9	13.9	-6.5	-25.2	-14.3	-17.5	-11.1	-35.3	-27.6
	10°	1	23.7	-9.8	15.7	-5.7	-25.2	-15.4	-17.5	-11.8	-35.3	-27.6
	15°	1	26.3	-8.7	17.5	-5.0	-25.2	-16.5	-17.5	-12.6	-35.3	-27.6
	20°	1	29.0	-7.7	19.4	-4.2	-25.2	-17.5	-17.5	-13.3	-35.3	-27.6
	25°	1	26.3	4.2	19.1	4.3	-11.7	-15.9	-8.5	-12.8	-21.8	-18.5
		2	-------	-------	-------	-------	-4.4	-8.7	-1.2	-5.5	-------	-------
	30 to 45	1	23.6	16.1	18.8	12.9	1.8	-14.3	0.6	-12.3	-8.3	-9.5
		2	23.6	16.1	18.8	12.9	9.1	-7.1	7.9	-5.0	-8.3	-9.5
120	0 to 5°	1	22.8	-11.9	15.1	-7.0	-27.4	-15.6	-19.1	-12.1	-38.4	-30.1
	10°	1	25.8	-10.7	17.1	-6.2	-27.4	-16.8	-19.1	-12.9	-38.4	-30.1
	15°	1	28.7	-9.5	19.1	-5.4	-27.4	-17.9	-19.1	-13.7	-38.4	-30.1
	20°	1	31.6	-8.3	21.1	-4.6	-27.4	-19.1	-19.1	-14.5	-38.4	-30.1
	25°	1	28.6	4.6	20.7	4.7	-12.7	-17.3	-9.2	-13.9	-23.7	-20.2
		2	-------	-------	-------	-------	-4.8	-9.4	-1.3	-6.0	-------	-------
	30 to 45	1	25.7	17.6	20.4	14.0	2.0	-15.6	0.7	-13.4	-9.0	-10.3
		2	25.7	17.6	20.4	14.0	9.9	-7.7	8.6	-5.5	-9.0	-10.3
130	0 to 5°	1	26.8	-13.9	17.8	-8.2	-32.2	-18.3	-22.4	-14.2	-45.1	-35.3
	10°	1	30.2	-12.5	20.1	-7.3	-32.2	-19.7	-22.4	-15.1	-45.1	-35.3
	15°	1	33.7	-11.2	22.4	-6.4	-32.2	-21.0	-22.4	-16.1	-45.1	-35.3
	20°	1	37.1	-9.8	24.7	-5.4	-32.2	-22.4	-22.4	-17.0	-45.1	-35.3
	25°	1	33.6	5.4	24.3	5.5	-14.9	-20.4	-10.8	-16.4	-27.8	-23.7
		2	-------	-------	-------	-------	-5.7	-11.1	-1.5	-7.1	-------	-------
	30 to 45	1	30.1	20.6	24.0	16.5	2.3	-18.3	0.8	-15.7	-10.6	-12.1
		2	30.1	20.6	24.0	16.5	11.6	-9.0	10.0	-6.4	-10.6	-12.1
140	0 to 5°	1	31.1	-16.1	20.6	-9.6	-37.3	-21.2	-26.0	-16.4	-52.3	-40.9
	10°	1	35.1	-14.5	23.3	-8.5	-37.3	-22.8	-26.0	-17.5	-52.3	-40.9
	15°	1	39.0	-12.9	26.0	-7.4	-37.3	-24.4	-26.0	-18.6	-52.3	-40.9
	20°	1	43.0	-11.4	28.7	-6.3	-37.3	-26.0	-26.0	-19.7	-52.3	-40.9
	25°	1	39.0	6.3	28.2	6.4	-17.3	-23.6	-12.5	-19.0	-32.3	-27.5
		2	-------	-------	-------	-------	-6.6	-12.8	-1.8	-8.2	-------	-------
	30 to 45	1	35.0	23.9	27.8	19.1	2.7	-21.2	0.9	-18.2	-12.3	-14.0
		2	35.0	23.9	27.8	19.1	13.4	-10.5	11.7	-7.5	-12.3	-14.0
150	0 to 5°	1	35.7	-18.5	23.7	-11.0	-42.9	-24.4	-29.8	-18.9	-60.0	-47.0
	10°	1	40.2	-16.7	26.8	-9.7	-42.9	-26.2	-29.8	-20.1	-60.0	-47.0
	15°	1	44.8	-14.9	29.8	-8.5	-42.9	-28.0	-29.8	-21.4	-60.0	-47.0
	20°	1	49.4	-13.0	32.9	-7.2	-42.9	-29.8	-29.8	-22.6	-60.0	-47.0
	25°	1	44.8	7.2	32.4	7.4	-19.9	-27.1	-14.4	-21.8	-37.0	-31.6
		2	-------	-------	-------	-------	-7.5	-14.7	-2.1	-9.4	-------	-------
	30 to 45	1	40.1	27.4	31.9	22.0	3.1	-24.4	1.0	-20.9	-14.1	-16.1
		2	40.1	27.4	31.9	22.0	15.4	-12.0	13.4	-8.6	-14.1	-16.1

Unit Conversions – 1.0 ft = 0.3048 m; 1.0 psf = 0.0479 kN/m²

Main Wind Force Resisting System – Method 2		$h \le 60$ ft.
Figure 28.6-1 (cont'd)	Design Wind Pressures	**Walls & Roofs**
Enclosed Buildings		

Simplified Design Wind Pressure , p_{S30} (psf) *(Exposure B at h = 30 ft.)*

Basic Wind Speed (mph)	Roof Angle (degrees)	Load Case	Zones									
			Horizontal Pressures				Vertical Pressures				Overhangs	
			A	B	C	D	E	F	G	H	E_{OH}	G_{OH}
160	0 to 5°	1	40.6	-21.1	26.9	-12.5	-48.8	-27.7	-34.0	-21.5	-68.3	-53.5
	10°	1	45.8	-19.0	30.4	-11.1	-48.8	-29.8	-34.0	-22.9	-68.3	-53.5
	15°	1	51.0	-16.9	34.0	-9.6	-48.8	-31.9	-34.0	-24.3	-68.3	-53.5
	20°	1	56.2	-14.8	37.5	-8.2	-48.8	-34.0	-34.0	-25.8	-68.3	-53.5
	25°	1	50.9	8.2	36.9	8.4	-22.6	-30.8	-16.4	-24.8	-42.1	-35.9
		2	-------	-------	-------	-------	-8.6	-16.8	-2.3	-10.7	-------	-------
	30 to 45	1	45.7	31.2	36.3	25.0	3.5	-27.7	1.2	-23.8	-16.0	-18.3
		2	45.7	31.2	36.3	25.0	17.6	-13.7	15.2	-9.8	-16.0	-18.3
180	0 to 5°	1	51.4	-26.7	34.1	-15.8	-61.7	-35.1	-43.0	-27.2	-86.4	-67.7
	10°	1	58.0	-24.0	38.5	-14.0	-61.7	-37.7	-43.0	-29.0	-86.4	-67.7
	15°	1	64.5	-21.4	43.0	-12.2	-61.7	-40.3	-43.0	-30.8	-86.4	-67.7
	20°	1	71.1	-18.8	47.4	-10.4	-61.7	-43.0	-43.0	-32.6	-86.4	-67.7
	25°	1	64.5	10.4	46.7	10.6	-28.6	-39.0	-20.7	-31.4	-53.3	-45.4
		2	-------	-------	-------	-------	-10.9	-21.2	-3.0	-13.6	-------	-------
	30 to 45	1	57.8	39.5	45.9	31.6	4.4	-35.1	1.5	-30.1	-20.3	-23.2
		2	57.8	39.5	45.9	31.6	22.2	-17.3	19.3	-12.3	-20.3	-23.2
200	0 to 5°	1	63.4	-32.9	42.1	-19.5	-76.2	-43.3	-53.1	-33.5	-106.7	-83.5
	10°	1	71.5	-29.7	47.6	-17.3	-76.2	-46.5	-53.1	-35.8	-106.7	-83.5
	15°	1	79.7	-26.4	53.1	-15.0	-76.2	-49.8	-53.1	-38.0	-106.7	-83.5
	20°	1	87.8	-23.2	58.5	-12.8	-76.2	-53.1	-53.1	-40.2	-106.7	-83.5
	25°	1	79.6	12.8	57.6	13.1	-35.4	-48.2	-25.6	-38.7	-65.9	-56.1
		2	-------	-------	-------	-------	-13.4	-26.2	-3.7	-16.8	-------	-------
	30 to 45	1	71.3	48.8	56.7	39.0	5.5	-43.3	1.8	-37.2	-25.0	-28.7
		2	71.3	48.8	56.7	39.0	27.4	-21.3	23.8	-15.2	-25.0	-28.7

Adjustment Factor
for Building Height and Exposure, λ

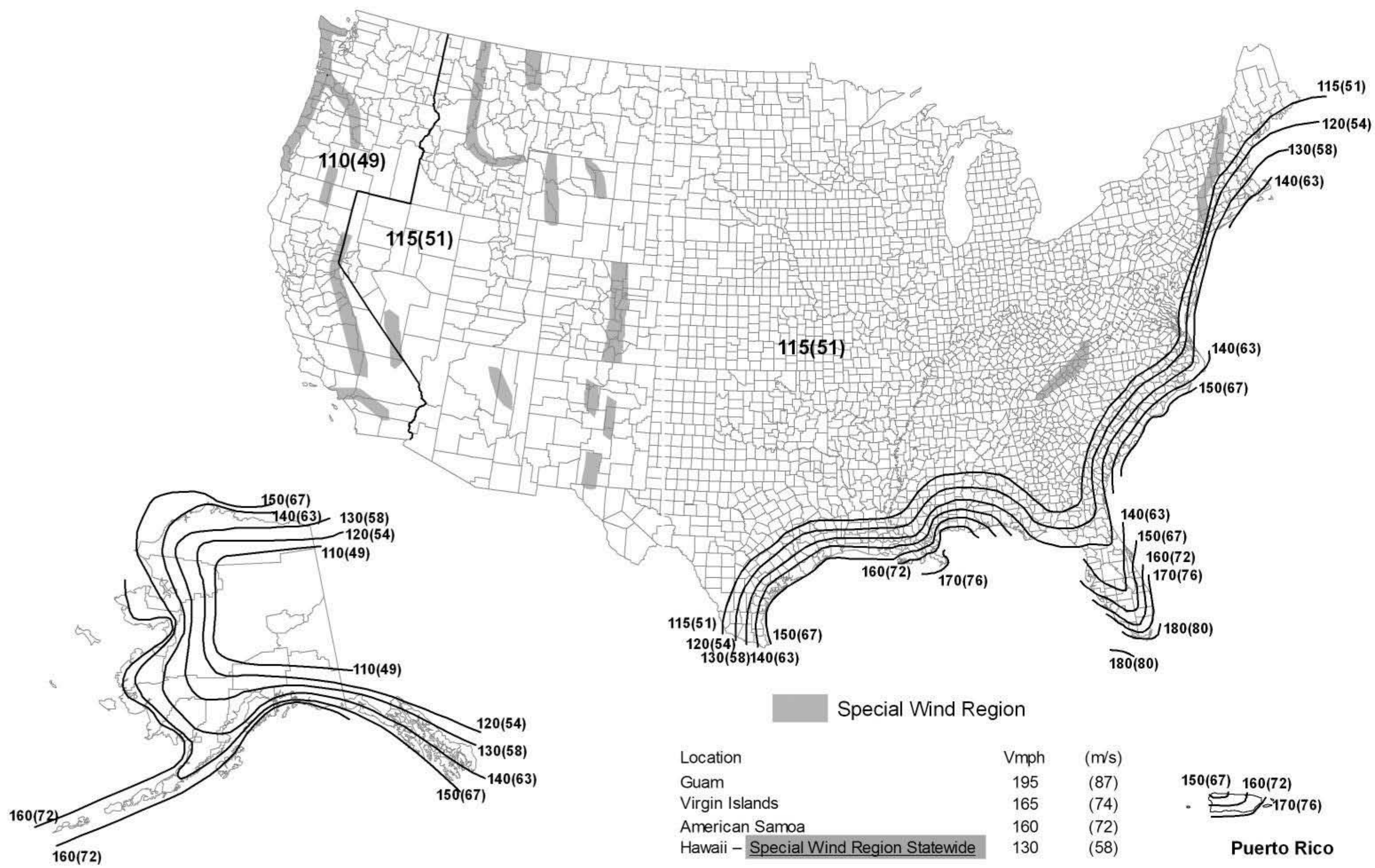
Mean roof height (ft)	Exposure		
	B	C	D
15	1.00	1.21	1.47
20	1.00	1.29	1.55
25	1.00	1.35	1.61
30	1.00	1.40	1.66
35	1.05	1.45	1.70
40	1.09	1.49	1.74
45	1.12	1.53	1.78
50	1.16	1.56	1.81
55	1.19	1.59	1.84
60	1.22	1.62	1.87

Unit Conversions – 1.0 ft = 0.3048 m; 1.0 psf = 0.0479 kN/m^2

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent a low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life. Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure. Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where their quantity exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.	III
Buildings and other structures designated as essential facilities. Buildings and other structures, the failure of which could pose a substantial hazard to the community. Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity exceeds a threshold quantity established by the authority having jurisdiction to be dangerous to the public if released and is sufficient to pose a threat to the public if released.[a] Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	IV

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the substances is commensurate with the risk associated with that Risk Category.



Location	Vmph	(m/s)
Guam	195	(87)
Virgin Islands	165	(74)
American Samoa	160	(72)
Hawaii – Special Wind Region Statewide	130	(58)

Figure 26.5-1A Basic Wind Speeds for Occupancy Category II Buildings and Other Structures.

Figure 26.5-1A *(Continued)*

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).

Residential Building Loads: Review and Roadmap for Future Progress, Crandell, Kenney & Rosowsky, ed., 2006 (ASCE)

APPENDIX B - Simplified ASCE 7-02 Wind Loads For Typical Low-Rise Buildings January 31, 2004

A.1 General. This appendix provides simplified wind loads that result in designs reasonably consistent with the requirements ASCE 7. It is intended for use by qualified design professionals and is subject to the limitations of Section A.2. In this method, a single wind pressure for each roof and wall vertical projected area and the roof horizontal projected area is used to determine main wind force resisting system loads. For components and cladding loads, surface pressures are determined for specific building elements such that multiple pressure zones are not required to be separately evaluated.

A.2 Limitations. These provisions are applicable to buildings meeting the following conditions:

- Light-frame, concrete, or masonry construction using shear walls and horizontal diaphragms to resist lateral loads.
- Mean roof height of 40 feet or less.
- One- and two-family dwellings, apartments, commercial buildings, and other building uses or occupancies with a wind load importance factor of 1.0.

A.3 Wind Design Criteria

A.3.1 Basic Wind Speed
The basic (design) wind speed shall be determined in accordance with Figure A1 or as required by the local governing building code.

A3.2 Wind Exposure and Topography. The provisions of this Appendix are based on wind exposure category B (suburban, urban, or wooded terrain) as defined in ASCE 7. For buildings located in wind exposure category C (open or coastal terrain), tabulated exposure B wind loads shall be increased by a factor 1.4 (see table footnotes as applicable in Section A4). Buildings sited within 10 building heights from the top edge of a prominent topographic feature shall be designed in accordance with ASCE 7. A prominent topographic feature has a ground slope of greater than 15 percent and a vertical rise of greater than 50 feet, and is separated from features of similar or greater height by a distance of more than approximately 100 times the height of the topographic feature.

A3.3 Wind-borne Debris Region.
The *wind-borne debris region* shall be defined in accordance with the Figure A1 for Atlantic Ocean and Gulf of Mexico coastal areas as follows:

Basic Wind Speed ≥ 120 mph – all areas.
110 mph ≤ Basic Wind Speed < 120 mph – all areas within 1 mile of coastline.

A3.4 Building Enclosure Condition
Building enclosure condition shall be classified in accordance with Table A1 for the purpose of determining wind loads in accordance with Section A4.2 and A4.3.

A3.5 Counteracting Dead Load
When dead load is used to counteract

41

effects of wind pressure, it shall be factored as follows for Allowable Strength Design (ASD) and Load and Resistance Factor or Strength Design (LRFD) methods:

ASD: W - 0.6D

LRFD: 1.6W - 0.9D

where W is wind load effect due to wind loads determined in accordance with Section A4 and D is dead load effect due to estimated actual dead load. Load effects include stresses in or forces applied to structural members, connections, or systems.

Other load combinations and design load effects shall be considered in accordance with ASCE 7, Chapter 2.

A.4 Wind Loads

A4.1 Lateral Force Resisting System Loads. Wind pressures from Table A2 shall be applied to building roof and wall vertical projected areas (VPA) corresponding to each of four elevations of the building to determine maximum lateral wind forces (shear) tributary to horizontal diaphragms, shear walls, and related connections.

A4.2 Roof System Uplift Loads. Wind pressures from Table A3 shall be applied to the horizontal projected area (HPA) of a roof system to determine uplift loads tributary to structural elements, assemblies, and connections that experience loads from multiple roof surfaces.

A4.3 Components and Cladding Loads. Table A4 shall be used to determine inward (positive) and outward (negative) acting wind loads tributary to wall and roof components, cladding, and related connections. Design wind pressures shall be applied perpendicular to the tributary area of the component, cladding, or connection under consideration.

Figure B1
Basic Gust Wind Speed (Gust), MPH

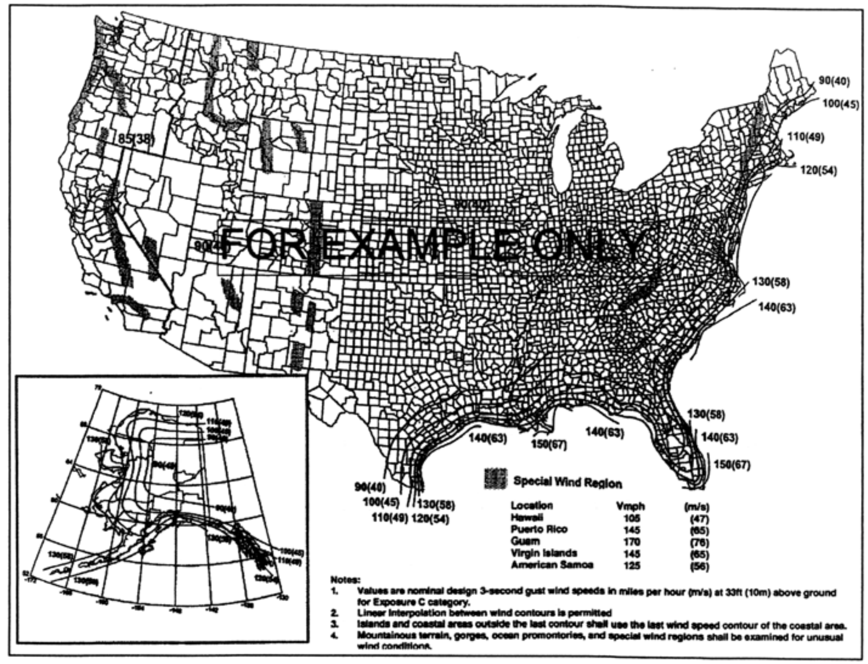


Table B1
Classification of Building Enclosure Condition

Partially-Enclosed Building	Enclosed Building
Buildings meeting one of the following: • All buildings with intentional openings in the exterior envelop exceeding the lesser of 4 ft^2 or 1 percent of the total projected wall or roof area on any building side, or • Buildings within the *wind-borne debris region* with conventional exterior glazing (unprotected from debris impact) exceeding the above opening amounts	All buildings not classified as 'partially enclosed' including: • Buildings not within the *wind-borne debris region*, and • Buildings within the *wind-borne debris region* with glazing protection or impact resistant glazing in accordance with ASCE 7 or the local governing building code.

Table B1 Notes:

1. Building enclosure condition affects internal pressures experienced within the building. Because internal pressure acts inward or outward on all exterior building surfaces simultaneously, the net effect on lateral building loads is zero. Therefore, building enclosure condition does not affect determination of lateral building loads in Section A4.1.
2. Open buildings are not addressed; refer to ASCE 7 for appropriate wind loads. Open buildings have openings in each wall which exceed 80 percent of the wall area.

TABLE B2
Lateral Wind Loads for Application to Vertical Projected Wall and Roof Area [Exposure B, Mean Roof Height 30 feet]

Basic Wind Speed (mph)	Design Wind Pressure (psf)					
	For Roof VPA by Roof Slope			For Wall VPA by Roof Slope		
	≤ 20° (4:12)	25° (5.6:12)	≥ 30° (7:12)	≤10° (2:12)	20° (4:12)	≥30° (7:12)
85	0	2.4	7.7	10.2	12.5	11.2
90	0	2.7	8.6	11.4	14.0	12.6
100	0	3.3	10.7	14.0	17.3	15.5
110	0	4.0	12.9	17.0	20.9	18.8
120	0	4.8	15.4	20.2	25.3	22.4
130	0	5.6	18.0	23.7	29.2	26.3
140	0	6.5	20.9	27.5	33.6	30.5
150	0	7.4	24.0	31.6	38.9	35.0

Table B2 Notes:

1. Table applies to wind exposure category B (urban, suburban, or wooded terrain). For exposure category C (open or coastal exposure), multiply table values by 1.4.
2. Table applies to a mean roof height of 30 feet. For other mean roof heights from 15 feet to 40 feet, multiply table values by the following factor: $f_h = 0.0087 (h) + 0.74$ where h is the mean roof height in feet.
3. Interpolation between reported wind speeds and roof slopes shall be permitted. For roof slopes greater than 45° (12:12), use wall VPA value.
4. Extrapolation to wind speeds other than shown shall be permitted by multiplying tabulated values by the ratio of squared wind speeds. For example, a wall VPA pressure of 20.9 psf at 110 mph from the table can be used to determine a pressure for a 170 mph wind speed by multiplying as follows: $(20.9 \text{ psf}) \times (170/110)^2 = 49.9$ psf.

TABLE B3
Wind Uplift Loads for Application to Roof System Horizontal Projected Area [Exposure B, Mean Roof Height 30 feet]

Basic Wind Speed (mph)	Roof Uplift Pressure (psf) by Building Enclosure Condition and Roof Slope				Overhang Uplift Pressure (psf) by Roof Slope	
	Partially-Enclosed		Enclosed			
	≤20° (4:12)	≥25° (5.6:12)	≤20° (4:12)	≥25° (5.6:12)	≤20° (4:12)	≥25° (5.6:12)
85	17	11	13	8	19	12
90	19	13	14	9	21	13
100	23	16	17	11	26	17
110	28	19	21	13	32	20
120	33	23	25	16	38	24
130	39	27	29	18	45	28
140	45	31	34	21	52	32
150	52	35	39	24	60	37

Table B3 Notes:

1. Table applies to wind exposure category B (urban, suburban, or wooded terrain). For exposure category C (open or coastal exposure), multiply table values by 1.4.
2. Table applies to a mean roof height of 30 feet. For other mean roof heights from 15 feet to 40 feet, multiply table values by the following factor: $f_h = 0.0087 (h) + 0.74$ where h is the mean roof height in feet.
3. For hip roofs, multiply roof uplift pressure by 0.9 for roof slope less than 25° (5.6:12) and 0.8 for roof slope greater than 25° (5.6:12). This adjustment does not apply to overhangs on hip roofs.
4. Apply roof uplift pressure to horizontal projected area bounded by exterior walls. Apply overhang uplift pressure to horizontal projected area of overhangs projecting outward from exterior walls.
5. Interpolation for roof slopes between 20° (4:12) and 25° (5.6:12) and reported wind speeds shall be permitted.
6. Extrapolation of tabulated pressures to wind speeds other than shown shall be done in accordance with note 4 of Table A2.

TABLE B4
Design Wind Pressures (psf)
For Components and Cladding
[Enclosed Building, Exposure B, Mean Roof Height 30 feet]

Component	85 mph		90 mph		100 mph		110 mph	
	Inward	Outward	Inward	Outward	Inward	Outward	Inward	Outward
Roof Components								
Roof Framing	12	-12	13	-13	17	-17	20	-20
Roof Sheathing, Purlins, etc.	12	-18	13	-20	17	-25	20	-30
Roof Skylights, Glazing, etc.	12	-18	13	-20	17	-25	20	-30
Roofing (non-air permeable)[6]	12	-21	13	-23	17	-29	20	-35
Roof Overhang (framing only)[7]	12	-24	13	-27	17	-33	n/a	-40
Wall Components								
Wall Framing	12	-12	13	-13	17	-17	20	-20
Wall Sheathing (panels, boards, girts)	12	-18	13	-20	17	-25	20	-30
Windows, Doors, Glazing	12	-18	13	-20	17	-25	20	-30
Garage Doors	12	-12	13	-13	17	-17	20	-20
Siding (non-air permeable)[6]	12	-18	13	-20	17	-25	20	-30

Component	120 mph		130 mph		140 mph		150 mph	
	Inward	Outward	Inward	Outward	Inward	Outward	Inward	Outward
Roof Components								
Roof Framing	24	-24	27	-27	32	-32	37	-37
Roof Sheathing, Purlins, etc.	24	-36	27	-42	32	-49	37	-56
Roof Skylights, Glazing, etc.	24	-36	27	-42	32	-49	37	-56
Roofing (non-air permeable)[5]	24	-42	27	-49	32	-57	37	-65
Roof Overhang (framing only)[6]	n/a	-48	n/a	-56	n/a	-65	n/a	-74
Wall Components								
Wall Framing	24	-24	27	-27	32	-32	37	-37
Wall Sheathing (panels, boards, girts)	24	-36	27	-42	32	-49	37	-56
Windows, Doors, Glazing	24	-36	27	-42	32	-49	37	-56
Garage Doors	24	-24	27	-27	32	-32	37	-37
Siding (non-air permeable)[5]	24	-36	27	-42	32	-49	37	-56

Table B4 Notes:

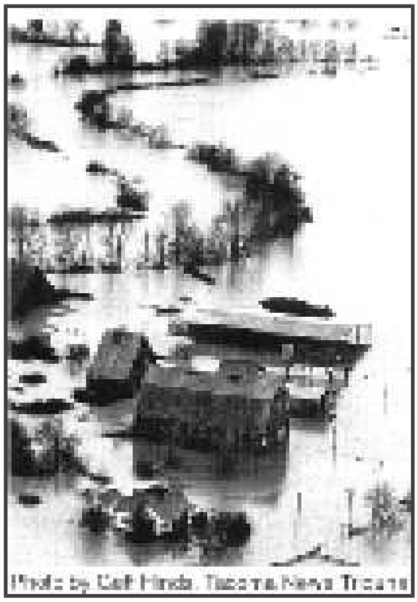
1. Table applies to wind exposure category B (urban, suburban, or wooded terrain). For exposure category C (open or coastal exposure), multiply table values by 1.4.
2. Table applies to enclosed buildings. For partially-enclosed buildings, multiply table values by 1.25.
3. Table applies to a mean roof height of 30 feet or less. For mean roof heights greater than 30 feet and not exceeding 40 feet, multiply table values by the following factor: $f_h = 0.0087\ (h) + 0.74$ where h is the mean roof height in feet.
4. Interpolation between reported wind speeds shall be permitted. Extrapolation of tabulated pressures to wind speeds other than shown shall be done in accordance with note 4 of Table A2.
5. Non-air permeable claddings (siding and roofing) do not allow venting of air either through the siding or through cavities behind the cladding that lead to vent openings on the same face of the building. Most claddings are air-permeable to some degree and provide some reduction in wind load, provided the supporting wall behind the cladding is relatively non-air permeable. For vinyl cladding, ASTM Standard D3679 permits a 50 percent reduction in wind load for this reason. Similarly, claddings such as brick veneer (with weeps and vent space) and hardboard lap siding have been reported to experience cladding wind load reductions of 30 percent or more. Wind loads on roofing, such as asphalt shingles, have been reported to experience wind load reductions of as much as 25 percent. Refer to the cladding manufacturer for an appropriate air-permeable cladding reduction factor to use. Consideration should also be given to the dynamic nature of wind pressures (e.g., fluttering) and its potential effect (e.g., fatigue) on some cladding systems and related connections.
6. Roof overhang pressure includes pressure from underside of the overhang as well as on the upper surface. If an "open soffit" is used, the roof overhang pressure should also apply to the roof sheathing (if sheathed) or the roofing (if not sheathed underneath).

References: *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-02, ASCE, Reston, VA. 2002.

Structural Loads for One- and Two-Family Dwellings, U.S. Dept. of Housing and Urban Development, Washington, DC. 2001.

Fact Sheet 229-96

The "1OO-Year Flood"



(Larger Version, 149K GIF)

Photo by Geff Hinds, Tacoma News Tribune

Flood designations are based on statistical averages, *not* on the number of years between big floods.

The estimates are only as good as the available data. Flood designations are updated as more data are collected or when the conditions change in a river basin.

BIG FLOODS COULD HAPPEN AGAIN IN WASHINGTON DURING ANY YEAR

Rivers across the Nation seem to be rising to record flood levels almost every year. In Washington, more than one 100-year flood has happened on a few rivers in just the past several years. How can 100-year floods happen so often?

WHY DON'T THESE FLOODS HAPPEN EVERY 100 YEARS?

The term "100-year flood" is misleading because it leads people to believe that it happens only once every 100 years. The truth is that an uncommonly big flood can happen any year. The term "100-year flood" is really a statistical designation, and there is a **1-in-100 chance** that a flood this size will happen during any year. Perhaps a better term would be the "1-in-100 chance flood."

The actual number of years between floods of any given size varies a lot. Big floods happen irregularly because the climate naturally varies over many years. We sometimes get big floods in successive or nearly successive years with several very wet years in a row.

HOW ARE FLOODS DESIGNATED?

Scientists collect data and study past floods to get a minimum of 10 years of information about the river; a longer record provides a better estimate of the "1-in-100 chance flood." Scientists use statistics and observe how frequently different sizes of floods occurred, and the average number of years between them, to determine the probability that a flood of any given size will be equalled or exceeded during any year.

MANY FLOOD DESIGNATIONS WILL CHANGE OVER TIME

As more data are collected, or when a river basin is altered in a way that affects the flow of water in the river, scientists re-evaluate the frequency of flooding. Dams and urban development are examples of some man-made changes in a basin that affect floods.

THE USGS COLLECTS ESSENTIAL DATA FOR UNDERSTANDING FLOODS

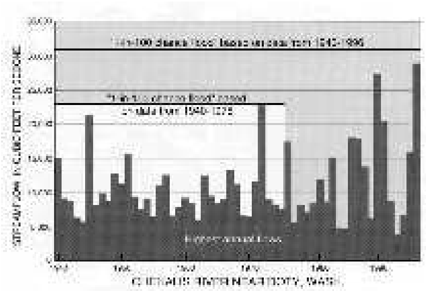
Scientists at the USGS measure streamflow in rivers across the State during every major flood. After flood waters recede, the USGS may be funded to locate and survey "high-water marks" where debris and mud lines indicate the highest extent of flood waters. These post-flood surveys are used to estimate maximum flows at sites that could not be reached during the floods and also to map the areas covered by the floods.

Streamflow data that have been collected since 1975 on the Chehalis River near Doty indicate that the estimated streamflow of "1-in-100 chance flood" is higher than it was 20 years ago.

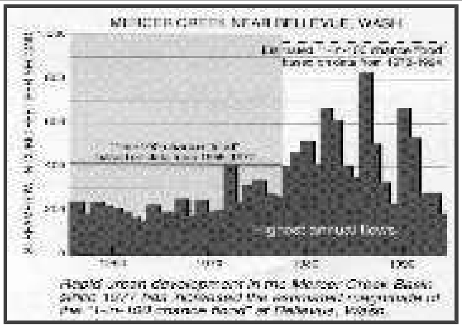
The eariler flood designation was accurate on the basis of the data that were available at the time; more large floods happened after 1975 than from 1940-1975.

The change in the flood designation after 20 years of additional data collection highlights the importance of continued river monitoring.

Annual peak flow data for 1995 and 1996 are provisional and may change.



(Larger Version, 182K GIF)



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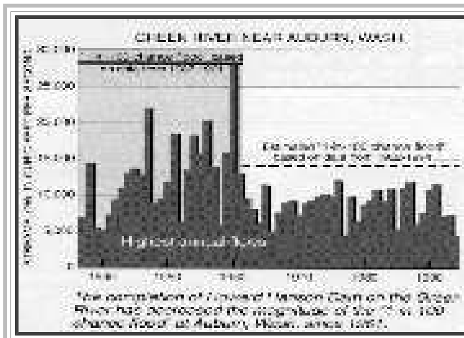
Rapid urban development in the Mercer Creek Basin since 1977 has increased the estimated magnitude of the "1-in-100 chance flood" at Bellevue, Wash.



(Larger Version, 182K GIF)

The completion of Howard Hanson Dam on the Green River has decreased the magnitude of the "1-in-100 chance flood" at Auburn, Wash. since 1961.

DO YOU LIVE ON THE FLOODPLAIN?

The areas affected by past floods have been mapped by the Federal Emergency Management Agency and many other government agencies. Because of continuing changes in river channels and land use in many basins, the maps may not reflect current information for your area. Inquire at your City or County Building or Planning Department.

If you live on the designated floodplain, the chances are about 1 in 2 that you will experience a flood during your lifetime. Prepare for a flood as you would for any natural disaster, and make evacuation plans for your family.

FLOODS WILL CONTINUE TO HAPPEN

Although we can lessen effects of some floods, they are part of the natural cycle of every river and benefit instream habitats by moving material downstream and renewing streambeds. As floods get bigger and spread farther, flood waters slow and deposit sediment on the floodplain. This natural process created valuable farmlands in river valleys of the Pacific Northwest over thousands of years.

Glossary of Flood Terms

A **flood** is any relatively high streamflow that overtops the natural or artificial banks of a river.

Discharge is another term for streamflow; it is the measured volume of water that moves past a point in the river in a given amount of time. Discharge is usually expressed in cubic feet per second.

The **floodplain** is the relatively flat lowland that borders a river, usually dry but subject to flooding. Floodplain soils actually are former flood deposits.

The *average* number of years between floods of a certain size is the **recurrence interval** or **return period.** The *actual* number of years between floods of any given size varies a lot because of the naturally changing climate.

A **hydrograph** is a graph that shows changes in discharge or river stage over time. The time scale may be in minutes, hours, days, months, One **cubic foot per second** (cfs) is about 450 gallons per minute. The average discharge of the Columbia ~ River in September at The Dalles, Oregon, is about 120,000 cfs, which would fill the Seattle Kingdome in less than 10 minutes. The average discharge of the Puyallup River in September is about 1,700 cfs at Puyallup, Wash.

years, or decades.

The **river stage** is the height of the water in the river, measured relative to an arbitrary fixed point.

--*Karen Dinicola*

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from U.S. Department of the Interior, U.S. Geological Survey, Fact Sheet FS-229-96

For more information contact any of the following:

The U.S. Geological Survey has served the public and Federal, State, and local goverments since 1879 by collecting, analyzing, and publishing detailed information about the Nation's mineral, land, and water resources. The USGS has been studying the water resources of Washington State since the turn of the century. This information is in a variety of map, book, electronic, and other formats and is available by contacting:

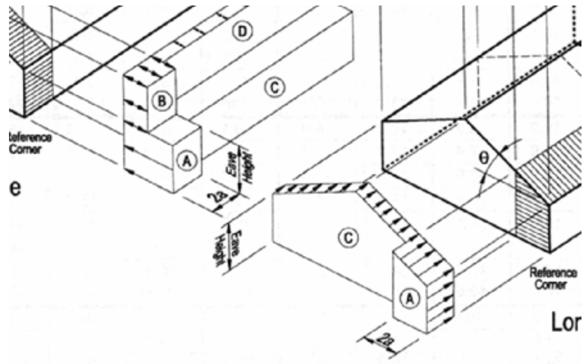
U.S. Geological Survey
1201 Pacific Ave., #600
Tacoma, WA 98402
(253) 428-3600
Fax: (253) 428-3614
Email:dc_wa@usgs.gov

Selected data and interpretive reports are available on the USGS Washington "home page" on the World Wide Web at http://wa.water.usgs.gov/

Examples:
Wind Loading

Example 1

Given the structure with three shear walls and rigid roof diaphragm, determine the horizontal shear distributed to the walls (and piers) with a static wind pressure and the overturning moment on each wall. The basic wind speed for the College Station area is within 90-100 mph from ASCE-7. (Use 100 mph)



Wind Pressure:
Flat roof (0°)
Zone A: 10% of 5 m = 0.5 m
Zone C (~10 ft height)
for simplicity use C
p_{s30} = 10.5 psf
x 0.0479 kN/m^2/psf
= 0.5 kN/m^2 (KPa)

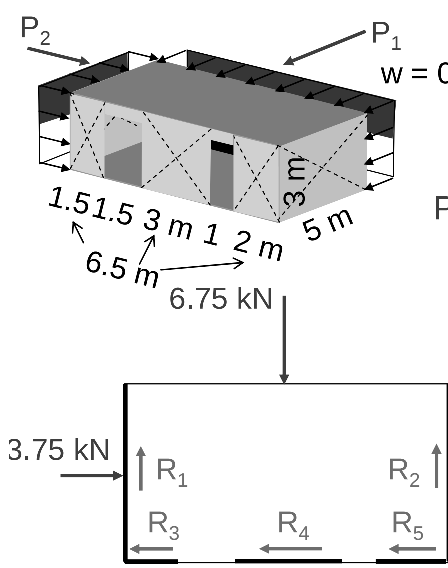
Simplified Design Wind Pressure , p_{s30} (psf) *(Expo*

Basic Wind Speed (mph)	Roof Angle (degrees)	Load Case	Zones: Horizontal Pressures A	B	C	D	Ver E	
100	0 to 5°	1	15.9	-8.2	10.5	-4.9	-19.1	-1
	10°	1	17.9	-7.4	11.9	-4.3	-19.1	-1
	15°	1	19.9	-6.6	13.3	-3.8	-19.1	-1
	20°	1	22.0	-5.8	14.6	-3.2	-19.1	-1
	25°	1	19.9	3.2	14.4	3.3	-8.8	-1
		2	-------	-------	-------	-------	-3.4	-
	30 to 45	1	17.8	12.2	14.2	9.8	1.4	-1
		2	17.8	12.2	14.2	9.8	6.9	-

SOLUTION:
The wind pressure needs to be turned into a static force by multiplying by the tributary area, which is half way from other "support" to "support" or top of parapet:

The force along the wide length, P_1, can be evenly resisted (split) by the end shear walls because they are the same size and stiffness.

In the long direction, the force P_2 must be resisted by the piers on one side only. The force should be distributed to each pier based on their stiffnesses (a function of h/L), but the calculation is laborious. This example splits the force proportionally by length.



w = 0.5 kPa

$$P_1 = w \cdot \frac{h}{2} \cdot l = 0.5 \cdot 1.5 \cdot 9 = 6.75\,\text{kN}$$

$$P_2 = w \cdot \frac{h}{2} \cdot d = 0.5 \cdot 1.5 \cdot 5 = 3.75\,\text{kN}$$

$$R_1 = R_2 = {}^{6.75}\!/_{2} = 3.38\,\text{kN}$$

symmetry

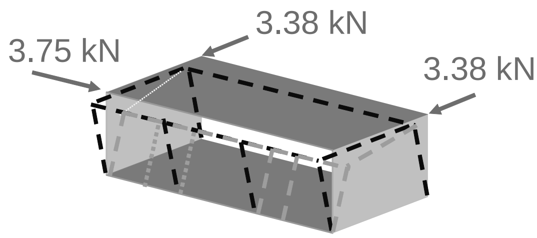
$$R_3 = 3.75\,\text{kN} \cdot {}^{1.5}\!/_{6.5} = 0.86\text{kN}$$

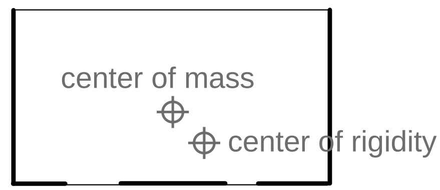
$$R_4 = 3.75\,\text{kN} \cdot {}^{3}\!/_{6.5} = 1.73\text{kN}$$

$$R_3 = 3.75\,\text{kN} \cdot {}^{2}\!/_{6.5} = 1.15\text{kN}$$

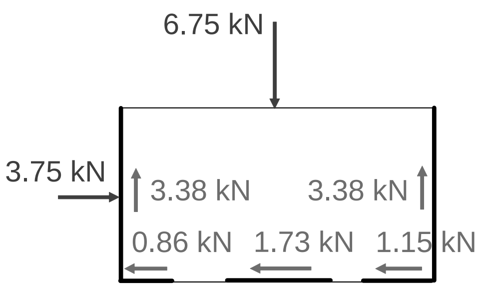
simplified stiffness

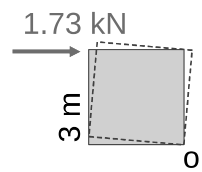
Would this want to twist? A torsional moment will result if the **center of rigidity**, which is the resulting location of the moments of the wall rigidities, does not coincide with the **center of mass** determined from the moments of the wall weight. There is, in effect, an eccentricity.



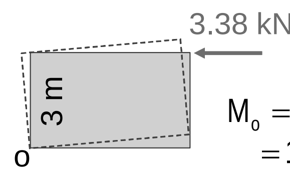


The overturning moments from the lateral forces at the top of the walls and piers about their bases (or toe) can be calculated.





$$M_o = 1.73\text{kN}\cdot 3\text{m} = 5.19\text{kN}\cdot\text{m}$$



$$M_o = 3.38\text{kN}\cdot 3\text{m} = 10.14\text{kN}\cdot\text{m}$$

Example 2

EXAMPLE 9.7 Header Acting as a Chord

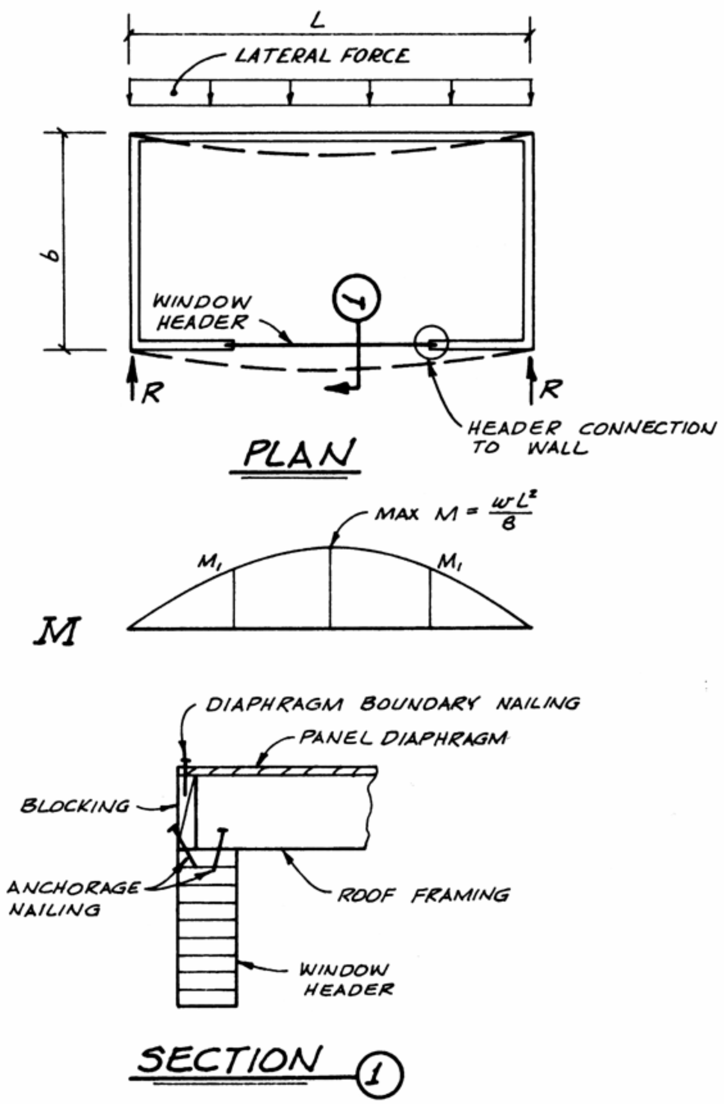


Figure 9.9 The header over an opening in a wall may be used as horizontal diaphragm chord.

Over the window the header serves as the chord. It must be capable of resisting the maximum chord force in addition to gravity loads. The maximum chord force is

$$T = C = \frac{\text{max. } M}{\text{b}}$$

The connection of the header to the wall must be designed for the chord force at that point:

$$T_1 = C_1 = \frac{M_1}{b}$$

NOTE: For simplicity, the examples in this book determine the chord forces using the dimension b as the width of the building. Theoretically b is the dimension between the centroids of the diaphragm chords, and the designer may choose to use this smaller, more conservative dimension.

Reducing Earthquake Losses Throughout the United States

Building Safer Structures

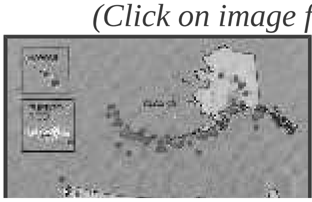
In this century, major earthquakes in the United States have damaged or destroyed numerous buildings, bridges, and other structures. By monitoring how structures respond to earthquakes and applying the knowledge gained, scientists and engineers are improving the ability of structures to survive major earthquakes. Many lives and millions of dollars have already been saved by this ongoing research.

(Click on image for a full size version - 216K)

The Transamerica Pyramid in San Francisco, built to withstand earthquakes, swayed more than 1 foot but was not damaged in the 1989 Loma Prieta, California, earthquake.

On October 17, 1989, the magnitude 7.1 Loma Prieta earthquake struck the Santa Cruz Mountains in central California. Sixty miles away, in downtown San Francisco, the occupants of the Transamerica Pyramid were unnerved as the 49-story office building shook for more than a minute. U.S. Geological Survey (USGS) instruments, installed years earlier, showed that the top floor swayed more than 1 foot from side to side. However, no one was seriously injured, and the Transamerica Pyramid was not damaged. This famous San Francisco landmark had been designed to withstand even greater earthquake stresses, and that design worked as planned during the earthquake.

(Click on image for a full size version - 128K)



Earthquakes are a widespread hazard in the United States. Colors show magnitudes of historical earthquakes: red, 7 or greater; orange, 5.5 to 7; yellow, 4.5 to 5.5. The U.S. Geological Survey operates instruments in many structures in the seismically active areas shown. These instruments measure *how structures respond to earthquake shaking.*

Designing and building large structures is always a challenge, and that challenge is compounded when they are built in earthquake-prone areas. More than 60 deaths and about $ 6 billion in property damage resulted from the Loma Prieta earthquake. As earth scientists learn more about ground motion during earthquakes and structural engineers use this information to design stronger buildings, such loss of life and property can be reduced.

To design structures that can withstand earthquakes, engineers must understand the stresses caused by shaking. To this end, scientists and engineers place instruments in structures and nearby on the ground to measure how the structures respond during an earthquake to the motion of the ground beneath. Every time a strong earthquake occurs, the new information gathered enables engineers to refine and improve structural designs and building codes. In 1984 the magnitude 6.2 Morgan Hill, California, earthquake shook the West Valley College campus, 20 miles away. Instruments in the college gymnasium showed that its roof was so flexible that in a stronger or closer earthquake the building might be severely damaged, threatening the safety of occupants. At that time, these flexible roof designs were permitted by the Uniform Building Code (a set of standards used in many states). Many industrial facilities nationwide were built with such roofs.

(Click on image for a full size version - 82K)

Seismic records (upper right) obtained during the 1984 Morgan Hill, California, earthquake led to an improvement in the Uniform Building Code (a set of standards used in many states). The center of the gym roof shook sideways three to four times as much as the edges. The Code has since been revised to reduce the flexibility of such large-span roof systems and thereby improve their seismic resistance.

Building codes provide the first line of defense against future earthquake damage and help to ensure public safety. Records of building response to earthquakes, especially those from structures that failed or were damaged, have led to many revisions and improvements in building codes. In 1991, as a direct result of what was learned about the West Valley College gymnasium roof, the Uniform Building Code was revised. It now recommends that such roofs be made less flexible and therefore better able to withstand large nearby earthquakes.

Earth scientists began recording earthquakes about 1880, but it was not until the 1940's that instruments were installed in buildings to measure their response to earthquakes. The number of instruments installed in strucures increased in the 1950's and 1960's. The first abundant data on the response of structures came from the devastating 1971 San Fernando, California, earthquake, which yielded several dozen records. These records were primitive by today's standards. The first records from instruments sophisticated enough to measure twisting of a building were obtained during the 1979 Imperial Valley, California, earthquake.

Today there are instruments installed in hospitals, bridges, dams, aqueducts, and other structures throughout the earthquake-prone areas of the United States, including Illinois, South Carolina, New York, Tennessee, Idaho, California, Washington, Alaska, and Hawaii. Both the California Division of Mines and Geology (CDMG) and the USGS operate instruments in California. The USGS also operates instruments in the other seismically active regions of the nation.



(Click on images for full size versions - 192K, 238K, 98K, 114K)

USGS scientists have installed instruments in a variety of structures across the United States to monitor their behavior during earthquakes. Examples shown include a dam, a bridge supporting a large aqueduct, a highway overpass, and a Veterans hospital.

The majority of deaths and injuries from earthquakes are caused by the damage or collapse of buildings and other structures. These losses can be reduced through documenting and understanding how structures respond to earthquakes. Gaining such knowledge requires a long-term commitment because large devastating earthquakes occur at irregular and often long intervals. Recording instruments must be in place and waiting, ready to capture the response to the next temblor whenever it occurs. The new information acquired by these instruments can then be used to better design earthquake-resistant structures. In this way, earth scientists and engineers help reduce loss of life and property in future earthquakes.

Mehmet Celebi, Robert A. Page, and Linda Seekins

COOPERATING AGENCIES, COMPANIES, AND INSTITUTIONS
California Department of Transportation
California Division of Mines and Geology
City of Los Angeles
General Services Administration
Metropolitan Water District of Southern California
Oregon Department of Highways
U.S. Army Corps of Engineers
U.S. Department of Energy
U.S. Department of Veterans Affairs
Washington Department of Highways
Washington Department of Natural Resources
Private building owners

For more information contact:
Earthquake Information Hotline (415) 329-4085
U.S. Geological Survey, MS 977
345 Middlefield Road, Menlo Park, CA 94025
USGS Menlo Park Earthquakes Home Page

U.S. Geological Survey Fact Sheet-167-95 1995

Web page by Will Prescott - 1996 April 9

Buildings at Risk: Seismic Design Basics for Practicing Architects, AIA, 1994

Chapter 1: The Nature of Ground Motion and its Effect on Buildings

GEOLOGIC BACKGROUND

According to the now generally accepted theory of Plate Tectonics, the earth's crust is divided into several major plates, some 50 miles (80km) thick, that move slowly and continuously over the interior of the earth.

Earthquakes are initiated when, due to slowly accumulating pressure, the ground slips abruptly along a geological fault plane on or near a plate boundary. The resulting waves of vibration within the earth create ground motion at the surface which begins to vibrate in a very complex manner. This, in turn, induces forces within buildings that are determined by the precise nature of the ground motion and the construction characteristics of the building.

The point where the fault first slips is termed the "focus" or "hypocenter." A theoretical point on the earth's surface directly above the focus is termed the "epicenter." (Figure 1.1) The epicenter for the January 17, 1994 Los Angeles earthquake was located in the city of Northridge in the San Fernando Valley.

The initial break in the fault moves rapidly along the line of the fault, and the distance of this movement largely determines the intensity of ground shaking. Thus the 1906 San Francisco earthquake ruptured along some 250 miles (400km) of the San Andreas fault. The Loma Prieta, California earthquake of 1989 was unusual since no surface faulting occured, although a broad area of ground cracking indicated a wide distribution of strain. The fault rupture moved upward to within about 6km of the ground surface area and then spread approximately 20km along the fault to each side of the initial rupture. (Figure 1.2)

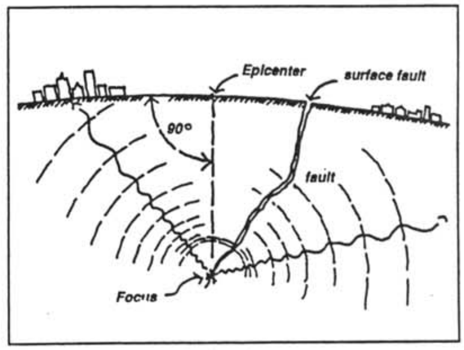


Figure 1.1: Earthquake location

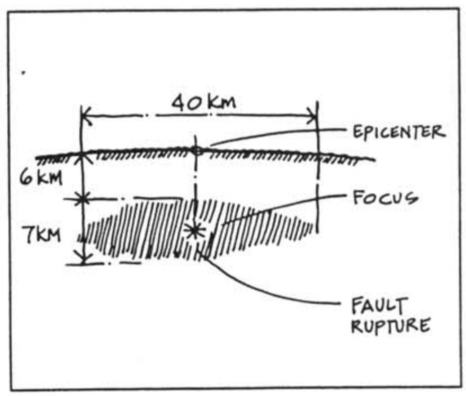


Figure 1.2: The Loma Prieta fault rupture, 1989

GROUND FAILURE

Surface Faulting

Slippage along a fault line deep in the earth's surface may eventually result in *surface faulting*, the crack or split on the earth's surface that provides the layperson's vision of earthquakes. Surface faulting may result in large earth movements: in the 1992 Landers earthquake east of Los Angeles, the earth offset as much as 18 feet at the surface. A building located across a surface fault, no matter how well designed, is almost certain to suffer very severe damage. (Figure 1.3) However, the large disturbance of the ground near a fault is generally quite narrow in width on either side of the fault: in Landers the maximum width of severely disturbed ground was only about 40 meters. Moreover, the probability that buildings will straddle a surface fault is very low

James Stratta

Figure 1.3: School straddling a landslide-induced rupture, Alaska

National Science Foundation

Figure 1.4: House on Turnagain slide

Unknown

Figure 1.5: Turnagain Heights, Alaska

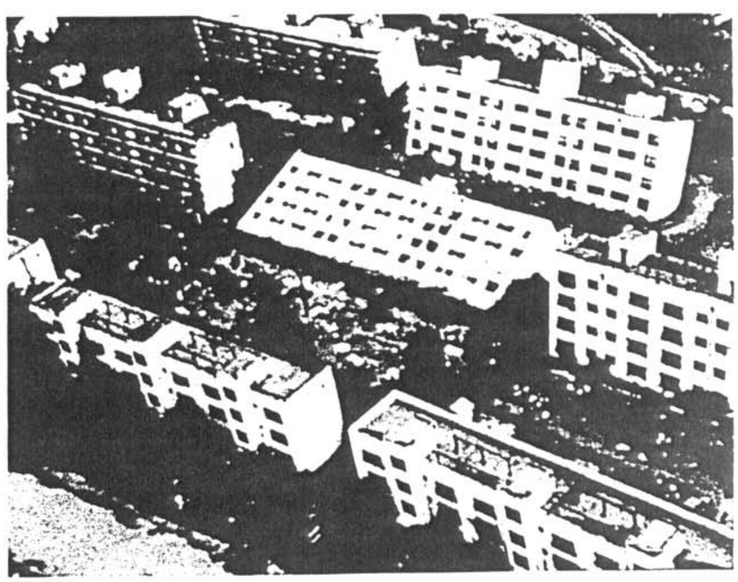
compared to the likelihood of significant ground motion. So, in seismic design, we design against the vibrations caused by fault slippage and try to ensure that buildings are not built over fault zones.

Landslides, Liquefaction and Subsidence

The energy released by an earthquake can also trigger ground failure in the form of landslides, liquefaction and subsidence which can have devastating effects on a structure. Even well-built structures, designed to withstand earthquake forces, if built on an unstable site or in the path of a landslide, can fall victim.

The Alaskan earthquake of 1964 provides examples of structures with the inherent strength to withstand ground shaking that were devastated as a result of the instability of the sites they were built on. (Figure 1.4) While an architect and contractor could take pride in the performance of their buildings on Turnagain Heights or on 4th Street in Anchorage, the decision to build on geologically unstable sites produced catastrophic results. (Figure 1.5) Avoidance of sites with a potential for liquefaction, landslides or subsidence represents the best design approach.

Ground shaking can also trigger subsidence and liquefaction in soils that are unconsolidated and/or saturated with water. When sandy, water saturated soils are shaken, the bearing capacity of the soil is reduced as the soil liquefies and flows laterally and vertically. Liquefied soils can produce volcano-like sandboils at the ground surface or flow laterally if the soil is not contained. The ground surface and structures built on shallow foundations can subside several feet or be torn apart as spreading occurs. Dramatic examples of liquefaction from recent earthquakes illustrate again, that even well built structures are vulnerable if adequate attention is not paid to site conditions and foundation design. (Figures 1.6 and 1.7)



H. B. Seed

Figure 1.6: Liquefaction, Niigata, Japan, 1964

GROUND MOTION

While ground failure can be an important consequence of any earthquake, the primary effect buildings are designed to resist is ground motion. During an earthquake, wave vibrations emanate from the line of fault rupture and so approach the building from a given direction. The waves begin like ripples in a still pond when a pebble is thrown into it, but the seismic waves rapidly become more complex.

There are four main wave types, of which "body" waves, within the earth, are the most important for design purposes. (Figure 1.8) First to arrive at the surface is the *P* or *primary* wave. In this wave the ground is successively pushed and pulled along the wave front. The effect is of a sharp punch - it feels as if a truck has hit the building. The P wave is followed by the *S, secondary* or *shear* wave, which is a lateral motion, back and forth (but sideways to the wave front).

The nature of the waves and their interactions are such that actual movement at the ground will be random: predominantly horizontal, often with considerable directional emphasis, but sometimes with a considerable vertical component. The actual horizontal ground displacement is small, only inches even in a large earthquake, except in the immediate area of the fault rupture where displacements of several feet may occur.

Unknown

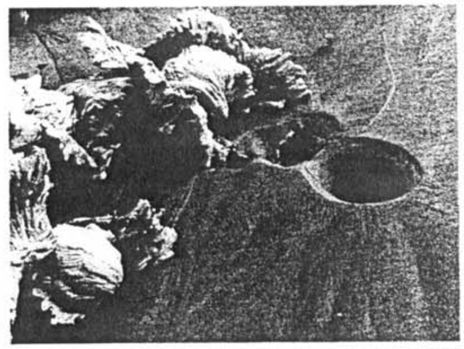


Figure 1.7: Sand boil in a lettuce field, Watsonville, 1989

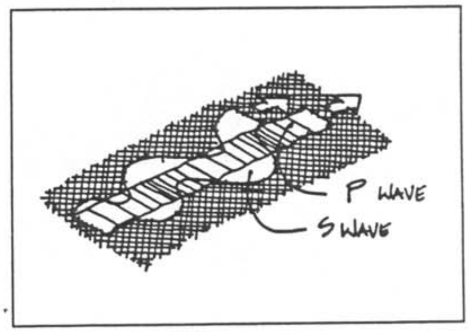
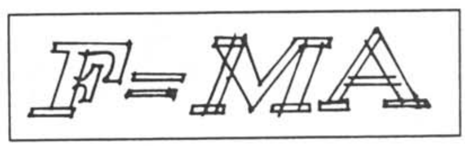


Figure 1.8: "P" and "S" waves



THE MEASUREMENT OF GROUND MOTION

Measurement of ground motion is important for design purposes because it provides the basis for determining forces, and assessing the relative seismic hazard at different locations.

Earthquake motion is recorded by a seismograph, an instrument that records the movement, over time, of a freely supported pendulum within a frame: the instrument may be placed on the ground or within a structure.

In modern seismographs, pendulum movement is converted into electronic signals on tape. Strong-motion seismographs, called accelerometers, are designed to directly record nearby rather than distant ground movement, and they produce a record called an accelerogram. Instruments are normally placed so as to measure movements along the two horizontal axes as well as one vertical. Three measures are of major interest: acceleration, velocity, and displacement.

Acceleration, Velocity, Displacement

Acceleration is the rate of change of velocity: when multiplied by mass it results in the inertial force that the building must resist. This is a key measure, and forms the basis of the estimation of earthquake forces on buildings: *Newton's Second Law of Motion* states in essence, that an *inertial force, F, equals mass (M) multiplied by the acceleration (A).*

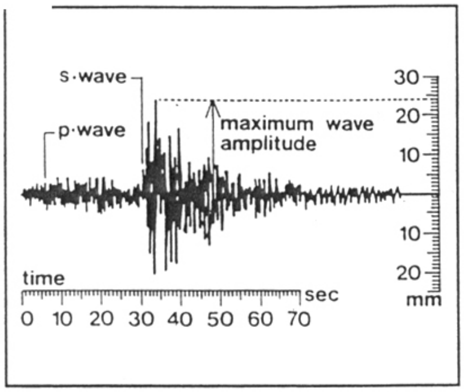


Figure 1.9: This accelerogram illustrates the size of the seismic waves and can be used to derive acceleration, velocity and displacement.

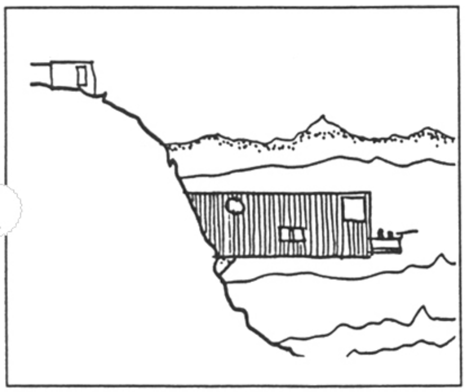


Figure 1.10: A 1.0g design

Acceleration is commonly measured in *"g's"* - the acceleration of a free falling body due to the earth's gravity (approx. 32ft/sec/sec., or 980 cm/sec/sec, or 1.0g.). *Velocity*, measured in inches or centimeters per second, refers to the rate of ground motion at any time. *Displacement*, measured in inches or centimeters, refers to the distance a particle is removed from its "at rest" position. (Figure 1.9)

The level of acceleration generally taken as sufficient to produce some damage to weak construction is 0.10g. The lower limit of acceleration perceptible to people is set by observation and experiment at approximately 0.001g or 1cm/sec^2; at around 0.20g and above most people will have difficulty keeping their footing and sickness symptoms may be induced. An earthquake causing acceleration approaching 0.5g on the ground is very high. On upper floors of buildings, maximum accelerations will often be higher, depending on the degree to which the mass and form of the building act to damp the vibratory effects. A figure of 1.00g, or 100% of gravity, may be reached, for a fraction of a second. To design for 1.00g is diagrammatically equivalent, in a static sense, to designing a building that projects horizontally from a vertical surface. (Figure 1.10) When the behavior of real buildings is observed, several factors modify this diagrammatic equivalence, and structures that could never cantilever from a vertical surface can briefly withstand 1.0g earthquake shaking.

Acceleration is the measure commonly used to indicate the possible destructive power of an earthquake in relation to a building. A more significant measure is that of acceleration combined with *duration*, which takes into account the impact of earthquake forces over time. In general, a number of cycles of moderate acceleration, sustained over time, can be much more difficult for a building to withstand than a single peak of much higher value. Seismic instrumentation also measures the duration of strong ground motion, which generally relates to the length of the fault break.

Typically the extreme vibration will occupy only a few seconds; both the 1989 Loma Prieta and 1994 Northridge earthquakes lasted only a little over ten seconds, yet they caused much destruction. In 1906, in San Francisco, the severe shaking lasted about 45 seconds; in Alaska in 1964 the severe earthquake motion lasted for over 3 minutes.

Two earthquake measurement systems are in common use and neither, for various reasons, is really satisfactory from the building design viewpoint.

Magnitude: The Size of the Wave

Earthquake *magnitude* is the first measure: it is expressed as Richter magnitude based on the scale devised by Professor Charles Richter of the California Institute of Technology in 1935. Richter's scale is based on the *maximum* amplitude of certain seismic waves recorded on a standard seismograph at a distance of 100 kilometers from the earthquake epicenter. The scale, however, tells nothing about duration, which may be of great significance in causing damage, nor does it tell anything about frequency content which, in its relationship to the building period, as discussed later, is also of great significance in determining damage. Because the instrument is unlikely to be exactly 100km from the source, Richter developed a method to allow for the diminishing of wave amplitude (or *"attenuation"*) with increased distance, just as the light of a star appears dimmer with distance. (Figure 1.11)

Because the size of earthquakes varies enormously, the graphic range of wave amplitude measured on seismographs is compressed by using, as a scale, the *logarithm to base ten* of the recorded wave amplitude. Hence, each unit of Richter magnitude indicates a *10* times increase in wave amplitude. But the *energy increase* represented by each unit of scale is estimated by seismologists as approximately *31* times. Since Richter magnitude is a measured quantity, the scale is open-ended, but seismologists believe that a Richter magnitude of about 9 represents the largest possible earthquake. A given earthquake will have only one Richter magnitude, though differences in recording result in some argument as to what this will be.*

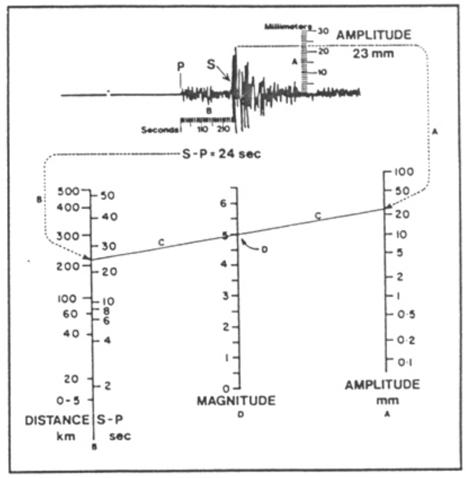


Figure 1.11: Richter magnitude

Figure 1.12: Damage to an older masonry building

Intensity: The Amount of Damage

To provide information directly related to local shaking and building damage, *intensity* scales are used. These scales are based on subjective observation of the effects of the earthquake on buildings, ground and people. In the United States the most commonly used scale is the *Modified Mercalli (MM)* originally developed in Europe in 1902, and modified in 1931 to fit construction conditions then prevalent in California and other parts of the United States.

As a result the MM scale is somewhat dated, with no references to common modern construction systems. This is not much of a disadvantage because earthquake damage is most likely to be concentrated in older buildings, often of the very type that the scale describes. (Figure 1.12) The MM Scale is a twelve point scale, *I - XII*. The descriptions for MM I are, in abbreviated form, "Not felt. Marginal and long-period effects of large earthquakes." For MM XII the descriptor reads, "Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air." Because earthquake effects vary depending on distance from the epicenter, nature of the ground, and magnitude, an earthquake will have many MM values. The MM scale has been correlated with ground acceleration. For example, MM VII corresponds to a peak acceleration between approximately 0.1g and 0.29g.

THE EFFECTS OF GROUND MOTION

Inertial Forces

While the effects of ground failure can be extremely severe, the most common and widespread cause of earthquake damage is ground shaking. Seismically induced shaking affects buildings in three primary ways: inertial forces, period and resonance, and torsion. Shaking causes damage by internally generated inertial forces generated by vibration of the building's mass.

* *The use of the term Richter Magnitude will eventually be replaced by the use of the terms 'preliminary magnitude' and 'moment magnitude.'*

Chapter 2: Site Issues

SITING OF A STRUCTURE - WHERE DOES THE SITE BEGIN?

From a seismic design standpoint, the "site" is the region within which a structure will be built; and while it is critical that a structure not be built across an active fault trace, it is equally important that siting and design decisions address the potential for increased intensity and duration of ground shaking, accessibility, survival of life lines and potentially hazardous adjacent land uses. Thus, seismic design is not limited to an analysis of the factors within the confines of the site boundary; it extends to a broad environmental analysis of regional and community vulnerability.

SEISMIC RISK AS A SITING CRITERIA

The factors that impact site vulnerability include proximity to active earthquake faults; susceptibility of the site to ground shaking; the potential for ground failure, including subsidence, lateral spreading, liquefaction, and landslides; adjacent structures and land uses that could pose a threat during or after an earthquakes; and, the potential for inundation resulting from tsunami or dam failure.

From a site and urban planning standpoint, however, concern should not be limited to the identification on the site of a fault or potential fault rupture, but to the broader impact of ground shaking and geologic failures that could occur in the region. The failure of the regional transportation network, disruption of power or water supply or the isolation of building as a result of ground failure, can be as devastating to a business as actual structural damage.

Therefore, seismic risks from beyond the building site property line must be considered as design criteria for a structure. These criteria address the relative desirability or risk of an individual site, that is, is one site safer for a particular use than another site; and what factors beyond the site boundary, such as adjacent land uses, geologic stability of adjacent land, or the survivability of lifelines or access, could impact site development?

ACTIVE EARTHQUAKE FAULTS

If a structure is built over an active fault trace, it should be designed to accommodate displacement or fault offset. (Figure 2.1)

This is both a challenging and costly effort, with no guarantee of success. The mapping of active faults has been a focus of geologists and urban planners for

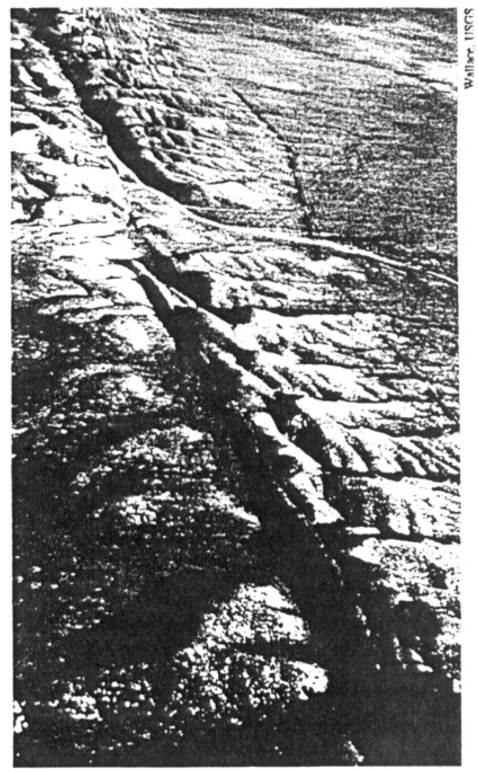


Figure 2.1: San Andreas fault in Central California

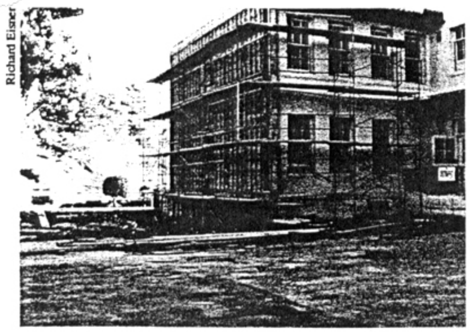


Figure 2.2: The section of Fairmont Hospital in San Leandro, California, built across a fault trace, was removed.

several decades. It has been a critical element increasing our understanding of regional seismicity: the frequency of seismic activity, the magnitude of previous seismic events, and the potential for future seismic activity. The fault maps indicate where active surface faulting is identified and where future offset potential exists. Where identified, designers should provide a setback from identified faults for new construction.

In many areas, development is limited or prohibited within defined zones adjacent to active faults. Programs to map fault zones and limit new construction within established zones have proven successful in reducing earthquake risks to new construction. Unfortunately, earthquake fault traces were often ignored when land was subdivided and developed, presenting a costly dilemma to owners of existing structures in a fault zone.

Where existing structures are built across fault lines, their structural performance, occupancy and continued use should be reviewed to evaluate the risk they pose. Those sections of structures built across a trace can be removed or occupancy types and loads can be reduced to reduce risk exposure. (Figure 2.2)

IMPACT OF REGIONAL GEOLOGY ON SITE PERFORMANCE

The geology of a region plays a significant role in determining the potential for shaking and ground failure damage. In relatively old geological regions, such as the eastern and midwestern United States where weathering and erosion have leveled the terrain and laid deep deposits of unconsolidated soils, violent ground shaking resulting from fault rupture thousands of feet below the earth's surface can extend for thousands of square miles. Deep soils can amplify ground shaking intensity, extend duration of violent shaking and limit attenuation of shaking; resulting in greater damage over a larger area than would result in younger or bedrock regions.

For example, in the central United States, the violent shaking of the New Madrid, Missouri earthquakes of 1811 and 1812 extended across the midwest and was felt as far away as the eastern seaboard. The earthquakes were felt over 2,000,000 square miles! In contrast, the 1906 San Francisco earthquake, estimated to have released 30 times more energy, was felt over only 375,000 square miles. It impacted a much smaller area because the regional geology in California limited propagation and increased attenuation of shaking. In both examples, one without surface manifestations of faulting, and the other with visible surface faulting, regional geology rather than presence of a surface fault determined the extent of potential damage. (Figure 2.3)

While not building across an earthquake fault is certainly a good rule, building adjacent to a fault may not pose as great a risk as one would expect. A number of recent earthquakes have emphasized that regional and local geology and the lack of attenuation of ground shaking are often more important than proximity to the earthquake's epicenter in determining the impact of an earthquake. The 1985 Mexican earthquake occurred on the coast of Mexico between Acapulco and Ixtapa. Damage close to the epicenter in the coastal resort areas was minor.

However, 250 miles away in Mexico City, the damage to midrise concrete structures was severe, resulting in several thousand deaths. Again in 1989, the Loma Prieta earthquake, centered in the Santa Cruz Mountains resulted in the deaths of more than 40 persons on the Cypress Viaduct, 60 miles north of Santa Cruz in Oakland. In both cases, the most violent ground shaking did not occur at the epicenter of the earthquakes, but a significant distance away as a result of the propagation of the ground waves, the geology of the region and local soil conditions. Understanding the regional and local geology can tell the designer a great deal about the relative risk of an individual site.

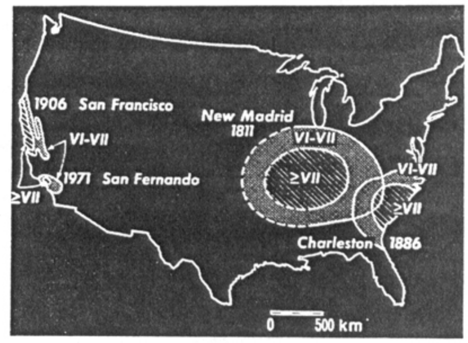


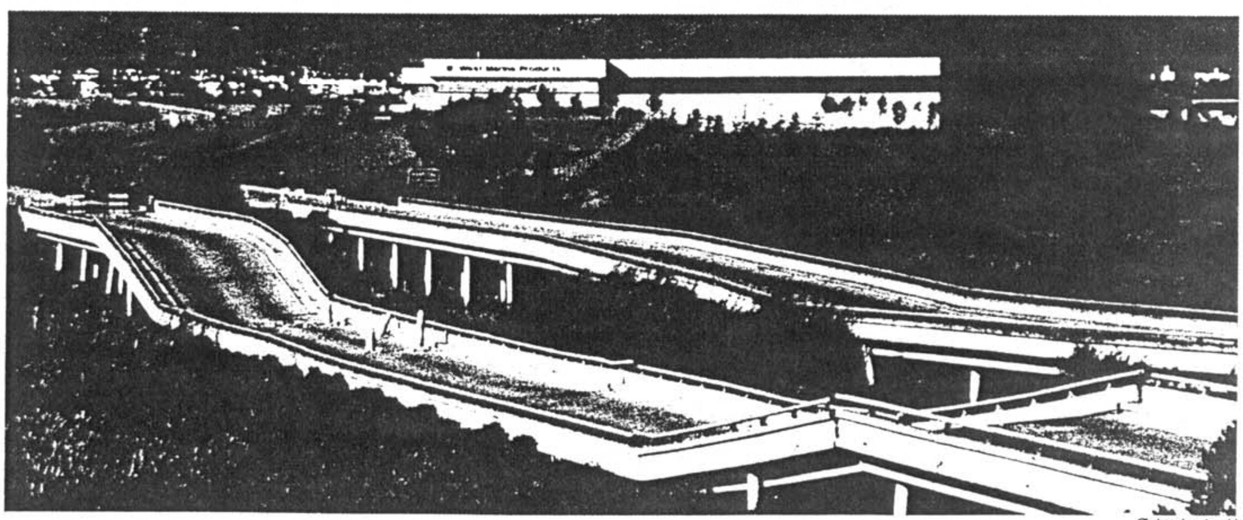
Figure 2.3: Comparison of isoseismals of large U.S. earthquakes

REGIONAL DAMAGE AND ITS IMPACT ON A SITE

Continued function and operation of a building depends on more than merely the performance of the structure. Damage to lifeline systems providing water, sewer, power, transportation and communication services can isolate a structure, cease operations or production, and leave the structure vulnerable to secondary hazards of fire and hazardous material releases. For buildings containing functions where power, water and/or communications is vital for continued operations or safety, analysis should address the vulnerability of regional lifelines serving the site. If access to the site or to regional transportation networks is critical for ongoing operations or for reaching and maintaining market deliveries, the designer should review the vulnerability of the regional transportation system. (Figure 2.4) While these issues cannot be addressed in building design, their identification for the clients will provide a basis for their understanding of the strengths and limits of a specific site, and for determining the need for back-up facilities, water and power sources, and communication systems that may prove critical to safety and post-earthquake response, recovery, and continued business operations.

Regional damage, well beyond the property line, can result in isolation of a facility from resources, market or employees, dislocation, and severe economic disruption, even without damage to the structure.

Figure 2.4: Ground failure occurred at this highway, overstressing column/slab connections.



Christopher Arnold

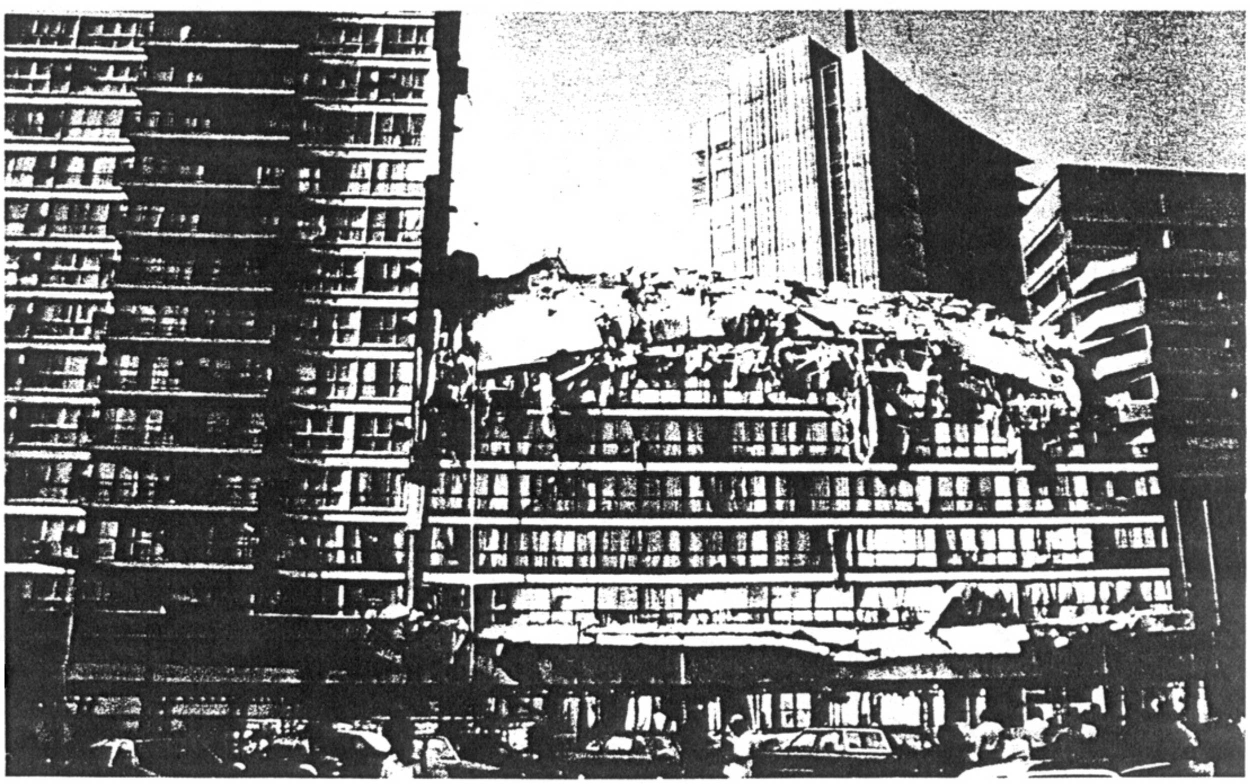
THE ARCHITECT'S ROLE IN SITE SELECTION AND EVALUATION

Only occasionally is the architect responsible for site selection. Most often, the architect is provided with a site by a client unaware of its vulnerability to seismic forces. The more traditional site analysis would include relevant information on zoning and planning restrictions on the site. A "seismic site analysis" should include an evaluation of local site conditions, adjacent hazards and regional geology, to assist the architect in briefing the client on the expected performance of the selected site, the survivability of transportation and access to the site, and the vulnerability of lifelines serving the site. This data can provide valuable insights for the client and design team in establishing design parameters and in defining expected seismic performance of the structure.

If, however, the architect participates in site selection, desired structural building performance and post earthquake function can be measured against expected site performance, life line survival and site access in determining the most appropriate location.

In either situation, a site analysis should include an assessment of the environment beyond the property line and include adjacent structures and site conditions that could "spill over" onto your site. (Figure 2.5) A complete analysis should address the issues identified in the Site Analysis Checklist.

Figure 2.5: Building adjacencies can have major impacts on performance during earthquakes. A large number of structures suffered pounding damage during the 1985 Mexico City event, leading in many cases to partial or full collapse.



Site Analysis Checklist

- ❑ Is there an active fault on or adjacent to the site?
- ❑ Will the site geology increase ground shaking?
  Does the site contain unconsolidated natural or man-made fills?
- ❑ Is the site geology stable?
- ❑ Is the site susceptible to liquefaction?
- ❑ Are adjacent up-slope and down-slope environments stable?
- ❑ Are post-earthquake access and egress secure?
- ❑ Are transportation, communication and utility lifelines vulnerable to disruption and failure?
- ❑ Are there adjacent land uses that could be hazardous after an earthquake?
- ❑ Are hazardous materials stored or used in the vicinity?
- ❑ Are building setbacks adequate to prevent battering from adjacent structures?
- ❑ Are adjacent structures collapse hazards? Would they collapse onto your site or would their failure otherwise impact the functions of your structure?
- ❑ Is the site subject to inundation from tsunami? Seiche? Dam failure flooding?
- ❑ Are there areas of the site that should be left undeveloped due to:
  - Landslide potential?
  - Inundation potential?
  - High potential for liquefaction?
  - Expected surface faulting?
  - More violent or longer duration ground shaking expected?
  - Areas necessary to provide separation from adjacent uses or structures?
- ❑ Is there adequate space on the site for a safe and "defensible" area of refuge from hazards for building occupants?
- ❑ Does the site plan increase potential for earthquake-induced landslides by:
  - Cutting unstable slopes?
  - Increasing surface runoff?
  - Increasing soil water content?

Chapter 5: The Basics Of Seismic Codes

BUILDING CODES AND SEISMIC PROVISIONS

The first seismic building code to be developed in the United States was the seismic portion of the Uniform Building Code (UBC) published by the International Conference of Building Officials (ICBO) in California. The seismic provisions of the UBC were developed on a volunteer basis by the Structural Engineers Association of California (SEAOC). Currently, in addition to the UBC, the following are important seismic codes in use:

- BOCA National Building Code
- SBCCI Standard Building Code
- GSA (Federal Buildings)
- Tri-services (Department of Defense-Military)
- Title 24, California (Hospitals and Schools)
- Veterans Administration (Veterans Hospitals)
- State Historic Building Code (SHBC) [California]
- City of Los Angeles, Section 88 (Existing URM Buildings)
- Uniform Code for Building Conservation (UCBC)

Most of the codes listed above have the stated goal of maintaining life safety; only Title 24 (California) has a higher performance goal of damage control to maintain post-earthquake function in hospitals. The last three listings, which relate to existing buildings, permit lower design force levels than those required for new buildings. (Figure 5.1)

Starting in the mid-1970s the Federal Government initiated a research program to develop a state-of-the-art approach to a seismic code that would have nationwide applicability. This effort resulted in the 1978 publication of the ATC-3 document (named after the Applied Technology Council, the non-profit engineering research group that developed it). Subsequently, the document has undergone several revisions and is now known as the *National Earthquake Hazards Reduction Program: Recommended Provisions for the Development of Seismic Regulations for New Buildings* or the *NEHRP Provisions*. Published by the Building Seismic Safety Council in Washington, and updated on a 3-year basis, the *NEHRP Provisions* document is not a code, but a technical resource document to assist in code development. In format, language and content, however, the document is very similar to a seismic code.

Richard Eisner

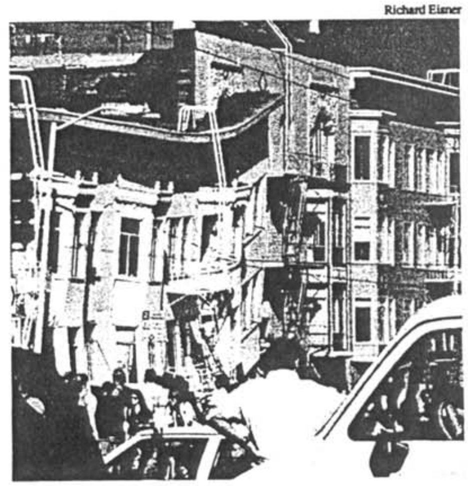


Figure 5.1: Advances in building code seismic provisions are intended to ensure life safety and prevent the types of failure and collapse that occur in pre-code buildings.

SUMMARY OF BUILDING CODE SEISMIC DESIGN CONCEPTS				
	Uniform Building Code(1991)		NEHRP Provisions(1991)	
Goal	Life Safety		Life Safety	
Seismic Load	Base Shear V (F=MA concept) $V = \frac{ZICW}{R_w}$ $(C = \frac{1.25S}{T^{2/3}})$		Base Shear V (F=MA Concept) $V = C_sW$ $(C_s = \frac{1.2A_vS}{RT^{2/3}})$	
Zone	Z	5 Zones 0.075, 0.15, 0.20, 0.30, 0.40	6 Zones 0.05, 0.10, 0.15, 0.20, 0.30, 0.40	
Import-ance	I	Building Occupancy (1.0, 1.25)	SHEG Exposure Groups (3 categories) and SPC Performance Categories (5 categories)	
Struct. Response	R_w	Response Modifications based on 5 basic Structural types	R	Response Modifications based on 6 basic Structural types
Soil	S	4 Soil Profiles (1.0, 1.2, 1.5, 2.0)	S	4 Soil Profiles (1.0, 1.2, 1.5, 2.0)
Mass	W	Building Weight	W	Building Weight
Period	T	Building Period	T	Building Period

Table 5.A

Table 5.A shows a comparison between the basic provisions of the 1991 UBC and the 1991 *NEHRP Provisions*. This summary shows that these two codes are very similar in concept and in the factors that are included.

Prior to 1988, the UBC and the *NEHRP Provisions* tended to pursue somewhat diverging approaches to code development and modification. However, in the 1988 edition of the UBC and the *NEHRP Provisions*, a notable merging of some concepts in the two documents occurred. While updating these documents continues independently, the concepts within them are subject to constant mutual review. Taken together, the SEAOC and NEHRP efforts represent probably the most influential and consistent effort in the world to provide a technical basis for seismic code development.

The UBC represents only one of the commonly used model codes in the U.S. The BOCA model code, developed by the Building Officials and Code Administrators organization is used extensively in the East and Midwest, and the Standard Building Code, developed by the Southern Building Code Congress International, is used extensively in the Southern states.

Until recently, the two latter model building codes groups have lagged behind in the development of seismic codes, primarily because these model codes were used in areas of little perceived seismic hazard. Concern for the seismic hazard present in other states in the U.S. besides California has resulted in a new interest in the development and adoption of appropriate codes, an interest which the development of the *NEHRP Provisions* was intended to support. Consequently, both the BOCA model code and the Standard Building Code now incorporate slightly modified versions of the *NEHRP Provisions* in their model building codes. (Figure 5.2)

Christopher Arnold

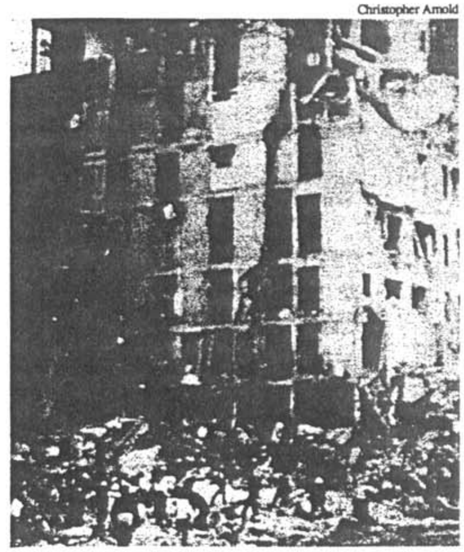


Figure 5.2: Seismic code provisions have undergone continuous development since the 1950's in response to both damaging earthquakes and to advances in engineering science.

Thus, on a national basis, the seismic code issue is basically accommodated by variations of the two main technical documents; the *NEHRP Provisions* and the UBC (or, more precisely, the SEAOC provisions upon which it is based.)

APPLYING CODES

The primary purpose of seismic building codes is to provide a simple uniform method to determine the seismic forces for any location with enough accuracy to ensure a safe and economical design. The code needs to provide for approximate uniformity of results so that no building owner, building type, or materials supplier is unfairly discriminated against.

In Chapter 1 it was shown that the earthquake forces on a building can be referred back to the basic formula for inertial forces - *F equals MA*. *M* is easy to obtain by calculating the weight of the building. How about *A*, the acceleration?

The *NEHRP Provisions* provide a number of sets of maps of the United States: these provide contour lines, or color codes of the counties in each state, so that the entire country is divided into seven areas. (Figure 5.3 shows a small scale reproduction of one of the maps provided with the *Provisions*.) Each area in turn is equivalent to a number from *0.05 to 0.40* in steps of *0.05, 0.10, 0.15, 0.20, 0.30,* and *0.40*. These represent *accelerations* in percentages of G - so that 0.40 represents 40% of G. This is the *A* for the $F = MA$ formula. It's not quite as simple as that, but nevertheless the relationship of the maps to the fundamental formula is quite direct and clear.

These maps reflect a number of assumptions. The general criterion is that the risk at any location has only a *10 percent probability of being exceeded in 50 years*, which translates into a *mean recurrence interval of 475 years*. This is a statistical number and not a prediction: the important thing is that the map is expressing a uniform risk, so that by looking at the different numbers you get an approximation of the relative risk among different regions of the country.

The *Provisions* state that, for most instances the horizontal force on the building can be represented as a horizontal shear force trying to push the base of the building across the ground where the building is attached to its foundation. This force is called the *base shear*, and a formula is provided for its estimation. Application of this formula is a key part of the code methodology and is called

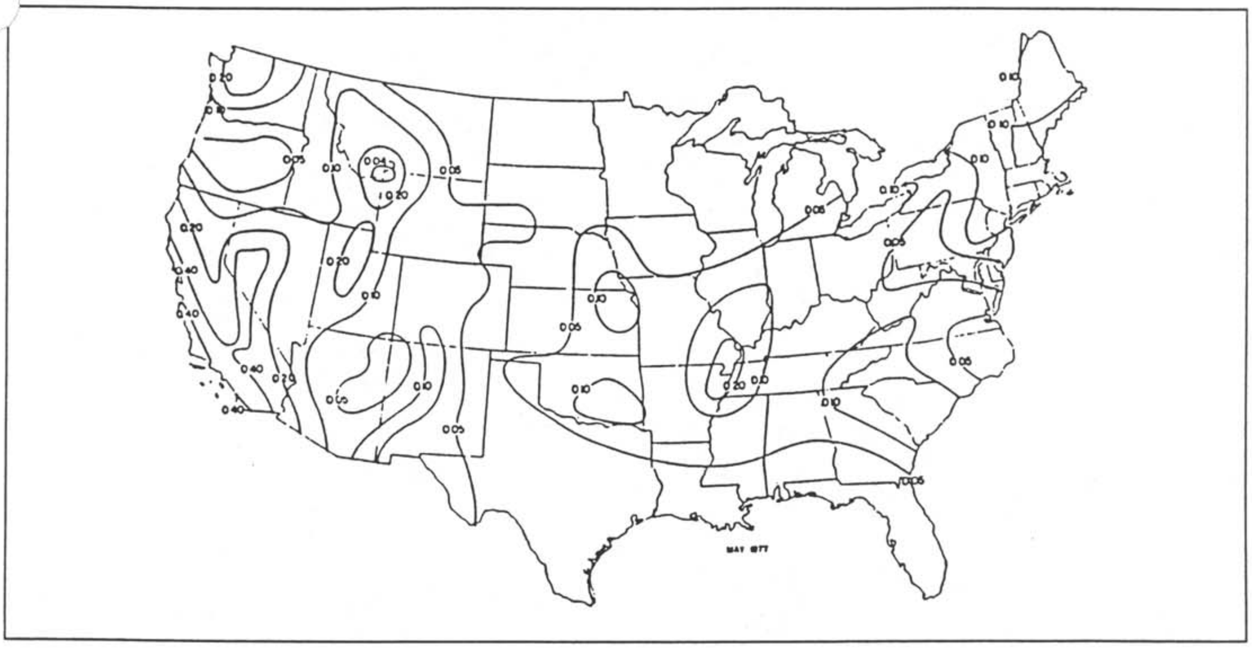


Figure 5.3: Contour map for coefficient A_a for the ontinental United States.*

the *equivalent lateral force procedure*. This general methodology is characteristic of all seismic codes throughout the world.

In the *Provisions* this formula is **$V=C_sW$** where:

C_s = the seismic design coefficient, which is related to an "expansion" of A, the acceleration number. The expansion adds other coefficients, or multipliers, which represent some of the other factors discussed in Chapter 1.

W = the building weight, which can easily be calculated.

$C_s = 1.2A_v\ S/RT^{2/3}$

where A_v is obtained from the contour map.*

S is the Coefficient for *soil profile type* (i.e. relating to soil amplification). This is obtained from a table in the *Provisions*. The coefficient varies from 1 to 2.0. For poor ground, where the coefficient is 2.0, the acceleration number is multiplied by two, thus increasing the *design force* - the forces for which the building must be designed.

R is a *response modification* coefficient, relating to the type and ductility of the chosen structural system. R factors are also obtained from a table in the *Provisions*. This is a number from 1.25 to 8: it is a divisor, so it has the

*A_v and A_a are two slightly different expressions of the acceleration factor used for design, and separate maps are provided for each in the *Provisions*.

effect of reducing the design forces, and the higher the number, the higher the reduction.

T is the *period* of the building (simple formulae for estimating this are provided in the *Provisions*.)

It can be seen that these coefficients, **A, W, S, R,** and **T** encompass most of the characteristics discussed in Chapters 1 and 3 that affect the building's earthquake performance.

For a really simple way of establishing the seismic force, the Equivalent Lateral Force method provides an alternative equation that can be used at the designer's option. This is:

$$C_s = \frac{2.5A_a}{R}$$

Note that to use this equation it is not necessary to calculate the building period or estimate the soil type. Use of this equation will generally result in a larger force factor; for a small structure, such as a house, this is not usually significant.

In addition to the equivalent lateral force equation, a formula is provided for calculating the *vertical distribution* of forces that makes some allowance for possible amplification, and allocates a higher proportion of the forces to the upper floors of the building.

Application of the equivalent lateral force formula to locations of maximum shaking (i.e: A_a=0.40 on the map) produces a coefficient C_s that varies approximately from *0.125* for a steel moment resistant frame building to *0.80* for an unreinforced masonry building. (Figure 5.4)

In other words, an unreinforced masonry building, which is a very poor seismic force resisting structure, would have to be designed to resist a base shear force equal to *80% of its weight* - a very high acceleration. (In fact, unreinforced masonry structures are not permitted to be constructed in California, and it would be very difficult, if not impossible, to design an unreinforced masonry structure for these forces). On the other hand, a moment resisting frame would only have to be designed to resist lateral forces equal to *12 1/2 % of its weight*.

So the equivalent lateral force equation provides a simple mathematical formula by which most of the factors that determine the lateral force on the building can be accounted for in a uniform way. Moreover, since the code defines a minimum force level, any of these coefficients can be revised upwards if the owner wishes to obtain a higher level of protection. This is a common practice.

Other parts of the *Provisions* set limits on *drift*, require the design to be checked for *overturning*, and require calculations for *torsion*. If severe *configuration irregularities* are present, the *Provisions* require that a more complex analysis be used instead of the simple equivalent lateral force procedure. There are, of

SAMPLE CALCULATION
(simple equation):

$V = C_sW$ and $C_s = 2.5A_a/R$

For San Francisco:
$A_a = 0.40$ (map)

For steel moment frame:
R = 8.0 (Table 3.3)

For URM:
R = 1.25 (Table 3.3)

Then:

For steel moment frame:
$C_s = 2.5 \text{ X } 0.40/8 = 0.0125$ (12.5% "G")

For URM:
$C_s = 2.5 \text{ X } 0.40/1.25 = 0.80$ (80% "G")

Figure 5.4

Eric Elsesser

Figure 5.5: Executive Order 12699 requires adoption of seismic standards in the design of all new buildings used, purchased or constructed with Federal assistance. The purpose is to avoid failures, such as that pictured above and opposite, and to reduce risks to occupants.

course many other issues presented in the *Provisions* that are not reflected in this simplified presentation. Nevertheless, the essence of any seismic code philosophy resides in the equivalent lateral force formula, and its relationship to the basic principles that have been discussed can readily be seen.

PERFORMANCE OBJECTIVES

One issue currently the focus of considerable effort is that of attempting to define performance objectives for seismic design, and ultimately to embody these in guidelines and codes. Performance objectives are statements of the limits of damage which a structure will be expected to sustain when subjected to specified earthquake demands, expressed in terms of defined ground motion. Performance objectives are expressed in terms of the performance of both the structural and nonstructural components.

The Guidelines for the Seismic Rehabilitation of Buildings, now under development by the Building Seismic Safety Council, defines three performance levels. Collapse Prevention requires that all significant components of the gravity load-resisting system must continue to carry their demands, although significant risk of injury due to falling hazards may exist. Life Safety requires that, while considerable structural damage may have occurred, major structural and nonstructural components have not become dislodged, creating a threat to life: the risk of life-threatening injury is very low. Immediate Occupancy is a damage state in which only very limited damage may have occurred. Nonstructural damage is minimized such that basic access and life safety systems including doors, elevators, emergency lighting, fire alarms, and suppression systems remain operable if power is available. Minor clean-up could be required.

While the specific terms for these damage states, and others, may change as work on this document proceeds, the philosophy of recognizing the inevitability of damage is characteristic of all the current focus on performance.

PRESIDENTIAL EXECUTIVE ORDER

An important development in the nationwide regulation of seismic building standards was the enactment into law in January 1990 of Executive Order 12699. This order requires that methods be taken to *reduce risks to the lives of occupants of buildings leased for federal uses or purchased or constructed with federal assistance, to reduce risks to the lives of persons who would be affected by engineering failures of federally assisted or regulated buildings, and to protect public investments, all in a cost-effective manner.*

The order directed federal agencies to issue regulations or procedures by February 1993 that incorporate seismic safety measures for all federal buildings that are owned, leased, assisted, or regulated by the federal government.

The link between seismic safety requirements and the availability of federal funds for new building construction was expected to encourage local governments and private sector building designers and contractors to update their codes and practices. (Figures 5.5, 5.6 and 5.7)

The order applies to any building located worldwide which is federally owned, lease constructed, leased (15 % or more of total space), regulated or financially assisted. This includes new construction financed with federal grants or loans, or federally insured or guaranteed loans or mortgages.

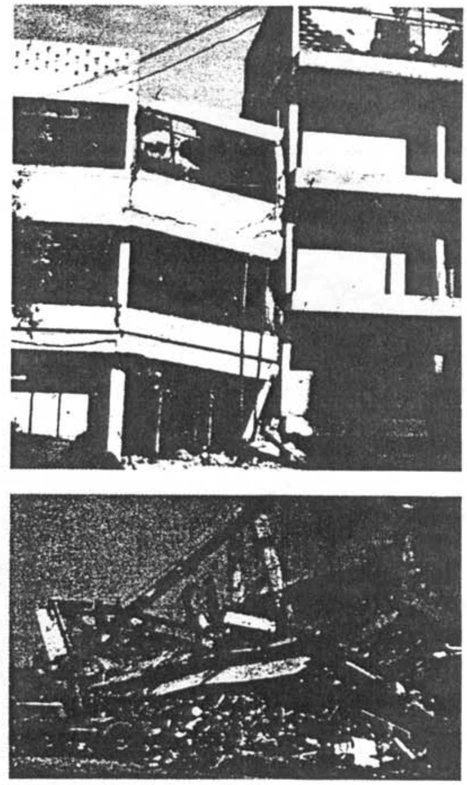
Individual federal agencies must ensure that building construction under their programs complies with the Executive Order. The Interagency Committee on Seismic Safety in Construction (ICSSC), which is a committee of federal agencies, recommends the use of seismic design and construction standards and practices equivalent to or exceeding those in the most recent (or immediately preceding) edition of the *NEHRP Provisions*.

The ICSSC determined that the following model building codes, including local codes that adopt and enforce these model codes in their entirety, are substantially equivalent to the *NEHRP Provisions*, and thus are appropriate for implementing the Executive Order.

- 1991 *Uniform Building Code* of the International Congress of Building Officials (ICBO)
- 1992 Supplement to the Building Officials and Code Administrators (BOCA) *National Building Code*, and
- 1992 Amendments to the Southern Building Code Congress (SBCCI) *Standard Building Code*.

Revisions of these model codes are considered appropriate for order implementation, as long as they are substantially equivalent to the latest version of the triennially published *NEHRP Provisions*. The order allows federal agencies to use local building codes if they, or the ICSSC, determine that the local codes provide adequately for seismic safety. Each federal agency must determine the steps that participants in its program must take to comply with the provisions of the Executive Order. FEMA has the responsibility of reporting every two years to the President and Congress on the execution of the order.

The implications of this Executive Order are far-reaching. In effect, the federal government is taking a leadership role in earthquake hazard mitigation by insisting that its own buildings, whether owned, leased or assisted, meet appropriate seismic standards. The results of the Executive Order will be watched with interest. Under normal rates of construction and retirement of buildings, a large proportion of federal buildings will be seismically resistant in 25 years.



Photos: Christopher Arnold

Figures 5.6 and 5.7

Earthquake History of Texas

The October 22, 1882 earthquake felt, in Texas, was probably centered in Oklahoma or Arkansas; the total felt area covered about 375,000 square kilometers. At Sherman, Texas, heavy machinery vibrated, bricks were thrown from chimneys, and movable objects overturned. A May 3, 1887, earthquake in Sonora, Mexico, caused damage at Bavispe and was felt strongly in parts or Arizona, New Mexico and Texas. The epicenter was in the Sierra Madre Occidental Range.

On January 8, 1891, violent shaking of buildings and a few toppled chimneys were reported from Rusk, Texas. These effects were evaluated as intensity VII, although other towns in eastern Texas along a northeast- southwest line through Rusk experienced tornadoes and sudden, violent wind storms producing effects similar to, and in some cases more damaging than, those in Rusk.

A locally damaging earthquake occurred at Panhandle, Texas, on March 28, 1917. Some cracked plaster was reported, and children were evacuated from a school building (VI).

Another disturbance occurred in the area on July 30, 1925. There were three distinct shocks over a period of 15 seconds. Major problems were the shaking of dishes from shelves and rattling and creaking of furniture (V). The shocks were felt over an area of approximately 518,000 square kilometers including distant points such as Roswell, New Mexico, 350 kilometers away; Tulsa, Oklahoma, 480 kilometers away; and Leavenworth, Kansas, 640 kilometers away.

The 1931 western Texas earthquake heavily damaged many buildings at Valentine. Also, many chimneys fell (VIII). The shock occurred at 5:40 a.m. on August 16; although people were panic stricken, there were no fatalities and only a few minor injuries from falling adobe. Adobe buildings suffered most, and cement and brick walls in many places were badly cracked. Even though Valentine bore the brunt of the shock, damage was reported from widely scattered points in Brewster, Culberson, Jeff Davis, and Presidio Counties. Cracked walls and damaged chimneys were reported from several towns. The total felt area covered about 647,000 square kilometers in Texas and New Mexico and an estimated 518,000 square kilometers in Mexico. The earthquake was accompanied by rumbling subterranean sounds heard over practically the entire affected area. The shock, measured at magnitude 6.4, was strongly recorded on all seismographs in North America and at stations all over the world. Numerous aftershocks were felt in the epicentral region; the strongest, on August 18, was intensity V at Alpine, Lobo, Pecos, and Valentine and intensity IV at Carlsbad, New Mexico. A minor aftershock was felt at Valentine on November 3.

Slight damage resulted from an earthquake in the Mexia - Wortham area on April 9, 1932. Loose bricks were thrown down, and some plaster cracked (V-VI). The shock was also felt at Coolidge, Currie, Groesbeck, Hillsboro, Teague, and Richland. A moderate earthquake affected an area of about 7700 square kilometers in northeastern Texas and an adjoining portion of Oklahoma on April 11, 1934. The tremor was most distinctly felt at Arthur City, Caviness, Chicota, Powderly, and Trout Switch (intensity V). Many persons who felt the shock reported having heard a roaring or rumbling noise. Two shocks were recognized by many observers.

A widely felt earthquake with an epicenter in the Panhandle region occurred on June 19, 1936. Intensity V effects were noted at Gruver, White Deer, and Whittenberg, Texas, Kenton, Oklahoma, and Elkhart, Kansas. The area of perceptibility covered about 103,000 square kilometers. On March 11, 1948, another shock in the Panhandle area caused minor damage, consisting mainly of cracked plaster, in northern Texas, a few places in northeastern New Mexico and northwestern Oklahoma, and one place in southeastern Colorado. The strongest effects (VI) were reported from Amarillo, Channing, Dalhart, Electric City, Panhandle, Perico, and Perryton. The felt area, which was slightly larger than that of the preceding earthquake, covered about 129,000 square kilometers. The Texas Panhandle area was the center for another moderate shock on June 20, 1951. Damage to plaster (VI) occurred at Amarillo and Hereford. The felt region extended from Lubbock to Borger.

Four shocks over 6 hours affected an area of about 26,000 square kilometers in northeastern Texas and bordering portions of Arkansas and Louisiana on March 19, 1957. Press reports noted that a few objects were upset and at least one or two windows were broken. Newspaper office and police station switchboards were swamped with calls from alarmed residents. Intensity V effects were felt at Diana, Elkhart, Gladewater, Marshall, Nacogdoches, and Troup, Texas, and Magnolia, Arkansas.

A series of moderate earthquakes in the Texas - Louisiana border region near Hemphill started on April 23, 1964. Epicenters were determined on April 23, 24, 27, and 28. There were numerous additional shocks reported felt at Pineland, Hemphill, and Milam. The only damage reported was from the magnitude 4.4 earthquake on April 28 - wall paper and plaster cracked at Hemphill (V). The magnitude of the other epicenters changed from 3.4 to 3.7. Shocks were also felt at Pineland on April 30 and May 7. On June 2, three more shocks were reported in the same area. The strongest was measured at magnitude 4.2; intensities did not exceed IV. Another moderate earthquake on August 16 awakened several people at Hemphill and there were some reports of cracked plaster (V). The shock was also felt at Bronson, Geneva, Milam, and Pineland.

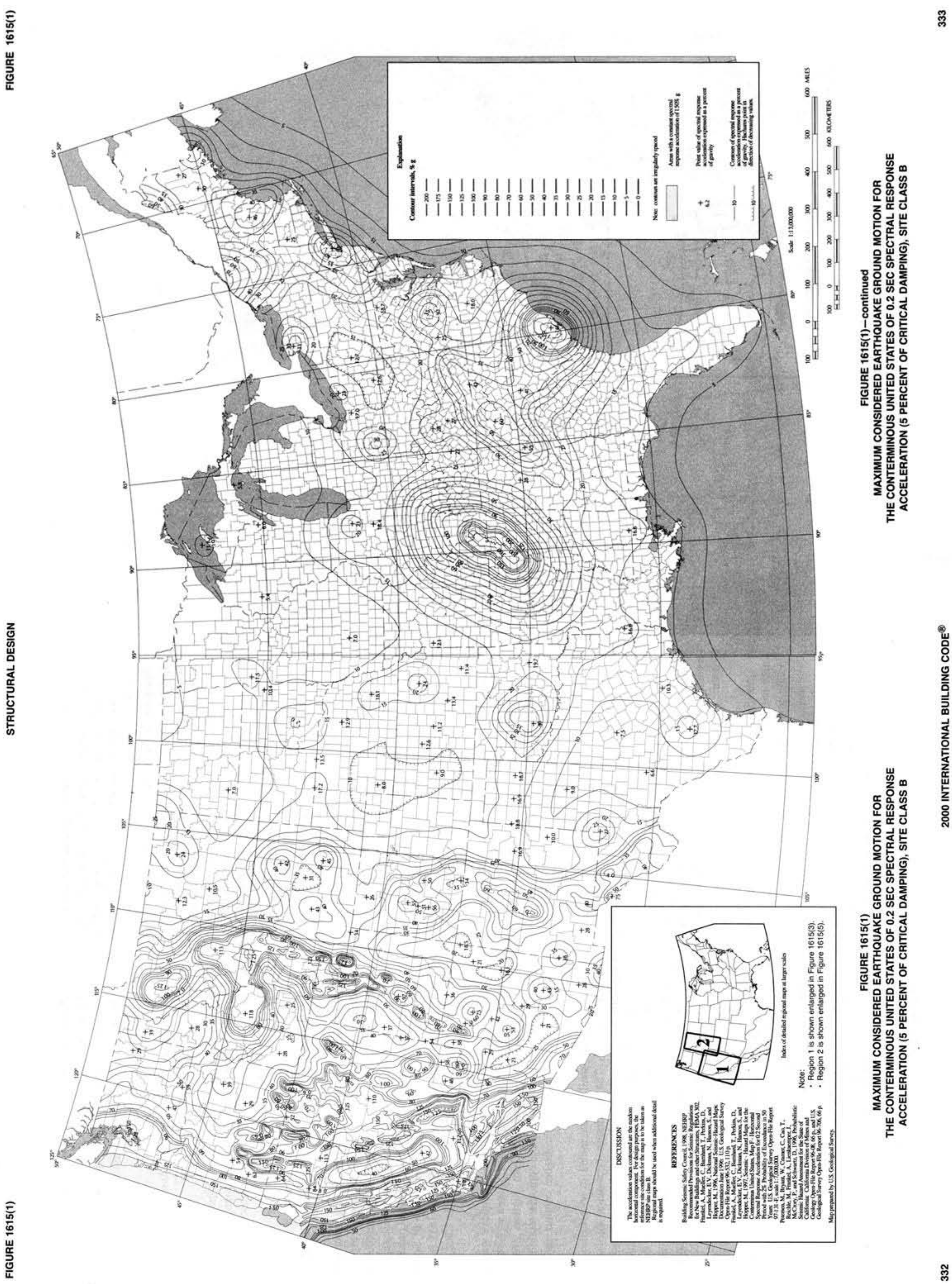
The Texas Panhandle region experienced another tremor on July 20, 1966. The magnitude 4.8 earthquake knocked books from a shelf in one home and was felt by nearly all (V) in Borger. At Amarillo, an observer in the courthouse reported a chair moved 4 or 5 inches. A similar effect was noted at the Federal Aviation Administration control tower at the Municipal Airport; observers thought a truck had hit the tower. Several street signs were knocked down and windows were broken (VI) at Kermit from a magnitude 3.4 earthquake on August 14, 1966. The shock was also felt at Wink, Texas, and Loco Hill, New Mexico.

Four small earthquakes occurred near El Paso on May 12, 1969. The first two shocks, 23 minutes apart, were measured at magnitude 3.3 and 3.4. One house in El Paso had hairline cracks in the ceiling and cracks in the cement driveway (VI). These earthquakes were also felt at Newman.

On February 15, 1974, an earthquake in the Texas Panhandle caused plaster cracks (V) at Booker, Darrovzett, and Perryton. Similar effects were noted at Liberal, Kansas, and Texhoma and Woodward, Oklahoma. The magnitude 4.5 shock was felt over an area of about 37,000 square kilometers.

Earthquake Information Bulletin, Volume 9, Number 3, May - June 1977, by Carl A. von Hake.

Earthquake Ground Motion, 0.2 Second Spectral Response
<u>International Building Code 2003:</u>



US Geological Survey, Earthquake Hazards Program, ShakeMap Scientific Background at http://earthquake.usgs.gov/eqcenter/shakemap/background.php

Spectral Response Maps

Following earthquakes larger than magnitude 5.5, spectral response maps are made. Response spectra portray the response of a damped, single-degree-of-freedom oscillator to the recorded ground motions. This data representation is useful for engineers determining how a structure will react to ground motions. The response is calculated for a range of periods. Within that range, the Uniform Building Code (UBC) refers to particular reference periods that help define the shape of the "design spectra" that reflects the building code.

Checklists for Seismic Inspection and Design
Applied Technology Council

Job Aid:
Inspection Checklist for Wood Frame Shear Walls

1. ■ **Verify from the structural framing plans and architectural floor plans the location and length of all shear walls**

2. ■ **Verify the nailing of the sheathing agrees with the shear wall schedule**

❍ Nail Type (common, galvanized box);

❍ Nail Diameter (8d or 10d);

❍ Nail Length (minimum penetration into framing 12 times nail diameter)

❍ Spacing Along Each Edge of Each Piece of Sheathing (6", 4", 3" etc.)

❍ Nail Head Shape (clipped heads not permitted)

❍ Nail Placement

__ Driven flush but not overdriven

__ Minimum 3/8" from sheathing edge to center of nail

__ View the stud side to check for nails that missed framing

__ Staggered along edges where spacing is 3 inches o.c. or less

__ Edge nails into hold-down post

3. ■ **Verify sheathing material agrees with the structural notes**

❍ Type (Plywood or OSB);

❍ Grade (APA Rated Panel or APA Rated Panel - Structural I) and

❍ Thickness (3/8", 15/32")

❍ Number of Plys (If specified for plywood)

4. ■ **Verify lumber size and grade agrees with the structural notes**

❍ Framing Grade of Studs & Posts (Stud, Construction, No. 2, No. 1);

❍ Lumber Species (Douglas Fir Larch, Hem-Fir)

❍ Framing Size (3x studs, sill at heavily nailed edges, 2-2x, 4x or 6x at HD posts)

5. ■ **Verify bottom of wall shear transfer (sill/sole plate) connection is based on the structural notes or specific sections and details**

❍ Nailing size and spacing of wall sole plate to floor framing below from shear wall schedule; verify nails penetrate framing below

❍ Foundation sill bolt diameter and spacing from shear wall schedule or notes

❍ Bolts not less than 7 bolt diameters from ends of sill piece; not more than 12 inches from ends; not less than 1 inch from edge of sill plate; not less than 1 1/2 inches to edge of concrete foundation.

❍ Verify square plate washer is used on bolts.

❍ Verify bolt hole in sill plate is not more than 1/16" larger than bolt diameter.

Job Aid:
Inspection Checklist for Wood Frame Shear Walls (continued)

6. ■ Verify top of wall shear transfer connection by looking at the shear wall schedule and typical sections at roof and floor level

- ❍ Location of edge nail row along top plate of lower wall and sole plate of upper wall, and if required, along the rim joist or blocking
- ❍ Size and spacing of framing clips, when required, from top plate to floor or roof framing, with all nail holes filled
- ❍ Where 10 d nails are required for the sheathing, and when edge nailing is required into the rim member, the minimum rim member thickness is 1 3/4 inch. Therefore a nominal 2x is NOT sufficient.

7. ■ Verify top plate splice connections along shear wall lines, not only those occurring directly above the shear wall

- ❍ Check for a detail or note on framing plans calling for typical or special plate splices.
- ❍ Verify the strap size (gage thickness and length) number of rows of nails, and total number of nails per the product manufacturer's catalogue
- ❍ Verify straps are centered on the splice and have all nail holes filled.
- ❍ Splices are needed anywhere that top plates are interrupted (by perpendicular beams or headers in the plane of the wall)

8. ■ Verify Hold-Down Installation

- ❍ Confirm locations per Framing and Foundation Plans (usually, but not always, are hold-downs required at each end of a shear wall)
- ❍ Verify minimum Post Size and Lumber Grade
- ❍ Verify equal number of nails to upper and lower wall framing for Nailed Strap Type Hold-downs Spanning Floor Framing
- ❍ Verify bolt hole diameter through posts is not more than 1/16 inch larger than the actual bolt diameter.
- ❍ Verify bolts heads or nuts are not countersunk into the post, unless specifically permitted
- ❍ Verify a washer is installed under the nut on side of the post opposite the HD
- ❍ Verify nuts are tight on all bolts, including the anchor bolt into the foundation and the ends of threaded rods spanning between floor levels.
- ❍ Anchor bolts and threaded rods should not be bent. HD location should be installed to minimize the length of threaded rods.
- ❍ Verify all bolt diameters are as specified either by the hold-down product manufacturer's catalogue or as specified on the drawings.
- ❍ Verify prior to concrete pour the length of embedment of anchor bolts and the embedded end condition (e.g., L-hook, J-hook, nut and square plate washer, hex headed bolt) match the drawings
- ❍ Verify anchor bolt clearance from edges and ends of footings as specified on the drawings.

Job Aid:
Checklist for Design of Masonry

Structural Notes

1. ❍ Applicable code specified (city and date).
2. ❍ Applied loads shown including wind, seismic and live loads.
3. ❍ Is the masonry strength f'm specified?
4. ❍ Is the method to verify the f'm specified? (Unit strength method).
5. ❍ Is type S mortar specified?
6. ❍ Is high or low lift grouting specified?
7. ❍ Are cleanouts required?
8. ❍ Is special inspection required? Are prism tests required?
9. ❍ Have full allowable stresses been used in the design?

Design

10. ❍ Is h/t less than 30? If not, verify calculations.
11. ❍ Is the wall laterally supported with straps or other methods capable of resisting at least 420 lb/ft?
12. ❍ Does the bar fit in the cell?
13. ❍ Are locations of laps shown (Min. 48 dia.)? Are they in locations were stresses are less than 80% of the allowable?
14. ❍ Are dowel laps sufficient (Min. 48 dia.)?
15. ❍ Is there continuous horizontal reinforcement at the window, and door head?
16. ❍ Is there continuous horizontal reinforcement at the floor?
17. ❍ Are window and door connections designed and shown on the drawings?
18. ❍ Are there expansion joints at the corners?
19. ❍ Are there provisions made in connections to accommodate thermal movement? (Steel roof rigidly attached at a masonry corner)?
20. ❍ Is the brick masonry confined between other materials without expansion joints?

Specifications

21. ❍ Is a color, pattern and workmanship panel required?
22. ❍ Is a grouting demonstration panel required?
23. ❍ Are materials specified in accordance with the correct standards? ❍ Brick?
    ❍ Is the Hollow clay brick of sufficient strength? ❍ Cement? ❍ Lime? ❍ Sand?
    ❍ Grout? ❍ Mortar? ❍ Is the mortar specified by proportions?
    ❍ Reinforcement? ❍ Is weldable steel required?
24. ❍ Are there requirements for handling and storage of materials?
25. ❍ Is there a requirement for a preconstruction meeting?
26. ❍ Are shop drawings required?
27. ❍ Are control joint size and materials specified?
28. ❍ Are sealant compatibility tests required?
29. ❍ Are the cleaning methods included?
30. ❍ Does the specification require wetting of the brick?
31. ❍ Are the joint finished specified? If raked joints are used is this in the analysis?
32. ❍ Are weep holes and fill materials specified?
33. ❍ Is the sealing procedures and materials specified?
34. ❍ Are cold weather and hot weather construction provisions included?
35. ❍ Are requirements for protecting the work included?
36. ❍ Is it required to verify dimensions prior to laying the masonry?
37. ❍ Is a written quality control procedure required?
38. ❍ Are prism test requirements included both prior to construction and during construction?

Job Aid:
Inspection Checklist for Masonry Construction

Plans

1.
- ❍ Is continuous inspection necessary?
- ❍ Are called inspections necessary?

Materials

2. **Concrete masonry units**:
- ❍ Type and quality
- ❍ Strength of the masonry complies with plans
- ❍ Is a laboratory test required?
- ❍ Correct size and type, (per UBC Standard Nos. 21-4, 21-5).
- ❍ Curing (UBC Standard Nos. 214, 21-5)
- ❍ Cleanliness.
- ❍ Soundness (UBC Standard Nos. 21-4, 21-5)
- ❍ Are required inspection holes provided?

3. **Sand:**
- ❍ Cleanliness
- ❍ Quality and fineness
- ❍ Compliance with code requirements (ASTM C144)

4. **Cement:**
- ❍ Meets requirements of the UBC Standards (UBC Standard No. 21-15).

5. **Aggregates:**
- ❍ Meet the requirements of UBC Standards (ASTM C144 and C404).

6. **Lime:**
- ❍ Conforms to the UBC Standards (UBC Standard No. 21-13).

7. **Water:**
- ❍ Is clean and free from harmful substances.

8. **Plasticizing agents:**
- ❍ Bonform to Standards.

9. **Admixtures conform to the following requirements:**
- ❍ Have been approved.
- ❍ Are of right quantity.
- ❍ Are not used with plastic cement.

10. **Reinforcing steel:**
- ❍ Kind and grade.
- ❍ Max. Size (UBC No. 2107.2.2. 1)

Workmanship

11.
- ❍ Sample panels have been provided and approved, if required.

12. **Mortar:**
- ❍ Proportions of the mortar mix and time Of mixing.
- ❍ Consistency of mortar.
- ❍ Clean water is used
- ❍ Mortar is properly handled in mixing
- ❍ Mortar is not excessively retempered.
- ❍ Work is kept dry at all times.
- ❍ Mortar classified by type and use (UBC Table No. 21-A)

13. **Grout:**
- ❍ Proportions (UBCTableNo.21-B).
- ❍ Consistency.
- ❍ Compressive strength (UBC Standard No. 21-19).
- ❍ Handling.
- ❍ Segregation.

Construction

14. **Bearing on solid masonry:**
- ❍ Suitability of bearing masonry
- ❍ Size of bearing masonry
- ❍ Location of bolt ties (UBC No. 2106.3.7)
- ❍ Size, length, placement and embedment of connectors.

15. **Masonry on concrete:**
- ❍ Width and depth of footing excavations.
- ❍ Anchorage around main steel
- ❍ Grouting and metal inserts
- ❍ Type, spacing and material of ties.
- ❍ Embedment of ties or connection to main steel.

16.
- ❍ Proper sill material and anchorage of supporting members to footings.

17. **Head, bed, end and wall joints:**
- ❍ Correct size and type
- ❍ Buttered where required
- ❍ Joints where fresh masonry is joined to set masonry.
- ❍ Properly filled with mortar. (Exception: UBC No. 2104.4.4).
- ❍ Watertight (bug holes filled).

Job Aid:
Inspection Checklist for Masonry Construction (continued)

■ **Construction** (continued)

18. Reinforced hollow unit masonry:
- ❍ Vertical alignment and continuity of cells
- ❍ Requirements when work is stopped for one hour or longer.
- ❍ Leakage of grout.
- ❍ Cleanout openings for pours over 5 ft. (15 m) (UBC No. 2104.6. 1).
- ❍ Overhanging mortar.
- ❍ Sealing of cleanout cells.
- ❍ Position of reinforcement.
- ❍ Reinforcing hooks and splices (UBC Nos. 2107.2.2.5, 2107.2.2.6).

19. ❍ Racking and toothing at wall intersections.

20. ❍ Corners and returns.

21. Reinforcing steel:
- ❍ Clearances.
- ❍ Deformation.
- ❍ Additional steel around openings (UBC No. 2106.1.12.3 Item 3)
- ❍ Placed within allowable tolerances (UBC No. 2104.5).

22. Connections:
- ❍ Size and location of joist anchors.
- ❍ Size, location and number of bolts
- ❍ Size and location of dowels
- ❍ Location of stirrups.
- ❍ Veneer ties (if any)

23. ❍ Separation between buildings.

24. ❍ Thickness of the walls.

25. ❍ Size of bond beams.

26. ❍ Placement of headers and lintels of material other than masonry.

27. ❍ Wall ties.

28. Unprotected steel supporting members:
- ❍ Correct location of mechanical installation supports.
- ❍ Size and location of bolts and connections.
- ❍ Size and spacing of bracing connections.
- ❍ Size and alignment of connection holes.
- ❍ Shims and dry packing.
- ❍ Location and size of stiffeners.
- ❍ Size and alignment of base plates.

29. Anchoring of wood floor joists to supporting masonry members:
- ❍ Required size of ledges.
- ❍ Required size, spacing and length of bolts and joist anchors.

30. Where floor joists are parallel to the wall:
- ❍ Placing of required blocking.
- ❍ Type of anchors required.
- ❍ Use of proper connections to anchors.

31. Floor joists tying to a masonry wall:
- ❍ Required size, spacing and bearing of joists
- ❍ Required air space around joists
- ❍ Required anchors
- ❍ Required bridging and/or blocking
- ❍ Connection to ledger
- ❍ Required connectors for anchors

32. Where fire-resistant floors are required.
- ❍ Proper material for fire resistance
- ❍ Required thickness of floor slab
- ❍ Required supports
- ❍ Required reinforcing
- ❍ Required time for supports and forms to remain in place for concrete floors

33. ❍ Contraction joints and control joints are located and provided as indicated or required.

34. ❍ Weepholes are provided if required.

35. Chases.
- ❍ Location and spacing on approved plans.
- ❍ Purpose.
- ❍ Maximum permitted depth.
- ❍ No reduction of the required strength and fire resistance of the wall.

36. Where there is a change of thickness in non-bearing walls
- ❍ Locate the position on plans.
- ❍ Required top plates comply
- ❍ location of ties, anchors, bolts and blocking.

37. Corbeling:
- ❍ Maximum projections
- ❍ Bonding and anchorage
- ❍ Required temporary supports
- ❍ Required reinforcing.

38. ❍ Pointing, replacement of defective units, and repair of other defects are promptly performed.

39. ❍ Waterproofing of walls is performed as required.

40. ❍ Methods of final cleaning are as required.

Example:
Seismic Loading

Example 1

Example 5

The floor plan of a single story commercial building located in Seismic Zone 3 is shown in Fig. 5-22. The 14-foot high masonry shear walls are load bearing and have a weight of 70 pounds per square foot. The weight of the roof is 50 pounds per square foot and all other weights may be neglected. Determine the seismic base shear.

Solution

The relevant dead loads are given by:

Roof = $W_R = 0.05 \times 40 \times 20 = 40$ kips

North wall = $W_3 = 0.07 \times 12 \times 14 = 11.76$ kips

South wall = $W_1 = 11.76$ kips

East wall = $W_2 = 0.07 \times 10 \times 14 = 9.80$ kips

West wall = $W_4 = 9.80$ kips

Total seismic dead load is then

$W = W_R + W_1 + W_2 + W_3 + W_4$
$= 83.12$ kips.

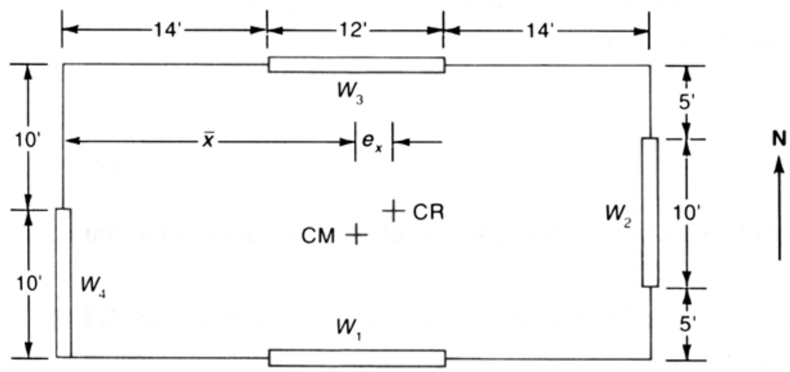


Fig. 5-22

The seismic base shear is given by Formula (28-1) as

$V = (ZIC/R_W)W$ where

$Z = 0.3$ for Zone 3 from Table 16-I

$I = 1.0$ for a standard occupancy structure as defined in Table 16-K

$C = 2.75$, the maximum value specified by UBC Section 1628.2.1

$R_W = 6$ from Table 16-N for a bearing wall system

$W = 83.12$ kips, as calculated

Then the seismic base shear is

$V = (0.3 \times 1 \times 2.75/6)W$
$= 0.1375\ W$
$= 11.43$ kips.

Connection and Tension Member Design

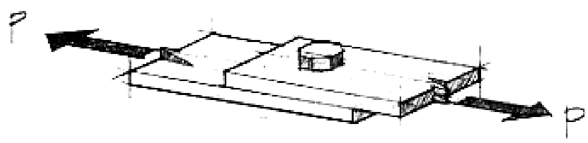
Notation:

A = area (net = with holes, bearing = in contact, etc...)
A_e = effective net area found from the product of the net area A_n by the shear lag factor U
A_b = area of a bolt
A_g = gross area, equal to the total area ignoring any holes
A_{gv} = gross area subjected to shear for block shear rupture
A_n = net area, equal to the gross area subtracting any holes, as is A_{net}
A_{nt} = net area subjected to tension for block shear rupture
A_{nv} = net area subjected to shear for block shear rupture
ASD = allowable stress design
d = diameter of a hole
f_p = bearing stress (see P)
f_t = tensile stress
f_v = shear stress
$F_{connector}$ = shear force capacity per connector
F_n = nominal tension or shear strength of a bolt
F_u = ultimate stress prior to failure
F_{EXX} = yield strength of weld material
F_y = yield strength
F_{yw} = yield strength of web material
g = gage spacing of staggered bolt holes
I = moment of inertia with respect to neutral axis bending
k = distance from outer face of W flange to the web toe of fillet
l = name for length
L = name for length
L_c = clear distance between the edge of a hole and edge of next hole or edge of the connected steel plate in the direction of the load
L' = length of an angle in a connector with staggered holes
$LRFD$ = load and resistance factor design
n = number of connectors across a joint
N = bearing length on a wide flange steel section
= bearing type connection with threads included in shear plane
p = pitch of connector spacing
P = name for axial force vector, as is T
R = generic load quantity (force, shear, moment, etc.) for LRFD design
R_a = required strength (ASD)
R_n = nominal value (capacity) to be multiplied by ϕ
R_u = factored design value for LRFD design
s = longitudinal center-to-center spacing of any two consecutive holes
S = allowable strength per length of a weld for a given size
SC = slip critical bolted connection
t = thickness of a hole or member
t_w = thickness of web of wide flange
T = throat size of a weld
V = internal shear force
$V_{longitudinal}$ = longitudinal shear force
U = shear lag factor for steel tension member design
U_{bs} = reduction coefficient for block shear rupture
X = bearing type connection with threads excluded from the shear plane
y = vertical distance
π = pi (3.1415 radians or 180°)
ϕ = resistance factor
= diameter symbol
γ = load factor in LRFD design
Ω = safety factor for ASD
Σ = summation symbol

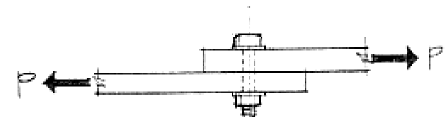
Connections

Connections must be able to transfer any axial force, shear, or moment from member to member or from beam to column. Steel construction accomplishes this with bolt and welds. Wood construction uses nails, bolts, shear plates, and split-ring connectors.

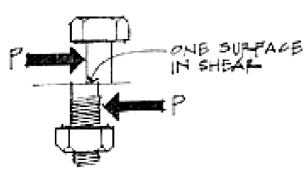
<u>Single Shear</u> - forces cause only one shear "drop" across the bolt.



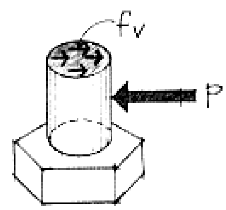
(a) Two steel plates bolted using one bolt.



(b) Elevation showing the bolt in shear.



(c)



(d)

f_v = Average shear stress through bolt cross section

A = Bolt cross-sectional area

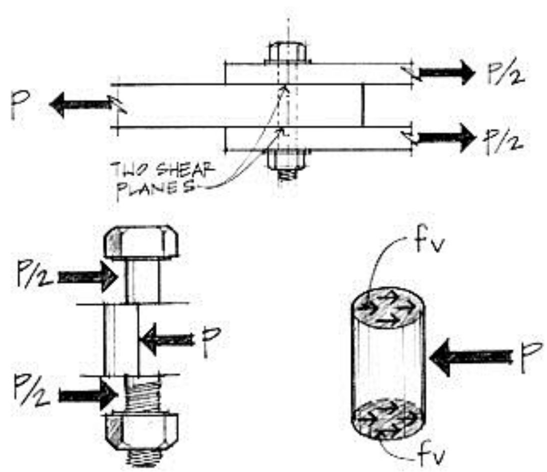
$$f_v = \frac{P}{A} = \frac{P}{\pi r^2}$$

Figure 5.11 A bolted connection—single shear.

<u>Double Shear</u> - forces cause two shear changes across the bolt.

$$f_v = \frac{P}{2A} = \frac{P}{2\pi r^2}$$

(two shear planes)

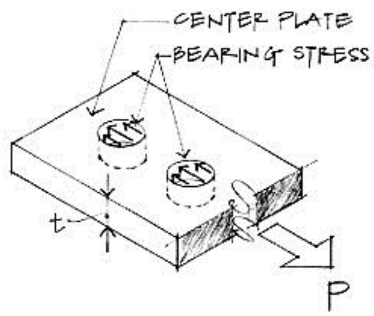


Free-body diagram of middle section of the bolt in shear.

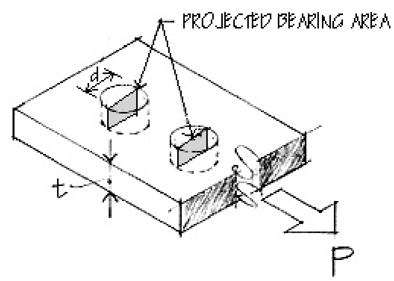
Figure 5.12 A bolted connection in double shear.

<u>Bearing of a Bolt on a Bolt Hole</u> – The bearing surface can be represented by *projecting* the cross section of the bolt hole on a plane (into a rectangle).

$$f_p = \frac{P}{A} = \frac{P}{td}$$

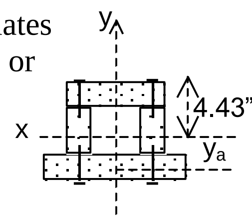


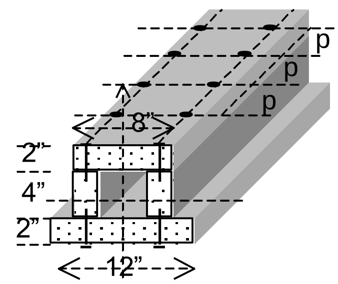
Bearing stress on plate.



<u>Horizontal Shear in Composite Beams</u>

Typical connections needing to resist shear are plates with nails or rivets or bolts in composite sections or splices.





The pitch (spacing) can be determined by the capacity in shear of the connector(s) to the shear flow over the spacing interval, p.

$$\frac{V_{longitudinal}}{p} = \frac{VQ}{I} \qquad V_{longitudinal} = \frac{VQ}{I} \cdot p$$

where

$$nF_{connector} \geq \frac{VQ_{connected\,area}}{I} \cdot p$$

p = pitch length

n = number of connectors connecting the connected area to the rest of the cross section

F = force capacity in one connector

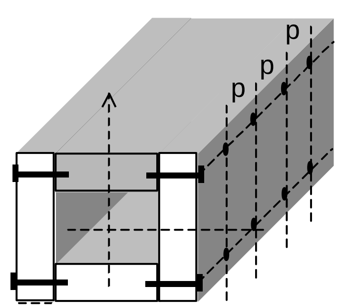
$Q_{connected\ area} = A_{connected\ area} \times y_{connected\ area}$

$y_{connected\ area}$ = distance from the centroid of the connected area to the neutral axis

<u>Connectors to Resist Horizontal Shear in Composite Beams</u>

Even vertical connectors have shear flow across them.

The spacing can be determined by the capacity in shear of the connector(s) to the shear flow over the spacing interval, p.



$$p \leq \frac{nF_{connector} I}{VQ_{connected\,area}}$$

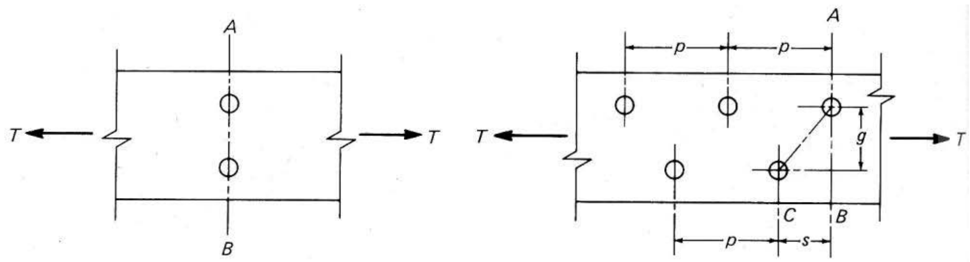
Tension Member Design

In tension members, there may be bolt holes that reduce the size of the cross section.

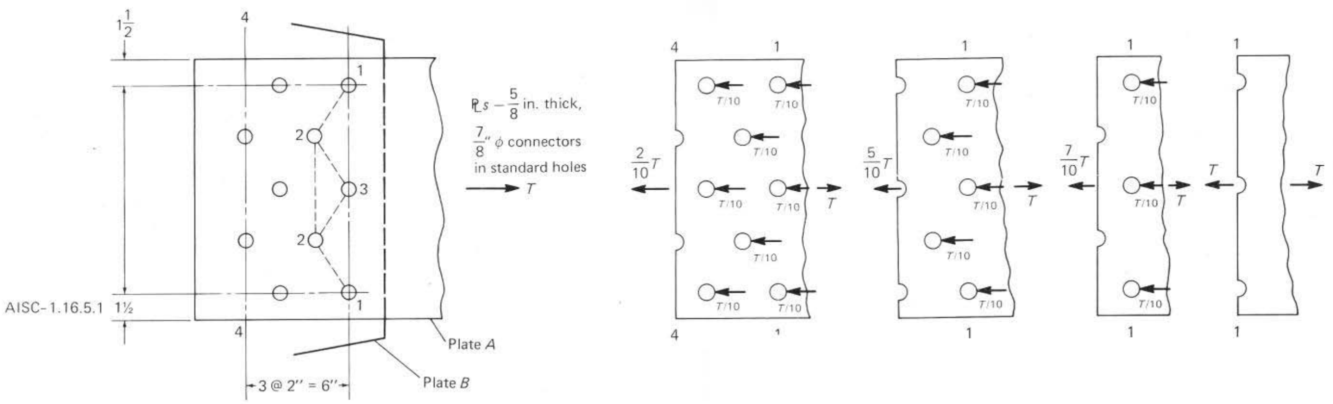
Effective Net Area:

The smallest effective are must be determined by subtracting the bolt hole areas. With staggered holes, the shortest length must be evaluated.

$$f_t = \frac{P}{A_e} \text{ or } \frac{T}{A_e}$$



A series of bolts can also transfer a portion of the tensile force, and some of the effective net areas see reduced stress.



Connections in Wood

Connections for wood are typically mechanical fasteners. Shear plates and split ring connectors are common in trusses. Bolts of metal bear on holes in wood, and nails rely on shear resistance transverse and parallel to the nail shaft.

<u>Bolted Joints</u>

Stress must be evaluated in the member being connected using the load being transferred and the reduced cross section area called *net area.* Bolt capacities are usually provided in tables and take into account the allowable shearing stress across the diameter for *single* and *double shear*, and the allowable bearing stress of the connected material based on the direction of the load with respect to the grain (parallel or perpendicular). Problems, such as ripping of the bolt hole at the end of the member, are avoided by following code guidelines on minimum edge distances and spacing.

Nailed Joints

Because nails rely on shear resistance, a common problem when nailing is splitting of the wood at the end of the member, which is a shear failure. Tables list the shear force capacity per unit length of embedment per nail. Jointed members used for beams will have shear stress across the connector, and the pitch spacing, p, can be determined from the shear stress equation when the capacity, F, is known.

Other Connectors

Screws - Range in sizes from #6 (0.138 in. shank diameter) to #24 (0.372 in. shank diameter) in lengths up to five inches. Like nails, they are best used laterally loaded in side grain rather than in withdrawal from side grain. Withdrawal from end is not permitted.

Lag screws (or bolts) – Similar to wood screw, but has a head like a bolt. It must have a load hole drilled and inserted along with a washer.

Split ring and shear plate connectors – Grooves are cut in each piece of the wood members to be joined so that half the ring is in each section. The members are held together with a bolt concentric with the ring. Shear plate connectors have a central plate within the ring.

Splice plates – These are common in pre-manufactured joists and consist of a sheet of metal with punched spikes.

Framing seats & anchors – for instance, joist hangers and post bases...

Connections in Steel

The limit state for connections depends on the loads:

1. tension yielding
2. shear yielding
3. bearing yielding
4. bending yielding due to eccentric loads
5. rupture

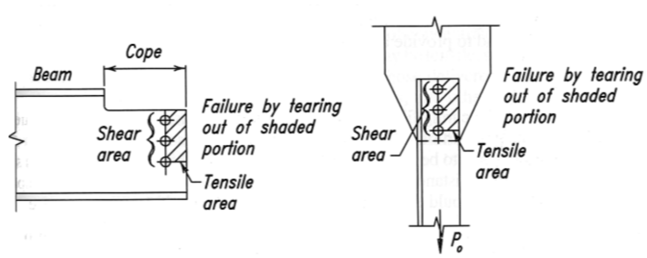


Fig. C-J4.1. Failure for block shear rupture limit state.

High strength bolts resist shear (primarily), while the connected part must resist yielding and rupture.

Welds must resist shear stress. The design strengths depend on the weld materials.

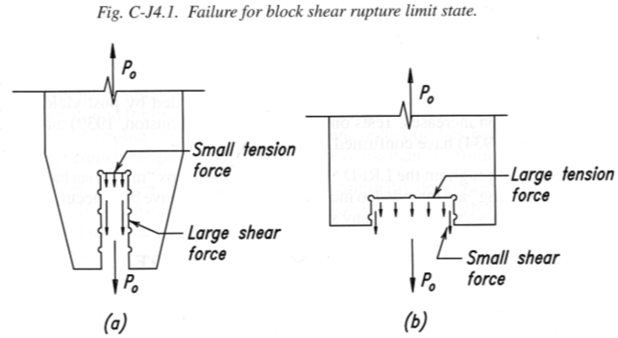
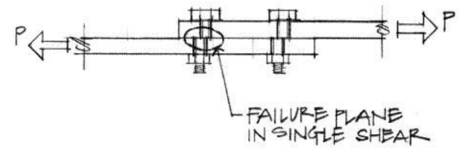


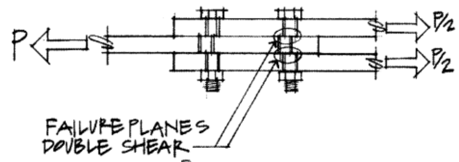
Fig. C-J4.2. Block shear rupture in tension.

Bolted Connection Design

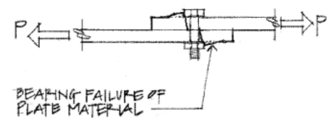


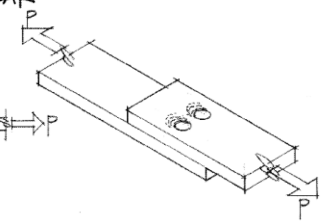
Bolt designations signify material and type of connection where

SC: slip critical
N: bearing-type connection with bolt threads *included* in shear plane
X: bearing-type connection with bolt threads *excluded* from shear plane



A307: similar in strength to A36 steel (also known as ordinary, common or unfinished bolts)
A325: high strength bolts (Group A)
A490: high strength bolts (higher than A325, Group B)





Bearing-type connection: no frictional resistance in the contact surfaces is assumed and slip between members occurs as the load is applied. (Load transfer through bolt only).
Slip-critical connections: bolts are torqued to a high tensile stress in the shank, resulting in a clamping force on the connected parts. (Shear resisted by clamping force). Requires inspections and is useful for structures seeing dynamic or fatigue loading.

Bolts rarely fail in **bearing**. The material with the hole will more likely yield first.

For the determination of the net area of a bolt hole the width is taken as *1/16" greater* than the nominal dimension of the hole. Standard diameters for bolt holes are *1/16"* larger than the bolt diameter. (This means the net width will be 1/*8"* larger than the bolt.)

Design for Bolts in Bearing, Shear and Tension

Available shear values are given by bolt type, diameter, and loading (Single or Double shear) in AISC manual tables. Available shear value for slip-critical connections are given for limit states of serviceability or strength by bolt type, hole type (standard, short-slotted, long-slotted or oversized), diameter, and loading. Available tension values are given by bolt type and diameter in AISC manual tables.

Available bearing force values are given by bolt diameter, ultimate tensile strength, F_u, of the connected part, and thickness of the connected part in AISC manual tables.

For shear OR tension (same equation) in bolts: $R_a \le R_n / \Omega$ or $R_u \le \phi R_n$

where $R_u = \Sigma \gamma_i R_i$

- single shear (or tension) $R_n = F_n A_b$
- double shear $R_n = F_n 2A_b$

where ϕ = the resistance factor
F_n = the nominal tension or shear strength of the bolt
A_b = the cross section area of the bolt

ϕ = 0.75 (LRFD) Ω = 2.00 (ASD)

Table 7-1
Available Shear Strength of Bolts, kips

ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	5/8 r_n/Ω	5/8 ϕr_n	3/4 r_n/Ω	3/4 ϕr_n	7/8 r_n/Ω	7/8 ϕr_n	1 r_n/Ω	1 ϕr_n
Nominal Bolt Diameter, *d*, in.					5/8		3/4		7/8		1	
Nominal Bolt Area, in.²					0.307		0.442		0.601		0.785	
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	27.0	40.5	S	8.29	12.4	11.9	17.9	16.2	24.3	21.2	31.8
				D	16.6	24.9	23.9	35.8	32.5	48.7	42.4	63.6
	X	34.0	51.0	S	10.4	15.7	15.0	22.5	20.4	30.7	26.7	40.0
				D	20.9	31.3	30.1	45.1	40.9	61.3	53.4	80.1
Group B	N	34.0	51.0	S	10.4	15.7	15.0	22.5	20.4	30.7	26.7	40.0
				D	20.9	31.3	30.1	45.1	40.9	61.3	53.4	80.1
	X	42.0	63.0	S	12.9	19.3	18.6	27.8	25.2	37.9	33.0	49.5
				D	25.8	38.7	37.1	55.7	50.5	75.7	65.9	98.9
A307	–	13.5	20.3	S	4.14	6.23	5.97	8.97	8.11	12.2	10.6	15.9
				D	8.29	12.5	11.9	17.9	16.2	24.4	21.2	31.9

ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	1 1/8 r_n/Ω	1 1/8 ϕr_n	1 1/4 r_n/Ω	1 1/4 ϕr_n	1 3/8 r_n/Ω	1 3/8 ϕr_n	1 1/2 r_n/Ω	1 1/2 ϕr_n
Nominal Bolt Diameter, *d*, in.					1 1/8		1 1/4		1 3/8		1 1/2	
Nominal Bolt Area, in.²					0.994		1.23		1.48		1.77	
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	27.0	40.5	S	26.8	40.3	33.2	49.8	40.0	59.9	47.8	71.7
				D	53.7	80.5	66.4	99.6	79.9	120	95.6	143
	X	34.0	51.0	S	33.8	50.7	41.8	62.7	50.3	75.5	60.2	90.3
				D	67.6	101	83.6	125	101	151	120	181
Group B	N	34.0	51.0	S	33.8	50.7	41.8	62.7	50.3	75.5	60.2	90.3
				D	67.6	101	83.6	125	101	151	120	181
	X	42.0	63.0	S	41.7	62.6	51.7	77.5	62.2	93.2	74.3	112
				D	83.5	125	103	155	124	186	149	223
A307	–	13.5	20.3	S	13.4	20.2	16.6	25.0	20.0	30.0	23.9	35.9
				D	26.8	40.4	33.2	49.9	40.0	60.1	47.8	71.9

ASD	LRFD	
Ω = 2.00	ϕ = 0.75	For end loaded connections greater than 38 in., see AISC *Specification* Table J3.2 footnote b.

Group A Bolts
A325, A325M
F1858
A354 Grade BC
A449

Table 7-3
Slip-Critical Connections
Available Shear Strength, kips
(Class A Faying Surface, μ = 0.30)

Group A Bolts

Hole Type	Loading	5/8 r_n/Ω	5/8 ϕr_n	3/4 r_n/Ω	3/4 ϕr_n	7/8 r_n/Ω	7/8 ϕr_n	1 r_n/Ω	1 ϕr_n
Nominal Bolt Diameter, *d*, in.		5/8		3/4		7/8		1	
Minimum Group A Bolt Pretension, kips		19		28		39		51	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	S	4.29	6.44	6.33	9.49	8.81	13.2	11.5	17.3
	D	8.59	12.9	12.7	19.0	17.6	26.4	23.1	34.6
OVS/SSLP	S	3.66	5.47	5.39	8.07	7.51	11.2	9.82	14.7
	D	7.32	10.9	10.8	16.1	15.0	22.5	19.6	29.4
LSL	S	3.01	4.51	4.44	6.64	6.18	9.25	8.08	12.1
	D	6.02	9.02	8.87	13.3	12.4	18.5	16.2	24.2

Hole Type	Loading	1 1/8 r_n/Ω	1 1/8 ϕr_n	1 1/4 r_n/Ω	1 1/4 ϕr_n	1 3/8 r_n/Ω	1 3/8 ϕr_n	1 1/2 r_n/Ω	1 1/2 ϕr_n
Nominal Bolt Diameter, *d*, in.		1 1/8		1 1/4		1 3/8		1 1/2	
Minimum Group A Bolt Pretension, kips		56		71		85		103	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	S	12.7	19.0	16.0	24.1	19.2	28.8	23.3	34.9
	D	25.3	38.0	32.1	48.1	38.4	57.6	46.6	69.8
OVS/SSLP	S	10.8	16.1	13.7	20.5	16.4	24.5	19.8	29.7
	D	21.6	32.3	27.4	40.9	32.7	49.0	39.7	59.4
LSL	S	8.87	13.3	11.2	16.8	13.5	20.2	16.3	24.4
	D	17.7	26.6	22.5	33.7	26.9	40.3	32.6	48.9

STD = standard hole
OVS = oversized hole
SSLT = short-slotted hole transverse to the line of force
SSLP = short-slotted hole parallel to the line of force
LSL = long-slotted hole transverse or parallel to the line of force

S = single shear
D = double shear

Hole Type	ASD	LRFD
STD and SSLT	Ω = 1.50	ϕ = 1.00
OVS and SSLP	Ω = 1.76	ϕ = 0.85
LSL	Ω = 2.14	ϕ = 0.70

Note: Slip-critical bolt values assume no more than one filler has been provided or bolts have been added to distribute loads in the fillers.
See AISC *Specification* Sections J3.8 and J5 for provisions when fillers are present.
For Class B faying surfaces, multiply the tabulated available strength by 1.67.

For bearing of plate material at bolt holes:

$$R_a \le R_n / \Omega \text{ or } R_u \le \phi R_n$$

where $R_u = \Sigma \gamma_i R_i$

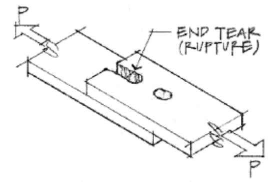


Figure 10.11 End tear-out.

- deformation at bolt hole is a concern

$$R_n = 1.2 L_c t F_u \le 2.4 d t F_u$$

- deformation at bolt hole is not a concern

$$R_n = 1.5 L_c t F_u \le 3.0 d t F_u$$

- long slotted holes with the slot perpendicular to the load

$$R_n = 1.0 L_c t F_u \le 2.0 d t F_u$$

where R_n = the nominal bearing strength
F_u = specified minimum tensile strength
L_c = clear distance between the edges of the hole and the next hole or edge in the direction of the load
d = nominal bolt diameter
t = thickness of connected material

ϕ = 0.75 (LRFD) Ω = 2.00 (ASD)

The *minimum* edge desistance from the center of the outer most bolt to the edge of a member is generally 1¾ times the bolt diameter for the sheared edge and 1¼ times the bolt diameter for the rolled or gas cut edges.

The *maximum* edge distance should not exceed 12 times the thickness of thinner member or 6 in.

Standard bolt hole spacing is 3 in. with the minimum spacing of $2\frac{2}{3}$ times the diameter of the bolt, d_b. Common edge distance from the center of last hole to the edge is 1¼ in..

Table 7-4
Available Bearing Strength at Bolt Holes Based on Bolt Spacing
kips/in. thickness

Hole Type	Bolt Spacing, s, in.	F_u, ksi	5/8 r_n/Ω ASD	5/8 ϕr_n LRFD	3/4 r_n/Ω ASD	3/4 ϕr_n LRFD	7/8 r_n/Ω ASD	7/8 ϕr_n LRFD	1 r_n/Ω ASD	1 ϕr_n LRFD
STD SSLT	$2^2/_3\ d_b$	58	34.1	51.1	41.3	62.0	48.6	72.9	55.8	83.7
		65	38.2	57.3	46.3	69.5	54.4	81.7	62.6	93.8
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	67.4	101
		65	48.8	73.1	58.5	87.8	68.3	102	75.6	113
SSLP	$2^2/_3\ d_b$	58	27.6	41.3	34.8	52.2	42.1	63.1	47.1	70.7
		65	30.9	46.3	39.0	58.5	47.1	70.7	52.8	79.2
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	58.7	88.1
		65	48.8	73.1	58.5	87.8	68.3	102	65.8	98.7
OVS	$2^2/_3\ d_b$	58	29.7	44.6	37.0	55.5	44.2	66.3	49.3	74.0
		65	33.3	50.0	41.4	62.2	49.6	74.3	55.3	82.9
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	60.9	91.4
		65	48.8	73.1	58.5	87.8	68.3	102	68.3	102
LSLP	$2^2/_3\ d_b$	58	3.62	5.44	4.35	6.53	5.08	7.61	5.80	8.70
		65	4.06	6.09	4.88	7.31	5.69	8.53	6.50	9.75
	3 in.	58	43.5	65.3	39.2	58.7	28.3	42.4	17.4	26.1
		65	48.8	73.1	43.9	65.8	31.7	47.5	19.5	29.3
LSLT	$2^2/_3\ d_b$	58	28.4	42.6	34.4	51.7	40.5	60.7	46.5	69.8
		65	31.8	47.7	38.6	57.9	45.4	68.0	52.1	78.2
	3 in.	58	36.3	54.4	43.5	65.3	50.8	76.1	56.2	84.3
		65	40.6	60.9	48.8	73.1	56.9	85.3	63.0	94.5
STD, SSLT, SSLP, OVS, LSLP	$s \ge s_{full}$	58	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104
		65	48.8	73.1	58.5	87.8	68.3	102	78.0	117
LSLT	$s \ge s_{full}$	58	36.3	54.4	43.5	65.3	50.8	76.1	58.0	87.0
		65	40.6	60.9	48.8	73.1	56.9	85.3	65.0	97.5
Spacing for full bearing strength s_{full}[a], in.		STD, SSLT, LSLT	$1^{15}/_{16}$		$2^5/_{16}$		$2^{11}/_{16}$		$3^1/_{16}$	
		OVS	$2^1/_{16}$		$2^7/_{16}$		$2^{13}/_{16}$		$3^1/_4$	
		SSLP	$2^1/_8$		$2^1/_2$		$2^7/_8$		$3^5/_{16}$	
		LSLP	$2^{13}/_{16}$		$3^3/_8$		$3^{15}/_{16}$		$4^1/_2$	
Minimum Spacing[a] = $2^2/_3 d$, in.			$1^{11}/_{16}$		2		$2^5/_{16}$		$2^{11}/_{16}$	

STD = standard hole
SSLT = short-slotted hole oriented transverse to the line of force
SSLP = short-slotted hole oriented parallel to the line of force
OVS = oversized hole
LSLP = long-slotted hole oriented parallel to the line of force
LSLT = long-slotted hole oriented transverse to the line of force

ASD	LRFD	
Ω = 2.00	ϕ = 0.75	Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC *Specification* Section J3.10. [a] Decimal value has been rounded to the nearest sixteenth of an inch.

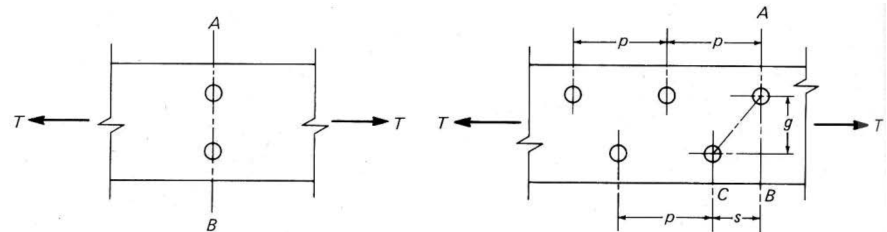
Table 7-5 (continued)
Available Bearing Strength at Bolt Holes Based on Edge Distance
kips/in. thickness

Hole Type	Edge Distance L_e, in.	F_u, ksi	5/8 r_n/Ω ASD	5/8 ϕr_n LRFD	3/4 r_n/Ω ASD	3/4 ϕr_n LRFD	7/8 r_n/Ω ASD	7/8 ϕr_n LRFD	1 r_n/Ω ASD	1 ϕr_n LRFD
STD SSLT	1 1/4	58	31.5	47.3	29.4	44.0	27.2	40.8	25.0	37.5
		65	35.3	53.0	32.9	49.4	30.5	45.7	28.0	42.0
	2	58	43.5	65.3	52.2	78.3	53.3	79.9	51.1	76.7
		65	48.8	73.1	58.5	87.8	59.7	89.6	57.3	85.9
SSLP	1 1/4	58	28.3	42.4	26.1	39.2	23.9	35.9	20.7	31.0
		65	31.7	47.5	29.3	43.9	26.8	40.2	23.2	34.7
	2	58	43.5	65.3	52.2	78.3	50.0	75.0	46.8	70.1
		65	48.8	73.1	58.5	87.8	56.1	84.1	52.4	78.6
OVS	1 1/4	58	29.4	44.0	27.2	40.8	25.0	37.5	21.8	32.6
		65	32.9	49.4	30.5	45.7	28.0	42.0	24.4	36.6
	2	58	43.5	65.3	52.2	78.3	51.1	76.7	47.9	71.8
		65	48.8	73.1	58.5	87.8	57.3	85.9	53.6	80.4
LSLP	1 1/4	58	16.3	24.5	10.9	16.3	5.44	8.16	—	—
		65	18.3	27.4	12.2	18.3	6.09	9.14	—	—
	2	58	42.4	63.6	37.0	55.5	31.5	47.3	26.1	39.2
		65	47.5	71.3	41.4	62.2	35.3	53.0	29.3	43.9
LSLT	1 1/4	58	26.3	39.4	24.5	36.7	22.7	34.0	20.8	31.3
		65	29.5	44.2	27.4	41.1	25.4	38.1	23.4	35.0
	2	58	36.3	54.4	43.5	65.3	44.4	66.6	42.6	63.9
		65	40.6	60.9	48.8	73.1	49.8	74.6	47.7	71.6
STD, SSLT, SSLP, OVS, LSLP	$L_e \geq L_{e\,full}$	58	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104
		65	48.8	73.1	58.5	87.8	68.3	102	78.0	117
LSLT	$L_e \geq L_{e\,full}$	58	36.3	54.4	43.5	65.3	50.8	76.1	58.0	87.0
		65	40.6	60.9	48.8	73.1	56.9	85.3	65.0	97.5
Edge distance for full bearing strength $L_e \geq L_{e\,full}$[a], in.		STD, SSLT, LSLT	1 5/8		1 15/16		2 1/4		2 9/16	
		OVS	1 11/16		2		2 5/16		2 5/8	
		SSLP	1 11/16		2		2 5/16		2 11/16	
		LSLP	2 1/16		2 7/16		2 7/8		3 1/4	

STD = standard hole
SSLT = short-slotted hole oriented transverse to the line of force
SSLP = short-slotted hole oriented parallel to the line of force
OVS = oversized hole
LSLP = long-slotted hole oriented parallel to the line of force
LSLT = long-slotted hole oriented transverse to the line of force

ASD	LRFD	
$\Omega = 2.00$	$\phi = 0.75$	— indicates spacing less than minimum spacing required per AISC *Specification* Section J3.3. Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC *Specification* Section J3.10. [a] Decimal value has been rounded to the nearest sixteenth of an inch.

<u>Tension Member Design</u>



In steel tension members, there may be bolt holes that reduce the size of the cross section.

g refers to the row spacing or *gage*
p refers to the bolt spacing or *pitch*
s refers to the longitudinal spacing of two consecutive holes

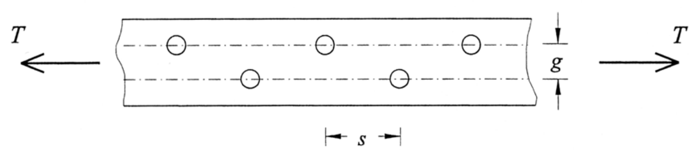
Effective Net Area:

The smallest effective are must be determined by subtracting the bolt hole areas. With staggered holes, the shortest length must be evaluated.

A series of bolts can also transfer a portion of the tensile force, and some of the effective net areas see reduced stress.

The effective net area, A_e, is determined from the net area, A_n, multiplied by a shear lag factor, U, which depends on the element type and connection configuration. If a portion of a connected member is not fully connected (like the leg of an angle), the unconnected part is not subject to the full stress and the shear lag factor can range from 0.6 to 1.0: $A_e = A_n U$

The staggered hole path area is determined by:



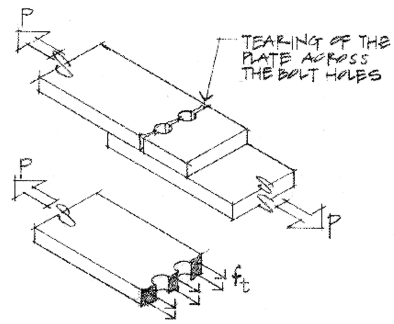
$$A_n = A_g - A_{of\ all\ holes} + t\Sigma\frac{s}{4g}$$

where t is the plate thickness, s is each stagger spacing, and g is the gage spacing.

For tension elements:

$$R_a \le R_n / \Omega \;\; or \;\; R_u \le \phi R_n$$

where $R_u = \Sigma \gamma_i R_i$



1. yielding $\quad R_n = F_y A_g$

   ϕ = 0.90 (LRFD) $\quad \Omega$ = 1.67 (ASD)

2. rupture $\quad R_n = F_u A_e$

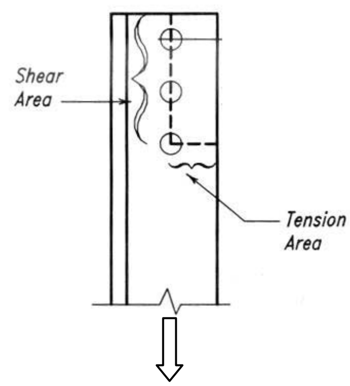
   ϕ = 0.75 (LRFD) $\quad \Omega$ = 2.00 (ASD)

where A_g = the gross area of the member (excluding holes)
A_e = the effective net area (with holes, etc.)
F_y = the yield strength of the steel
F_u = the tensile strength of the steel (ultimate)

For shear elements:

$$R_a \le R_n / \Omega \;\; or \;\; R_u \le \phi R_n$$

where $R_u = \Sigma \gamma_i R_i$



1. yielding $\quad R_n = 0.6 F_y A_g$

   ϕ = 1.00 (LRFD) $\quad \Omega$ = 1.50 (ASD)

2. rupture $\quad R_n = 0.6 F_u A_{nv}$

   ϕ = 0.75 (LRFD) $\quad \Omega$ = 2.00 (ASD)

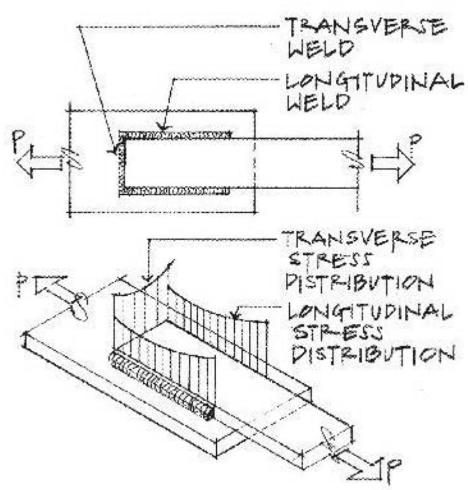
where A_g = the gross area of the member (excluding holes)
A_{nv} = the net area subject to shear (with holes, etc.)
F_y = the yield strength of the steel
F_u = the tensile strength of the steel (ultimate)

Welded Connections

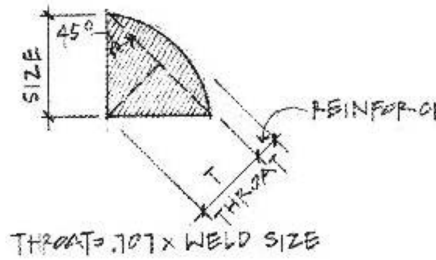
Weld designations include the strength in the name, i.e. E70XX has F_y = 70 ksi. Welds are weakest in shear and are assumed to always fail in the shear mode.

The throat size, T, of a fillet weld is determined trigonometry by: T = 0.707 ×weld size*

* When the submerged arc weld process is used, welds over 3/8" will have a throat thickness of 0.11 in. larger than the formula.



Weld sizes are limited by the size of the parts being put together and are given in AISC manual table J2.4 along with the allowable strength per length of fillet weld, referred to as *S*.



The maximum size of a fillet weld permitted along edges of connected parts shall be:

- Material less than ¼ in. thick, not greater than the thickness of the material.

- Material ¼ in. or more in thickness, not greater than the thickness of the material minus 1/16 in., unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness.

The *minimum length* of a fillet weld is 4 times the nominal size. If it is not, then the weld size used for design is ¼ the length.

Intermittent fillet welds cannot be less than four times the weld size, not to be less than 1 ½".

TABLE J2.4
Minimum Size of Fillet Welds

Material Thickness of Thicker Part Joined (in.)	Minimum Size of Fillet Weld[a] (in.)
To ¼ inclusive	⅛
Over ¼ to ½	3/16
Over ½ to ¾	¼
Over ¾	5/16

[a]Leg dimension of fillet welds. Single-pass welds must be used.

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<u>*For fillet welds:*</u> $R_a \le R_n / \Omega$ or $R_u \le \phi R_n$

where $R_u = \Sigma \gamma_i R_i$

for the weld metal: $R_n = 0.6 F_{EXX} Tl = Sl$

ϕ = 0.75 (LRFD) Ω = 2.00 (ASD)

where:

T is throat thickness
l is length of the weld

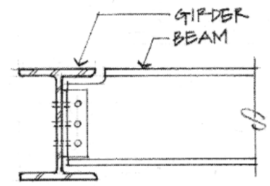
For a connected part, the other limit states for the base metal, such as tension yield, tension rupture, shear yield, or shear rupture **must** be considered.

Available Strength of Fillet Welds per inch of weld (ϕS)

Weld Size (in.)	E60XX (k/in.)	E70XX (k/in.)
3/16	3.58	4.18
¼	4.77	5.57
5/16	5.97	6.96
3/8	7.16	8.35
7/16	8.35	9.74
½	9.55	11.14
5/8	11.93	13.92
¾	14.32	16.70

(not considering increase in throat with submerged arc weld process)

Framed Beam Connections

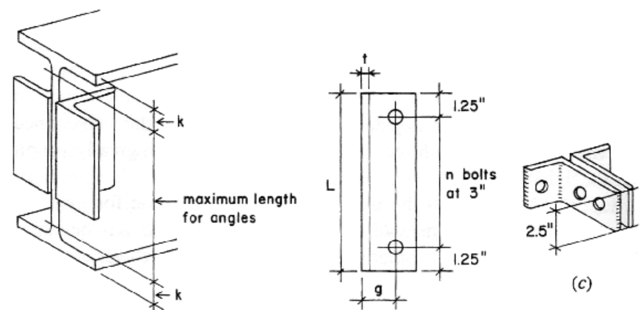


Coping is the term for cutting away part of the flange to connect a beam to another beam using welded or bolted angles.

AISC provides tables that give bolt and angle available strength knowing number of bolts, bolt type, bolt diameter, angle leg thickness, hole type and coping, *and* the wide flange beam being connected. For the connections the limit-state of bolt shear, bolts bearing on the angles, shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles, and bolt bearing on the beam web are considered.

Group A bolts include A325, while Group B includes A490.

here are also tables for bolted/welded double-angle connections and all-welded double-angle connections.



(a) (b) (c)

Sample AISC Table for Bolt and Angle Available Strength in All-Bolted Double-Angle Connections

Beam: F_y = 50 ksi, F_u = 65 ksi
Angle: F_y = 36 ksi, F_u = 58 ksi

Table 10-1 (continued)
All-Bolted Double-Angle Connections
3/4-in. Bolts

Bolt and Angle Available Strength, kips

4 Rows — W24, 21, 18, 16

Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD	67.1	101	83.9	126	95.5	143	95.5	143
	X	STD	67.1	101	83.9	126	101	151	120	180
	SC Class A	STD	50.6	75.9	50.6	75.9	50.6	75.9	50.6	75.9
		OVS	43.1	64.5	43.1	64.5	43.1	64.5	43.1	64.5
		SSLT	50.6	75.9	50.6	75.9	50.6	75.9	50.6	75.9
	SC Class B	STD	67.1	101	83.9	126	84.4	127	84.4	127
		OVS	65.3	97.9	71.9	108	71.9	108	71.9	108
		SSLT	65.8	98.7	82.2	123	84.4	127	84.4	127
Group B	N	STD	67.1	101	83.9	126	101	151	120	180
	X	STD	67.1	101	83.9	126	101	151	134	201
	SC Class A	STD	63.3	94.9	63.3	94.9	63.3	94.9	63.3	94.9
		OVS	53.9	80.7	53.9	80.7	53.9	80.7	53.9	80.7
		SSLT	63.3	94.9	63.3	94.9	63.3	94.9	63.3	94.9
	SC Class B	STD	67.1	101	83.9	126	101	151	105	158
		OVS	65.3	97.9	81.6	122	89.9	134	89.9	134
		SSLT	65.8	98.7	82.2	123	98.7	148	105	158

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
L_{eh}*, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
	L_{ev}, in.	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	167	250	175	262	156	234	164	246	164	245	172	257
	1 3/8	169	254	177	266	158	238	167	250	166	249	174	261
	1 1/2	171	257	180	269	161	241	169	254	168	253	177	265
	1 5/8	174	261	182	273	163	245	171	257	171	256	179	268
	2	181	272	189	284	171	256	179	268	178	267	186	279
	3	201	301	209	313	190	285	198	297	198	296	206	309
Coped at Both Flanges	1 1/4	156	234	156	234	146	219	146	219	156	234	156	234
	1 3/8	161	241	161	241	151	227	151	227	161	241	161	241
	1 1/2	166	249	166	249	156	234	156	234	166	249	166	249
	1 5/8	171	256	171	256	161	241	161	241	171	256	171	256
	2	181	272	185	278	171	256	176	263	178	267	185	278
	3	201	301	209	313	190	285	198	297	198	296	206	309
Uncoped		234	351	234	351	234	351	234	351	234	351	234	351

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	468	702

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

* Tabulated values include 1/4-in. reduction in end distance, L_{eh}, to account for possible underrun in beam length.

Note: Slip-critical bolt values assume no more than one filler has been provided or bolts have been added to distribute loads in the fillers.

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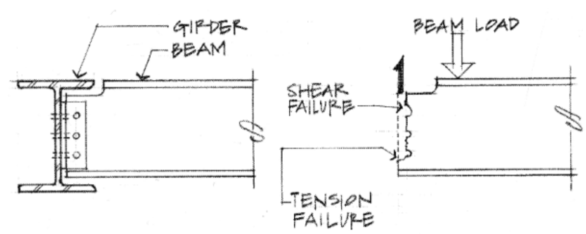
Limiting Strength or Stability States

In addition to resisting shear and tension in bolts and shear in welds, the connected materials may be subjected to shear, bearing, tension, flexure and even prying action. Coping can significantly reduce design strengths and may require web reinforcement. All the following must be considered:

- shear yielding
- shear rupture
- block shear rupture -
  failure of a block at a beam as a result of shear and tension
- tension yielding
- tension rupture
- local web buckling
- lateral torsional buckling



Block Shear Strength (or Rupture): $R_a \le R_n / \Omega$ or $R_u \le \phi R_n$

where $R_u = \Sigma \gamma_i R_i$

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$$

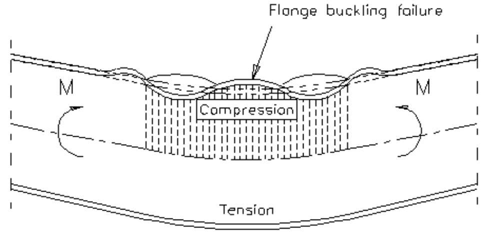
$\phi = 0.75$ (LRFD) $\Omega = 2.00$ (ASD)

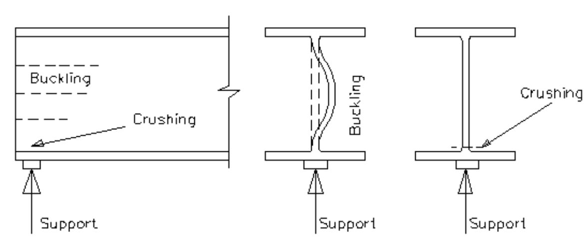
where:

A_{nv} is the net area subjected to shear
A_{nt} is the net area subjected to tension
A_{gv} is the gross area subjected to shear
U_{bs} = 1.0 when the tensile stress is uniform (most cases)
= 0.5 when the tensile stress is non-uniform

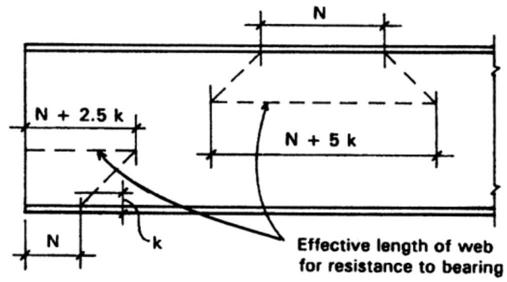
<u>Local Buckling in Steel I Beams– Web Crippling or Flange Buckling</u>

Concentrated forces on a steel beam can cause the web to buckle (called web crippling). Web stiffeners under the beam loads and bearing plates at the supports reduce that tendency. Web stiffeners also prevent the web from shearing in plate girders.





The maximum support load and interior load can be determined from:



$$P_{n(\max-\text{end})} = (2.5k + N)F_{yw}t_w$$

$$P_{n(\text{interior})} = (5k + N)F_{yw}t_w$$

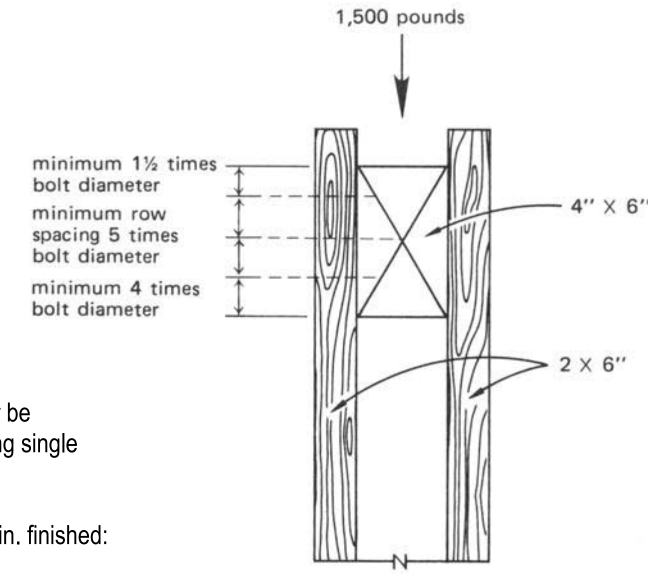
where t_w = thickness of the web
N = bearing length
k = dimension to fillet found in beam section tables

ϕ = 1.00 (LRFD) Ω = 1.50 (ASD)

Examples:
Connections and Tension Members

Example 1

A nominal 4 x 6 in. redwood beam is to be supported by two 2 x 6 in. members acting as a spaced column. The minimum spacing and edge distances for the ½ inch bolts are shown. How many ½ in. bolts will be required to safely carry a load of 1500 lb? Use the chart provided.



SOLUTION:
The table requires that the length of the bolt in the main wood member be known, along with the diameter of bolt in inches, and if the bolt is seeing single shear or double shear and what direction it is bearing on the grain.

The main member is the beam. The 4 in. nominal size is actually 3 ½ in. finished:

The bolt is ½ inches in diameter, and sees **two** planes of shear at the interfaces with the 2 x 6's. This means double shear.

The vertical force is pushing the beam down onto the bolt, so the bolt is in contact with the grain running horizontally. That means the bolt is bearing **perpendicular** to the grain, and we should look up *q*.

The allowable load per bolt multiplied by the number of bolts will determine the capacity, which we need to be at least 1500 lb:

$q \times n \geq P$

knowing q & P, the equation for n becomes:

$n \geq \frac{P}{q} = \frac{1500lb}{980\, {}^{lb}\!/_{bolt}} = 1.5$ bolt *rounded up* = 2 bolts required

Table: Holding Power of Bolts

p = Safe loads parallel to grain in pounds
q = Safe loads perpendicular to grain in pounds

Length of Bolt in Main Wood Member[3] (in Inches)		DIAMETER OF BOLT (IN INCHES)								
		3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4	1 1/2
1½	Single *p*	325	470	590	710	830	945			
	Shear *q*	185	215	245	270	300	325			
	Double *p*	650	940	1180	1420	1660	1890			
	Shear *q*	370	430	490	540	600	650			
2½	Single *p*		630	910	1155	1370	1575			
	Shear *q*		360	405	450	495	540			
	Double *p*	710	1260	1820	2310	2740	3150			
	Shear *q*	620	720	810	900	990	1080			
3½	Single *p*			990	1400	1790	2135	2455	2740	3305
	Shear *q*			565	630	695	760	825	895	1020
	Double *p*	710	1270	1980	2800	3580	4270	4910	5480	6610
	Shear *q*	640	980	1130	1260	1390	1520	1650	1780	2040

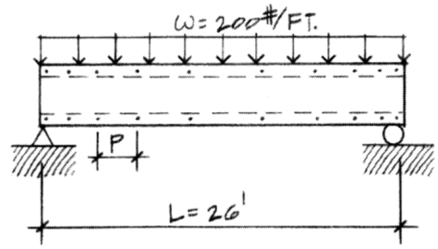
[1]Tabulated values are on a normal load-duration basis and apply to joints made of seasoned lumber used in dry locations. See U.B.C. Standard No. 25-17 for other service conditions.

[2]Double shear values are for joints consisting of three wood members in which the side members are one half the thickness of the main member. Single shear values are for joints consisting of two wood members having a minimum thickness not less than that specified.

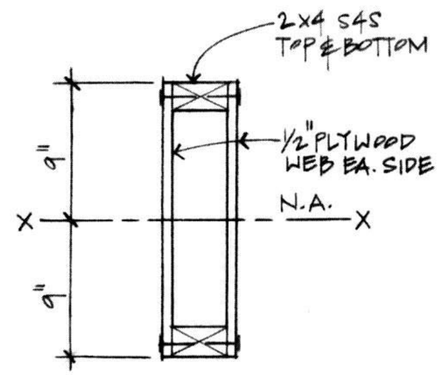
[3]The length specified is the length of the bolt in the main member of double shear joints or the length of the bolt in the thinner member of single shear joints.

[4]See U.B.C. Standard No. 25-17 for wood-to-metal bolted joints.

Example 2



8.11 A built-up plywood box beam with 2 × 4 S4S top and bottom flanges is held together by nails. Determine the pitch (spacing) of the nails if the beam supports a uniform load of 200 #/ft. along the 26-foot span. Assume the nails have a shear capacity of 80# each.



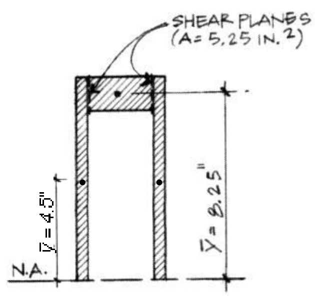
Solution:

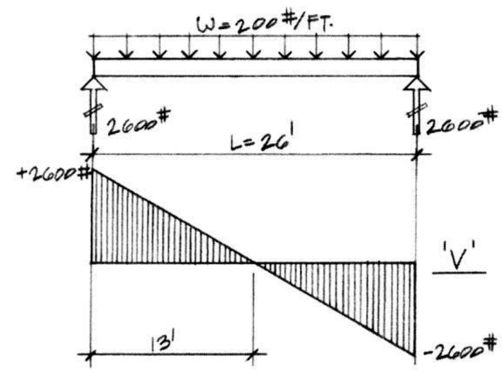
Construct the shear (V) diagram to obtain the critical shear condition and its location

Note that the condition of shear is critical at the supports, and the shear intensity decreases as you approach the center line of the beam. This would indicate that the nail spacing P varies from the support to midspan. Nails are closely spaced at the support, but increasing spacing occurs toward midspan, following the shear diagram.

$$f_v = \frac{VQ}{Ib}$$

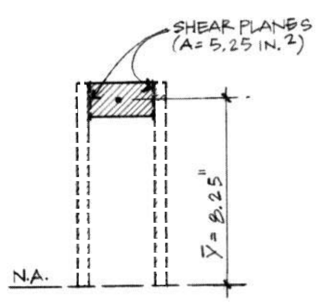
$$I_x = \frac{(4.5'')(18'')^3}{12} - \frac{(3.5'')(15'')^3}{12} = 1{,}202.6\text{ in.}^4$$





$Q = \Sigma A\bar{y} = (9")(½")(4.5")+(9")(½")(4.5")+(1.5")(3.5")(8.25") = 83.8\text{ in}^3$

$$f_{v-\max} = \frac{(2{,}600\#)(83.3in.^3)}{(1{,}202.6in.^4)(\tfrac{1}{2}"+\tfrac{1}{2}")} = 180.2\,psi$$



$Q = A\bar{y} = (5.25\text{ in.}^2)(8.25'') = 43.3\text{ in.}^3$

Shear force $= f_v \times A_v$

where:

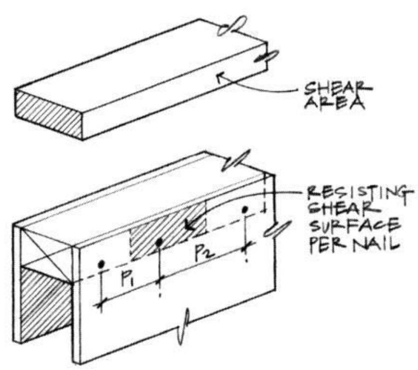
A_v = shear area

Assume:

F = Capacity of two nails (one each side) at the flange; representing two shear surfaces

$$(n)F \geq f_v \times b \times p = \frac{VQ}{Ib} \times bp$$

$$\therefore\ (n)F \geq p \times \frac{VQ}{I}; \quad p \leq \frac{(n)FI}{VQ}$$

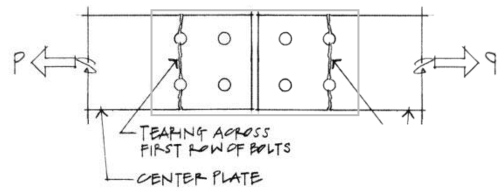


At the maximum shear location (support) where V = 2,600#

$$p = \frac{(2\text{ nails} \times 80\text{ \#/nail})(1{,}202.6\text{ in.}^4)}{(2{,}600\#)(43.3\text{ in.}^3)} = 1.71''$$

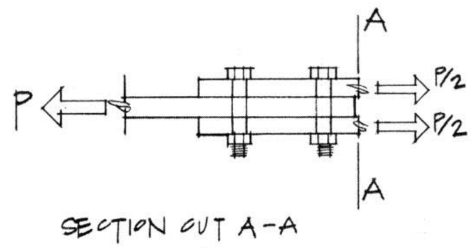
Example 3

10.2 The butt splice shown in Figure 10.22 uses two 8 × ⅜" plates to "sandwich" in the 8 × ½" plates being joined. Four ⅞"ϕ A325-SC bolts are used on both sides of the splice. Assuming A36 steel and standard round holes, determine the allowable capacity of the connection.



SOLUTION:

Shear, bearing and net tension will be checked to determine the critical conditions that governs the capacity of the connection. (The edge distance to the holes is presumed to be adequate.)

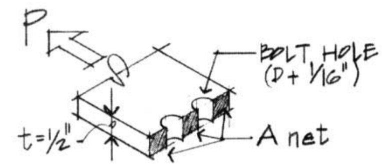


Shear: Using the AISC available shear in Table 7-3 (Group A):

ϕR_n = 26.4 k/bolt x 4 bolts = 105.6 k

Bearing: Using the AISC available bearing in Table 7-4:

There are 4 bolts bearing on the center (1/2") plate, while there are 4 bolts bearing on a total width of two sandwich plates (3/4" total). The thinner bearing width will govern. Assume 3 in. spacing (center to center) of bolts. For A36 steel, F_u = 58 ksi.



ϕR_n = 91.4 k/bolt/in. x 0.5 in. x 4 bolts = 182.8 k

Tension: The center plate is critical, again, because its thickness is less than the combined thicknesses of the two outer plates. We must consider tension yielding and tension rupture:

$\phi R_n = \phi F_y A_g$ *and* $\phi R_n = \phi F_u A_e$ where $A_e = A_{net}U$

A_g = 8 in. x ½ in. = 4 in^2

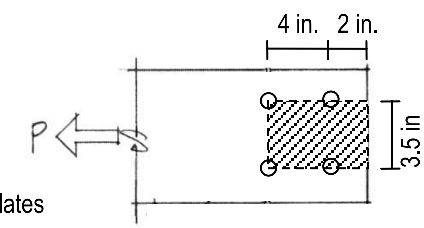
The holes are considered 1/8 in. larger than the bolt hole diameter = (7/8 + 1/8) = 1.0 in.

A_n = (8 in. – 2 holes x 1.0 in.) x ½ in. = 3.0 in^2

The whole cross section sees tension, so the shear lag factor U = 1

$\phi F_y A_g$ = 0.9 x 36 ksi x 4 in^2 = 129.6 k

$\phi F_u A_e$ = 0.75 x 58 ksi x (1) x 3.0 in^2 = 130.5 k



Block Shear Rupture: It is possible for the center plate to rip away from the sandwich plates leaving the block (shown hatched) behind:

$\phi R_n = \phi(0.6F_uA_{nv} + U_{bs}F_uA_{nt}) \leq \phi(0.6F_yA_{gv} + U_{bs}F_uA_{nt})$

where A_{nv} is the area resisting shear, A_{nt} is the area resisting tension, A_{gv} is the gross area resisting shear, and U_{bs} = 1 when the tensile stress is uniform.

A_{gv} = (4 + 2 in.) x ½ in. = 3 in^2

$A_{nv} = A_{gv}$ – 1 ½ holes area = 3 in^2 – 1.5 x 1 in. x ½ in. = 2.25 in^2

A_{nt} = 3.5 in. x t – 1 holes = 3.5 in. x ½ in – 1 x 1 in. x ½ in. = 1.25 in^2

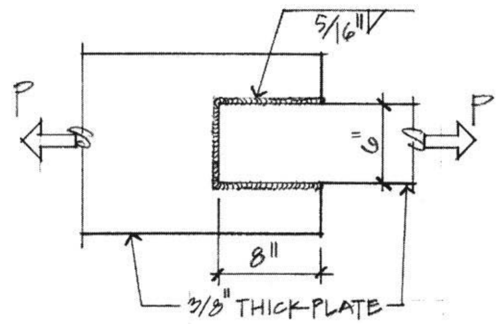
$\phi(0.6F_uA_{nv} + U_{bs}F_uA_{nt})$ = 0.75 x (0.6 x 58 ksi x 2.25 in^2 + 1 x 58 ksi x 1.25 in^2) = 113.1 k

$\phi(0.6F_yA_{gv} + U_{bs}F_uA_{nt})$ = 0.75 x (0.6 x 36 ksi x 3 in^2 + 1 x 58 ksi x 1.25 in^2) = 103.0 k

The maximum connection capacity (*smallest value*) is governed by block shear rupture.

ϕR_n = 103.0 k

Example 4



10.7 Determine the capacity of the connection in Figure 10.44 assuming A36 steel with E70XX electrodes.

Solution:

Capacity of weld:

For a 5⁄16″ fillet weld, ϕS = 6.96 k/in

Weld length = 8 in + 6 in + 8 in = 22 in.

Weld capacity = 22″ × 6.96 k/in = 153.1 k

Capacity of plate: 0.9 x 36 k/in² x 3/8″ x 6″= 72.9 k

$\phi P_n = \phi F_y A_g \quad \phi = 0.9$

Plate capacity = 0.9 x 36 k/in² x 3/8″ x 6″= 72.9 k

∴ Plate capacity governs, P_u = 72.9 k

The weld size used is obviously too strong. What size, then, can the weld be reduced to so that the weld strength is more compatible to the plate capacity? To make the weld capacity ≈ plate capacity:

22″ × (weld capacity per in.) = 72.9 k

$$\text{Weld capacity per inch} = \frac{72.9\text{ k}}{22\text{ in.}} = 3.31\text{ k/in.}$$

From Available Strength table, use 3/16″ weld (ϕS = 4.18 k/in.)

Minimum size fillet = 3⁄16″ based on a 3⁄8″ thick plate.

Available Strength of Fillet Welds per inch of weld (ϕS)

Weld Size (in.)	E60XX (k/in.)	E70XX (k/in.)
3⁄16	3.58	4.18
¼	4.77	5.57
5⁄16	5.97	6.96
3⁄8	7.16	8.35
7⁄16	8.35	9.74
½	9.55	11.14
5⁄8	11.93	13.92
¾	14.32	16.70

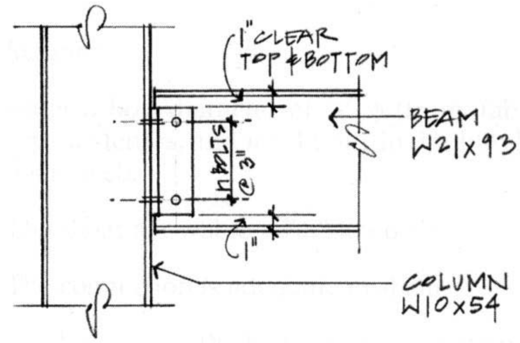
(not considering increase in throat with submerged arc weld process)

Table 7-1
Available Shear Strength of Bolts, kips

Nominal Bolt Diameter, d, in.					5/8		3/4		7/8		1	
Nominal Bolt Area, in.²					0.307		0.442		0.601		0.785	
ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	27.0	40.5	S	8.29	12.4	11.9	17.9	16.2	24.3	21.2	31.8
				D	16.6	24.9	23.9	35.8	32.5	48.7	42.4	63.6
	X	34.0	51.0	S	10.4	15.7	15.0	22.5	20.4	30.7	26.7	40.0
				D	20.9	31.3	30.1	45.1	40.9	61.3	53.4	80.1
Group B	N	34.0	51.0	S	10.4	15.7	15.0	22.5	20.4	30.7	26.7	40.0
				D	20.9	31.3	30.1	45.1	40.9	61.3	53.4	80.1
	X	42.0	63.0	S	12.9	19.3	18.6	27.8	25.2	37.9	33.0	49.5
				D	25.8	38.7	37.1	55.7	50.5	75.7	65.9	98.9
A307	–	13.5	20.3	S	4.14	6.23	5.97	8.97	8.11	12.2	10.6	15.9
				D	8.29	12.5	11.9	17.9	16.2	24.4	21.2	31.9
Nominal Bolt Diameter, d, in.					1 1/8		1 1/4		1 3/8		1 1/2	
Nominal Bolt Area, in.²					0.994		1.23		1.48		1.77	
ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	27.0	40.5	S	26.8	40.3	33.2	49.8	40.0	59.9	47.8	71.7
				D	53.7	80.5	66.4	99.6	79.9	120	95.6	143
	X	34.0	51.0	S	33.8	50.7	41.8	62.7	50.3	75.5	60.2	90.3
				D	67.6	101	83.6	125	101	151	120	181
Group B	N	34.0	51.0	S	33.8	50.7	41.8	62.7	50.3	75.5	60.2	90.3
				D	67.6	101	83.6	125	101	151	120	181
	X	42.0	63.0	S	41.7	62.6	51.7	77.5	62.2	93.2	74.3	112
				D	83.5	125	103	155	124	186	149	223
A307	–	13.5	20.3	S	13.4	20.2	16.6	25.0	20.0	30.0	23.9	35.9
				D	26.8	40.4	33.2	49.9	40.0	60.1	47.8	71.9
ASD	LRFD	For end loaded connections greater than 38 in., see AISC *Specification* Table J3.2 footnote b.										
$\Omega = 2.00$	$\phi = 0.75$											

Example 5

The steel used in the connection and beams is A992 with F_y = 50 ksi, and F_u = 65 ksi. Using A490-N bolt material, determine the maximum capacity of the connection based on shear in the bolts, bearing in all materials and pick the number of bolts and angle length (not staggered). Use A36 steel for the angles.



W21x93: d = 21.62 in, t_w = 0.58 in, t_f = 0.93 in
W10x54: t_f = 0.615 in

SOLUTION:

The maximum length the angles can be depends on how it fits between the top and bottom flange with some clearance allowed for the fillet to the flange, and getting an air wrench in to tighten the bolts. This example uses 1" of clearance:

Available length = beam depth – both flange thicknesses – 1" clearance at top & 1" at bottom

= 21.62 in – 2(0.93 in) – 2(1 in) = 17.76 in.

With the spaced at 3 in. and 1 ¼ in. end lengths (each end), the maximum number of bolts can be determined:

Available length ≥ 1.25 in. + 1.25 in. + 3 in. x (number of bolts – 1)

number of bolts ≤ (17.76 in – 2.5 in. - (-3 in.))/3 in. = 6.1, so 6 bolts.

It is helpful to have the All-bolted Double-Angle Connection Tables 10-1. They are available for ¾", 7/8", and 1" bolt diameters and list angle thicknesses of ¼", 5/16", 3/8", and ½". Increasing the angle thickness is likely to increase the angle strength, although the limit states include shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles.

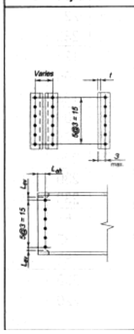
For these diameters the available **shear** (double) from Table 7-1 for 6 bolts is (6)41.5 k/bolt = 270.6 kips, (6)61.3 k/bolt = 367.8 kips, and (6)80.1 k/bolt = 480.6 kips.

Tables 10-1 (not all provided here) list a bolt and angle available strength of 271 kips for the ¾" bolts, 296 kips for the 7/8" bolts, and 281 kips for the 1" bolts. It appears that increasing the bolt diameter to 1" will not gain additional load. Use 7/8" bolts.

Beam: F_y = 50 ksi, F_u = 65 ksi
Angle: F_y = 36 ksi, F_u = 58 ksi

Table 10-1 (continued)
All-Bolted Double-Angle Connections
7/8-in. Bolts

Bolt and Angle Available Strength, kips



6 Rows W40, 36, 33, 30, 27, 24, 21	Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD	98.6	148	123	185	148	222	195	292
		X	STD	98.6	148	123	185	148	222	197	296
		SC Class A	STD	98.6	148	106	159	106	159	106	159
			OVS	90.1	135	90.1	135	90.1	135	90.1	135
			SSLT	97.3	146	106	159	106	159	106	159
		SC Class B	STD	98.6	148	123	185	148	222	176	264
			OVS	93.5	140	117	175	140	210	150	225
			SSLT	97.3	146	122	182	146	219	176	264
	Group B	N	STD	98.6	148	123	185	148	222	197	296
		X	STD	98.6	148	123	185	148	222	197	296
		SC Class A	STD	98.6	148	123	185	133	199	133	199
			OVS	93.5	140	113	169	113	169	113	169
			SSLT	97.3	146	122	182	133	199	133	199
		SC Class B	STD	98.6	148	123	185	148	222	197	296
			OVS	93.5	140	117	175	140	210	187	281
			SSLT	97.3	146	122	182	146	219	195	292

ϕR_n = 368.7 kips for double shear of 7/8" bolts ϕR_n = 296 kips for limit state in angles

We also need to evaluate **bearing** of bolts on the beam web, and column flange where there are bolt holes. Table 7-5 provides available bearing strength for the material type, bolt diameter, hole type, and spacing per inch of material thicknesses.

a) Bearing for beam web: There are *6* bolt holes through the beam web. This is typically the critical bearing limit value because there are two angle legs that resist bolt bearing and twice as many bolt holes to the column. The material is A992 (F_u = 65 ksi), 0.58" thick, with 7/8" bolt diameters at 3 in. spacing.

ϕR_n = 6 bolts·(102 k/bolt/inch)·(0.58 in) = 355.0 kips

b) Bearing for column flange: There are 12 bolt holes through the column. The material is A992 (F_u = 65 ksi), 0.615" thick, with 1" bolt diameters.

ϕR_n = 12 bolts·(102 k/bolt/inch)·(0.615 in) = 752.8 kips

Although, the bearing in the beam web is the smallest at 355 kips, with the shear on the bolts even smaller at 324.6 kips, *the maximum capacity for the simple-shear connector is 296 kips* limited by the critical capacity of the angles.

Group A Bolts

A325, A325M
F1858
A354 Grade BC
A449

Table 7-3
Slip-Critical Connections
Available Shear Strength, kips
(Class A Faying Surface, $\mu = 0.30$)

Group A Bolts

Hole Type	Loading	Nominal Bolt Diameter, d, in.							
		5/8		3/4		7/8		1	
		Minimum Group A Bolt Pretension, kips							
		19		28		39		51	
		r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	S	4.29	6.44	6.33	9.49	8.81	13.2	11.5	17.3
	D	8.59	12.9	12.7	19.0	17.6	26.4	23.1	34.6
OVS/SSLP	S	3.66	5.47	5.39	8.07	7.51	11.2	9.82	14.7
	D	7.32	10.9	10.8	16.1	15.0	22.5	19.6	29.4
LSL	S	3.01	4.51	4.44	6.64	6.18	9.25	8.08	12.1
	D	6.02	9.02	8.87	13.3	12.4	18.5	16.2	24.2

Hole Type	Loading	Nominal Bolt Diameter, d, in.							
		1 1/8		1 1/4		1 3/8		1 1/2	
		Minimum Group A Bolt Pretension, kips							
		56		71		85		103	
		r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	S	12.7	19.0	16.0	24.1	19.2	28.8	23.3	34.9
	D	25.3	38.0	32.1	48.1	38.4	57.6	46.6	69.8
OVS/SSLP	S	10.8	16.1	13.7	20.5	16.4	24.5	19.8	29.7
	D	21.6	32.3	27.4	40.9	32.7	49.0	39.7	59.4
LSL	S	8.87	13.3	11.2	16.8	13.5	20.2	16.3	24.4
	D	17.7	26.6	22.5	33.7	26.9	40.3	32.6	48.9

STD = standard hole
OVS = oversized hole
SSLT = short-slotted hole transverse to the line of force
SSLP = short-slotted hole parallel to the line of force
LSL = long-slotted hole transverse or parallel to the line of force

S = single shear
D = double shear

Hole Type	ASD	LRFD
STD and SSLT	$\Omega = 1.50$	$\phi = 1.00$
OVS and SSLP	$\Omega = 1.76$	$\phi = 0.85$
LSL	$\Omega = 2.14$	$\phi = 0.70$

Note: Slip-critical bolt values assume no more than one filler has been provided or bolts have been added to distribute loads in the fillers.
See AISC *Specification* Sections J3.8 and J5 for provisions when fillers are present.
For Class B faying surfaces, multiply the tabulated available strength by 1.67.

Table 7-4
Available Bearing Strength at Bolt Holes Based on Bolt Spacing
kips/in. thickness

Hole Type	Bolt Spacing, s, in.	F_u, ksi	Nominal Bolt Diameter, d, in.							
			5/8		3/4		7/8		1	
			r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD SSLT	$2^2/_3\ d_b$	58	34.1	51.1	41.3	62.0	48.6	72.9	55.8	83.7
		65	38.2	57.3	46.3	69.5	54.4	81.7	62.6	93.8
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	67.4	101
		65	48.8	73.1	58.5	87.8	68.3	102	75.6	113
SSLP	$2^2/_3\ d_b$	58	27.6	41.3	34.8	52.2	42.1	63.1	47.1	70.7
		65	30.9	46.3	39.0	58.5	47.1	70.7	52.8	79.2
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	58.7	88.1
		65	48.8	73.1	58.5	87.8	68.3	102	65.8	98.7
OVS	$2^2/_3\ d_b$	58	29.7	44.6	37.0	55.5	44.2	66.3	49.3	74.0
		65	33.3	50.0	41.4	62.2	49.6	74.3	55.3	82.9
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	60.9	91.4
		65	48.8	73.1	58.5	87.8	68.3	102	68.3	102
LSLP	$2^2/_3\ d_b$	58	3.62	5.44	4.35	6.53	5.08	7.61	5.80	8.70
		65	4.06	6.09	4.88	7.31	5.69	8.53	6.50	9.75
	3 in.	58	43.5	65.3	39.2	58.7	28.3	42.4	17.4	26.1
		65	48.8	73.1	43.9	65.8	31.7	47.5	19.5	29.3
LSLT	$2^2/_3\ d_b$	58	28.4	42.6	34.4	51.7	40.5	60.7	46.5	69.8
		65	31.8	47.7	38.6	57.9	45.4	68.0	52.1	78.2
	3 in.	58	36.3	54.4	43.5	65.3	50.8	76.1	56.2	84.3
		65	40.6	60.9	48.8	73.1	56.9	85.3	63.0	94.5
STD, SSLT, SSLP, OVS, LSLP	$s \ge s_{full}$	58	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104
		65	48.8	73.1	58.5	87.8	68.3	102	78.0	117
LSLT	$s \ge s_{full}$	58	36.3	54.4	43.5	65.3	50.8	76.1	58.0	87.0
		65	40.6	60.9	48.8	73.1	56.9	85.3	65.0	97.5
Spacing for full bearing strength s_{full}[a], in.		STD, SSLT, LSLT	1 15/16		2 5/16		2 11/16		3 1/16	
		OVS	2 1/16		2 7/16		2 13/16		3 1/4	
		SSLP	2 1/8		2 1/2		2 7/8		3 5/16	
		LSLP	2 13/16		3 3/8		3 15/16		4 1/2	
Minimum Spacing[a] $= 2^2/_3 d$, in.			1 11/16		2		2 5/16		2 11/16	

STD = standard hole
SSLT = short-slotted hole oriented transverse to the line of force
SSLP = short-slotted hole oriented parallel to the line of force
OVS = oversized hole
LSLP = long-slotted hole oriented parallel to the line of force
LSLT = long-slotted hole oriented transverse to the line of force

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC *Specification* Section J3.10.
[a] Decimal value has been rounded to the nearest sixteenth of an inch.

Wood Design

Notation:

a = name for width dimension
A = name for area
$A_{req'd\text{-}adj}$ = area required at allowable stress when shear is adjusted to include self weight
b = width of a rectangle
= name for height dimension
c_1 = coefficient for shear stress for a rectangular bar in torsion
C_C = curvature factor for laminated arches
C_D = load duration factor
C_{fu} = flat use factor for other than decks
C_F = size factor
C_H = shear stress factor
C_i = incising factor
C_L = beam stability factor
C_M = wet service factor
C_p = column stability factor for wood design
C_r = repetitive member factor for wood design
C_V = volume factor for glue laminated timber design
C_t = temperature factor for wood design
d = name for depth
d_{min} = dimension of timber critical for buckling
DL = shorthand for dead load
E = modulus of elasticity
f = stress (strength is a stress limit)
f_b = bending stress
$f_{from\ table}$ = tabular strength (from table)
f_p = bearing stress
f_r = radial stress for a glulam timber
f_v = shear stress
$f_{v\text{-}max}$ = maximum shear stress
F_b = tabular bending strength
= allowable bending stress
F'_b = allowable bending stress (adjusted)
F_c = tabular compression strength parallel to the grain
F'_c = allowable compressive stress (adjusted)
F^*_c = intermediate compressive stress for column design dependent on load duration
F_{cE} = theoretical allowed buckling stress
$F_{c\perp}$ = tabular compression strength perpendicular to the grain
F_p = tabular bearing strength parallel to the grain
= allowable bearing stress
F_R = allowable radial stress
F_t = tabular tensile strength
F_u = ultimate strength
F_v = tabular bending strength
= allowable shear stress
h = height of a rectangle
I = moment of inertia with respect to neutral axis bending
I_{trial} = moment of inertia of trial section
$I_{req'd}$ = moment of inertia required at limiting deflection
I_y = moment of inertia with respect to an y-axis
J = polar moment of inertia
K_{cE} = material factor for wood column design
L_e = effective length that can buckle for column design, as is ℓ_e
L = name for length or span length
LL = shorthand for live load
$LRFD$ = load and resistance factor design
M = internal bending moment
M_{max} = maximum internal bending moment
$M_{max\text{-}adj}$ = maximum bending moment adjusted to include self weight
P = name for axial force vector
R = radius of curvature of a deformed beam
= radius of curvature of a laminated arch
= name for a reaction force
S = section modulus
$S_{req'd}$ = section modulus required at allowable stress

$S_{req'd\text{-}adj}$ = section modulus required at allowable stress when moment is adjusted to include self weight
T = torque (axial moment)
V = internal shear force
V_{max} = maximum internal shear force
$V_{max\text{-}adj}$ = maximum internal shear force adjusted to include self weight
w = name for distributed load
$w_{self\ wt}$ = name for distributed load from self weight of member
$\Delta_{allowable}$ = allowable beam deflection
Δ_{limit} = allowable beam deflection limit
Δ_{max} = maximum beam deflection
κ = slenderness ratio limit for long columns
γ = density or unit weight
ρ = radial distance

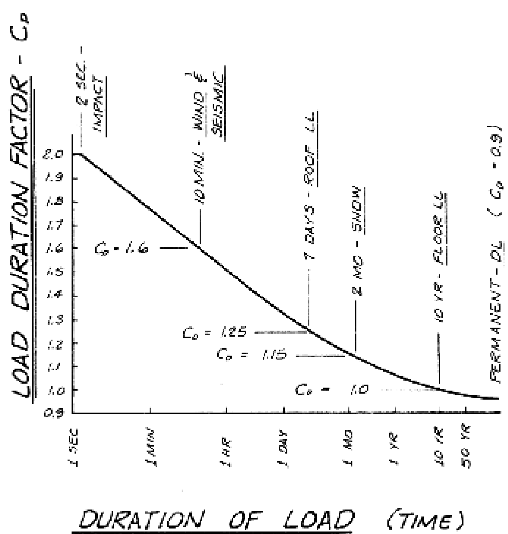
Wood or Timber Design

Structural design standards for wood are established by the *National Design Specification (NDS)* published by the National Forest Products Association. There is a combined specification (from 2005) for **Allowable** Stress Design and limit state design (LRFD).

Tabulated wood strength values are used as the base allowable strength (ASD) and modified by appropriate adjustment factors:

$$f = C_D C_M C_F \ldots \times f_{from\,table}$$

Adjustment Factors



C_D load duration factor

C_M wet service factor (1.0 dry < 16% moisture content)

C_t temperature factor (at high temperatures strength decreases)

C_L beam stability factor (for beams without full lateral support)

C_F size factor for visually graded sawn lumber and round timber > 12" depth

$$C_F = (12/d)^{1/9} \le 1.0$$

C_V volume factor for glued laminated timber (similar to C_F)

C_{fu} flat use factor (excluding decking)

C_r repetitive member factor (1.15 for three or more parallel members of Dimension lumber spaced not more than 24 in. on center, connected together by a load-distributing element such as roof, floor, or wall sheathing)

C_c curvature factor for glued laminated timber (1.0 straight & cambered)
t/R ≤ 1/100 for hardwoods & southern pine or 1/125 other softwoods

$$C_c = 1 - 2000(t/R)^2$$

C_i incising factor (0.85 incised sawn lumber, 1 for sawn lumber not incised and glulam)

C_H shear stress factor (amount of splitting)

C_P column stability factor (1.0 for fully supported columns)

Design Values

- F_b: bending stress
- F_t: tensile stress
- F_v: horizontal shear stress
- $F_{c\perp}$: compression stress (perpendicular to grain)
- F_c: compression stress (parallel to grain)
- E: modulus of elasticity
- F_p: bearing stress (parallel to grain)

Wood is significantly weakest in shear and strongest along the direction of the grain (tension and compression).

Load Combinations and Deflection

The critical load combination is determined by the largest of either:

$$\frac{dead\ load}{0.9}\ or\ \frac{(dead\ load + any\ combination\ of\ live\ load\ )}{C_D}$$

The deflection limits may be increased for less stiffness with total load: LL + 0.5(DL)

Criteria for Beam Design

Allowable normal stress or normal stress from LRFD should not be exceeded:

Knowing M and F_b, the minimum section modulus fitting the limit is: $$S_{req'd} \geq \frac{M}{F_b}$$

Besides strength, we also need to be concerned about *serviceability*. This involves things like limiting deflections & cracking, controlling noise and vibrations, preventing excessive settlements of foundations and durability. When we know about a beam section and its material, we can determine beam deformations.

Determining Maximum Bending Moment

Drawing V and M diagrams will show us the maximum values for design. Computer applications are very helpful.

Determining Maximum Bending Stress

For a prismatic member (constant cross section), the maximum normal stress will occur at the maximum moment.

For a *non-prismatic* member, the stress varies with the cross section AND the moment.

Deflections

Elastic curve equations can be found in handbooks, textbooks, design manuals, etc...Computer programs can be used as well.

Elastic curve equations can be superpositioned ONLY if the stresses are in the elastic range. *The deflected shape is roughly the same shape flipped as the bending moment diagram but is constrained by supports and geometry.*

Allowable Deflection Limits

All building codes and design codes limit deflection for beam types and damage that could happen based on service condition and severity.

Use	LL only	DL+LL
Roof beams:		
Industrial	L/180	L/120
Commercial		
plaster ceiling	L/240	L/180
no plaster	L/360	L/240
Floor beams:		
Ordinary Usage	L/360	L/240
Roof or floor (damageable elements)		L/480

Lateral Buckling

With compression stresses in the top of a beam, a sudden "popping" or buckling can happen even at low stresses. In order to prevent it, we need to brace it along the top, or laterally brace it, or provide a bigger I_y.

Beam Loads & Load Tracing

In order to determine the loads on a beam (or girder, joist, column, frame, foundation...) we can start at the top of a structure and determine the *tributary area* that a load acts over and the beam needs to support. Loads come from material weights, people, and the environment. This area is assumed to be from half the distance to the next beam over to halfway to the next beam.

The reactions must be supported by the next lower structural element *ad infinitum*, to the ground.

Design Procedure

The intent is to find the most light weight member satisfying the section modulus size.

1. Know F_{all} for the material or F_U for LRFD.
2. Draw V & M, finding M_{max}.

3. Calculate $S_{req'd}$. This step is equivalent to determining $f_b = \frac{M_{max}}{S} \leq F_b'$
4. For rectangular beams $S = \frac{bh^2}{6}$
   - For timber: use the section charts to find S that will work *and remember that the beam self weight will increase $S_{req'd.}$*

*****Determine the "updated" V_{max} and M_{max} including the beam self weight, and verify that the updated $S_{req'd}$ has been met.*******

5. Consider lateral stability.

6. Evaluate horizontal shear stresses using V_{max} to determine if $f_v \leq F_v'$

   For rectangular beams $$f_{v-\max} = \frac{3V}{2A} = 1.5\frac{V}{A}$$

7. Provide adequate bearing area at supports: $$f_p = \frac{P}{A} \leq F_p'$$

8. Evaluate shear due to torsion $$f_v = \frac{T\rho}{J} \text{ or } \frac{T}{c_1 ab^2} \leq F_v'$$
   (circular section or rectangular)

9. Evaluate the deflection to determine if $\Delta_{max LL} \leq \Delta_{LL-allowed}$ and/or $\Delta_{\max Total} \leq \Delta_{Total-allowed}$

***** note: when $\Delta_{calculated} > \Delta_{limit}$, $I_{required}$ can be found with: and $S_{req'd}$ will be satisfied for similar self weight ******

$$I_{req'd} \geq \frac{\Delta_{too\,big}}{\Delta_{limit}} I_{trial}$$

FOR ANY EVALUATION:

Redesign (with a new section) at any point that a stress or serviceability criteria is NOT satisfied and re-evaluate each condition until it is satisfactory.

Column Design

National Design Specification for Wood Construction (1992):

<u>Any slenderness ratio, l_e/d ≤ 50:</u>

$$f_c = \frac{P}{A} \leq F_c' \qquad F_c' = F_c(C_D)(C_M)(C_t)(C_F)(C_p)$$

For preliminary column design:

$$F_c' = F_c^* C_p = (F_c C_D) C_p$$

Procedure

1. Obtain F'$_c$

   find l_e/d or assume (l_e/d ≤ 50)

   compute $F_{cE} = \frac{K_{cE}E}{\left(\frac{l_e}{d}\right)^2}$ with K_{cE} =0.3 for sawn, = 0.418 for glu-lam

   compute $F_c^* \cong F_c C_D$ with C_D = 1, normal, C_D =1.25 for 7 day roof...

   find F_{cE}/F_c^* and get C_p

2. Select a section

   If the load and area of the column are known, set the stress equal to the allowable stress, and solve for l_e, l, or d_{min}

   If the load and length of the column are known, set the stress equal to the allowable stress, and solve for A or d_{min} and select a section that satisfies the values found.

3. Continue from 2 until F_c' is satisfied: $F_c' = F_c^* C_p$

Alternate Column Allowable Stress

For *intermediate length* columns with 11<L/d<κ, where $\kappa = 0.67\sqrt{\frac{E}{F_c}}$:

$$F_c' = F_c\left\{1 - \left(\frac{1}{3}\right)\left[\left(\frac{L_e}{d}\right)\kappa\right]^4\right\}$$

For *long* columns with L/d>κ, and an assumed safety factor of 2.73: the allowable stress is:

$$F_c = \frac{0.3E}{\left(\frac{L_e}{d}\right)^2}$$

***Table 9.3** Column stability factor* C_p.

Statics and Strength of Materials for Architecture and Building Construction, 2nd ed., Onouye & Kane

Column Stability Factor C_p

"C_p"

$F_c' = C_p \cdot F_c^*$ $F_{CE} = \frac{30\,E}{(L/d)^2}$ for sawn posts $F_{CE} = \frac{418\,E}{(L/d)^2}$ for Glu-Lam posts

$\frac{F_{CE}}{F_c^*}$	Sawn C_p	Glu-Lam C_p	$\frac{F_{CE}}{F_c^*}$	Sawn C_p	Glu-Lam C_p	$\frac{F_{CE}}{F_c^*}$	Sawn C_p	Glu-Lam C_p	$\frac{F_{CE}}{F_c^*}$	Sawn C_p	Glu-Lam C_p
0.00	0.000	0.000	0.60	0.500	0.538	1.20	0.750	0.822	2.40	0.894	0.940
0.01	0.010	0.010	0.61	0.506	0.545	1.22	0.755	0.826	2.45	0.897	0.941
0.02	0.020	0.020	0.62	0.512	0.552	1.24	0.760	0.831	2.50	0.899	0.943
0.03	0.030	0.030	0.63	0.518	0.559	1.26	0.764	0.836	2.55	0.901	0.944
0.04	0.040	0.040	0.64	0.524	0.566	1.28	0.769	0.840	2.60	0.904	0.946
0.05	0.049	0.050	0.65	0.530	0.573	1.30	0.773	0.844	2.65	0.906	0.947
0.06	0.059	0.060	0.66	0.536	0.580	1.32	0.777	0.848	2.70	0.908	0.949
0.07	0.069	0.069	0.67	0.542	0.587	1.34	0.781	0.852	2.75	0.910	0.950
0.08	0.079	0.079	0.68	0.548	0.593	1.36	0.785	0.855	2.80	0.912	0.951
0.09	0.088	0.089	0.69	0.553	0.600	1.38	0.789	0.859	2.85	0.914	0.952
0.10	0.098	0.099	0.70	0.559	0.607	1.40	0.793	0.862	2.90	0.916	0.953
0.11	0.107	0.109	0.71	0.564	0.613	1.42	0.796	0.865	2.95	0.917	0.954
0.12	0.117	0.118	0.72	0.569	0.619	1.44	0.800	0.868	3.00	0.919	0.955
0.13	0.126	0.128	0.73	0.575	0.626	1.46	0.803	0.871	3.05	0.920	0.956
0.14	0.136	0.138	0.74	0.580	0.632	1.48	0.807	0.874	3.10	0.922	0.957
0.15	0.145	0.147	0.75	0.585	0.638	1.50	0.810	0.877	3.15	0.923	0.958
0.16	0.154	0.157	0.76	0.590	0.644	1.52	0.813	0.879	3.20	0.925	0.959
0.17	0.164	0.167	0.77	0.595	0.650	1.54	0.816	0.882	3.25	0.926	0.960
0.18	0.173	0.176	0.78	0.600	0.655	1.56	0.819	0.884	3.30	0.927	0.961
0.19	0.182	0.186	0.79	0.605	0.661	1.58	0.822	0.887	3.35	0.929	0.961
0.20	0.191	0.195	0.80	0.610	0.667	1.60	0.825	0.889	3.40	0.930	0.962
0.21	0.200	0.205	0.81	0.614	0.672	1.62	0.827	0.891	3.45	0.931	0.963
0.22	0.209	0.214	0.82	0.619	0.678	1.64	0.830	0.893	3.50	0.932	0.963
0.23	0.218	0.224	0.83	0.623	0.683	1.66	0.832	0.895	3.55	0.933	0.964
0.24	0.227	0.233	0.84	0.628	0.688	1.68	0.835	0.897	3.60	0.934	0.965
0.25	0.235	0.242	0.85	0.632	0.693	1.70	0.837	0.899	3.65	0.936	0.965
0.26	0.244	0.252	0.86	0.637	0.698	1.72	0.840	0.901	3.70	0.937	0.966
0.27	0.253	0.261	0.87	0.641	0.703	1.74	0.842	0.903	3.75	0.938	0.966
0.28	0.261	0.270	0.88	0.645	0.708	1.76	0.844	0.904	3.80	0.938	0.967
0.29	0.270	0.279	0.89	0.649	0.713	1.78	0.846	0.906	3.85	0.939	0.968
0.30	0.278	0.288	0.90	0.653	0.718	1.80	0.849	0.908	3.90	0.940	0.968
0.31	0.287	0.297	0.91	0.658	0.722	1.82	0.851	0.909	3.95	0.941	0.969
0.32	0.295	0.306	0.92	0.661	0.727	1.84	0.853	0.911	4.00	0.942	0.969
0.33	0.304	0.315	0.93	0.665	0.731	1.86	0.855	0.912	4.05	0.943	0.969
0.34	0.312	0.324	0.94	0.669	0.735	1.88	0.857	0.914	4.10	0.944	0.970
0.35	0.320	0.333	0.95	0.673	0.740	1.90	0.858	0.915	4.15	0.944	0.970
0.36	0.328	0.342	0.96	0.677	0.744	1.92	0.860	0.916	4.20	0.945	0.971
0.37	0.336	0.351	0.97	0.680	0.748	1.94	0.862	0.918	4.25	0.946	0.971
0.38	0.344	0.360	0.98	0.684	0.752	1.96	0.864	0.919	4.30	0.947	0.972
0.39	0.352	0.368	0.99	0.688	0.756	1.98	0.866	0.920	4.35	0.947	0.972
0.40	0.360	0.377	1.00	0.691	0.760	2.00	0.867	0.921	4.40	0.948	0.972
0.41	0.367	0.386	1.01	0.694	0.764	2.02	0.869	0.922	4.45	0.949	0.973
0.42	0.375	0.394	1.02	0.698	0.767	2.04	0.870	0.924	4.50	0.949	0.973
0.43	0.383	0.403	1.03	0.701	0.771	2.06	0.872	0.925	4.55	0.950	0.974
0.44	0.390	0.411	1.04	0.704	0.774	2.08	0.874	0.926	4.60	0.950	0.974
0.45	0.398	0.420	1.05	0.708	0.778	2.10	0.875	0.927	4.65	0.951	0.974
0.46	0.405	0.428	1.06	0.711	0.781	2.12	0.876	0.928	4.70	0.952	0.975
0.47	0.412	0.436	1.07	0.714	0.784	2.14	0.878	0.929	4.75	0.952	0.975
0.48	0.419	0.444	1.08	0.717	0.788	2.16	0.879	0.930	4.80	0.953	0.975
0.49	0.427	0.453	1.09	0.720	0.791	2.18	0.881	0.931	4.85	0.953	0.975
0.50	0.434	0.461	1.10	0.723	0.794	2.20	0.882	0.932	4.90	0.954	0.976
0.51	0.441	0.469	1.11	0.726	0.797	2.22	0.883	0.932	5.00	0.955	0.976
0.52	0.448	0.477	1.12	0.729	0.800	2.24	0.885	0.933	6.00	0.963	0.981
0.53	0.454	0.484	1.13	0.731	0.803	2.26	0.886	0.934	8.00	0.973	0.986
0.54	0.461	0.492	1.14	0.734	0.806	2.28	0.887	0.935	10.00	0.979	0.989
0.55	0.468	0.500	1.15	0.737	0.809	2.30	0.888	0.936	20.00	0.990	0.995
0.56	0.474	0.508	1.16	0.740	0.811	2.32	0.889	0.937	40.00	0.995	0.997
0.57	0.481	0.515	1.17	0.742	0.814	2.34	0.891	0.937	60.00	0.997	0.998
0.58	0.487	0.523	1.18	0.745	0.817	2.36	0.892	0.938	100.00	0.998	0.999
0.59	0.494	0.530	1.19	0.747	0.819	2.38	0.893	0.939	200.00	0.999	0.999

Table developed and permission for use granted by Professor Ed Lebert, Dept. of Architecture, University of Washington.

SECTION PROPERTIES / STANDARD SIZES

To the extent that other considerations will permit, the finished sizes of structural glued laminated timber as given in Table B constitute normal industry practice. Industry standards do, however, permit the use of any depth or width of glued laminated timber. Dimension lumber of 1½ in. net thickness is normally used for laminating straight members.

The modified section modulus includes size factor (C_f), and no further reduction of bending stress for size is needed.

DEPTH, d in.	AREA, A in.^{2}	MODIFIED SECTION MODULUS, SC_F in.^{3}	MOMENT OF INERTIA, I in.^{4}
3⅛" WIDTH			
6.0	18.8	18.8	56
7.5	23.4	29.3	110
9.0	28.1	42.2	190
10.5	32.8	57.4	302
12.0	37.5	75.0	450
13.5	42.2	93.7	641
15.0	46.9	114.3	879
16.5	51.6	136.9	1,170
18.0	56.3	161.3	1,519
19.5	60.9	187.6	1,931
21.0	65.6	215.8	2,412
22.5	70.3	245.9	2,966
24.0	75.0	277.8	3,600
5⅛" WIDTH			
7.5	38.4	48.0	180
9.0	46.1	69.2	311
10.5	53.8	94.2	494
12.0	61.5	123.0	738
13.5	69.2	153.6	1,051
15.0	76.9	187.5	1,441
16.5	84.6	224.5	1,919
18.0	92.3	264.6	2,491
19.5	99.9	307.7	3,167
21.0	107.6	354.0	3,955
22.5	115.3	403.2	4,865
24.0	123.0	455.5	5,904
25.5	130.7	510.8	7,082
27.0	138.4	569.0	8,406
28.5	146.1	630.2	9,887
30.0	153.8	694.3	11,531
31.5	161.4	761.4	13,349
33.0	169.1	831.3	15,348
34.5	176.8	904.1	17,538
36.0	184.5	979.8	19,926
6¾" WIDTH			
12.0	81.0	162.0	972
13.5	91.1	202.4	1,384
15.0	101.3	246.9	1,898
16.5	111.4	295.6	2,527
18.0	121.5	348.4	3,280
19.5	131.6	405.3	4,171
21.0	141.8	466.2	5,209
22.5	151.9	531.1	6,407
24.0	162.0	600.0	7,776
25.5	172.1	672.8	9,327
27.0	182.3	749.5	11,072
28.5	192.4	830.0	13,021
30.0	202.5	914.5	15,188
31.5	212.6	1,002.8	17,581
33.0	222.8	1,094.9	20,215
34.5	232.9	1,190.8	23,098
36.0	243.0	1,290.5	26,244
37.5	253.1	1,393.9	29,663
39.0	263.3	1,501.1	33,367
40.5	273.4	1,612.0	37,367
42.0	283.5	1,726.6	41,674
43.5	293.6	1,845.0	46,301
45.0	303.8	1,967.0	51,258
46.5	313.9	2,092.6	56,556
48.0	324.0	2,222.0	62,208
8¾" WIDTH			
12.0	105.0	210.0	1,260
13.5	118.1	262.3	1,794
15.0	131.3	320.1	2,461
16.5	144.4	383.2	3,276
18.0	157.5	451.7	4,252
19.5	170.6	525.4	5,407
21.0	183.8	604.4	6,753
22.5	196.9	688.5	8,306
24.0	210.0	777.7	10,080
25.5	223.1	872.1	12,091
27.0	236.3	971.5	14,352
28.5	249.4	1,076.0	16,880
30.0	262.5	1,185.5	19,688
31.5	275.6	1,299.9	22,791
33.0	288.8	1,419.3	26,204
34.5	301.9	1,543.6	29,942
36.0	315.0	1,672.8	34,020
37.5	328.1	1,806.9	38,452
39.0	341.3	1,945.9	43,253
40.5	354.4	2,089.6	48,439
42.0	367.5	2,238.2	54,022
43.5	380.6	2,391.6	60,020
45.0	393.8	2,549.8	66,445
46.5	406.9	2,712.7	73,314
48.0	420.0	2,880.3	80,640
49.5	433.1	3,052.7	88,439
51.0	446.3	3,229.8	96,725
52.5	459.4	3,411.6	105,513
54.0	472.5	3,598.0	114,818
55.5	485.6	3,789.1	124,654
57.0	498.8	3,984.9	135,037
58.5	511.9	4,185.3	145,980
60.0	525.0	4,390.3	157,500
10¾" WIDTH			
15.0	161.3	393.3	3,023
16.5	177.4	470.8	4,024
18.0	193.5	554.9	5,224
19.5	209.6	645.5	6,642
21.0	225.8	742.5	8,296
22.5	241.9	845.8	10,204
24.0	258.0	955.5	12,384
25.5	274.1	1,071.4	14,854
27.0	290.3	1,193.6	17,633
28.5	306.4	1,321.9	20,738
30.0	322.5	1,456.4	24,188
31.5	338.6	1,597.0	28,000
33.0	354.8	1,743.7	32,194
34.5	370.9	1,896.4	36,786
36.0	387.0	2,055.2	41,796
37.5	403.1	2,219.9	47,241
39.0	419.3	2,390.6	53,140
40.5	435.4	2,567.3	59,510
42.0	451.5	2,749.8	66,370
43.5	467.6	2,938.3	73,739
45.0	483.8	3,132.6	81,633
46.5	499.9	3,332.7	90,071
48.0	516.0	3,538.7	99,072
49.5	532.1	3,750.5	108,653
51.0	548.3	3,968.0	118,833
52.5	564.4	4,191.4	129,630
54.0	580.5	4,420.4	141,062
55.5	596.6	4,655.2	153,146
57.0	612.8	4,895.7	165,902
58.5	628.9	5,141.9	179,347
60.0	645.0	5,398.8	193,500
61.5	661.1	5,651.4	208,379
63.0	677.3	5,914.5	224,000
64.5	693.4	6,183.3	240,384
66.0	709.5	6,457.8	257,548
67.5	725.6	6,737.8	275,511
69.0	741.8	7,023.4	294,289
70.5	757.9	7,314.6	313,902
72.0	774.0	7,611.3	334,368
73.5	790.1	7,913.6	355,704

Glue Laminated Timber

These members come in nominal widths of 3, 4, 6, 8, 10, 12, 14 and 16 inches. The depth can exceed 12 inches, so the size factor, C_F must be used. The formula is based on a uniformly loaded beam, simply supported with an l/d ratio of 21. With a single midspan load, multiply C_F by 1.078. With two loads at third points, multiply C_F by 0.968. (Note: the table on page 4 provides section modulus that include C_F).

$$C_F = (12/d)^{1/9} \le 1.0$$

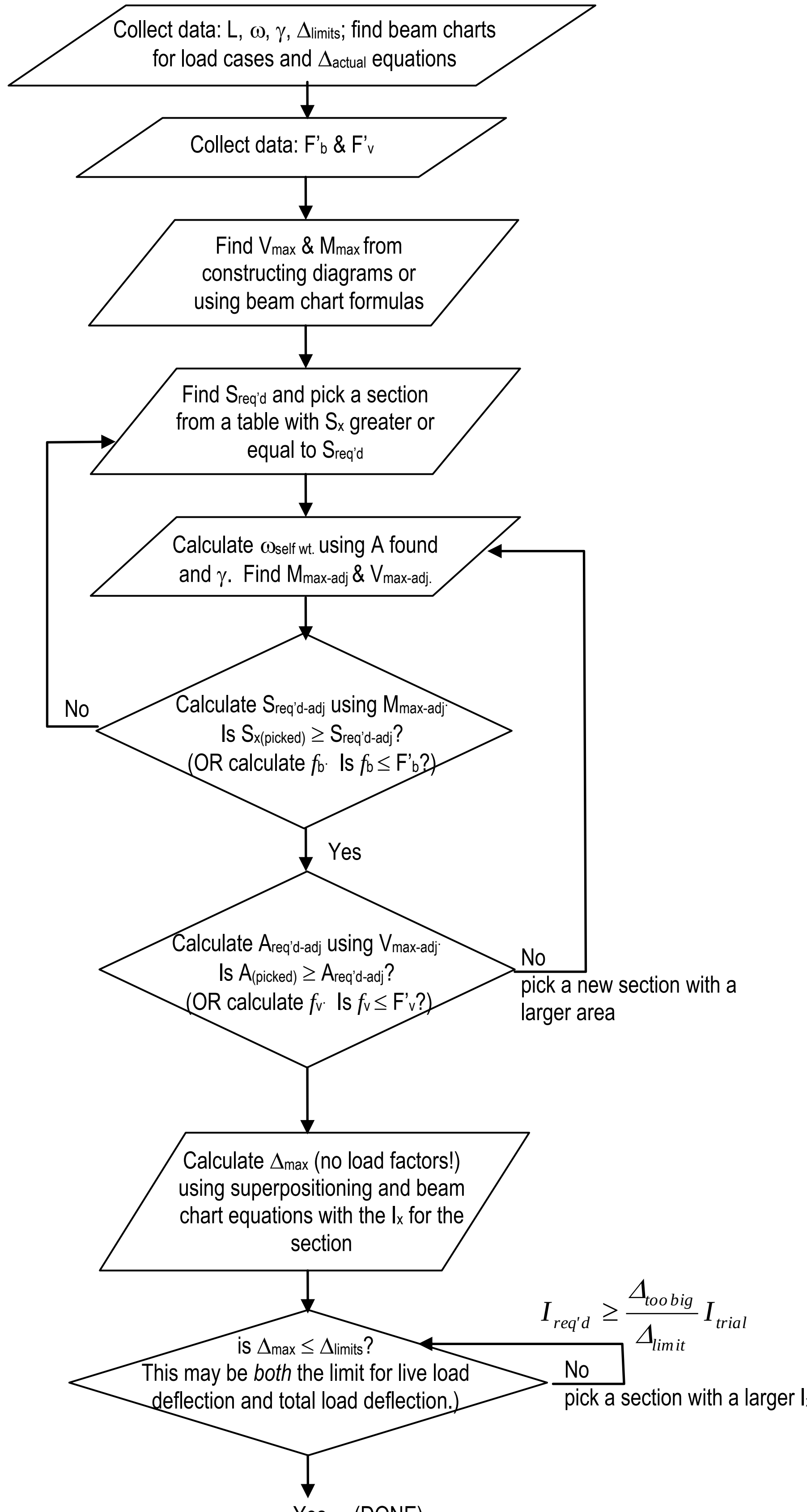
If a glulam is subject to lateral buckling, the slenderness factor is used, and the size factor is not.

Bending of a curved glulam causes radial stresses (like membrane pressures) in tension and compression which can be evaluated for an arc with a radius of R at the neutral axis from:

$$f_r = {}^{3M}\!/_{2Rbd} \quad \text{for constant rectangular cross section}$$

$$f_r \le F_R \quad \text{where} \quad F_R = \begin{cases} F_{C\perp} \\ {}^{1}\!/_{3} F_V \end{cases}$$

ASD Beam Design Flow Chart



Laminated Timber Design Guide

AITC QUALITY INSPECTED®

THE SYMBOL OF QUALITY IN ENGINEERED TIMBER

AITC

American Institute of Timber Construction

American Institute of Timber Construction (AITC)

Representing the glued laminated timber industry since 1952, AITC provides technical support to manufacturers and the design community, and third party quality control manufacturing plants. AITC members design, manufacture, fabricate,or erect wood structural systems.

Boathouse, Boston University, Cambridge, MA.; Architect–Architectural Resources.; Structural Engineer–John Born Associates; Contractor–Walsh Brothers Construction.

Glued Laminated Timber

Glued laminated timber, often referred to as glulam, permits new uses, enhances the natural beauty and extends the enduring qualities of wood. The laminating process makes possible the production of structural timber in a wide variety of sizes and shapes and allows design creativity. The advantages of using glued laminated timber are as varied as your imagination and your specific applications.

Product Standards

AITC recommends and establishes standards and specifications that guide building officials and industry professionals in the design or use of laminated timber.

AITC is the sponsor of the American National Standard, ANSI/AITC A190. This includes plant qualifications, a quality control system, inspection, testing, certification and identification.

AITC's certification and quality assurance programs have proven effective for over 40 years.

Product Identification

Laminated structural members manufactured to the Industry Standard are identified with the AITC Quality Inspection Mark. To assure compliance to the Standard, AITC maintains a staff of highly experienced inspectors.

Species, Sizes and Grades

Species: Laminated timber is manufactured in many species, including softwoods and hardwoods. The most popular softwood species are Douglas Fir/Larch, Southern Pine and Alaskan Yellow Cedar. Hem-Fir, Spruce-Pine-Fir (SPF) and Ponderosa Pine are also frequently used. AITC Standard 117 *Design Specifications for Structural Glued Laminated Timber of Softwood Species,* provides detailed design information.

Sizes: Standard widths for Douglas-Fir are 3 1/8", 5 1/8", 6 3/4", 8 3/4", 10 3/4", 12 1/4" and 14 1/4". Standard widths for Southern Pine are 3", 3 1/8", 5", 5 1/8", 6 3/4", 8 1/2", 10 1/2", 12" and 14". Other widths are available upon request. Depths and lengths of glulam members are limited only by the capability of the individual manufacturer.

Grades: There are four appearance grades -- Industrial, Framing (formerly Industrial S), Architectural and Premium. *Industrial grade* is suitable where appearance is not a primary concern, or the members will not be exposed to view. *Framing grade* matches the width of conventional framing for use as window and door headers where appearance is not a concern. *Architectural grade* is suitable for construction where appearance is an important requirement. *Premium* is the highest grade and is specified where appearance is of utmost importance. Appearance grades do not modify design values, grades of lumber used or other provisions governing the manufacture or use of glued laminated timber.

Textured surfaces, such as rough sawn, are also available from most manufacturers. See AITC Standard 110 for detailed specifications.

Strong, Durable and Beautiful

Because glued laminated timber is fabricated from dry lumber, the resulting higher dimensional stablility reduces checking, twisting, warping and shrinkage. The result is a stable and beautiful installation.

Easy To Install

Laminated timbers can be prefabricated at a plant so they arrive at the job site ready for immediate installation. Most timbers are installed with mobile construction equipment and connections are made by using conventional power and hand tools.

Cost Effective

Laminated timber construction is economical. It does not require the added expense of false ceilings to cover structural framework. Accurate manufacturing reduces the need for on-site fabrication, minimizing waste and installation costs. Equally important, Engineered Timber is more adaptable to construction design changes than are other framing systems.

Availablity

AITC manufacturing plants are located throughout the country. Many straight beam sizes are available through local building material suppliers. Custom glued laminated members can be obtained from a laminator or a local representative.

Custom fabrication for assembly with power tools.

Glued laminated timber truss.

Energy Efficient

Wood's natural insulating properties help reduce building heating and cooling expense. Wood has less thermal expansion than steel or concrete, and its insulating value is many times higher. It also has excellent electrical insulating qualities.

High Resilience

Wood absorbs shocks and provides high resistance to hurricane force winds and earthquake forces.

A naturally renewable resource.

A Renewable Resource

Only one primary building material comes from a renewable resource; cleans the air and water, providing habitat, scenic beauty and recreation as it grows; utilizes nearly 100% of its resource for products; is the lowest of all in energy requirements for its manufacturing; creates fewer air and water emissions than any of its alternatives; and is totally reusable, recyclable and 100% biodegradable: wood. And it has been increasing in U.S. net reserves since 1952, with growth exceeding harvest in the U.S. by more than 30%.

Availability

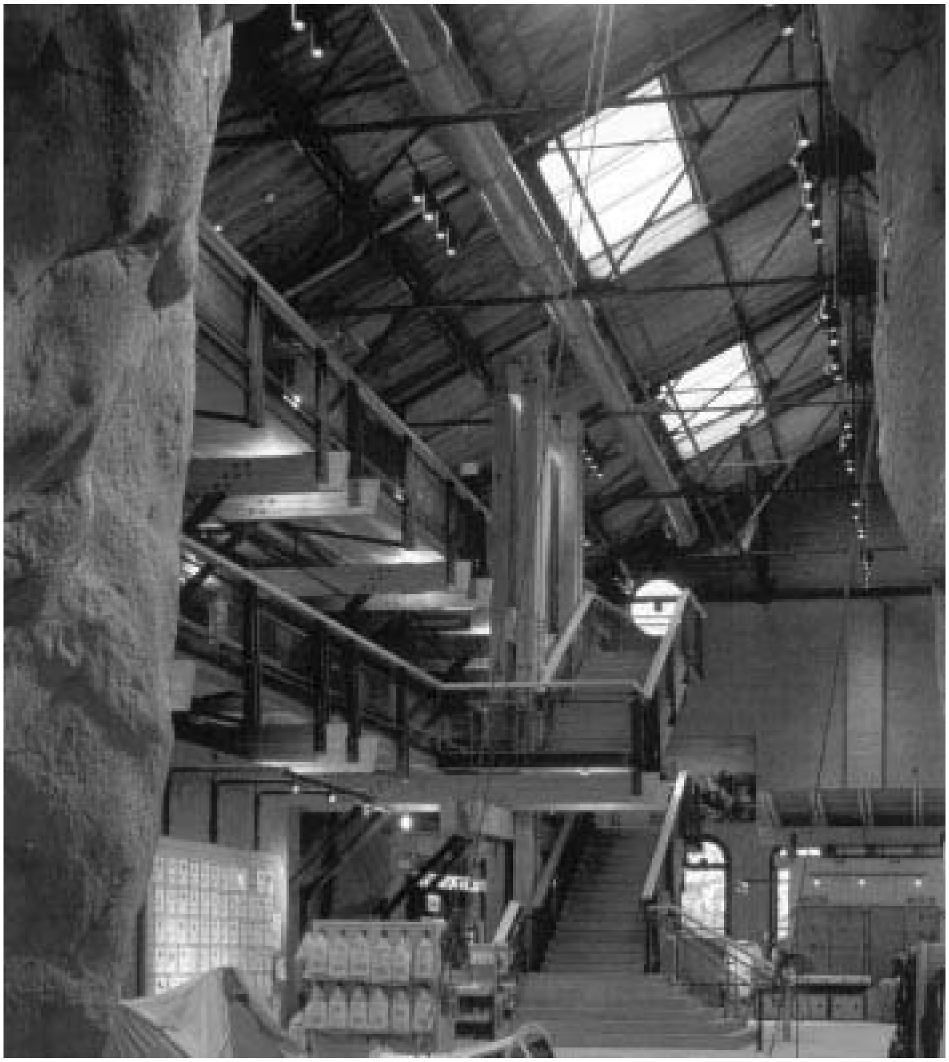
Straight beams in most tabulated sizes are mass produced and readily available at many building products and lumber distribution centers across the country.

Typical structural uses:

- Complete structural systems
- Ridge beams
- Garage door headers
- Door and window headers
- Long span girders
- Stair treads and stringers

Laminated timbers permit large rooms with minimal columns while providing the warmth of wood for living or working environments.

Renovating with laminated timber is easy as beams can be modified at the jobsite to fit existing conditions. Laminated timber can be textured, stained, or painted to match or meet traditional or historic appearance requirements.



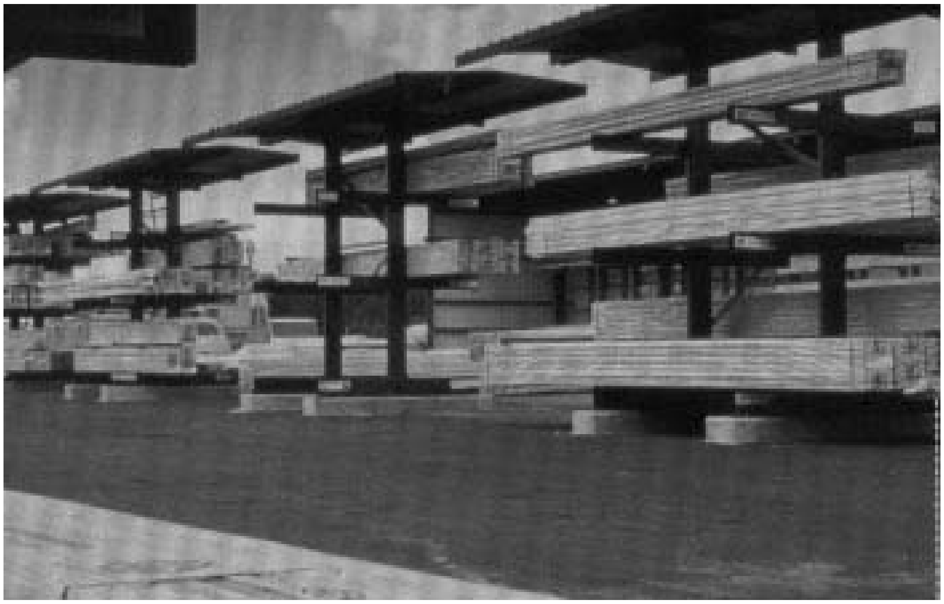
Historic Preservation Award, REI, Denver, CO; Architect–Mithun Partners; Structural Engineer–Skilling Ward Magnusson Barkshire, Inc.

Garage door header.

Residence, Eagle, ID; Architect–Olsen and Associates; Contractor–Gordon Jensen Construction

Residence, Aurora, OR; Architect–Jack Smith F.A.I.A.; Engineer–Bouiss and Associates; Contractor–Busic Construction Company.

Sports Complex, Coronado, CA; Architect–SHWC Architects; Engineer–Ramierez and Associates Engineers; Contractor–Taylor Ball Contractors, Inc.

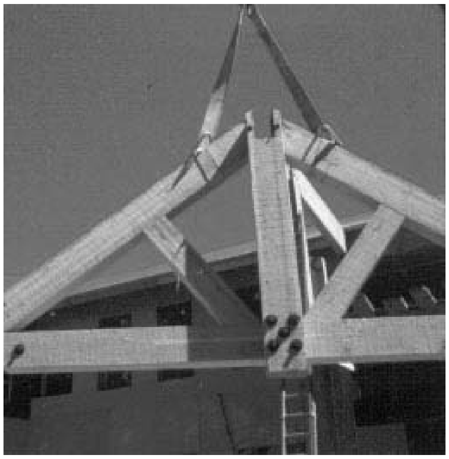


Inventory readily available from local distributors for prompt delivery to job site.

Laminated Timbers Are Easy to Field

Field cutting a stock beam.

Installing a residential ridge beam.



Saw textured beams add warmth and beauty, and are available from most manufacturers.

Sun Valley, ID Residence; Architect–Jack Smith, F.A.I.A.; Structural Engineer–Purdy and Associates; Contractor–Premier Construction Company.

Albertson Shopping Center, Jackson, WY; Architect–Jeffrey A. Shneider; Structural Engineer–Rex Harrison Engineering; Contractor–Bateman Hall

Airport Terminal, Jackson, WY

Ceiling beams compliment rustic design of this McCall, ID home.

Office Building Remodel, Jackson, WY.

Custom Laminated Timber

Laminated timber permits long, clear spans, majestic soaring arches -- tudor, radial, gothic, or parabolic, and many special shapes.

Cut to size and framed for connections at the plant to exact specifications and shapes, laminated timber requires less on-site fabrication which minimizes waste and installation costs.

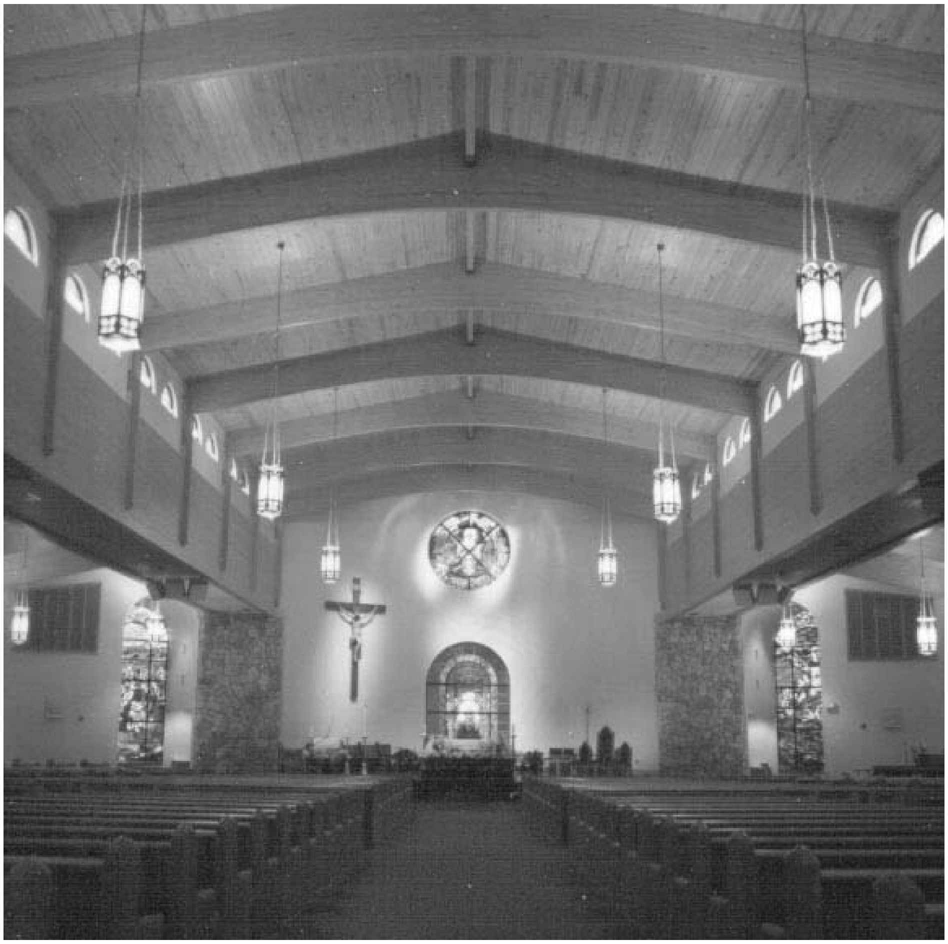
Grant Creek Shopping Mall, Missoula, MT; Architect–Fehlman-Labarre Architects; Contractor–Quality Construction

Great Buddha Hall, Carmel, NY; Architect–Edward A. Valeri; Structural Engineer–Enterprise Engineering Consultants, Ltd.

Newport Beach, CA, 30,000 sq. ft. residence; Architect–Brian Jeannete and Associates; Structural Engineer–Omnispan Corporation; Contractor–Buwalda Construction

Ross High School, East Hampton, Long Island, NY; Architect–Richard Cook & Associates; Contractor–Telemar Construction.

Animal Science Center, 172 ft. span arches, Univ. of Arkansas, Little Rock, AR; Architect–AMR Architect; Structural Engineer–Engineering Consultants, Inc.; Contractor–Harrison Davis Construction



St. Anastasia Catholic Church, St. Augustine, FL; Architect–Richard L. San Giovanni; Engineer–C. Louis Structural Engineers; Contractor–Hall Construction Company, Inc.

Cabella Outdoor Recreation Store, Owatonna, MN; Architect–Nielsen and Mayne Architects; Engineer–Kirkham Michael Engineers; Contractor–Kraus-Anderson.

Custom Shapes

Laminated timber arches or pitched and curved beams can be made in almost any shape. A Tudor type three-hinge arch is favored for many ecclesiastical designs. Radial arches are well suited to large unobstructed clear spans, as are pitched and tapered curved beams.

North Syracuse Baptist Church, Syracuse, NY; Architect–RSA Architects; Contractor–Butterfield Construction

Long Span Structures

Laminated timber beams, arches and other shapes are widely used to provide efficient enclosure of large areas such as gymnasiums, auditoriums and indoor pools. While indoor pools generate high humidity, pressure treatment is not required when the building is adequately ventilated to control humidity, or where a highly durable species, such as Alaska Yellow cedar is used.

Ross High School, East Hampton, Long Island, NY; Architect–Richard Cook & Associates; Contractor–Telemar Construction.

Wood ceilings and beams were selected for acoustical control. Performing Arts Center, North Texas University, Denton, TX; Architect–KVG Gideon Toll Architects; Engineer–Freese and Nichols, Inc.; Contractor–Huber, Hunt and Nichols, Inc.

Exploration Place, Wichita, KS; Achitect–Moshe Safdie & Associates with Schaeffer, Johnson, Cox, Frey; Structural Engineer–Dudley Williams & Associates; Contractor–Dondlinger & Sons Construction

YMCA Pool, Brewton, AL; Architect–Dampier and Associates; Structural Engineer–Joseph and Spain; Contractor–Stuart Construction

Michigan arch bridge; Engineer–Northwest Design Group, Inc.

Bridge in Nature Park, Portland, OR, pressure treated with environmentally friendly copper naphthenate. Structural Engineer–Ceccacci Associates.

Golf course bridge.

Water Oriented Installations

Experience shows that wood is one of the materials most suitable for construction in and around the water.

Wood is resilient enough to resist battering by the ocean and docking ships, and it is naturally resistant to the destructiveness of salt water. It won't rust or spall, and is not affected by corrosion.

Where wood is fully exposed to weather, or where weather protection cannot ensure a moisture content of less than 20%, pressure treatment is required. Buildings housing wet processes, or where wood is in direct contact with the ground or water also require pressure treatment.

See AITC Standard 109 for specific recommendations.

New covered bridge with laminated trusses; Engineer–USDA Forest Service.

Load Tables

Span and load tables are available on AITC's web site or may be obtained by calling AITC. See back page.

Design Properties

Bending members are typically specified on the basis of the maximum allowable bending stress of the member.

A 24 F designation indicates a member with an allowable bending stress of 2400 psi.

See AITC Standard 117 for allowable design stresses.

Cantilevered Beams

Cantilever beam systems are highly efficient for large flat roofs as the continuity across supports permits smaller beams than required for simple spans.

For most residential applications where cantilever lengths are relatively short, a stock unbalanced glulam can be used. Cantilever roof overhangs up to 20 percent of the main span can be accommodated using an unbalanced beam without special layups. For longer length cantilevers, balanced beams should be specified.

Balanced and Unbalanced Sections

Glued laminated timbers are manufactured with both balanced and unbalanced layups. Balanced layups are made with identical lumber grades in the outer laminations, placed symmetrically about the neutral axis. Consequently, balanced layups have equal bending strength for both positive and negative bending. Balanced layups are recommended for beams that are continuous across supports and for cantilevered beams.

Unbalanced layups utilize higher grade lumber in the bottom (tension) side of the beam and are stamped with the word "TOP" on the upper surface. This unsymmetrical configuration results in higher strength for positive bending (tension on bottom) than for negative bending. Unbalanced layups are primarily intended for simple span beams, but can also be used for short cantilevers.

Equivalent Glulam Sections for Dimension Lumber/Timber Beams

Sawn[4] Sections	Roof Beams[1,2]				Floor Beams[1,3]			
	Select Structural		No. 1		Select Structural		No. 1	
Nominal Size	Douglas Fir/Larch	Southern Pine	Douglas Fir/Larch	Southern Pine	Douglas Fir/Larch	Southern Pine	Douglas Fir/Larch	Southern Pine
3x8	3 1/8x6	3x6 7/8	3 1/8x6	3x5 1/2	3 1/8x7 1/2	3x6 7/8	3 1/8x7 1/2	3x6 7/8
3x10	3 1/8x7 1/2	3x8 1/4	3 1/8x6	3x6 7/8	3 1/8x9	3x9 5/8	3 1/8x9	3x9 5/8
3x12	3 1/8x9	3x9 5/8	3 1/8x7 1/2	3x8 1/4	3 1/8x12	3x11	3 1/8x10 1/2	3x11
3x14	3 1/8x9	3x11	3 1/8x7 1/2	3x9 5/8	3 1/8x13 1/2	3x13 3/4	3 1/8x13 1/2	3x12 3/8
4x6	3 1/8x6	3x6 7/8	3 1/8x6	3x5 1/2	3 1/8x6	3x6 7/8	3 1/8x6	3x6 7/8
4x8	3 1/8x7 1/2	3x8 1/4	3 1/8x6	3x6 7/8	3 1/8x9	3x8 1/4	3 1/8x7 1/2	3x8 1/4
4x10	3 1/8x9	3x11	3 1/8x7 1/2	3x8 1/4	3 1/8x10 1/2	3x11	3 1/8x10 1/2	3x9 5/8
4x12	3 1/8x10 1/2	3x12 3/8	3 1/8x9	3x9 5/8	3 1/8x12	3x12 3/8	3 1/8x12	3x12 3/8
4x14	3 1/8x12	3x13 3/4	3 1/8x10 1/2	3x11	3 1/8x15	3x15 1/8	3 1/8x15	3x13 3/4
4x16	3 1/8x13 1/2	3x15 1/8	3 1/8x10 1/2	3x12 3/8	3 1/8x16 1/2	3x16 1/2	3 1/8x16 1/2	3x16 1/2
6x8	5 1/8x7 1/2	5x6 7/8	5 1/8x7 1/2	5x6 7/8	5 1/8x7 1/2	5x8 1/4	5 1/8x7 1/2	5x8 1/4
6x10	5 1/8x9	5x8 1/4	5 1/8x7 1/2	5x8 1/4	5 1/8x10 1/2	5x9 5/8	5 1/8x10 1/2	5x9 5/8
6x12	5 1/8x10 1/2	5x9 5/8	5 1/8x9	5x9 5/8	5 1/8x12	5x12 3/8	5 1/8x12	5x12 3/8
6x14	5 1/8x12	5x12 3/8	5 1/8x10 1/2	5x11	5 1/8x13 1/2	5x13 3/4	5 1/8x13 1/2	5x13 3/4
6x16	5 1/8x13 1/2	5x13 3/4	5 1/8x12	5x12 3/8	5 1/8x16 1/2	5x15 1/8	5 1/8x16 1/2	5x15 1/8
6x18	5 1/8x15	5x15 1/8	5 1/8x13 1/2	5x13 3/4	5 1/8x18	5x17 7/8	5 1/8x18	5x17 7/8
6x20	5 1/8x18	5x16 1/2	5 1/8x16 1/2	5x15 1/8	5 1/8x19 1/2	5x19 1/4	5 1/8x19 1/2	5x19 1/4
8x10	6 3/4x9	6 3/4x8 1/4	6 3/4x9	6 3/4x8 1/4	6 3/4x10 1/2	6 3/4x9 5/8	6 3/4x10 1/2	6 3/4x9 5/8
8x12	6 3/4x10 1/2	6 3/4x9 5/8	6 3/4x10 1/2	6 3/4x9 5/8	6 3/4x12	6 3/4x12 3/8	6 3/4x12	6 3/4x12 3/8
8x14	6 3/4x12	6 3/4x12 3/8	6 3/4x12	6 3/4x11	6 3/4x13 1/2	6 3/4x13 3/4	6 3/4x13 1/2	6 3/4x13 3/4
8x16	6 3/4x13 1/2	6 3/4x13 3/4	6 3/4x13 1/2	6 3/4x12 3/8	6 3/4x16 1/2	6 3/4x15 1/8	6 3/4x16 1/2	6 3/4x15 1/8
8x18	6 3/4x16 1/2	6 3/4x15 1/8	6 3/4x15	6 3/4x13 3/4	6 3/4x18	6 3/4x17 7/8	6 3/4x18	6 3/4x17 7/8
8x20	6 3/4x18	6 3/4x16 1/2	6 3/4x16 1/2	6 3/4x16 1/2	6 3/4x19 1/2	6 3/4x19 1/4	6 3/4x19 1/2	6 3/4x19 1/4
8x22	6 3/4x19 1/2	6 3/4x17 7/8	6 3/4x18	6 3/4x17 7/8	6 3/4x22 1/2	6 3/4x22	6 3/4x22 1/2	6 3/4x22

AITC mark of quality. The word "TOP" identifies beams with unbalanced sections.

AITC is approved for inspection under this Japanese Agricultural Standard. Laminated timber exported to Japan is identified with this label.

Sawn, Steel, LVL and PSL

Equivalent Glulam Sections for Steel Beams

Steel[5] Section	Roof Beams[1, 2] Douglas Fir/Larch		Southern Pine[8]		Floor Beams[1, 3] Douglas Fir/Larch		Southern Pine[8]	
W 6x9	3 1/8x10 1/2 or	5 1/8x9	3x11 or	5x8 1/4	3 1/8x10 1/2 or	5 1/8x9	3x11 or	5x9 5/8
W 8x10	3 1/8x12	5 1/8x9	3x12 3/8	5x9 5/8	3 1/8x13 1/2	5 1/8x12	3x13 3/4	5x11
W 12x14	3 1/8x16 1/2	5 1/8x13 1/2	3x16 1/2	5x13 3/4	3 1/8x18	5 1/8x15	3x17 7/8	5x15 1/8
W 12x16	3 1/8x18	5 1/8x13 1/2	3x17 7/8	5x13 3/4	3 1/8x19 1/2	5 1/8x16 1/2	3x19 1/4	5x16 1/2
W 12x19	3 1/8x19 1/2	5 1/8x16 1/2	3x20 5/8	5x15 1/8	3 1/8x21	5 1/8x18	3x20 5/8	5x17 7/8
W 10x22	3 1/8x21	5 1/8x16 1/2	3x20 5/8	5x16 1/2	3 1/8x19 1/2	5 1/8x16 1/2	3x20 5/8	5x17 7/8
W 12x22	5 1/8x18	6 3/4x15	3x22	5x17 7/8	5 1/8x19 1/2	6 3/4x16 1/2	5x19 1/4	6 3/4x16 1/2
W 14x22	5 1/8x18	6 3/4x16 1/2	3x23 3/8	5x17 7/8	5 1/8x21	6 3/4x18	5x20 5/8	6 3/4x17 7/8
W 12x26	5 1/8x19 1/2	6 3/4x18	5x19 1/4	6 3/4x16 1/2	5 1/8x21	6 3/4x19 1/2	5x20 5/8	6 3/4x19 1/4
W 14x26	5 1/8x21	6 3/4x18	5x20 5/8	6 3/4x17 7/8	5 1/8x21	6 3/4x19 1/2	5x22	6 3/4x19 1/4
W 16x26	5 1/8x21	6 3/4x19 1/2	5x20 5/8	6 3/4x17 7/8	5 1/8 22 1/2	6 3/4x21	5x23 3/8	6 3/4x20 5/8
W 12x30	5 1/8x21	6 3/4x19 1/2	5x20 5/8	6 3/4x17 7/8	5 1/8x21	6 3/4x19 1/2	5x22	6 3/4x19 1/4
W 14x30	5 1/8x22 1/ 2	6 3/4x19 1/2	5x22	6 3/4x19 1/4	5 1/8x22 1/2	6 3/4x21	5x23 3/8	6 3/4x20 5/8
W 16x31	5 1/8x24	6 3/4x21	5x23 3/8	6 3/4x20 5/8	5 1/8x25 1/2	6 3/4x22 1/2	5x24 3/4	6 3/4x23 3/8
W 14x34	5 1/8x24	6 3/4x21	5x23 3/8	6 3/4x20 5/8	5 1/8x24	6 3/4x22 1/2	5x24 3/4	6 3/4x22
W 18x35	5 1/8x27	6 3/4x24	5x26 1/8	6 3/4x22	5 1/8x27	6 3/4x25 1/2	5x27 1/2	6 3/4x24 3/4
W 16x40	5 1/8x28 1/2	6 3/4x25 1/2	5x27 1/2	6 3/4x23 3/8	5 1/8x27	6 3/4x25 1/2	5x27 1/2	6 3/4x24 3/4
W 21x44	5 1/8x33	6 3/4x28 1/2	5x31 5/8	6 3/4x27 1/2	5 1/8x33	6 3/4x30	5x33	6 3/4x30 1/4
W 18x50	5 1/8x34 1/2	6 3/4x30	5x33	6 3/4x28 7/8	5 1/8x31 1/2	6 3/4x28 1/2	5x31 5/8	6 3/4x28 7/8
W 21x50	5 1/8x34 1/2	6 3/4x31 1/2	5x34 3/8	6 3/4x28 7/8	5 1/8x34 1/2	6 3/4x31 1/2	5x34 3/8	6 3/4x31 5/8
W 18x55	5 1/8x36	6 3/4x31 1/2	5x34 3/8	6 3/4x30 1/4	5 1/8x33	6 3/4x30	5x33	6 3/4x30 1/4
W 21x62		6 3/4x36		6 3/4x34 3/8	5 1/8x37 1/2	6 3/4x34 1/2		6 3/4x34 3/8

Equivalent Glulam Sections for Laminated Veneer Lumber (LVL)

LVL[6] Sections	Roof Beams[1, 2] Douglas Fir/Larch		Southern Pine[8]		Floor Beams[1, 3] Douglas Fir/Larch		Southern Pine[8]	
2pcs 1 3/4x9 1/2	3 1/8x10 1/2 or	5 1/8x9	3x11 or	5x8 1/4	3 1/8x10 1/2 or	5 1/8x9	3x11 or	5x9 5/8
2pcs 1 3/4x11 7/8	3 1/8x13 1/2	5 1/8x10 1/2	3x13 3/4	5x11	3 1/8x13 1/2	5 1/8x12	3x13 3/4	5x11
2pcs 1 3/4x14	3 1/8x16 1/2	5 1/8x12	3x16 1/2	5x12 3/8	3 1/8x16 1/2	5 1/8x13 1/2	3x16 1/2	5x13 3/4
2pcs 1 3/4x16	3 1/8x18	5 1/8x15	3x17 7/8	5x13 3/4	3 1/8x18	5 1/8x15	3x17 7/8	5x15 1/8
2pcs 1 3/4x18	3 1/8x21	5 1/8x16 1/2	3x20 5/8	5x16 1/2	3 1/8x19 1/2	5 1/8x16 1/2	3x20 5/8	5x17 7/8
3pcs 1 3/4x9 1/2	3 1/8x13 1/2	5 1/8x10 1/2	3x13 3/4	5x11	3 1/8x12	5 1/8x10 1/2	3x12 3/8	5x11
3pcs 1 3/4x11 7/8	3 1/8x16 1/2	5 1/8x13 1/2	3x16 1/2	5x13 3/4	3 1/8x15	5 1/8x13 1/2	3x15 1/8	5x13 3/4
3pcs 1 3/4x14	3 1/8x19 1/2	5 1/8x15	3x19 1/4	5x15 1/8	3 1/8x18	5 1/8x15	3x17 7/8	5x15 1/8
3pcs 1 3/4x16	3 1/8x22 1/2	5 1/8x18	3x22	5x17 7/8	3 1/8x21	5 1/8x18	3x20 5/8	5x17 7/8
3pcs 1 3/4x18	3 1/8x25 1/2	5 1/8x19 1/2	3x24 3/4	5x19 1/4	3 1/8x22 1/2	5 1/8x19 1/2	3x23 3/8	5x19 1/4

Equivalent Glulam Sections for Parallel Strand Lumber (PSL)

PSL[7] Sections	Roof Beams[1, 2] Douglas Fir/Larch		Southern Pine[8]		Floor Beams[1, 3] Douglas Fir/Larch		Southern Pine[8]	
3 1/2x9 1/2	3 1/8x10 1/2 or	5 1/8x9	3x11 or	5x9 5/8	3 1/8x10 1/2 or	5 1/8x9	3x11 or	5x9 5/8
3 1/2x11 7/8	3 1/8x13 1/2	5 1/8x10 1/2	3x13 3/4	5x11	3 1/8x13 1/2	5 1/8x12	3x13 3/4	5x11
3 1/2x14	3 1/8x16 1/2	5 1/8x12	3x16 1/2	5x12 3/8	3 1/8x16 1/2	5 1/8x13 1/2	3x16 1/2	5x13 3/4
3 1/2x16	3 1/8x18	5 1/8x13 1/2	3x17 7/8	5x13 3/4	3 1/8x18	5 1/8x15	3x17 7/8	5x15 1/8
3 1/2x18	3 1/8x21	5 1/8x16 1/2	3x20 5/8	5x16 1/2	3 1/8x19 1/2	5 1/8x16 1/2	3x20 5/8	5x17 7/8
5 1/4x9 1/2	3 1/8x13 1/2	5 1/8x10 1/2	3x13 3/4	5x11	3 1/8x12	5 1/8x10 1/2	3x12 3/8	5x11
5 1/4x11 7/8	3 1/8x16 1/2	5 1/8x13 1/2	3x16 1/2	5x13 3/4	3 1/8x15	5 1/8x13 1/2	3x15 1/8	5x13 3/4
5 1/4x14	3 1/8x19 1/2	5 1/8x15	3x19 1/4	5x15 1/8	3 1/8x18	5 1/8x15	3x17 7/8	5x15 1/8
5 1/4x16	3 1/8x22 1/2	5 1/8x18	3x22	5x17 7/8	3 1/8x21	5 1/8x18	3x20 5/8	5x17 7/8
5 1/4x18	3 1/8x25 1/2	5 1/8x19 1/2	3x24 3/4	5x19 1/4	3 1/8x22 1/2	5 1/8x19 1/2	3x23 3/8	5x19 1/4
7x9 1/2	5 1/8x12	6 3/4x10 1/2	5x12 3/8	6 3/4x11	5 1/8x12	6 3/4x10 1/2	5x12 3/8	6 3/4x11
7x11 7/8	5 1/8x15	6 3/4x13 1/2	5x15 1/8	6 3/4x13 3/4	5 1/8x15	6 3/4x13 1/2	5x15 1/8	6 3/4x13 3/4
7x14	5 1/8x18	6 3/4x16 1/2	5x17 7/8	6 3/4x15 1/8	5 1/8x16 1/2	6 3/4x15	5x16 1/2	6 3/4x15 1/8
7x16	5 1/8x21	6 3/4x18	5x20 5/8	6 3/4x17 7/8	5 1/8x19 1/2	6 3/4x18	5x19 1/4	6 3/4x17 7/8
7x18	5 1/8x22 1/2	6 3/4x21	5x22	6 3/4x19 1/4	5 1/8x21	6 3/4x19 1/2	5x22	6 3/4x19 1/4

Footnotes For All Tables:

1. Glued laminated timber beam sizes are based on a span to depth ratio, L/d of 21. When the span to depth ratio is different, sizes should be determined by engineering calculations.

2. Roof beam sections are compared on the basis of equivalent bending resistance only. These sizes use a dry service condition and a 1.15 increase for duration of load (as for snow loading) as applicable to wood members. Sizes shown should also be checked for shear, deflection, and other applicable strength properties and design considerations. For determining glulam roof beam sections, the bending design value, F_b was adjusted by the volume factor.

3. Floor beam sections are compared on the basis of equivalent stiffness (EI) only, using a dry service condition for the wood members. Sizes shown should also be checked for shear, bending, and other applicable strength properties and design considerations.

4. Solid sawn sections are shown for select structural or No. 1 grade. Design values used are from the 1997 NDS.

5. Steel sections were selected as the most economical from the "Manual of Steel Construction," AISC, 9th Edition. Design values used were:
F_y = 36 ksi,
$F_b = 0.66 \times F_y$
E = 29,000 ksi.

6. LVL sections are based on the following design values:
F_b = 2350 psi (adjusted for $C_f = (12/d)^{1/7.5}$ for depths greater than 12 in.)
E = 2,000,000 psi.

7. PSL sections are based on the following design values:
F_b = 2900 psi (adjusted for $C_f = (12/d)^{1/9.0}$ for depths greater than 12 in.)
E = 2,000,000 psi.

8. 3 1/8" width Southern Pine beams are also available.

Glulam beam sections are based on the following design values:
F_{bx} = 2400 psi (dry service conditions)
E_x = 1,800,000 psi
30F, 3000 psi beams are also available.

While these design conversions have been prepared in accordance with recognized engineering principles, and are based on accurate tech-nical data available, conversions should not be used without competent professional examination and verification of the accuracy, suitability, and applicability by a licensed design professional.

Any user of this information assumes all risks and liability arising from such use.

Connection Details and Other Applications

Posts and Columns

Laminated posts and columns are available in long length members, eliminating the need to splice short timber sections.

Due to its dimensional stability and close manufacturing tolerances, a glued laminated timber column will remain straight and square. Other framing members, such as beams, can easily be attached with simple connection detailing.

Other Applications

The use of laminated stair stringers is a good choice when long stringers are required, or when the stair framing will be exposed. Custom curved members are an option when special architectural considerations need to be met.

Stair stringers should not be notched for installation of risers, because it could compromise the stringers structural performance. Steel angles or ledgers may be used to support risers.

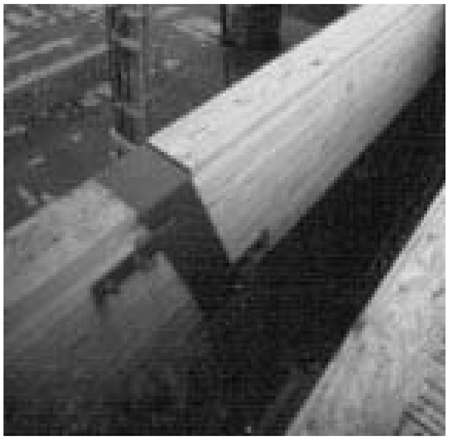
Connection Details

Some typical connection details are shown on this page. For more information, request AITC Standard 104, Typical Construction Details.

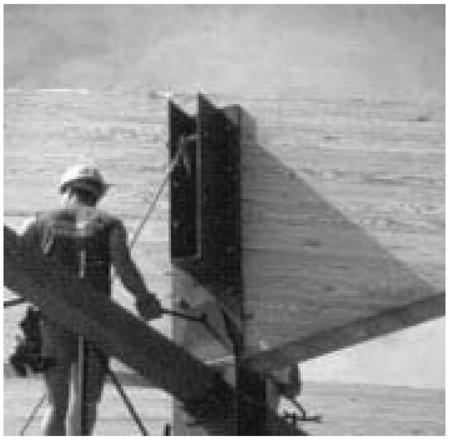
Corrosion Resistance

Wood has excellent chemical and corrosion resistance and is used in installations such as fertilizer storage buildings.

Beam hangers.



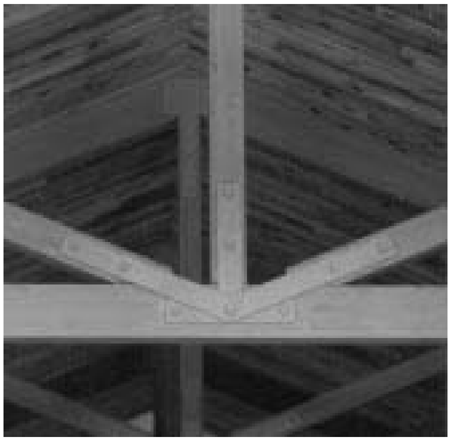
Cantilever connection.



Column connection.

Beam Intersection connection.

Corner support.



Truss connection.

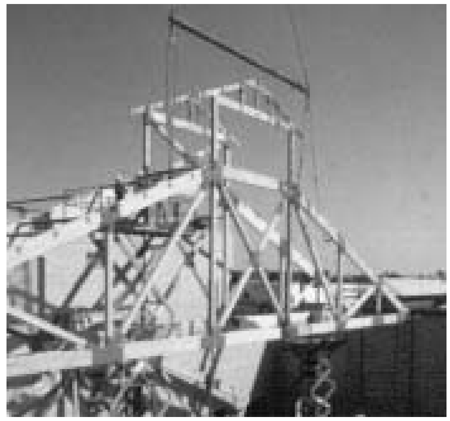
Heavy Timber Construction

Heavy timber construction has long been recognized by the model building codes as fire resistant. To receive building code acceptance as "heavy timber," limitations are placed upon size and thickness or composition of all load carrying wood members. Heavy timber also avoids concealed spaces under floors and roofs and requires the use of approved fastenings, construction details and adhesives.

The performance of heavy timber construction under fire conditions is markedly superior to most unprotected non-combustible structures. Fire fighting is simpler and safer due to elimination of concealed spaces and the inherent structural integrity of large glued laminated timbers.

Unprotected metals lose their strength quickly and collapse suddenly under extreme heat. Steel weakens dramatically as its temperature climbs above 450° Fahrenheit, retaining only 10% of its strength at about 1380°F. The average building fire temperatures range from 1290°F to 1650°F.

Wood retains a significantly higher percentage of its original strength for a longer period of time, losing strength only as material is lost through surface charring.



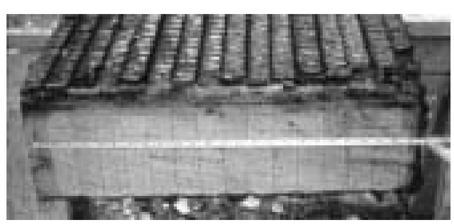
Heavy timber contruction.

Fire Resistance

The fire resistance rating is the time a member can support full design load without collapsing or spreading fire, either directly or indirectly through heat transfer. For example, a one-hour rating means the assembly should be capable of supporting its full load without collapsing for at least one hour after the fire starts.

Fire Design Method

Fire tests jointly sponsored by the American Forest & Paper Association and AITC led to a fire design methodology which allows the designer to calculate a specific fire rating for a glulam member. See AITC Technical Note No. 7.

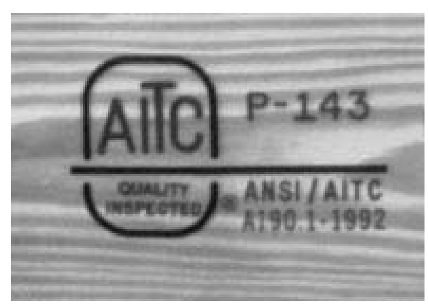


A typical glulam beam following a fire test. The outer surface of the beam has charred, while the inner areas remain unburned. The charred outer material acts as an insulator during fire, reducing the rate at which the inner material burns.

Quality Control and Inspection

As a service to the construction industry, AITC provides a quality control and inspection system based on three elements:

1. **Licensing of manufacturers.** AITC licenses qualified laminators whose personnel procedures and facilities have complied with the requirements of ANSI/AITC A190.1.
2. **Quality control maintenance.** Each licensee agrees to accept responsibility for maintaining a quality control system which is in compliance with ANSI/AITC A190.1, AITC standards, and AITC 200-- Inspection Manual.
3. **Periodic plant inspection.** AITC's Inspection Bureau, a nationwide team of qualified inspectors, conducts frequent, unannounced inspection and verification checks of laminators' in-plant quality control system, procedures and production.



AITC Publications

- *Timber Construction Manual – This 904 page handbook for timber design includes design methods and examples for laminated beams, columns, arches, trusses, single and double tapered beams, curved beams, and pitched and tapered curved beams.*
- *Bridge Systems Manual*
- *Structural Glued Laminated Timber in Religious Structures*
- *Glued Laminated Timbers for Residential Construction*
- *Glulam–Superior Fire Resistance*
- *Pitched and Curved Glulam Beams*
- *Pitched and Tapered Curved Beams*
- *WoodWorks® Sizer for AITC Software*

See our web site for publication prices.

AITC Standards

AITC 104-84 Typical Construction Details.
AITC 109-98 Standard for Preservative Treatment of Structural Glued Laminated Timber.
AITC 110-97 Standard Appearance Grades for Structural Glued Laminated Timber.
AITC 111-79 Recommended Practice for Protection of Structural Glued Laminated Timber During Transit, Storage and Erection.
AITC 113-93 Standard for Dimensions of Structural Glued Laminated Timber.
AITC 117-93 (Design) Standard Specifications for Structural Glued Laminated Timber of Softwood Species, Design Requirements.
AITC 119-96 Standard Specifications for Structural Glued Laminated Timber of Hardwood Species.

A number of Technical Notes cover subjects such as checking, drilling, notching and fire performance. These are available through our web site.

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BE CONSTRUCTIVE™
WOOD

Examples: Timber

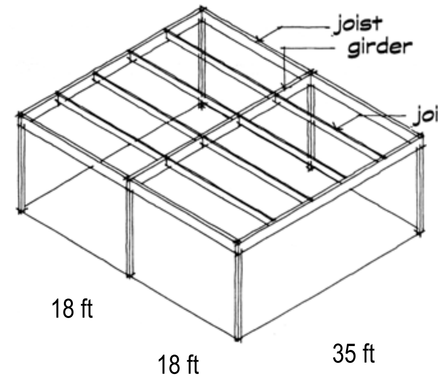
Example 1

Design a Flat Roof joist, 16 in. on center (o.c.) , 18 ft span with Douglas fir-larch No. 2. Snow load is 30 psf. Dead load (including ballast, roofing, sheathing, joists & ceiling) = 18.9 psf. C_r = 1.15 *for bending only*.

F_b = 875 psi; F_v = 95 psi; E = 1.6 x 10^6 psi

Also design the glulam girder supporting the joists if it spans 35 ft (simply supported) and F_b = 2400 psi.

Assume the density of the glulam timber is 32 lb/ft^3.



SOLUTION:
The load case that is most likely to govern the design is Dead + Live. Because the live load is from snow, C_D = 1.15:

$$\frac{18.9psf}{0.9} = 21psf < \frac{(18.9psf + 30psf)}{1.15} = 42.5$$

Joist
The distributed load for each joist needs to be found by multiplying the area load by the tributary width:

w = (30 lb/ft^2 + 18.9 lb/ft^2)(16in)(1ft/12in) = 65.2 lb/ft

$$M_{max} = \frac{wl^2}{8} = \frac{(65.2\,{}^{lb}\!/_{ft})(18\,ft)^2}{8} = 2641^{lb-ft}$$

Allowable stress is the tabulated stress multiplied by all applicable adjustment factors, which would be C_D and C_r:

$$F_b' = F_b C_D C_r = 875\,{}^{lb}\!/_{in^2}(1.15)(1.15) = 1157 \text{ lb/in}^2$$

$$S_{req'd} \geq \frac{M}{F_b'} = \frac{2641^{lb-ft}}{1157\,{}^{lb}\!/_{in^2}} \cdot (12\,{}^{in}\!/_{ft}) = 27.4in^3$$

Shear can quite often govern the design of timber beams:

$$V_{max} = \frac{wl}{2} = \frac{(65.2\,{}^{lb}\!/_{ft})(18\,ft)}{2} = 587^{lb}$$

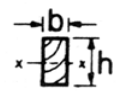
Allowable stress is the tabulated stress multiplied by all applicable adjustment factors, which would be C_D *only:*

$$F_v' = F_v C_D = 95\,{}^{lb}\!/_{in^2}(1.15) = 109 \text{ lb/in}^2$$

Shear stress in a rectangular beam is found from 3V/2A:

$$A_{req'd} \geq \frac{3V}{2F_v'} = \frac{3(587^{lb})}{2(109\,{}^{lb}\!/_{in^2})} = 8.1in^2$$

SECTION PROPERTIES JOISTS AND BEAMS



Nominal Size In Inches b h	Surfaced Size In Inches For Design b h	Area (A) $A = bh$ (In^2)	Section Modulus (S) $S = \frac{bh^2}{6}$ (In^3)	Moment of Inertia (I) $I = \frac{bh^3}{12}$ (In^4)	Board Feet Per Linear Foot of Piece
2 x 2	1.5 x 1.5	2.25	0.562	0.422	0.33
2 x 3	1.5 x 2.5	3.75	1.56	1.95	0.50
2 x 4	1.5 x 3.5	5.25	3.06	5.36	0.67
2 x 5	1.5 x 4.5	6.75	5.06	11.39	.83
2 x 6	1.5 x 5.5	8.25	7.56	20.80	1.00
2 x 8	1.5 x 7.25	10.88	13.14	47.63	1.33
2 x 10	1.5 x 9.25	13.88	21.39	98.93	1.67
2 x 12	1.5 x 11.25	16.88	31.64	177.98	2.00
2 x 14	1.5 x 13.25	19.88	43.89	290.78	2.33
3 x 3	2.5 x 2.5	6.25	2.60	3.26	0.75
3 x 4	2.5 x 3.5	8.75	5.10	8.93	1.00
3 x 5	2.5 x 4.5	11.25	8.44	18.98	1.25
3 x 6	2.5 x 5.5	13.75	12.60	34.66	1.50
3 x 8	2.5 x 7.25	18.12	21.90	79.39	2.00
3 x 10	2.5 x 9.25	23.12	35.65	164.89	2.50
3 x 12	2.5 x 11.25	28.12	52.73	296.63	3.00
3 x 14	2.5 x 13.25	33.12	73.15	484.63	3.50
3 x 16	2.5 x 15.25	38.12	96.90	738.87	4.00

Allowable deflection is not known, but $I_{req'd}$ could be determined from $\Delta = \dfrac{5wl^4}{384EI} \le \Delta_{limit}$ then $I_{req'd} \ge \dfrac{5wl^4}{384E\Delta_{limit}}$

From the section property table, a 2 x 12 satisfies $A_{req'd}$ and $I_{req'd.}$ (bending governs)

Girder

The distributed load on the girder is the reaction of each joist over the 16 inch spacing plus the self weight of the girder.

Guessing a self weight of 40 lb/ft (≈ 32 lb/ft³x1ft²):

$$w = \frac{V}{spacing} + s.w. = \frac{587lb}{16in}\cdot\frac{12in}{ft} + 40\,{}^{lb}\!/\!_{ft} = 480\,{}^{lb}\!/\!_{ft}$$

$$M_{max} = \frac{wl^2}{8} = \frac{(480\,{}^{lb}\!/\!_{ft})(35\,ft)^2}{8} = 73{,}500^{lb-ft}$$

Allowable stress is the tabulated stress multiplied by all applicable adjustment factors, which would be C_F. The charts provided say that C_F has been included in the section modulus. If we didn't have a chart that included C_F and we don't know the depth, we could guess - say 18 inches:

$C_F = \left(\dfrac{12}{d}\right)^{1/9} = \left(\dfrac{12}{18}\right)^{1/9} = 0.956$ (< 1) which would need to be multiplied with all the other adjustment factors by F_b to find F'_b

$$S_{req'd} \ge \frac{M}{F'_b} = \frac{73{,}500^{lb-ft}}{2400\,{}^{lb}\!/\!_{in^2}}\cdot(12\,{}^{in}\!/\!_{ft}) = 367.5in^3$$

No information is available to evaluate shear or deflection. Based on that, try a 5 1/8 x 22.5. It has a smaller area than the 8 ¾ section with a big enough adjusted S. (Real S = 5.125x22.5²/6 = 432.42 in³, C_F = 0.932, $S_{adjusted}$ = 403.2 in³)

Check self weight: $=\gamma\cdot A = 32\,{}^{lb}\!/\!_{ft^3}\,(115.3in^2)\left(\dfrac{1ft}{12in}\right)^2 = 26\,{}^{lb}\!/\!_{ft}$ which is less than what was used.

We could try a smaller section, which would mean calculating a new self weight, then moment, then $S_{req'd}$ and comparing S_{actual} to $S_{req'd.}$

The lower self weight means a lower design moment, but the smaller C_F means a smaller allowed stress, so we might end up with the same section.

$w_{revised}$ = 480 lb/ft + (26-40 lb/ft), $M_{revised}$ = 71,356 lb-ft , $S_{req'd\ now}$ = 356.8 in³ and the 5 1/8 x 22.5 is the choice for bending.

Of course, we need to satisfy shear and deflection criteria as well.

Width	DEPTH, d in.	AREA, A in.²	MODIFIED SECTION MODULUS, S_{CF} in.³	MOMENT OF INERTIA, I in.⁴
3⅛" WIDTH	6.0	18.8	18.8	56
	7.5	23.4	29.3	110
	9.0	28.1	42.2	190
	10.5	32.8	57.4	302
	12.0	37.5	75.0	450
	13.5	42.2	93.7	641
	15.0	46.9	114.3	879
	16.5	51.6	136.9	1,170
	18.0	56.3	161.3	1,519
	19.5	60.9	187.6	1,931
	21.0	65.6	215.8	2,412
	22.5	70.3	245.9	2,966
	24.0	75.0	277.3	3,600
5⅛" WIDTH	7.5	38.4	48.0	180
	9.0	46.1	69.2	311
	10.5	53.8	94.2	494
	12.0	61.5	123.0	738
	13.5	69.2	153.6	1,051
	15.0	76.9	187.5	1,441
	16.5	84.6	224.5	1,919
	18.0	92.3	264.6	2,491
	19.5	99.9	307.7	3,167
	21.0	107.6	354.0	3,955
	22.5	115.3	403.2	4,865
	24.0	123.0	455.5	5,904
	25.5	130.7	510.8	7,082
	27.0	138.4	569.0	8,406
	28.5	146.1	630.2	9,887
	30.0	153.8	694.3	11,531
	31.5	161.4	761.4	13,349
	33.0	169.1	831.3	15,348
	34.5	176.8	904.1	17,538
	36.0	184.5	979.8	19,926
6¾" WIDTH	12.0	81.0	162.0	972
	13.5	91.1	202.4	1,384
	15.0	101.3	246.9	1,898
	16.5	111.4	295.6	2,527
	18.0	121.5	348.4	3,280
	19.5	131.6	405.3	4,171
	21.0	141.8	466.2	5,209
	22.5	151.9	531.1	6,407

Example 2

Example Problem 10.20:
Design of Wood Columns(Figure 10.66)

A 22'-tall glu-lam column is required to support a roof load (including snow) of 40 kips. Assuming $8\frac{3}{4}$" in one dimension (to match the beam width above), determine the minimum column size if the top and bottom are pin supported.

Select from the following sizes:

$8\frac{3}{4}" \times 9"$ ($A = 78.75$ in.^{2})

$8\frac{3}{4}" \times 10\frac{1}{2}"$ ($A = 91.88$ in.^{2})

$8\frac{3}{4}" \times 12"$ ($A = 105.00$ in.^{2})

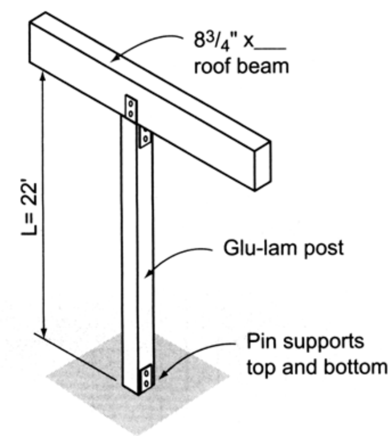


Figure 10.66 Glu-lam column design.

Solution:

Glu-lam column: (F_c = 1650 psi, $E = 1.8 \times 10^6$ psi)

Try $8\frac{3}{4}" \times 10\frac{1}{2}"$ ($A = 105.00$ in.^{2})

$$\frac{L_e}{d} = \frac{(22' \times 12 \text{ in./ft.})}{8.75 \text{ in.}}$$
$$= 30.2 < 50 \text{ (max. slenderness ratio)}$$

$$F_{cE} = \frac{0.418E}{(L_e/d)^2} = \frac{0.418(1.8 \times 10^6 \text{ lb./in.}^2)}{(30.2)^2} = 825 \text{ psi}$$

$$F_c^* \cong F_c C_D = (1650 \text{ psi}) \times \underset{\text{(snow)}}{(1.15)} = 1900 \text{ psi}$$

$$\frac{F_{cE}}{F_c^*} = \frac{825}{1900} = 0.43$$

From Appendix Table 14: $C_p = 0.403$

$$F_c' = F_c^* C_p = (1900 \text{ lb./in.}^2) \times (0.403) = 765 \text{ psi}$$
$$P_a = F_c' \times A = (765 \text{ lb./in.}^2) \times (91.9 \text{ in.}^2)$$
$$= 70{,}300 \text{ lb.} > 40{,}000 \text{ lb.}$$

Cycle again, trying a smaller, more economical section. Try $8\frac{3}{4}" \times 9"$ ($A = 78.8$ in.^{2})

Since the critical dimension is still $8\frac{3}{4}$", the values for F_{cE}, F_c^*, and F_c' all remain the same as in trial 1. The only change that affects the capability of the column is the available cross-sectional area.

$$\therefore P_a = F_c' \times A = (765 \text{ lb./in.}^2) \times (78.8 \text{ in.}^2)$$
$$= 60{,}300 \text{ lb.}$$
$$P_a = 60.3 \text{ k} > 40 \text{ k}$$

Use $8\frac{3}{4}" \times 9"$ glu-lam section.

Case Study in Timber

adapted from Simplified Design of Wood Structures, James Ambrose, 5th ed.

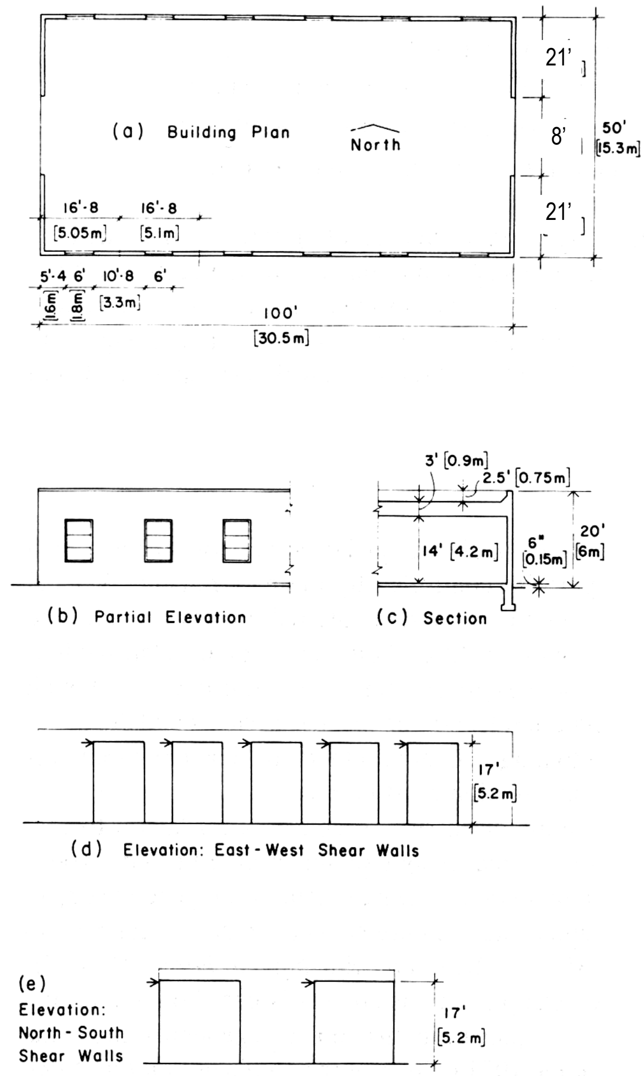


Figure 16.1 Building One, general form.

Building description

The building is a one-story building intended for commercial occupancy. Figure 16.1 presents a building plan, partial elevation, section and elevation of the perimeter shear walls. Light wood framing (assuming the fire resistance requirements have been met) will be used.

Loads

Live Loads:

Roof: 20 lb/ft^2 (0.96 kPa)

Wind: critical at 20 lb/ft^2 (0.96 kPa) on vertical exterior surfaces.

Dead Loads:

Roofing & deck: 7.5 lb/ft^2 (0.36 kPa)

Ceiling joists, ceiling & fixtures:
6.5 lb/ft^2 (0.31 kPa)

Total: 14 lb/ft^2 (0.67 kPa)

Materials

Wood framing of Douglas fir-larch, structural grades No. 1 & 2 having a density of 32 lb/ft^3, and AITC glulam timber.

Structural Elements/Plan

If the interior partition walls are arranged as in Figure 16.3a, there are options on the arrangement of the roof structure. We will analyze case 16.3b consisting of roof deck and rafters, stud walls, continuous (two span) beams, and columns.

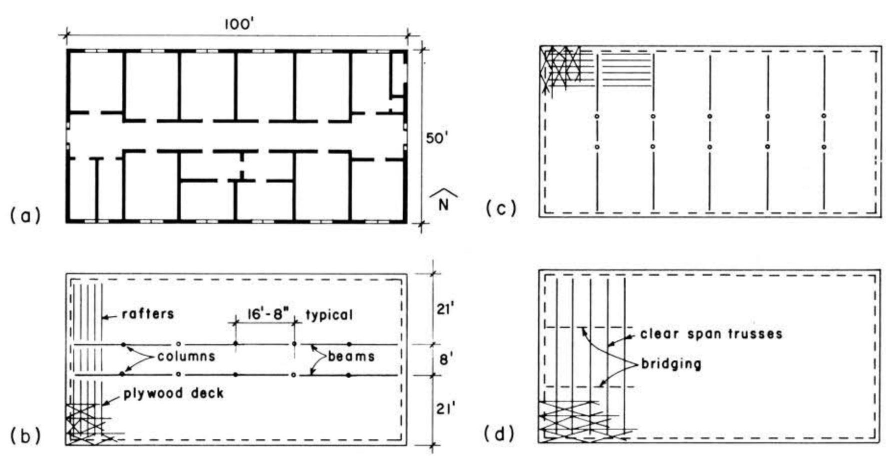


Figure 16.3 Developed plan for interior partitioning and alternatives for the roof framing.

Decking & Rafters:

The standard size of plywood or structural deck panel is 4 ft x 8 ft. The typical orientation is with the long direction with the face grain perpendicular to the rafters or floor joists. (See cross hatching in Figure 16.3.) Typical joist and rafter spacings are 12 in., 16 in., and 24 in. on center. If we use 16 in. on center, the total distributed roof loads (with allowable stress design) with an assumed self weight of 4 lb/ft is:

$$w = (20\ \text{lb/ft}^2 + 14\ \text{lb/ft}^2)\cdot 16\ \text{in}/12\ \text{in/ft} + 4\ \text{lb/ft} = 49.3\ \text{lb/ft}$$

$$M_{max} = \frac{wL^2}{8} = \frac{49.3^{lb/ft}(21^{ft})^2}{8} = 2718^{\ lb\text{-}ft}$$

Tabular allowable stresses for No. 2 Douglas fir-larch, 2"-4" thick and 2" to 4" wide are:

$$F_{b-single} = 875^{psi},\ F_v = 95^{psi},\ F_{c\perp} = 625^{psi},\ F_c = 1300^{psi},\ E = 1{,}600{,}000^{psi}$$

The load duration for roof loads, $C_D = 1.25$. The repetitive member factor, $C_r = 1.15$, applies and the adjusted allowed stress for a fully braced 2x is:

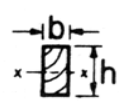
$$F_b' = C_D C_r F_b = (1.25)(1.15)(875\ \text{psi}) = 1258\ \text{psi}$$

The required section modulus is

$$S_{req'd} \geq \frac{M}{F_b'} = \frac{2718^{lb-ft}\cdot 12^{in/ft}}{1258^{psi}} = 25.9\ \text{in}^3$$

A 2x12 will work if the deflection is limited to allowable for the building code. (This tends to govern for floors. Shear stress should also be checked).

SECTION PROPERTIES JOISTS AND BEAMS



Nominal Size In Inches b h	Surfaced Size In Inches For Design b h	Area (A) $A = bh$ (In²)	Section Modulus (S) $S = \frac{bh^2}{6}$ (In³)	Moment of Inertia (I) $I = \frac{bh^3}{12}$ (In⁴)	Board Feet Per Linear Foot of Piece
2 x 2	1.5 x 1.5	2.25	0.562	0.422	0.33
2 x 3	1.5 x 2.5	3.75	1.56	1.95	0.50
2 x 4	1.5 x 3.5	5.25	3.06	5.36	0.67
2 x 5	1.5 x 4.5	6.75	5.06	11.39	.83
2 x 6	1.5 x 5.5	8.25	7.56	20.80	1.00
2 x 8	1.5 x 7.25	10.88	13.14	47.63	1.33
2 x 10	1.5 x 9.25	13.88	21.39	98.93	1.67
2 x 12	1.5 x 11.25	16.88	31.64	177.98	2.00
2 x 14	1.5 x 13.25	19.88	43.89	290.78	2.33
3 x 3	2.5 x 2.5	6.25	2.60	3.26	0.75
3 x 4	2.5 x 3.5	8.75	5.10	8.93	1.00
3 x 5	2.5 x 4.5	11.25	8.44	18.98	1.25
3 x 6	2.5 x 5.5	13.75	12.60	34.66	1.50
3 x 8	2.5 x 7.25	18.12	21.90	79.39	2.00
3 x 10	2.5 x 9.25	23.12	35.65	164.89	2.50
3 x 12	2.5 x 11.25	28.12	52.73	296.63	3.00
3 x 14	2.5 x 13.25	33.12	73.15	484.63	3.50
3 x 16	2.5 x 15.25	38.12	96.90	738.87	4.00

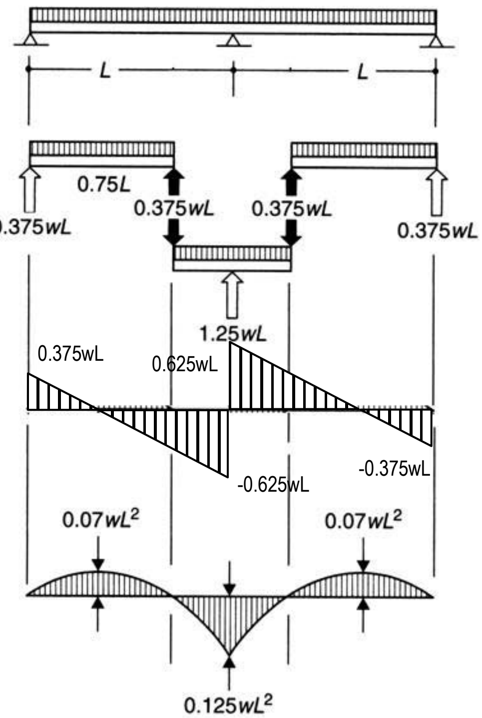
Continuous Beams:

The distributed load, including an estimated self weight of 11 lb/ft (about a 6 in x 12 in section) of a glulam beam can be found from:

rafter distributed load:

$$\frac{\gamma \cdot A \cdot trib.\,width}{rafter\ spacing} = \frac{(32\,{}^{lb}\!/\!_{ft^3})(16.88in^2)(21\,{}^{ft}\!/\!_{2} + 8\,{}^{ft}\!/\!_{2})}{16in} \cdot \left(\frac{1ft}{12in}\right)^2 \cdot \frac{12in}{ft} = 40.8\,{}^{lb}\!/\!_{ft}$$

roof load:



$(20\ lb/ft^2 + 14\ lb/ft^2)\cdot(21ft/2+8ft/2) = 493\ ^{lb/ft}$

total distributed load:

w = 40.8 lb/ft + 493 lb/ft + 11 lb/ft = 545 lb/ft

The maximum positive moment is $0.07wL^2$ and the maximum negative moment over the support is $0.125wL^2$, where L is the length of one span. $V_{max} = 0.625wL$. (These values come from a beam diagram.)

$M_{max} = 0.125(545\ lb/ft)(16.67\ ft)^2 = 18{,}931\ ^{lb\text{-}ft}$

$V_{max} = 0.625(545\ lb/ft)(16.67\ ft) = 5678\ lb$

$F_b' = C_D F_b = (1.25)(2400\ psi) = 3000\ psi$

$$S_{req'd} \geq \frac{M}{F_b'} = \frac{18931^{lb-ft}}{3000^{psi}} \cdot 12\,{}^{in}\!/\!_{ft} = 75.7\ in^3$$

From SECTION PROPERTIES/STANDARD SIZES, the 5 ⅛ " x 10.5" is adequate, although a 3 ⅛ " x 13.5" could be evaluated.

DEPTH, d in.	AREA, A in.²	MODIFIED SECTION MODULUS, SC_F in.³	MOMENT OF INERTIA, I in.⁴
3⅛" WIDTH			
6.0	18.8	18.8	56
7.5	23.4	29.3	110
9.0	28.1	42.2	190
10.5	32.8	57.4	302
12.0	37.5	75.0	450
13.5	42.2	93.7	641
15.0	46.9	114.3	879
16.5	51.6	136.9	1,170
18.0	56.3	161.3	1,519
19.5	60.9	187.6	1,931
21.0	65.6	215.8	2,412
22.5	70.3	245.9	2,966
24.0	75.0	277.8	3,600
5⅛" WIDTH			
7.5	38.4	48.0	180
9.0	46.1	69.2	311
10.5	53.8	94.2	494
12.0	61.5	123.0	738
13.5	69.2	153.6	1,051
15.0	76.9	187.5	1,441
16.5	84.6	224.5	1,919
18.0	92.3	264.6	2,491
19.5	99.9	307.7	3,167
21.0	107.6	354.0	3,955
22.5	115.3	403.2	4,865
24.0	123.0	455.5	5,904
25.5	130.7	510.8	7,082
27.0	138.4	569.0	8,406
28.5	146.1	630.2	9,887
30.0	153.8	694.3	11,531
31.5	161.4	761.4	13,349
33.0	169.1	831.3	15,348
34.5	176.8	904.1	17,538
36.0	184.5	979.8	19,926
6¾" WIDTH			
12.0	81.0	162.0	972
13.5	91.1	202.4	1,384
15.0	101.3	246.9	1,898
16.5	111.4	295.6	2,597
18.0	121.5	348.4	3,280
19.5	131.6	405.3	4,171
21.0	141.8	466.2	5,209
22.5	151.9	531.1	6,407

TABLE DF-25

DOUGLAS FIR - LARCH

THE AMERICAN INSTITUTE OF TIMBER CONSTRUCTION

Structural Glued Laminated Timber

ROOF BEAMS
CONSTRUCTION LOAD

F_b	F_v	E	C_D	Deflection limit
2400 psi	240 psi	1.8 million psi	1.25	Span / 180 for TOTAL LOAD

Simple Span Beams
For Preliminary Design Purposes
Lamination thickness: 1.500 in.

BEAM SIZE		BEAM	BEAM CAPACITY, UNIFORM LOAD w, plf																
Width b, in.	Depth d, in.	WEIGHT plf	SPAN, ft 8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
3 1/8	6	4.6	586 D	412 D	300 D	225 D	174 D	137 D	109 D	89 D	--	--	--	--	--	--	--	--	--
3 1/8	7 1/2	5.7	916 B	723 B	586 D	440 D	339 D	267 D	214 D	174 D	143 D	119 D	100 D	--	--	--	--	--	--
3 1/8	9	6.8	1318 B	1042 B	844 B	697 B	586 D	461 D	369 D	300 D	247 D	206 D	174 D	148 D	127 D	109 D	95 D	--	--
3 1/8	10 1/2	8.0	1794 B	1418 B	1148 B	949 B	798 B	680 B	586 D	476 D	393 D	327 D	276 D	234 D	201 D	174 D	151 D	132 D	116 D
3 1/8	12	9.1	2344 B	1852 B	1500 B	1240 B	1042 B	888 B	765 B	667 B	586 D	488 D	412 D	350 D	300 D	259 D	225 D	197 D	174 D
3 1/8	13 1/2	10.3	2935 S	2344 B	1898 B	1569 B	1318 B	1123 B	969 B	844 B	742 B	657 B	586 D	498 D	427 D	369 D	321 D	281 D	247 D
3 1/8	15	11.4	3000 *	2885 S	2344 B	1937 B	1628 B	1387 B	1196 B	1042 B	916 B	811 B	723 B	649 B	586 D	506 D	440 D	385 D	339 D
3 1/8	16 1/2	12.5	3000 *	3000 *	2836 B	2344 B	1969 B	1678 B	1447 B	1260 B	1108 B	981 B	875 B	786 B	709 B	643 B	586 D	513 D	451 D
3 1/8	18	13.7	3000 *	3000 *	3000 *	2789 B	2344 B	1997 B	1722 B	1500 B	1318 B	1168 B	1042 B	935 B	844 B	765 B	697 B	638 B	583 B
3 1/8	19 1/2	14.8	3000 *	3000 *	3000 *	3000 *	2751 B	2344 B	2021 B	1760 B	1547 B	1371 B	1223 B	1097 B	990 B	898 B	815 B	743 B	679 B
5 1/8	6	7.5	961 D	675 D	492 D	370 D	285 D	224 D	179 D	146 D	--	--	--	--	--	--	--	--	--
5 1/8	7 1/2	9.3	1501 B	1186 B	961 D	722 D	556 D	437 D	350 D	285 D	235 D	196 D	165 D	--	--	--	--	--	--
5 1/8	9	11.2	2162 B	1708 B	1384 B	1144 B	961 D	756 D	605 D	492 D	405 D	338 D	285 D	242 D	208 D	179 D	156 D	--	--
5 1/8	10 1/2	13.1	2943 B	2325 B	1883 B	1557 B	1308 B	1114 B	961 D	781 D	644 D	537 D	452 D	384 D	330 D	285 D	248 D	217 D	191 D
5 1/8	12	14.9	3844 B	3037 B	2460 B	2033 B	1708 B	1456 B	1255 B	1093 B	961 D	801 D	675 D	574 D	492 D	425 D	370 D	323 D	285 D
5 1/8	13 1/2	16.8	4813 S	3844 B	3113 B	2573 B	2162 B	1842 B	1588 B	1384 B	1216 B	1077 B	961 D	817 D	701 D	605 D	526 D	461 D	405 D
5 1/8	15	18.7	5591 S	4731 S	3844 B	3177 B	2669 B	2274 B	1961 B	1708 B	1501 B	1328 B	1178 B	1052 B	944 B	830 D	722 D	632 D	556 D
5 1/8	16 1/2	20.6	6000 *	5412 S	4651 B	3844 B	3230 B	2752 B	2373 B	2067 B	1808 B	1592 B	1412 B	1261 B	1132 B	1022 B	926 B	841 D	740 D
5 1/8	18	22.4	6000 *	6000 *	5271 S	4574 B	3844 B	3275 B	2824 B	2443 B	2133 B	1878 B	1666 B	1487 B	1335 B	1205 B	1093 B	996 B	911 B
5 1/8	19 1/2	24.3	6000 *	6000 *	5922 S	5158 S	4511 B	3841 B	3288 B	2844 B	2484 B	2187 B	1940 B	1731 B	1555 B	1403 B	1273 B	1159 B	1060 B
5 1/8	21	26.2	6000 *	6000 *	6000 *	5740 S	5065 S	4422 B	3785 B	3274 B	2859 B	2518 B	2233 B	1993 B	1790 B	1615 B	1465 B	1334 B	1220 B
5 1/8	22 1/2	28.0	6000 *	6000 *	6000 *	6000 *	5591 S	4986 S	4315 B	3733 B	3260 B	2870 B	2546 B	2272 B	2040 B	1842 B	1670 B	1521 B	1391 B
5 1/8	24	29.9	6000 *	6000 *	6000 *	6000 *	6000 *	5467 S	4878 B	4220 B	3685 B	3245 B	2878 B	2569 B	2306 B	2082 B	1888 B	1720 B	1573 B
5 1/8	25 1/2	31.8	6000 *	6000 *	6000 *	6000 *	6000 *	5974 S	5362 S	4735 B	4135 B	3641 B	3229 B	2882 B	2588 B	2336 B	2119 B	1930 B	1765 B
6 3/4	6	9.8	1266 D	889 D	648 D	487 D	375 D	295 D	236 D	192 D	--	--	--	--	--	--	--	--	--
6 3/4	7 1/2	12.3	1978 B	1563 B	1266 D	951 D	732 D	576 D	461 D	375 D	309 D	258 D	217 D	--	--	--	--	--	--
6 3/4	9	14.8	2848 B	2250 B	1823 B	1506 B	1266 D	995 D	797 D	648 D	534 D	445 D	375 D	319 D	273 D	236 D	205 D	--	--
6 3/4	10 1/2	17.2	3876 B	3063 B	2481 B	2050 B	1723 B	1468 B	1266 D	1029 D	848 D	707 D	595 D	506 D	434 D	375 D	326 D	285 D	251 D
6 3/4	12	19.7	5063 B	4000 B	3240 B	2678 B	2250 B	1917 B	1653 B	1440 B	1265 B	1055 D	889 D	756 D	648 D	560 D	487 D	426 D	375 D
6 3/4	13 1/2	22.1	6339 S	5063 B	4101 B	3389 B	2848 B	2426 B	2092 B	1812 B	1583 B	1393 B	1236 B	1076 D	923 D	797 D	693 D	607 D	534 D
6 3/4	15	24.6	7364 S	6231 S	5063 B	4184 B	3516 B	2990 B	2559 B	2214 B	1933 B	1702 B	1510 B	1348 B	1210 B	1092 B	951 D	832 D	732 D
6 3/4	16 1/2	27.1	8000 *	7128 S	6126 B	5063 B	4239 B	3583 B	3067 B	2653 B	2317 B	2040 B	1809 B	1615 B	1450 B	1309 B	1187 B	1081 B	975 D
6 3/4	18	29.5	8000 *	8000 *	6943 S	6004 B	5001 B	4228 B	3618 B	3130 B	2734 B	2407 B	2135 B	1905 B	1711 B	1544 B	1401 B	1276 B	1167 B
6 3/4	19 1/2	32.0	8000 *	8000 *	7800 S	6794 S	5823 B	4922 B	4213 B	3644 B	3183 B	2802 B	2485 B	2218 B	1992 B	1798 B	1631 B	1485 B	1358 B
6 3/4	21	34.5	8000 *	8000 *	8000 *	7560 S	6671 S	5666 B	4850 B	4196 B	3664 B	3226 B	2861 B	2554 B	2293 B	2070 B	1877 B	1710 B	1564 B
6 3/4	22 1/2	36.9	8000 *	8000 *	8000 *	8000 *	7364 S	6460 B	5529 B	4783 B	4177 B	3678 B	3262 B	2912 B	2614 B	2360 B	2140 B	1949 B	1783 B
6 3/4	24	39.4	8000 *	8000 *	8000 *	8000 *	8000 *	7200 S	6250 B	5407 B	4722 B	4157 B	3687 B	3291 B	2955 B	2667 B	2419 B	2204 B	2015 B
6 3/4	25 1/2	41.8	8000 *	8000 *	8000 *	8000 *	8000 *	7869 S	7013 B	6067 B	5298 B	4665 B	4137 B	3693 B	3316 B	2993 B	2715 B	2473 B	2261 B
6 3/4	27	44.3	8000 *	8000 *	8000 *	8000 *	8000 *	8000 *	7674 S	6763 B	5906 B	5200 B	4612 B	4117 B	3696 B	3337 B	3026 B	2756 B	2521 B
6 3/4	28 1/2	46.8	8000 *	8000 *	8000 *	8000 *	8000 *	8000 *	8000 *	7495 B	6545 B	5763 B	5111 B	4562 B	4096 B	3697 B	3353 B	3054 B	2793 B

TABLE SPECIFICATIONS: This table applies to straight, simply supported glued laminated timber beams under dry conditions of use. Beams must be laterally supported at the top along the length of the beam and at the top and bottom at the ends. The load carrying capacities tabulated are for total load including the weight of the member.

BEAM WEIGHT: 35.0 pounds per cubic foot was used to determine beam weight per lineal foot shown in the table.

DESIGN VALUE MODIFICATIONS: The allowable stress in bending , Fb , has been adjusted by the AITC volume factor, CV .

For determination of load carrying capacities governed by shear, loads within a distance "d" (the depth of the beam) from the ends have been neglected.

DEFLECTION LIMITS: For roof beams, deflection is limited to span /180 for total load.

CONTROLLING VALUES: Values marked with a D are controlled by deflection, B are bending controlled, and S are shear controlled.

SPAN: Span is defined as the length from centerline to centerline of bearing. This span is the length used in standard engineering equations to calculate deflection, bending and shear.

* The values have been limited to reasonable capacities. Engineering calculations may allow for greater capacities.

Feb-2001

The self weight should be determined to compare to the assumption. Table DF-25 indicates the self weight is 13 lb/ft, and that size at our span is controlled by deflection (I for Δ=L/180), but this chart is for *simply supported beams* and $\Delta_{max} = \frac{5wL^4}{384EI}$.

The maximum deflection for a two span beam can be found with $\Delta_{max} = \frac{wL^4}{185EI}$, which is only 0.415x the deflection of a simply supported span.

For sawn lumber, a 6x14 would be required from the comparison chart.

Equivalent Glulam Sections for Dimension Lumber/Timber Beams

Sawn[4] Sections Nominal Size	Roof Beams[1, 2] Select Structural Douglas Fir/Larch	Select Structural Southern Pine	No. 1 Douglas Fir/Larch	No. 1 Southern Pine
3x8	3 1/8x6	3x6 7/8	3 1/8x6	3x5 1/2
3x10	3 1/8x7 1/2	3x8 1/4	3 1/8x6	3x6 7/8
3x12	3 1/8x9	3x9 5/8	3 1/8x7 1/2	3x8 1/4
3x14	3 1/8x9	3x11	3 1/8x7 1/2	3x9 5/8
4x6	3 1/8x6	3x6 7/8	3 1/8x6	3x5 1/2
4x8	3 1/8x7 1/2	3x8 1/4	3 1/8x6	3x6 7/8
4x10	3 1/8x9	3x11	3 1/8x7 1/2	3x8 1/4
4x12	3 1/8x10 1/2	3x12 3/8	3 1/8x9	3x9 5/8
4x14	3 1/8x12	3x13 3/4	3 1/8x10 1/2	3x11
4x16	3 1/8x13 1/2	3x15 1/8	3 1/8x10 1/2	3x12 3/8
6x8	5 1/8x7 1/2	5x6 7/8	5 1/8x7 1/2	5x6 7/8
6x10	5 1/8x9	5x8 1/4	5 1/8x7 1/2	5x8 1/4
6x12	5 1/8x10 1/2	5x9 5/8	5 1/8x9	5x9 5/8
6x14	5 1/8x12	5x12 3/8	5 1/8x10 1/2	5x11
6x16	5 1/8x13 1/2	5x13 3/4	5 1/8x12	5x12 3/8
6x18	5 1/8x15	5x15 1/8	5 1/8x13 1/2	5x13 3/4
6x20	5 1/8x18	5x16 1/2	5 1/8x16 1/2	5x15 1/8

Evaluate shear strength:

$$F'_v = C_D F_v = (1.25)240 \text{ psi} = 300 \text{ psi}$$

$$f_v = \frac{3V}{2A} = \frac{3(5678lb)}{2(53.8in^2)} = 158psi$$

which is less than the allowable of 300 psi (OK).

Stud Walls & Columns:

Building codes dictate the maximum height for slenderness (10 ft typical), and the spacing of wall studs depending on what they support (roof, roof and one floor, roof and two floors). Structural design focuses on shear wall behavior.

The interior column load is:

$$P = 1.25wL = 1.25(545 \text{ lb/ft} + 2 \text{ lb/ft of extra beam self weight})(16.67 \text{ ft}) = 11.4^{\text{kips}}$$

For a 10 ft braced column height, choose a 6 x 6.

TABLE 10.1 Safe Loads for Wood Columns[a]

Column Section Nominal Size	Area (in.²)	Unbraced Length (ft) 6	8	10	12	14	16	18	20	22	24	26
4 × 4	12.25	11.1	7.28	4.94	3.50	2.63						
4 × 6	19.25	17.4	11.4	7.76	5.51	4.14						
4 × 8	25.375	22.9	15.1	10.2	7.26	6.46						
6 × 6	30.25	27.6	24.8	20.9	16.9	13.4	10.7	8.71	7.17	6.53		
6 × 8	41.25	37.6	33.9	28.5	23.1	18.3	14.6	11.9	9.78	8.91		
6 × 10	52.25	47.6	43.0	36.1	29.2	23.1	18.5	15.0	13.4	11.3		
8 × 8	56.25	54.0	51.5	48.1	43.5	38.0	32.3	27.4	23.1	19.7	16.9	14.6
8 × 10	71.25	68.4	65.3	61.0	55.1	48.1	41.0	34.7	29.3	24.9	21.4	18.4
8 × 12	86.25	82.8	79.0	73.8	66.7	58.2	49.6	42.0	35.4	30.2	26.0	22.3
10 × 10	90.25	88.4	85.9	83.0	79.0	73.6	67.0	60.0	52.9	46.4	40.4	35.5
10 × 12	109.25	107	104	100	95.6	89.1	81.2	72.6	64.0	56.1	48.9	42.9
10 × 14	128.25	126	122	118	112	105	95.3	85.3	75.1	65.9	57.5	50.4
12 × 12	132.25	130	128	125	122	117	111	104	95.6	86.9	78.3	70.2
14 × 14	182.25	180	178	176	172	168	163	156	148	139	129	119
16 × 16	240.25	238	236	234	230	226	222	216	208	200	190	179

[a]Load capacity in kips for solid-sawn sections of No. 1 grade Douglas fir–larch under normal moisture and load duration conditions.

Wind Design:

Diaphragms are categorized as flexible or rigid and must resist lateral forces in both transverse and longitudinal directions. A diaphragm is made up of a shear-resisting element (sheathing) and boundary members called *chords* and *collectors (struts or drag struts)*. The chords are designed to carry the moment in the diaphragm. The collectors are designed to transmit the horizontal reactions to the shear walls. The structural behavior is often compared to that of a steel I section on its side (Figure 15.6).

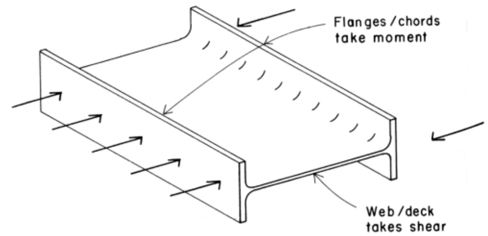


Figure 15.6 Flanged and webbed beam analogy for a horizontal, wood-framed diaphragm.

Tables in building codes for combinations of plywood grade, common nail size, plywood thickness, how the panels are arrayed and if blocking is used provide allowable shear in pounds per foot.

Consideration of lateral wind loads will be presented, but uplift on the roof must be accounted for with anchorage if the live load exceeds the downward gravity loads.

Selected Tables from the *Uniform Building Code*, 1997 Edition C.23

TABLE 23-II-H—ALLOWABLE SHEAR IN POUNDS PER FOOT FOR HORIZONTAL WOOD STRUCTURAL PANEL DIAPHRAGMS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1]

PANEL GRADE	COMMON NAIL SIZE	MINIMUM NAIL PENETRATION IN FRAMING (inches)	MINIMUM NOMINAL PANEL THICKNESS (inches)	MINIMUM NOMINAL WIDTH OF FRAMING MEMBER (inches)	BLOCKED DIAPHRAGMS: Nail spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 and 4) and at all panel edges (Cases 5 and 6) (× 25.4 for mm): 6	4	$2^1/_2$[2]	2[2]	UNBLOCKED DIAPHRAGMS: Nails spaced 6" (152 mm) max. at supported edges: Case 1 (No unblocked edges or continuous joints parallel to load)	All other configurations (Cases 2, 3, 4, 5 and 6)
					Nail spacing (in.) at other panel edges (× 25.4 for mm): 6	6	4	3		
		× 25.4 for mm			× 0.0146 for N/mm					
Structural 1	6d	$1^1/_4$	$^5/_{16}$	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	$1^1/_2$	$^3/_8$	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d[3]	$1^5/_8$	$^{15}/_{32}$	2	320	425	640	730	285	215
				3	360	480	720	820	320	240
C-D, C-C, Sheathing, and other grades covered in UBC Standard 23-2 or 23-3	6d	$1^1/_4$	$^5/_{16}$	2	170	225	335	380	150	110
				3	190	250	380	430	170	125
			$^3/_8$	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	$1^1/_2$	$^3/_8$	2	240	320	480	545	215	160
				3	270	360	540	610	240	180
			$^7/_{16}$	2	255	340	505	575	230	170
				3	285	380	570	645	255	190
			$^{15}/_{32}$	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d[3]	$1^5/_8$	$^{15}/_{32}$	2	290	385	575	655	255	190
				3	325	430	650	735	290	215
			$^{19}/_{32}$	2	320	425	640	730	285	215
				3	360	480	720	820	320	240

[1]These values are for short-time loads due to wind or earthquake and must be reduced 25 percent for normal loading. Space nails 12 inches (305 mm) on center along intermediate framing members.

Allowable shear values for nails in framing members of other species set forth in Division III, Part III, shall be calculated for all other grades by multiplying the shear capacities for nails in Structural I by the following factors: 0.82 for species with specific gravity greater than or equal to 0.42 but less than 0.49, and 0.65 for species with a specific gravity less than 0.42.

[2]Framing at adjoining panel edges shall be 3-inch (76 mm) nominal or wider and nails shall be staggered where nails are spaced 2 inches (51 mm) or $2^1/_2$ inches (64 mm) on center.

[3]Framing at adjoining panel edges shall be 3-inch (76 mm) nominal or wider and nails shall be staggered where 10d nails having penetration into framing of more than $1^5/_8$ inches (41 mm) are spaced 3 inches (76 mm) or less on center.

North-South

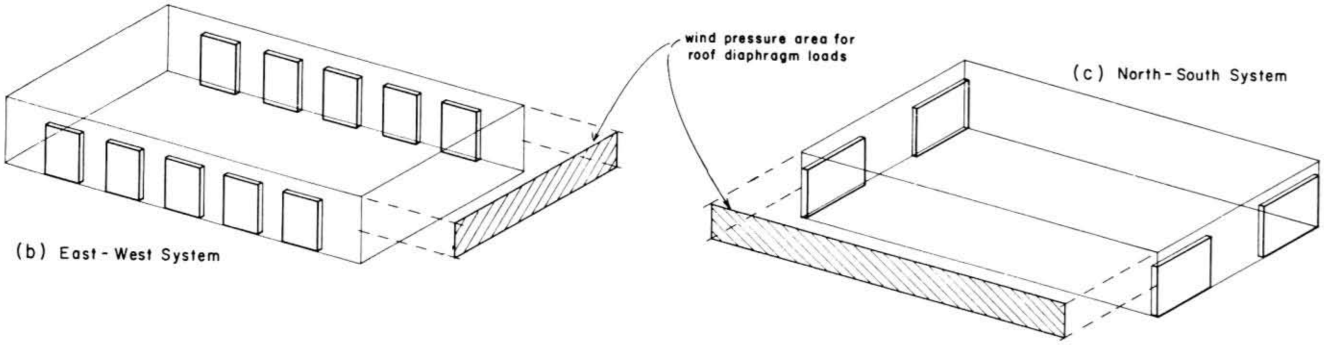
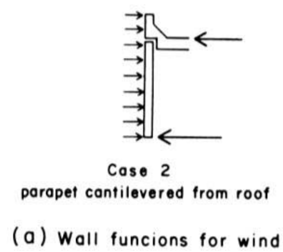


Figure 16.5 Building One, wall functions and wind pressure development.

The tributary height for the wall and parapet is 17.5 ft/2 + 2.5 ft = 11.25 ft



The distributed lateral wind load = (20 lb/ft^2)11.25ft = 225 lb/ft

The total lateral wind load = (225 lb/ft)(100 ft) = 22,500 lb

The end reactions to the lateral load = 22,500 lb/2 = 11,250 lb

The *unit shear* (or distributed shear) in the **diaphragm** = 11,250 lb/(50 ft) = 225 lb/ft;

so a roof deck can be chosen that has an allowable shear > 225 lb/ft.

Knowing that ½ in decking is the minimum for a membrane-type roof, we use table 23-II-H to select $^{15}/_{32}$ in. sheathing with 2 x framing and 8d nails at 6 in. at all panel edges and a blocked diaphragm having an allowable shear in pounds per foot of 270 lb/ft.

The moment of the "deep beam" is used to determine the force in the top and bottom chords as show in Figure 16.6 which is 5.62 kips.

The *unit shear* in the two **shear walls** of 21 ft each = 11,250 lb/(2·21 ft) = 268 lb/ft;

so a stud wall can be chosen that has an allowable shear > 268 lb/ft.

Using table 23-II-I-1, $^{3}/_{8}$ in. plywood sheathing with 6d nails at 4 in. at all panel edges directly applied to framing (not over gypsum sheathing) has an allowable shear in pounds per foot of 300 lb/ft.

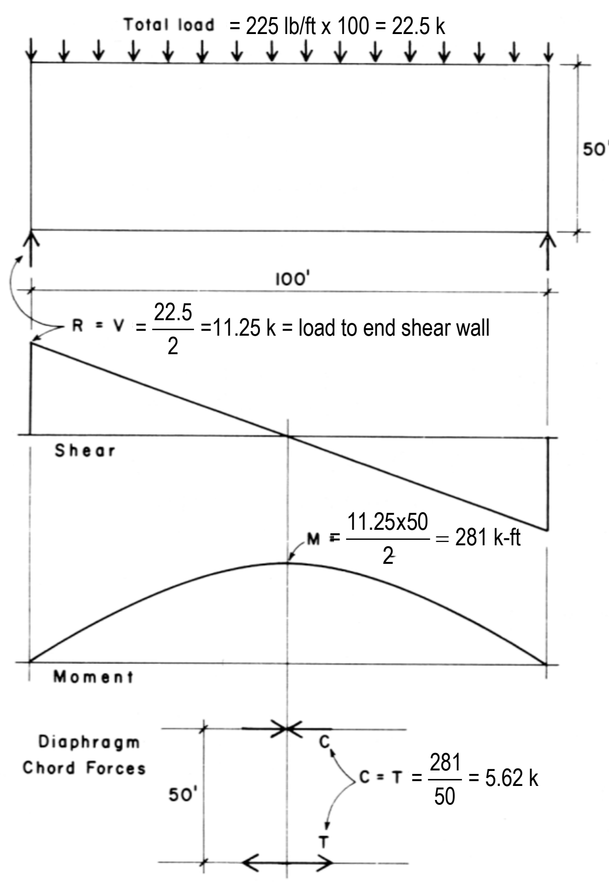


Figure 16.6 Spanning functions of the roof diaphragm.

TABLE 23-II-I-1—ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES IN POUNDS PER FOOT FOR WOOD STRUCTURAL PANEL SHEAR WALLS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1,2,3]

PANEL GRADE	MINIMUM NOMINAL PANEL THICKNESS (inches)	MINIMUM NAIL PENETRATION IN FRAMING (inches)	PANELS APPLIED DIRECTLY TO FRAMING: Nail Size (Common or Galvanized Box)[5]	Nail Spacing at Panel Edges (in.) 6	4	3	2	PANELS APPLIED OVER 1/2-INCH (13 mm) OR 5/8-INCH (16 mm) GYPSUM SHEATHING: Nail Size (Common or Galvanized Box)[5]	Nail Spacing at Panel Edges (in.) 6	4	3	2
	× 25.4 for mm			× 25.4 for mm; × 0.0146 for N/mm					× 25.4 for mm; × 0.0146 for N/mm			
Structural I	5/16	1 1/4	6d	200	300	390	510	8d	200	300	390	510
	3/8	1 1/2	8d	230[4]	360[4]	460[4]	610[4]	10d	280	430	550	730
	7/16			255[4]	395[4]	505[4]	670[4]					
	15/32			280	430	550	730					
	15/32	1 5/8	10d	340	510	665	870	—	—	—	—	—
C-D, C-C Sheathing, plywood panel siding and other grades covered in UBC Standard 23-2 or 23-3	5/16	1 1/4	6d	180	270	350	450	8d	180	270	350	450
	3/8			200	300	390	510		200	300	390	510
	3/8	1 1/2	8d	220[4]	320[4]	410[4]	530[4]	10d	260	380	490	640
	7/16			240[4]	350[4]	450[4]	585[4]					
	15/32			260	380	490	640					
	15/32	1 5/8	10d	310	460	600	770	—	—	—	—	—
	19/32			340	510	665	870					
			Nail Size (Galvanized Casing)					Nail Size (Galvanized Casing)				
Plywood panel siding in grades covered in UBC Standard 23-2	5/16	1 1/4	6d	140	210	275	360	8d	140	210	275	360
	3/8	1 1/2	8d	160	240	310	410	10d	160	240	310	410

[1]All panel edges backed with 2-inch (51 mm) nominal or wider framing. Panels installed either horizontally or vertically. Space nails at 6 inches (152 mm) on center along intermediate framing members for 3/8-inch (9.5 mm) and 7/16-inch (11 mm) panels installed on studs spaced 24 inches (610 mm) on center and 12 inches (305 mm) on center for other conditions and panel thicknesses. These values are for short-time loads due to wind or earthquake and must be reduced 25 percent for normal loading.

Allowable shear values for nails in framing members of other species set forth in Division III, Part III, shall be calculated for all other grades by multiplying the shear capacities for nails in Structural I by the following factors: 0.82 for species with specific gravity greater than or equal to 0.42 but less than 0.49, and 0.65 for species with a specific gravity less than 0.42.

[2]Where panels are applied on both faces of a wall and nail spacing is less than 6 inches (152 mm) on center on either side, panel joints shall be offset to fall on different framing members or framing shall be 3-inch (76 mm) nominal or thicker and nails on each side shall be staggered.

[3]Where allowable shear values exceed 350 pounds per foot (5.11 N/mm), foundation sill plates and all framing members receiving edge nailing from abutting panels shall not be less than a single 3-inch (76 mm) nominal member. Nails shall be staggered.

[4]The values for 3/8-inch (9.5 mm) and 7/16-inch (11 mm) panels applied direct to framing may be increased to values shown for 15/32-inch (12 mm) panels, provided studs are spaced a maximum of 16 inches (406 mm) on center or panels are applied with long dimension across studs.

[5]Galvanized nails shall be hot-dipped or tumbled.

Wall overturning must be considered from the shear and compared to the resisting moment from gravity loads and proper anchorage must be provided to keep the wall from sliding off the foundation. Referring to Figure 16.7:

$V = 11.25 \text{ k}/ 2 = 5.625 \text{ k}$

Roof dead load is determined from a tributary area of half a rafter spacing width, one rafter, and the wall length

roof DL = $(14 \text{ lb/ft}^2 \cdot 16 \text{ in}/12 \text{ in/ft}/2 + 4 \text{ lb/ft})21 \text{ ft} = 280 \text{ lb}$

Wall dead load can be determined with the material weights for stud walls, sheathing, gypsum board and wood shingles:

wall DL= $(2 \text{ lb/ft}^2 + 3 \text{ lb/ft}^2 + 5 \text{ lb/ft}^2 + 2 \text{ lb/ft}^2)(21 \text{ ft})(17 \text{ ft}) = 4.3 \text{ k}$

overturning moment = $(5.625\text{k})(17 \text{ ft}) = 95.6 \text{ k-ft}$

resisting moment = $(4.6 \text{ k})(21 \text{ ft})/2 = 48.4 \text{ k-ft}$

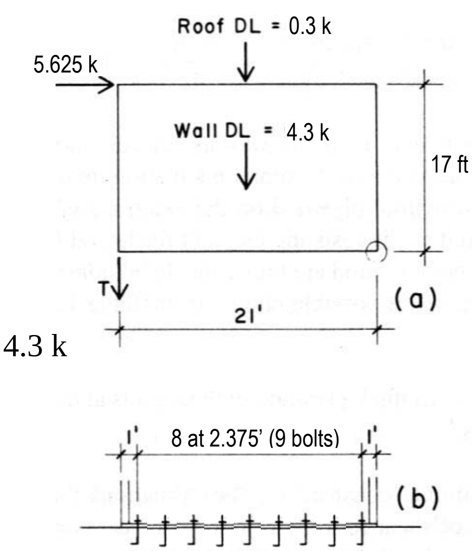


Figure 16.7

Table 17-13.
Weights of Building Materials

Materials	Weight lb per sq ft	Materials	Weight lb per sq ft
CEILINGS		**PARTITIONS**	
Channel suspended system	1	Clay tile	
Lathing and plastering	See Partitions	3 in.	17
Acoustical fiber tile	1	4 in.	18
		6 in.	28
		8 in.	34
		10 in.	40
FLOORS		Gypsum block	
Steel deck	See Manufacturer	2 in.	$9\frac{1}{2}$
		3 in.	$10\frac{1}{2}$
Concrete-Reinforced 1 in.		4 in.	$12\frac{1}{2}$
Stone	$12\frac{1}{2}$	5 in.	14
Slag	$11\frac{1}{2}$	6 in.	$18\frac{1}{2}$
Lightweight	6 to 10	Wood studs 2×4	
		12–16 in. o.c.	2
Concrete-Plain 1 in.		Steel partitions	4
Stone	12	Plaster 1 in.	
Slag	11	Cement	10
Lightweight	3 to 9	Gypsum	5
		Lathing	
Fills 1 inch		Metal	$\frac{1}{2}$
Gypsum	6	Gypsum board $\frac{1}{2}$ in.	2
Sand	8		
Cinders	4		
Finishes			
Terrazzo 1 in.	13		
Ceramic or Quarry Tile $\frac{3}{4}$-in.	10	**WALLS**	
Linoleum $\frac{1}{4}$-in.	1	Brick	
Mastic $\frac{3}{4}$-in.	9	4 in.	40
Hardwood $\frac{7}{8}$-in.	4	8 in.	80
Softwood $\frac{3}{4}$-in.	$2\frac{1}{2}$	12 in.	120
		Hollow concrete block	
		(Heavy aggregate)	
ROOFS		4 in.	30
Copper or tin	1	6 in.	43
Corrugated steel	See Manufactuer	8 in.	55
3-ply ready roofing	1	$12\frac{1}{2}$ in.	80
3-ply felt and gravel	$5\frac{1}{2}$	Hollow concrete block	
5-ply felt and gravel	6	(Light aggregate)	
		4 in.	21
Shingles		6 in.	30
Wood	2	8 in.	38
Asphalt	3	12 in.	55
Clay tile	9 to 14	Clay tile (Load bearing)	
Slate $\frac{1}{4}$ in.	10	4 in.	25
		6 in.	30
Sheathing		8 in.	33
Wood $\frac{3}{4}$ in.	3	12 in.	45
Gypsum 1 in.	4	Stone 4 in.	55
		Glass block 4 in.	18
Insulation 1 in.		Window, Glass, Frame, & Sash	8
Loose	$\frac{1}{2}$	Curtain walls	See Manufacturer
Poured	2	Structural glass 1 in.	15
Rigid	$1\frac{1}{2}$	Corrugated Cement Asbestos $\frac{1}{4}$ in.	3

For weights of other materials used in building construction, see Table 17-12.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

The resisting moment is not enough to compensate for the overturning moment. We like the factor of safety for overturning to be 1.5, and there *is no safety* in this case, which means we must provide a tie down in tension (T). The L shape of the corner will help some resisting overturning, as well as the glulam beam reaction.

For equilibrium of moments (positive = negative)

$$SF = \frac{M_{resist}}{M_{overturning}} \geq 1.5$$

$T(21ft) + 48.4$ k-ft $= (95.6$ k-ft$)1.5$; $T_{req'd} = 4.52$ k

The shear must be resisted, and the code minimum bolting usually consists of ½ in. diameter bolts at 1 ft from the wall ends and at a maximum of 6 ft on center for the remainder of the wall length. If design for wind loading allows us to increase the allowable stress by 1/3, the number of bolts from single shear in a 2" sill plate parallel to the grain will be:

$(1.33)(480 \text{ lb/bolt})(n) \geq 5{,}625$ lb

$n \geq 8.8$ bolts

Use 9 bolts, spaced at 2.375 ft

(see next page for description of design value symbols)

TABLE 11.1 Bolt Design Values for Wood Joints with Douglas Fir–Larch (lb/bolt)

THICKNESS			DOUGLAS FIR-LARCH					
MAIN MEMBER	SIDE MEMBER	BOLT DIAMETER	SINGLE SHEAR			DOUBLE SHEAR		
t_m inches	t_s inches	D inches	$Z_{\parallel}$ lbs.	$Z_{s\perp}$ lbs.	$Z_{m\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{s\perp}$ lbs.	$Z_{m\perp}$ lbs.
		1/2	480	300	300	1050	730	470
		5/8	600	360	360	1310	1040	530
1-1/2	1-1/2	3/4	720	420	420	1580	1170	590
		7/8	850	470	470	1840	1260	630
		1	970	530	530	2100	1350	680

$Z_{||}$ = nominal lateral design value for single bolt in connection with all wood members loaded parallel to grain

$Z_{s\perp}$ = nominal lateral design value for single bolt in wood-to-wood connection with main member loaded parallel to grain and side member loaded perpendicular to grain

$Z_{m\perp}$ = nominal lateral design value for single bolt in wood-to-wood connection with main member loaded parallel to grain and side member loaded perpendicular to grain and side member loaded parallel to grain

East-West

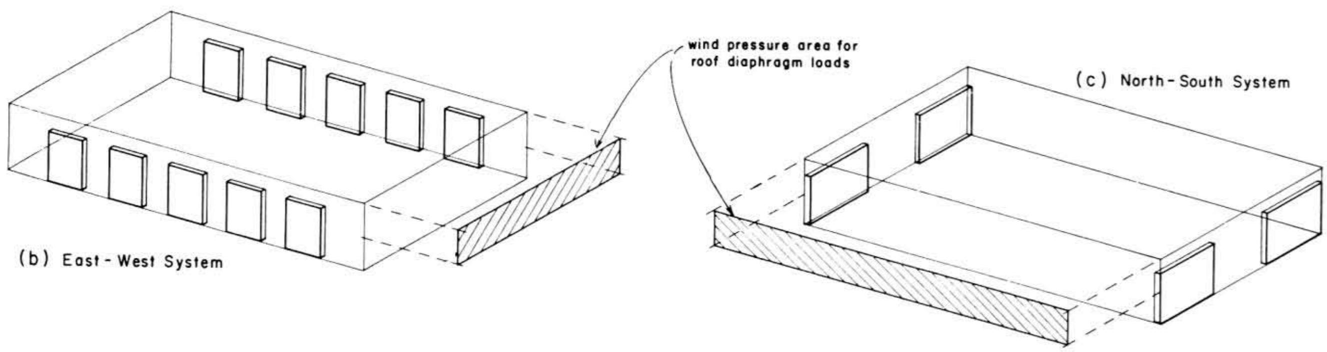


Figure 16.5 Building One, wall functions and wind pressure development.

The tributary height for the wall and parapet and the distributed lateral wind load are the same as in the North-South direction.

The total lateral wind load = (225 lb/ft)(50 ft) = 11,250 lb

The end reactions to the lateral load = 11,250 lb/2 = 5,625 lb

The *unit shear* (or distributed shear) in the **diaphragm** = 5,625 lb/(100 ft) = 56.25 lb/ft.

It is convenient to use the diaphragm structural panel construction chosen in the North-South direction with a capacity of 270 lb/ft.

The *unit shear* (or distributed shear) in the five **shear walls** of 10.67 ft each:

= 5,625 lb/(5·10.67 ft) = 105 lb/ft.

It is convenient to use the shear wall structural panel construction chosen in the North-South direction with a capacity of 300 lb/ft.

Steel Design

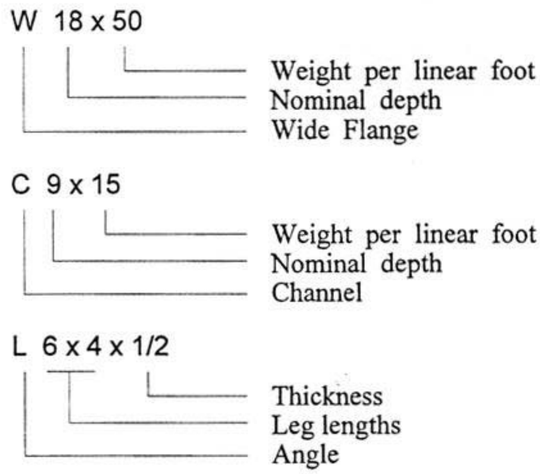
Notation:

a = name for width dimension
A = name for area
A_g = gross area, equal to the total area ignoring any holes
$A_{req'd\text{-}adj}$ = area required at allowable stress when shear is adjusted to include self weight
A_w = area of the web of a wide flange section, as is A_{web}
$AISC$ = American Institute of Steel Construction
ASD = allowable stress design
b = name for a (base) width
= name for height dimension
b_f = width of the flange of a steel beam cross section
B = width of a column base plate
B_1 = factor for determining M_u for combined bending and compression
c = largest distance from the neutral axis to the top or bottom edge of a beam. as is c_{max}
c_1 = coefficient for shear stress for a rectangular bar in torsion
C_b = modification factor for moment in ASD & LRFD steel beam design
C_m = modification factor accounting for combined stress in steel design
C_v = web shear coefficient
d = name for depth
= depth of a wide flange section
D = shorthand for dead load
DL = shorthand for dead load
E = shorthand for earthquake load
= modulus of elasticity
f_a = axial stress
f_b = bending stress
f_p = bearing stress
f_v = shear stress
$f_{v\text{-}max}$ = maximum shear stress
f_y = yield stress
F = shorthand for fluid load
F_a = allowable axial (compressive) stress
F_b = allowable bending stress
F_{cr} = flexural buckling stress
F_e = elastic critical buckling stress
F_p = allowable bearing stress
F_u = ultimate stress prior to failure
F_y = yield strength
F_{yw} = yield strength of web material
h = name for a height
h_c = height of the web of a wide flange steel section
H = shorthand for lateral pressure load
I = moment of inertia with respect to neutral axis bending
I_y = moment of inertia about the y axis
J = polar moment of inertia
k = distance from outer face of W flange to the web toe of fillet
= shape factor for plastic design of steel beams
K = effective length factor for columns, as is k
l = name for length, as is L
= column base plate design variable
L = name for length or span length, as is l
= shorthand for live load
L_b = unbraced length of a steel beam in LRFD design
L_e = effective length that can buckle for column design, as is ℓ_e
L_r = shorthand for live roof load
= maximum unbraced length of a steel beam in LRFD design for inelastic lateral-torsional buckling
L_p = maximum unbraced length of a steel beam in LRFD design for full plastic flexural strength
LL = shorthand for live load
$LRFD$ = load and resistance factor design
m = edge distance for a column base plate
M = internal bending moment
M_a = required bending moment (ASD)
M_{max} = maximum internal bending moment
$M_{max\text{-}adj}$ = maximum bending moment adjusted to include self weight

M_n = nominal flexure strength with the full section at the yield stress for LRFD beam design
M_p = internal bending moment when all fibers in a cross section reach the yield stress
M_u = maximum moment from factored loads for LRFD beam design
M_y = internal bending moment when the extreme fibers in a cross section reach the yield stress
n = edge distance for a column base plate
n' = column base plate design value
$n.a.$ = shorthand for neutral axis
N = bearing length on a wide flange steel section
= depth of a column base plate
P = name for load or axial force vector
P_a = required axial force (ASD)
P_c = available axial strength
P_{e1} = Euler buckling strength
P_r = required axial force
P_n = nominal column load capacity in LRFD steel design
P_p = nominal bearing capacity of concrete under base plate
P_u = factored column load calculated from load factors in LRFD steel design
r = radius of gyration
R = generic load quantity (force, shear, moment, etc.) for LRFD design
= shorthand for rain or ice load
R_a = required strength (ASD)
R_n = nominal value (capacity) to be multiplied by ϕ in LRFD and divided by the safety factor Ω in ASD
R_u = factored design value for LRFD design
S = shorthand for snow load
= section modulus
$S_{req'd}$ = section modulus required at allowable stress
$S_{req'd\text{-}adj}$ = section modulus required at allowable stress when moment is adjusted to include self weight
t_f = thickness of flange of wide flange
t_{min} = minimum thickness of column base plate
t_w = thickness of web of wide flange
T = torque (axial moment)
= shorthand for thermal load
V = internal shear force
V_a = required shear (ASD)
V_{max} = maximum internal shear force
$V_{max\text{-}adj}$ = maximum internal shear force adjusted to include self weight
V_n = nominal shear strength capacity for LRFD beam design
V_u = maximum shear from factored loads for LRFD beam design
$w_{equivalent}$ = the equivalent distributed load derived from the maximum bending moment
$w_{self\ wt}$ = name for distributed load from self weight of member
W = shorthand for wind load
X = column base plate design value
Z = plastic section modulus of a steel beam
Δ_{actual} = actual beam deflection
$\Delta_{allowable}$ = allowable beam deflection
Δ_{limit} = allowable beam deflection limit
Δ_{max} = maximum beam deflection
ε_y = yield strain (no units)
ϕ = resistance factor
ϕ_b = resistance factor for bending for LRFD
ϕ_c = resistance factor for compression for LRFD
ϕ_v = resistance factor for shear for LRFD
λ = column base plate design value
γ = load factor in LRFD design
π = pi (3.1415 radians or 180°)
ρ = radial distance
Ω = safety factor for ASD

Steel Design

Structural design standards for steel are established by the *Manual of Steel Construction* published by the American Institute of Steel Construction, and uses **Allowable Stress Design** and **Load and Factor Resistance Design**. The 13th edition combines both methods in one volume and provides common requirements for analyses and design and requires the application of the same set of specifications.



<u>Materials</u>

American Society for Testing Materials (ASTM) is the organization responsible for material and other standards related to manufacturing. Materials meeting their standards are guaranteed to have the published strength and material properties for a designation.

A36 – carbon steel used for plates, angles	F_y = 36 ksi, F_u = 58 ksi, E = 29,000 ksi
A572 – high strength low-alloy used for some beams	F_y = 60 ksi, F_u = 75 ksi, E = 30,000 ksi
A992 – for building framing used for <u>most beams</u>	F_y = 50 ksi, F_u = 65 ksi, E = 30,000 ksi
(A572 Grade 60 has the same properties as A992)	

<u>ASD</u> $R_a \leq R_n/\Omega$

where R_a = required strength (dead or live; force, moment or stress)
R_n = nominal strength specified for ASD
Ω = safety factor

Factors of Safety are applied to the limit stresses for allowable stress values:

bending (braced, $L_b < L_p$)	Ω = 1.67
bending (unbraced, $L_p < L_b$ and $L_b > L_r$)	Ω = 1.67 (nominal moment reduces)
shear (beams)	Ω = 1.67
shear (bolts)	Ω = 2.00 (tabular nominal strength)
shear (welds)	Ω = 2.00

- L_b is the unbraced length between bracing points, laterally
- L_p is the limiting laterally unbraced length for the limit state of yielding
- L_r is the limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling

LRFD $R_u \leq \phi R_n$ $\quad where \cdots R_u = \Sigma \gamma_i R_i$

where ϕ = resistance factor
γ = load factor for the type of load
R = load (dead or live; force, moment or stress)
R_u = factored load (moment or stress)
R_n = nominal load (ultimate capacity; force, moment or stress)

Nominal strength is defined as the

> capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths (such as yield strength, F_y, or ultimate strength, F_u) and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions

Factored Load Combinations

The design strength, ϕR_n, of each structural element or structural assembly must equal or exceed the design strength based on the ASCE-7 combinations of factored nominal loads:

$1.4(D + F)$
$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
$1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
$1.2D + 1.0E + L + 0.2S$
$0.9D + 1.6W + 1.6\text{ H}$
$0.9D + 1.0E + 1.6\text{ H}$

Criteria for Design of Beams

Allowable normal stress or normal stress from LRFD should not be exceeded:

$$F_b \text{ or } \phi F_n \geq f_b = \frac{Mc}{I}$$
$$(M_a \leq M_n / \Omega \text{ or } M_u \leq \phi_b M_n)$$

Knowing M and F_b, the minimum section modulus fitting the limit is:

$$Z_{req'd} \geq \frac{M_a}{F_y \Omega} \quad \left(S_{req'd} \geq \frac{M}{F_b} \right)$$

Besides strength, we also need to be concerned about *serviceability*. This involves things like limiting deflections & cracking, controlling noise and vibrations, preventing excessive settlements of foundations and durability. When we know about a beam section and its material, we can determine beam deformations.

Determining Maximum Bending Moment

Drawing V and M diagrams will show us the maximum values for design. Computer applications are very helpful.

Determining Maximum Bending Stress

For a prismatic member (constant cross section), the maximum normal stress will occur at the maximum moment.

For a *non-prismatic* member, the stress varies with the cross section AND the moment.

Deflections

Elastic curve equations can be found in handbooks, textbooks, design manuals, etc...Computer programs can be used as well.

Elastic curve equations can be superpositioned ONLY if the stresses are in the elastic range. *The deflected shape is roughly the same shape flipped as the bending moment diagram but is constrained by supports and geometry.*

Allowable Deflection Limits

All building codes and design codes limit deflection for beam types and damage that could happen based on service condition and severity.

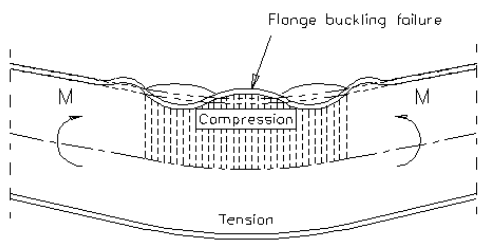
$$\Delta_{actual} \leq \Delta_{allowable} = L/_{value}$$

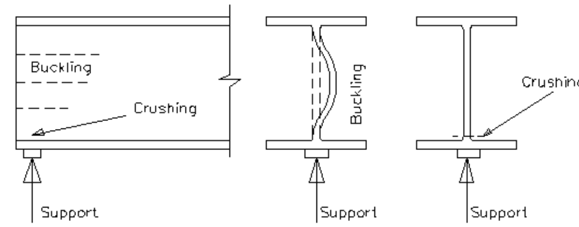
Use	LL only	DL+LL
Roof beams:		
Industrial	L/180	L/120
Commercial		
plaster ceiling	L/240	L/180
no plaster	L/360	L/240
Floor beams:		
Ordinary Usage	L/360	L/240
Roof or floor (damageable elements)		L/480

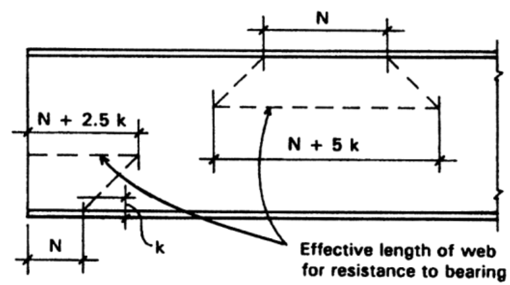
Lateral Buckling

With compression stresses in the top of a beam, a sudden "popping" or buckling can happen even at low stresses. In order to prevent it, we need to brace it along the top, or laterally brace it, or provide a bigger I_y.

Local Buckling in Steel I Beams– Web Crippling or Flange Buckling







Concentrated forces on a steel beam can cause the web to buckle (called web crippling). Web stiffeners under the beam loads and bearing plates at the supports reduce that tendency. Web stiffeners also prevent the web from shearing in plate girders.

The maximum support load and interior load can be determined from:

$$P_{n(\text{max-end})} = (2.5k + N)F_{yw}t_w$$

$$P_{n(\text{interior})} = (5k + N)F_{yw}t_w$$

where t_w = thickness of the web
N = bearing length
k = dimension to fillet found in beam section tables

$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)

Beam Loads & Load Tracing

In order to determine the loads on a beam (or girder, joist, column, frame, foundation...) we can start at the top of a structure and determine the *tributary area* that a load acts over and the beam needs to support. Loads come from material weights, people, and the environment. This area is assumed to be from half the distance to the next beam over to halfway to the next beam.

The reactions must be supported by the next lower structural element *ad infinitum*, to the ground.

LRFD Bending or Flexure

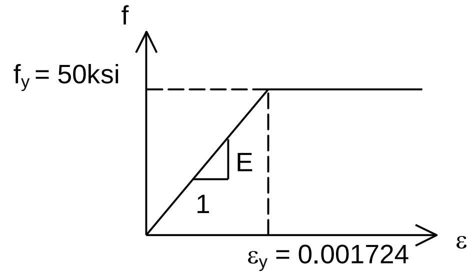
For determining the flexural design strength, $\phi_b M_n$, for resistance to pure bending (no axial load) in most flexural members where the following conditions exist, a single calculation will suffice:

$$\Sigma\gamma_i R_i = M_u \leq \phi_b M_n = 0.9F_y Z$$

where M_u = maximum moment from factored loads
ϕ_b = resistance factor for bending = 0.9

M_n = nominal moment (ultimate capacity)
F_y = yield strength of the steel
Z = plastic section modulus

Plastic Section Modulus



Plastic behavior is characterized by a yield point and an increase in strain with no increase in stress.

Internal Moments and Plastic Hinges

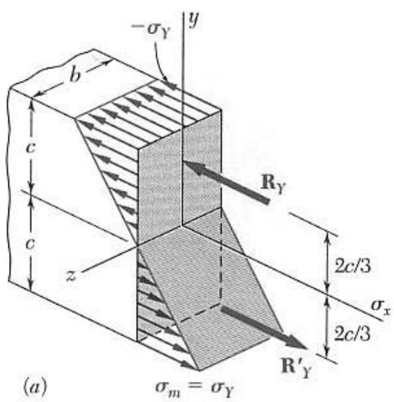
Plastic hinges can develop when all of the material in a cross section sees the yield stress. Because all the material at that section can strain without any additional load, the member segments on either side of the hinge can rotate, possibly causing instability.

For a rectangular section:

Elastic to f_y:

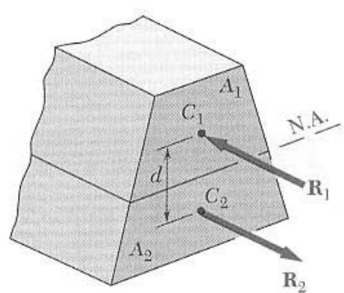
$$M_y = \frac{I}{c} f_y = \frac{bh^2}{6} f_y = \frac{b(2c)^2}{6} f_y = \frac{2bc^2}{3} f_y$$

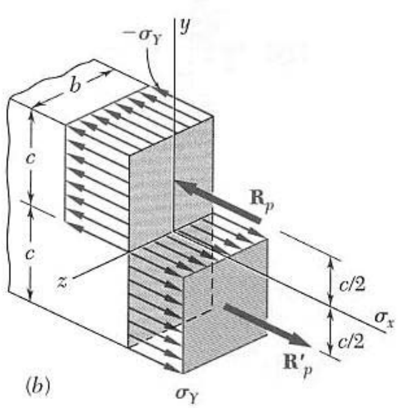
Fully Plastic: M_{ult} *or* $M_p = bc^2 f_y = \frac{3}{2} M_y$

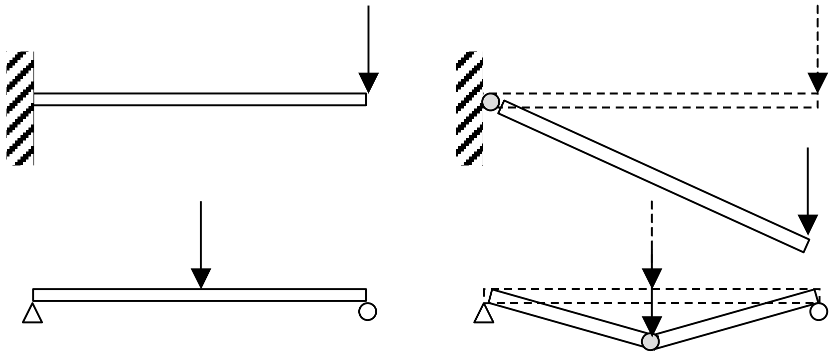


For a non-rectangular section and internal equilibrium at σ_y, the n.a. *will not necessarily be at the centroid.* The n.a. occurs where the $A_{tension} = A_{compression}$. The reactions occur at the centroids of the tension and compression areas.

$$A_{tension} = A_{compression}$$







Instability from Plastic Hinges

Shape Factor:

The ratio of the plastic moment to the elastic moment at yield:

$$k = M_p / M_y$$

k = 3/2 for a rectangle
k ≈ 1.1 for an I beam

Plastic Section Modulus

$$Z = \frac{M_p}{f_y} \quad \textit{and} \quad k = Z/S$$

Design for Shear

$$V_a \le V_n / \Omega \text{ or } V_u \le \phi_v V_n$$

The nominal shear strength is dependent on the cross section shape. Case 1: With a thick or stiff web, the shear stress is resisted by the web of a wide flange shape (with the exception of a handful of W's). Case 2: When the web is not stiff for doubly symmetric shapes, singly symmetric shapes (like channels) (excluding round high strength steel shapes), inelastic web buckling occurs. When the web is very slender, elastic web buckling occurs, reducing the capacity even more:

1. For $h/t_w \le 2.24\sqrt{\frac{E}{F_y}}$ $\quad V_n = 0.6F_{yw}A_w$ $\quad \phi_v = 1.00$ (LRFD) $\quad \Omega = 1.50$ (ASD)

   where h equals the clear distance between flanges less the fillet or corner radius for rolled shapes
   V_n = nominal shear strength
   F_{yw} = yield strength of the steel in the web
   $A_w = t_w d$ = area of the web

2. For $h/t_w > 2.24\sqrt{\frac{E}{F_y}}$ $\quad V_n = 0.6F_{yw}A_wC_v$ $\quad \phi_v = 0.9$ (LRFD) $\quad \Omega = 1.67$ (ASD)

   where C_v is a reduction factor (1.0 or less by equation)

Design for Flexure

$M_a \leq M_n / \Omega$ or $M_u \leq \phi_b M_n$ ϕ_b = 0.90 (LRFD) Ω = 1.67 (ASD)

The nominal flexural strength M_n is the *lowest* value obtained according to the limit states of

1. yielding, limited at length $L_p = 1.76 r_y \sqrt{\frac{E}{F_y}}$, where r_y is the radius of gyration in *y*
2. lateral-torsional buckling limited at length L_r
3. flange local buckling
4. web local buckling

Beam design charts show available moment, M_n/Ω and $\phi_b M_n$, for unbraced length, L_b, of the compression flange in one-foot increments from 1 to 50 ft. for values of the bending coefficient C_b = 1. For values of $1<C_b\leq 2.3$, the required flexural strength M_u can be reduced by dividing it by C_b. (C_b = 1 when the bending moment at any point within an unbraced length is larger than that at both ends of the length. C_b of 1 is conservative and permitted to be used in any case. When the free end is unbraced in a cantilever or overhang, C_b = 1. The full formula is provided below.)

NOTE: the self weight <u>is not</u> included in determination of $\phi_b M_n$

Compact Sections

For a laterally braced *compact* section (one for which the plastic moment can be reached before local buckling) only the limit state of yielding is applicable. For unbraced compact beams and non-compact tees and double angles, only the limit states of yielding and lateral-torsional buckling are applicable.

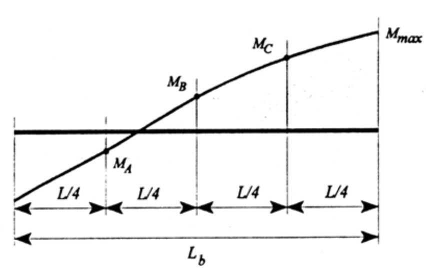
Compact sections meet the following criteria: $\frac{b_f}{2t_f} \leq 0.38\sqrt{\frac{E}{F_y}}$ and $\frac{h_c}{t_w} \leq 3.76\sqrt{\frac{E}{F_y}}$

where:

b_f = flange width in inches
t_f = flange thickness in inches
E = modulus of elasticity in ksi
F_y = minimum yield stress in ksi
h_c = height of the web in inches
t_w = web thickness in inches

With lateral-torsional buckling the nominal flexural strength is

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p$$



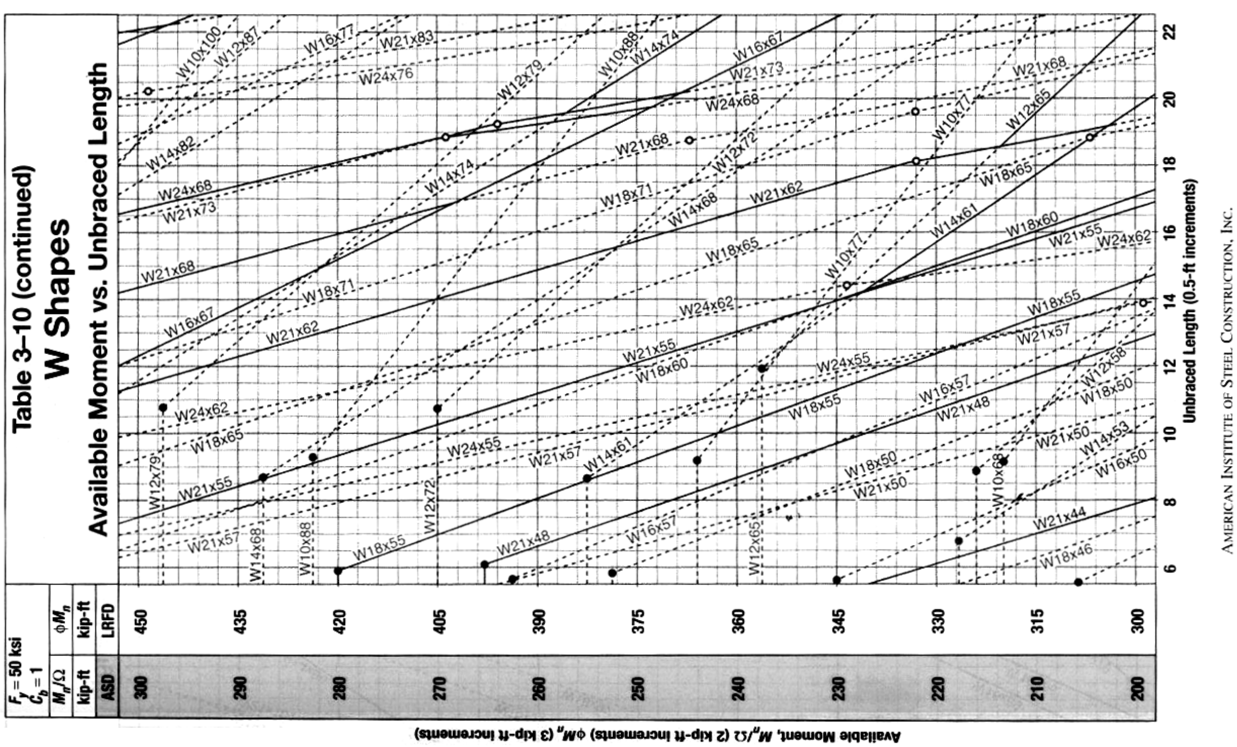
where C_b is a modification factor for non-uniform moment diagrams where, when both ends of the beam segment are braced:

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$

M_{max} = absolute value of the maximum moment in the unbraced beam segment
M_A = absolute value of the moment at the quarter point of the unbraced beam segment
M_B = absolute value of the moment at the center point of the unbraced beam segment
M_C = absolute value of the moment at the three quarter point of the unbraced beam segment length.

Available Flexural Strength Plots

Plots of the available moment for the unbraced length for wide flange sections are useful to find sections to satisfy the design criteria of $M_a \le M_n/\Omega$ *or* $M_u \le \phi_b M_n$. The maximum moment that can be applied on a beam (taking self weight into account), M_a or M_u, can be plotted against the unbraced length, L_b. The limit L_p is indicated by a solid dot (•), while L_r is indicated by an open dot (○). Solid lines indicate the most economical, while dashed lines indicate there is a lighter section that could be used. C_b, which is a modification factor for non-zero moments at the ends, is 1 for simply supported beams (0 moments at the ends). (see *figure)*



Design Procedure

The intent is to find the most light weight member (which is economical) satisfying the section modulus size.

1. Determine the unbraced length to choose the limit state (yielding, lateral torsional buckling or more extreme) and the factor of safety and limiting moments. Determine the material.
2. Draw V & M, finding V_{max} and M_{max}.for unfactored loads (ASD, V_a & M_a) or from factored loads (LRFD, V_u & M_u)
3. Calculate $S_{req'd}$ *or* Z when yielding is the limit state. This step is equivalent to determining if $f_b = \frac{M_{max}}{S} \le F_b$, $S_{req'd} \ge \frac{M_{max}}{F_b} = \frac{M_{max}}{F_y/\Omega}$ and $Z \ge \frac{M_u}{\phi_b F_b}$ to meet the design criteria that

   $M_a \le M_n / \Omega$ or $M_u \le \phi_b M_n$

   If the limit state *is something other than yielding*, determine the nominal moment, M_n, or use plots of available moment to unbraced length, L_b.

4. For steel: use the section charts to find a trial S *or Z and remember that the beam self weight (the second number in the section designation) will increase $S_{req'd.}$* or *Z* The design charts show the lightest section within a grouping of similar S's or Z's.

TABLE 9.1 Load Factor Resistance Design Selection

		F_y = 36 ksi			
Designation	Z_x in.^{3}	L_p ft	L_r ft	M_p kip-ft	M_r kip-ft
W 33 × 141	**514**	**10.1**	**30.1**	**1,542**	**971**
W 30 × 148	500	9.50	30.6	1,500	945
W 24 × 162	468	12.7	45.2	1,404	897
W 24 × 146	418	12.5	42.0	1,254	804
W 33 × 118	**415**	**9.67**	**27.8**	**1,245**	**778**
W 30 × 124	408	9.29	28.2	1,224	769
W 21 × 147	373	12.3	46.4	1,119	713
W 24 × 131	370	12.4	39.3	1,110	713
W 18 × 158	356	11.4	56.5	1,068	672

*****Determine the "updated" V_{max} and M_{max} including the beam self weight, and verify that the updated $S_{req'd}$ has been met.*******

5. Evaluate horizontal shear using V_{max}. This step is equivalent to determining if $f_v \le F_v$ is satisfied to meet the design criteria that $V_a \le V_n / \Omega$ or $V_u \le \phi_v V_n$

   For I beams: $$f_{v-\max} = \frac{3V}{2A} \approx \frac{V}{A_{web}} = \frac{V}{t_w d} \qquad V_n = 0.6F_{yw}A_w \quad or\ V_n = 0.6F_{yw}A_wC_v$$

   Others: $$f_{v-max} = \frac{VQ}{Ib}$$

6. Provide adequate bearing area at supports. This step is equivalent to determining if $f_p = \frac{P}{A} \le F_p$ is satisfied to meet the design criteria that $P_a \le P_n / \Omega$ or $P_u \le \phi P_n$
7. Evaluate shear due to torsion $f_v = \frac{T\rho}{J}$ *or* $\frac{T}{c_1 ab^2} \le F_v$ (circular section or rectangular)
8. Evaluate the deflection to determine if $\Delta_{max LL} \le \Delta_{LL-allowed}$ and/or $\Delta_{\max Total} \le \Delta_{Tota-lallowed}$

***** note: when $\Delta_{calculated} > \Delta_{limit}$, $I_{required}$ can be found with: and $S_{req'd}$ will be satisfied for similar self weight ******

$$I_{req'd} \ge \frac{\Delta_{too\ big}}{\Delta_{limit}} I_{trial}$$

FOR ANY EVALUATION:

Redesign (with a new section) at any point that a stress or serviceability criteria is NOT satisfied and re-evaluate each condition until it is satisfactory.

Load Tables for Uniformly Loaded Joists & Beams

Tables exist for the common loading situation of uniformly distributed load. The tables either provide the safe distributed load based on bending and deflection limits, they give the allowable span for specific live and dead loads including live load deflection limits.
If the load is *not uniform*, an *equivalent uniform load* can be calculated from the maximum moment equation:

$$M_{max} = \frac{w_{equivalent}L^2}{8}$$

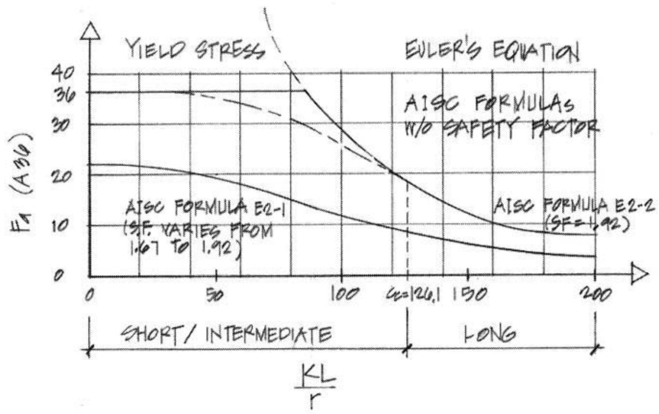
If the deflection limit is less, the design live load to check against allowable must be increased, ex.

$$w_{adjusted} = w_{ll-have}\left(\frac{L/360}{L/400}\right) \begin{matrix} \textit{table limit} \\ \textit{wanted} \end{matrix}$$

Criteria for Design of Columns

If we know the loads, we can select a section that is adequate for strength & buckling.

If we know the length, we can find the limiting load satisfying strength & buckling.



Design for Compression

American Institute of Steel Construction (AISC) Manual 13th ed:

$P_a \le P_n/\Omega$ or $P_u \le \phi_c P_n$ where

$P_u = \Sigma\gamma_i P_i$

γ is a load factor
P is a load type
ϕ is a resistance factor
P_n is the nominal load capacity (strength)

ϕ = 0.90 (LRFD) Ω = 1.67 (ASD)

For compression $P_n = F_{cr}A_g$

where : A_g is the cross section area and F_{cr} is the flexural buckling stress

The flexural buckling stress, F_{cr}, is determined as follows:

when $\dfrac{KL}{r} \le 4.71\sqrt{\dfrac{E}{F_y}}$ or ($F_e \ge 0.44F_y$):

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}} \right] F_y$$

when $\dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{F_y}}$ or ($F_e < 0.44F_y$):

$$F_{cr} = 0.877F_e$$

where F_e is the elastic critical buckling stress: $F_e = \dfrac{\pi^2 E}{\left(KL/r\right)^2}$

Design Aids

Tables exist for the value of the flexural buckling stress based on slenderness ratio. In addition, tables are provided in the AISC Manual for Available Strength in Axial Compression based on the effective length with respect to least radius of gyration, r_y. If the critical effective length is about the largest radius of gyration, r_x, it can be turned into an effective length about the y axis with the fraction r_x/r_y.

Sample AISC Table for Available Strength in Axial

F_y = 50 ksi

Table 4-1 (continued)
Available Strength in Axial Compression, kips
W Shapes

W12

Shape	W12×									
Wt/ft	96		87		79		72		65	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
0	844	1270	766	1150	694	1040	633	951	571	859
6	811	1220	735	1110	667	1000	607	913	548	824
7	800	1200	725	1090	657	987	598	899	540	811
8	787	1180	713	1070	646	971	588	884	531	798
9	772	1160	699	1050	634	952	577	867	520	782
10	756	1140	685	1030	620	932	565	849	509	765
11	739	1110	669	1010	606	910	551	828	497	747
12	720	1080	652	980	590	887	537	807	484	727
13	701	1050	634	953	573	862	522	784	470	706
14	680	1020	615	924	556	836	506	761	456	685
15	659	990	595	895	538	809	490	736	441	662
16	637	957	575	864	520	781	473	710	425	639
17	614	923	554	833	501	752	455	684	409	615
18	591	888	533	801	481	723	437	657	393	591
19	567	852	511	769	461	694	419	630	377	566
20	543	816	490	736	442	664	401	603	360	541
22	495	744	446	670	402	603	365	548	327	491
24	447	672	402	605	362	544	328	493	294	442
26	401	602	360	541	323	486	293	440	262	393
28	356	534	319	479	286	430	259	389	231	347
30	312	469	279	420	250	376	226	340	202	303
32	274	412	246	369	220	331	199	299	177	267
34	243	365	218	327	195	293	176	265	157	236
36	217	326	194	292	174	261	157	236	140	211
38	195	292	174	262	156	234	141	212	126	189
40	176	264	157	236	141	212	127	191	114	171

Effective length KL (ft) with respect to least radius of gyration, r_y

Properties

Property	96 ASD	96 LRFD	87 ASD	87 LRFD	79 ASD	79 LRFD	72 ASD	72 LRFD	65 ASD	65 LRFD
P_{wo} (kips)	137	206	121	181	104	157	90.9	136	78.2	117
P_{wi} (kips/in.)	18.3	27.5	17.2	25.8	15.7	23.5	14.3	21.5	13.0	19.5
P_{wb} (kips)	296	445	243	366	185	278	142	213	106	159
P_{fb} (kips)	152	228	123	185	101	152	84.0	126	68.5	103
L_p (ft)	10.9		10.8		10.8		10.7		11.9	
L_r (ft)	46.6		43.0		39.9		37.4		35.1	
A_g (in.^{2})	28.2		25.6		23.2		21.1		19.1	
I_x (in.^{4})	833		740		662		597		533	
I_y (in.^{4})	270		241		216		195		174	
r_y (in.)	3.09		3.07		3.05		3.04		3.02	
Ratio r_x/r_y	1.76		1.75		1.75		1.75		1.75	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})	23800		21200		18900		17100		15300	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})	7730		6900		6180		5580		4980	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Procedure for Analysis

1. Calculate KL/r for each axis (if necessary). The largest will govern the buckling load.
2. Find F_{cr} as a function of KL/r from the appropriate equation (above) or table.
3. Compute $P_n = F_{cr} \cdot A_g$

   or alternatively compute $f_c = P_a/A$ or P_u/A
4. Is the design satisfactory?

   Is $P_a \leq P_n/\Omega$ or $P_u \leq \phi_c P_n$? $\Rightarrow$ yes, it is; no, it is no good

   or Is $f_c \leq F_{cr}/\Omega$ or $\phi_c F_{cr}$? $\Rightarrow$ yes, it is; no, it is no good

Procedure for Design

1. Guess a size by picking a section.
2. Calculate KL/r for each axis (if necessary). The largest will govern the buckling load.
3. Find F_{cr} as a function of KL/r from appropriate equation (above) or table.
4. Compute $P_n = F_{cr} \cdot A_g$

   or alternatively compute $f_c = P_a/A$ or P_u/A
5. Is the design satisfactory?

   Is $P \leq P_n/\Omega$ or $P_u \leq \phi_c P_n$? yes, it is; *no, pick a bigger section and go back to step 2.*

   Is $f_c \leq F_{cr}/\Omega$ or $\phi_c F_{cr}$? $\Rightarrow$ yes, it is; *no, pick a bigger section and go back to step 2.*
6. Check design efficiency by calculating percentage of stress used:=

$$\frac{P_a}{P_n/\Omega} \cdot 100\% \; or \; \frac{P_u}{\phi_c P_n} \cdot 100\%$$

   If value is between 90-100%, it is efficient.

   If values is less than 90%, *pick a smaller section and go back to step 2.*

Columns with Bending (Beam-Columns)

In order to *design* an adequate section for allowable stress, we have to start somewhere:

1. Make assumptions about the limiting stress from:
   - buckling
   - axial stress
   - combined stress

1. See if we can find values for r or A or Z (S for ASD)

2. Pick a trial section based on if we think r or A is going to govern the section size.

3. Analyze the stresses and compare to allowable using the allowable stress method or interaction formula for eccentric columns.

4. Did the section pass the stress test?
   - If not, do you *increase* r or A or S?
   - If so, is the difference really big so that you could *decrease* r or A or S to make it more efficient (economical)?

5. Change the section choice and go back to step 4. Repeat until the section meets the stress criteria.

Design for Combined Compression and Flexure:

The interaction of compression and bending are included in the form for two conditions based on the size of the required axial force to the available axial strength. This is notated as P_r (either P from ASD or P_u from LRFD) for the axial force being supported, and P_c (either P_n/Ω for ASD or $\phi_c P_n$ for LRFD). The increased bending moment due to the P-Δ effect must be determined and used as the moment to resist.

For $\frac{P_r}{P_c} \geq 0.2$:

$$\frac{P}{P_n/\Omega} + \frac{8}{9}\left(\frac{M_x}{M_{nx}/\Omega} + \frac{M_y}{M_{ny}/\Omega}\right) \leq 1.0 \qquad \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

(ASD) (LRFD)

For $\frac{P_r}{P_c} < 0.2$:

$$\frac{P}{2P_n/\Omega} + \left(\frac{M_x}{M_{nx}/\Omega} + \frac{M_y}{M_{ny}/\Omega}\right) \leq 1.0 \qquad \frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

(ASD) (LRFD)

where:

for compression	ϕ_c = 0.90 (LRFD)	Ω = 1.67 (ASD)
for bending	ϕ_b = 0.90 (LRFD)	Ω = 1.67 (ASD)

For a <u>braced </u>condition, the moment magnification factor B_1 is determined by $B_1 = \frac{C_m}{1-(P_u/P_{e1})} \leq 1.0$

where C_m is a modification factor accounting for end conditions

When not subject to transverse loading between supports in plane of bending:

= 0.6 – 0.4 (M_1/M_2) where M_1 and M_2 are the end moments and $M_1<M_2$. M_1/M_2 is positive when the member is bent in reverse curvature (same direction), negative when bent in single curvature.

When there is transverse loading between the two ends of a member:

= 0.85, members with restrained (fixed) ends

= 1.00, members with unrestrained ends

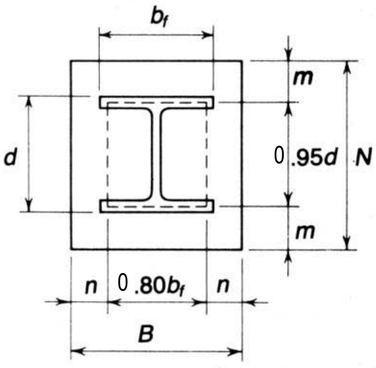
P_{e1} =Euler buckling strength

$$P_{e1} = \frac{\pi^2 EA}{\left(Kl/r\right)^2}$$

Criteria for Design of Connections and Tension Members

Refer to the specific note set.

Criteria for Design of Column Base Plates



Column base plates are designed for bearing on the concrete (concrete capacity) and flexure because the column "punches" down the plate and it could bend upward near the edges of the column (shown as *0.8b$_f$* and *0.95d*). The plate dimensions are B and N and are preferably in full inches with thicknesses in multiples of 1/8 inches.

LRFD minimum thickness: $t_{min} = l\sqrt{\dfrac{2P_u}{0.9F_y BN}}$

where l is the larger of m, n and $\lambda n'$

$$m = \frac{N - 0.95d}{2} \qquad n = \frac{B - 0.8b_f}{2}$$

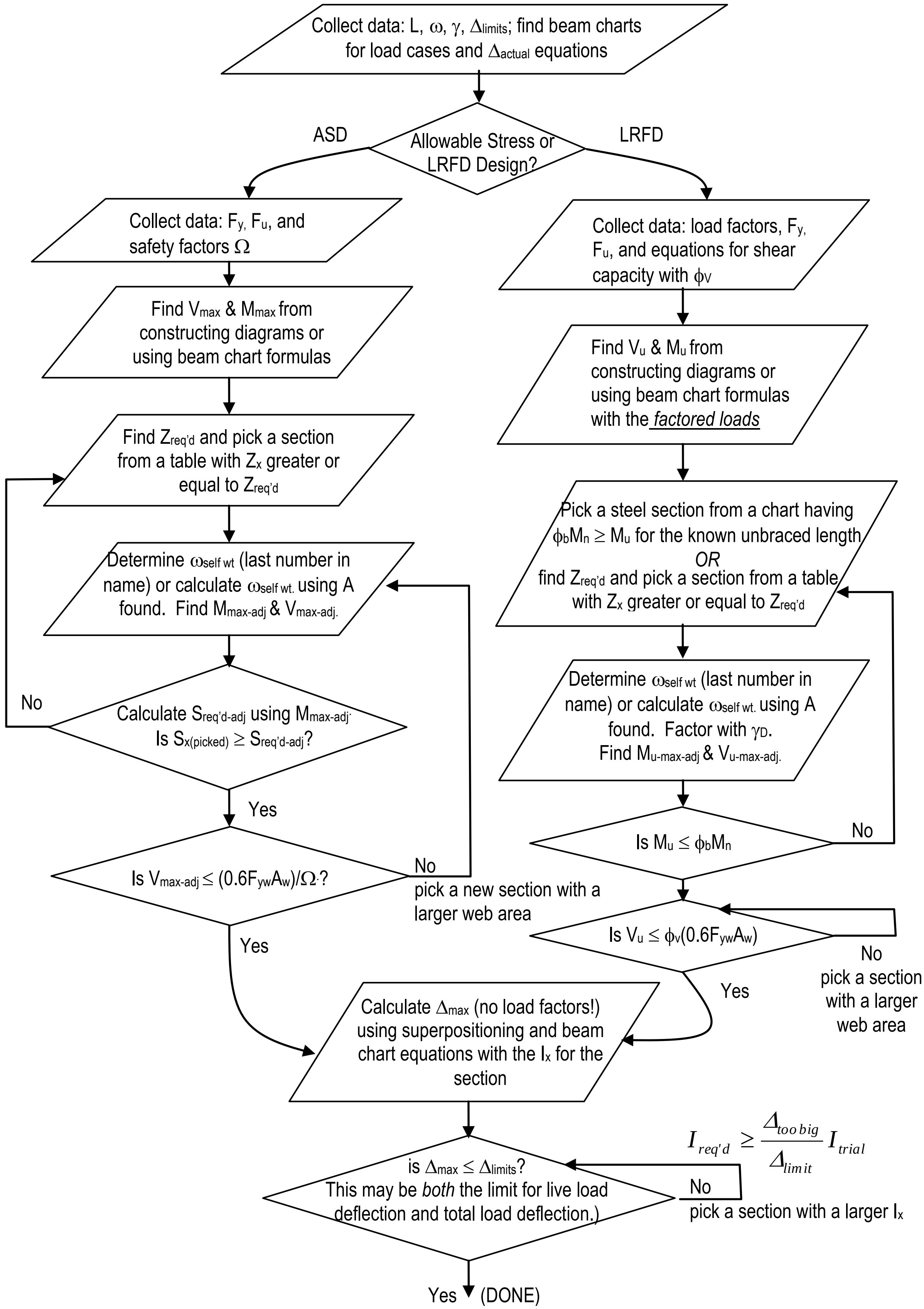
$$n' = \frac{\sqrt{db_f}}{4} \qquad \lambda = \frac{2\sqrt{X}}{(1+\sqrt{1-X})} \leq 1$$

where X depends on the concrete bearing capacity of $\phi_c P_p$, with

ϕ_c = 0.65 and $P_p = 0.85f'_c A$

$$X = \frac{4db_f}{(d+b_f)^2}\cdot\frac{P_u}{\phi_c P_p} = \frac{4db_f}{(d+b_f)^2}\cdot\frac{P_u}{\phi_c(0.85f_c')BN}$$

Beam Design Flow Chart



Listing of W shapes in Descending Order of Z_x for Beam Design

Z_x – US (in.^{3})	I_x – US (in.^{4})	Section	I_x – SI (10^6mm.^{4})	Z_x – SI (10^3mm.3)
514	**7450**	**W33X141**	**3100**	**8420**
511	5680	W24X176	2360	8370
509	**7800**	**W36X135**	**3250**	**8340**
500	6680	W30X148	2780	8190
490	4330	W18X211	1800	8030
487	3400	W14X257	1420	7980
481	3110	W12X279	1290	7880
476	4730	W21X182	1970	7800
468	5170	W24X162	2150	7670
467	**6710**	**W33X130**	**2790**	**7650**
464	5660	W27X146	2360	7600
442	3870	W18X192	1610	7240
437	5770	W30X132	2400	7160
436	3010	W14X233	1250	7140
432	4280	W21X166	1780	7080
428	2720	W12X252	1130	7010
418	4580	W24X146	1910	6850
415	**5900**	**W33X118**	**2460**	**6800**
408	5360	W30X124	2230	6690
398	3450	W18X175	1440	6520
395	4760	W27X129	1980	6470
390	2660	W14X211	1110	6390
386	2420	W12X230	1010	6330
378	**4930**	**W30X116**	**2050**	**6190**
373	3630	W21X147	1510	6110
370	4020	W24X131	1670	6060
356	3060	W18X158	1270	5830
355	2400	W14X193	999	5820
348	2140	W12X210	891	5700
346	**4470**	**W30X108**	**1860**	**5670**
343	4080	W27X114	1700	5620
333	3220	W21X132	1340	5460
327	3540	W24X117	1470	5360
322	2750	W18X143	1140	5280
320	2140	W14X176	891	5240
312	**3990**	**W30X99**	**1660**	**5110**
311	1890	W12X190	787	5100
307	2960	W21X122	1230	5030
305	3620	W27X102	1510	5000
290	2460	W18X130	1020	4750

Z_x – US (in.^{3})	I_x – US (in.^{4})	Section	I_x – SI (10^6mm.^{4})	Z_x – SI (10^3mm.3)
289	3100	W24X104	1290	4740
287	1900	W14X159	791	4700
283	**3610**	**W30X90**	**1500**	**4640**
280	3000	W24X103	1250	4590
279	2670	W21X111	1110	4570
278	3270	W27X94	1360	4560
275	1650	W12X170	687	4510
262	2190	W18X119	912	4290
260	1710	W14X145	712	4260
254	2700	W24X94	1120	4160
253	2420	W21X101	1010	4150
244	**2850**	**W27X84**	**1190**	**4000**
243	1430	W12X152	595	3980
234	1530	W14X132	637	3830
230	1910	W18X106	795	3770
224	**2370**	**W24X84**	**986**	**3670**
221	2070	W21X93	862	3620
214	1240	W12X136	516	3510
212	1380	W14X120	574	3470
211	1750	W18X97	728	3460
200	**2100**	**W24X76**	**874**	**3280**
198	1490	W16X100	620	3240
196	1830	W21X83	762	3210
192	1240	W14X109	516	3150
186	1530	W18X86	637	3050
185	1070	W12X120	445	3050
177	**1830**	**W24X68**	**762**	**2900**
174	1300	W16X89	541	2870
173	1110	W14X99	462	2830
169	1600	W21X73	666	2820
164	933	W12X106	388	2690
160	1330	W18X76	554	2670
159	**1480**	**W21X68**	**616**	**2620**
157	999	W14X90	416	2570
153	**1550**	**W24X62**	**645**	**2510**
147	1110	W16X77	462	2460
147	833	W12X96	347	2410
146	716	W10X112	298	2410
146	1170	W18X71	487	2390

(continued)

Listing of W Shapes in Descending order of Z_x for Beam Design (Continued)

Z_x – US (in.^{3})	I_x – US (in.^{4})	Section	I_x – SI (10^6mm.^{4})	Z_x – SI (10^3mm.3)
144	**1330**	**W21X62**	**554**	**2360**
139	881	W14X82	367	2280
133	**1350**	**W24X55**	**562**	**2200**
132	1070	W18X65	445	2180
131	740	W12X87	308	2160
130	954	W16X67	397	2130
129	623	W10X100	259	2130
129	1170	W21X57	487	2110
126	**1140**	**W21X55**	**475**	**2060**
126	795	W14X74	331	2060
123	984	W18X60	410	2020
118	662	W12X79	276	1950
115	722	W14X68	301	1880
113	534	W10X88	222	1850
112	**890**	**W18X55**	**370**	**1840**
110	**984**	**W21X50**	**410**	**1800**
108	597	W12X72	248	1770
107	**959**	**W21X48**	**399**	**1750**
105	758	W16X57	316	1720
102	640	W14X61	266	1670
100	800	W18X50	333	1660
96.8	455	W10X77	189	1600
95.5	533	W12X65	222	1590
95.4	**843**	**W21X44**	**351**	**1560**
91.7	659	W16X50	274	1510
90.6	712	W18X46	296	1490
86.5	541	W14X53	225	1430
86.4	475	W12X58	198	1420
85.2	394	W10X68	164	1400
82.1	586	W16X45	244	1350
78.4	**612**	**W18X40**	**255**	**1280**
78.1	484	W14X48	201	1280
77.3	425	W12X53	177	1280
74.4	341	W10X60	142	1220
72.2	**518**	**W16X40**	**216**	**1200**
71.8	391	W12X50	163	1180
69.6	272	W8X67	113	1150
69.4	428	W14X43	178	1140
66.5	303	W10X54	126	1090
66.5	**510**	**W18X35**	**212**	**1090**
64.0	348	W12X45	145	1050
63.5	448	W16X36	186	1050
61.5	385	W14X38	160	1010
59.4	228	W8X58	94.9	980
57.0	307	W12X40	128	934
54.7	248	W10X45	103	900
54.5	**340**	**W14X34**	**142**	**895**
53.7	**375**	**W16X31**	**156**	**885**
51.2	285	W12X35	119	839
49.0	184	W8X48	76.6	803
47.2	**291**	**W14X30**	**121**	**775**
46.7	209	W10X39	87.0	767
44.2	**301**	**W16X26**	**125**	**724**
43.0	238	W12X30	99.1	706
40.1	**245**	**W14X26**	**102**	**659**
39.7	146	W8X40	60.8	652
38.5	171	W10X33	71.2	636
37.1	204	W12X26	84.9	610
36.6	170	W10X30	70.8	600
34.7	127	W8X35	52.9	569
33.2	**199**	**W14X22**	**82.8**	**544**
31.3	144	W10X26	59.9	513
30.4	110	W8X31	45.8	498
29.2	**156**	**W12X22**	**64.9**	**480**
27.1	98.0	W8X28	40.8	446
26.0	**118**	**W10X22**	**49.1**	**426**
24.6	**130**	**W12X19**	**54.1**	**405**
23.1	82.7	W8X24	34.4	379
21.4	**96.3**	**W10X19**	**40.1**	**354**
20.4	75.3	W8X21	31.3	334
20.1	**103**	**W12x16**	**42.9**	**329**
18.6	81.9	W10X17	34.1	306
17.3	**88.6**	**W12X14**	**36.9**	**285**
17.0	61.9	W8X18	25.8	279
15.9	68.9	W10X15	28.7	262
13.6	48.0	W8X15	20.0	223
12.6	**53.8**	**W10X12**	**22.4**	**206**
11.4	39.6	W8X13	16.5	187
8.87	**30.8**	**W8X10**	**12.8**	**145**

Available Critical Stress, $\phi_c F_{cr}$, for Compression Members, ksi (F_y = 36 ksi and ϕ_c = 0.90)

KL/r	$\phi_c F_{cr}$	KL/r	$\phi_c F_{cr}$	KL/r	$\phi_c F_{cr}$	KL/r	$\phi_c F_{cr}$	KL/r	$\phi_c F_{cr}$
1	32.4	41	29.7	81	22.9	121	15.0	161	8.72
2	32.4	42	29.5	82	22.7	122	14.8	162	8.61
3	32.4	43	29.4	83	22.5	123	14.6	163	8.50
4	32.4	44	29.3	84	22.3	124	14.4	164	8.40
5	32.4	45	29.1	85	22.1	125	14.2	165	8.30
6	32.3	46	29.0	86	22.0	126	14.0	166	8.20
7	32.3	47	28.8	87	21.8	127	13.9	167	8.10
8	32.3	48	28.7	88	21.6	128	13.7	168	8.00
9	32.3	49	28.6	89	21.4	129	13.5	169	7.91
10	32.2	50	28.4	90	21.2	130	13.3	170	7.82
11	32.2	51	28.3	91	21.0	131	13.1	171	7.73
12	32.2	52	28.1	92	20.8	132	12.9	172	7.64
13	32.1	53	27.9	93	20.5	133	12.8	173	7.55
14	32.1	54	27.8	94	20.3	134	12.6	174	7.46
15	32.0	55	27.6	95	20.1	135	12.4	175	7.38
16	32.0	56	27.5	96	19.9	136	12.2	176	7.29
17	31.9	57	27.3	97	19.7	137	12.0	177	7.21
18	31.9	58	27.1	98	19.5	138	11.9	178	7.13
19	31.8	59	27.0	99	19.3	139	11.7	179	7.05
20	31.7	60	26.8	100	19.1	140	11.5	180	6.97
21	31.7	61	26.6	101	18.9	141	11.4	181	6.90
22	31.6	62	26.5	102	18.7	142	11.2	182	6.82
23	31.5	63	26.3	103	18.5	143	11.0	183	6.75
24	31.4	64	26.1	104	18.3	144	10.9	184	6.67
25	31.4	65	25.9	105	18.1	145	10.7	185	6.60
26	31.3	66	25.8	106	17.9	146	10.6	186	6.53
27	31.2	67	25.6	107	17.7	147	10.5	187	6.46
28	31.1	68	25.4	108	17.5	148	10.3	188	6.39
29	31.0	69	25.2	109	17.3	149	10.2	189	6.32
30	30.9	70	25.0	110	17.1	150	10.0	190	6.26
31	30.8	71	24.8	111	16.9	151	9.91	191	6.19
32	30.7	72	24.7	112	16.7	152	9.78	192	6.13
33	30.6	73	24.5	113	16.5	153	9.65	193	6.06
34	30.5	74	24.3	114	16.3	154	9.53	194	6.00
35	30.4	75	24.1	115	16.2	155	9.40	195	5.94
36	30.3	76	23.9	116	16.0	156	9.28	196	5.88
37	30.1	77	23.7	117	15.8	157	9.17	197	5.82
38	30.0	78	23.5	118	15.6	158	9.05	198	5.76
39	29.9	79	23.3	119	15.4	159	8.94	199	5.70
40	29.8	80	23.1	120	15.2	160	8.82	200	5.65

Available Critical Stress, $\phi_c F_{cr}$, for Compression Members, ksi (F_y = 50 ksi and ϕ_c = 0.90)

KL/r	$\phi_c F_{cr}$	*KL/r*	$\phi_c F_{cr}$	*KL/r*	$\phi_c F_{cr}$	*KL/r*	$\phi_c F_{cr}$	*KL/r*	$\phi_c F_{cr}$
1	45.0	41	39.8	81	27.9	121	15.4	161	8.72
2	45.0	42	39.6	82	27.5	122	15.2	162	8.61
3	45.0	43	39.3	83	27.2	123	14.9	163	8.50
4	44.9	44	39.1	84	26.9	124	14.7	164	8.40
5	44.9	45	38.8	85	26.5	125	14.5	165	8.30
6	44.9	46	38.5	86	26.2	126	14.2	166	8.20
7	44.8	47	38.3	87	25.9	127	14.0	167	8.10
8	44.8	48	38.0	88	25.5	128	13.8	168	8.00
9	44.7	49	37.8	89	25.2	129	13.6	169	7.91
10	44.7	50	37.5	90	24.9	130	13.4	170	7.82
11	44.6	51	37.2	91	24.6	131	13.2	171	7.73
12	44.5	52	36.9	92	24.2	132	13.0	172	7.64
13	44.4	53	36.6	93	23.9	133	12.8	173	7.55
14	44.4	54	36.4	94	23.6	134	12.6	174	7.46
15	44.3	55	36.1	95	23.3	135	12.4	175	7.38
16	44.2	56	35.8	96	22.9	136	12.2	176	7.29
17	44.1	57	35.5	97	22.6	137	12.0	177	7.21
18	43.9	58	35.2	98	22.3	138	11.9	178	7.13
19	43.8	59	34.9	99	22.0	139	11.7	179	7.05
20	43.7	60	34.6	100	21.7	140	11.5	180	6.97
21	43.6	61	34.3	101	21.3	141	11.4	181	6.90
22	43.4	62	34.0	102	21.0	142	11.2	182	6.82
23	43.3	63	33.7	103	20.7	143	11.0	183	6.75
24	43.1	64	33.4	104	20.4	144	10.9	184	6.67
25	43.0	65	33.0	105	20.1	145	10.7	185	6.60
26	42.8	66	32.7	106	19.8	146	10.6	186	6.53
27	42.7	67	32.4	107	19.5	147	10.5	187	6.46
28	42.5	68	32.1	108	19.2	148	10.3	188	6.39
29	42.3	69	31.8	109	18.9	149	10.2	189	6.32
30	42.1	70	31.4	110	18.6	150	10.0	190	6.26
31	41.9	71	31.1	111	18.3	151	9.91	191	6.19
32	41.8	72	30.8	112	18.0	152	9.78	192	6.13
33	41.6	73	30.5	113	17.7	153	9.65	193	6.06
34	41.4	74	30.2	114	17.4	154	9.53	194	6.00
35	41.1	75	29.8	115	17.1	155	9.40	195	5.94
36	40.9	76	29.5	116	16.8	156	9.28	196	5.88
37	40.7	77	29.2	117	16.5	157	9.17	197	5.82
38	40.5	78	28.8	118	16.2	158	9.05	198	5.76
39	40.3	79	28.5	119	16.0	159	8.94	199	5.70
40	40.0	80	28.2	120	15.7	160	8.82	200	5.65

STANDARD SPECIFICATIONS
FOR OPEN WEB STEEL JOISTS, K-SERIES

Adopted by the Steel Joist Institute November 4, 1985
Revised to November 10, 2003 - Effective March 01, 2005

SECTION 1.
SCOPE

This specification covers the design, manufacture and use of Open Web Steel Joists, **K-**Series. Load and Resistance Factor Design (LRFD) and Allowable Strength Design (ASD) are included in this specification.

SECTION 2.
DEFINITION

The term "Open Web Steel Joists **K-**Series," as used herein, refers to open web, parallel chord, load-carrying members suitable for the direct support of floors and roof decks in buildings, utilizing hot-rolled or cold-formed steel, including cold-formed steel whose yield strength* has been attained by cold working. **K-**Series Joists shall be designed in accordance with this specification to support the uniformly distributed loads given in the Standard Load Tables for Open Web Steel Joists, **K-**Series, attached hereto.

The KCS Joist is a **K-**Series Joist which is provided to address the problem faced by specifying professionals when trying to select joists to support uniform plus concentrated loads or other non-uniform loads.

The design of chord sections for **K-**Series Joists shall be based on a yield strength of 50 ksi (345 MPa). The design of web sections for **K-**Series Joists shall be based on a yield strength of either 36 ksi (250 MPa) or 50 ksi (345 MPa). Steel used for **K-**Series Joists chord or web sections shall have a minimum yield strength determined in accordance with one of the procedures specified in Section 3.2, which is equal to the yield strength assumed in the design.

* The term "Yield Strength" as used herein shall designate the yield level of a material as determined by the applicable method outlined in paragraph 13.1 "Yield Point", and in paragraph 13.2 "Yield Strength", of ASTM A370, *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*, or as specified in paragraph 3.2 of this specification.

SECTION 3.
MATERIALS

3.1 STEEL

The steel used in the manufacture of chord and web sections shall conform to one of the following ASTM Specifications:

- Carbon Structural Steel, ASTM A36/A36M.
- High-Strength, Low-Alloy Structural Steel, ASTM A242/A242M.
- High-Strength Carbon-Manganese Steel of Structural Quality, ASTM A529/A529M, Grade 50.
- High-Strength Low-Alloy Columbium-Vanadium Structural Steel, ASTM A572/A572M, Grade 42 and 50.
- High-Strength Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4 inches (100 mm) Thick, ASTM A588/A588M.
- Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Corrosion Resistance, ASTM A606.
- Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability, ASTM A1008/A1008M
- Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability, ASTM A1011/A1011M

or shall be of suitable quality ordered or produced to other than the listed specifications, provided that such material in the state used for final assembly and manufacture is weldable and is proved by tests performed by the producer or manufacturer to have the properties specified in Section 3.2.

3.2 MECHANICAL PROPERTIES

The yield strength used as a basis for the design stresses prescribed in Section 4 shall be either 36 ksi (250 MPa) or 50 ksi (345 MPa). Evidence that the steel furnished meets or exceeds the design yield strength shall, if requested, be provided in the form of an affidavit or by witnessed or certified test reports.

For material used without consideration of increase in yield strength resulting from cold forming, the specimens shall be taken from as-rolled material. In the case of material, the mechanical properties of which conform to the requirements of one of the listed specifications, the test specimens and procedures shall conform to those of such specifications and to ASTM A370.

In the case of material, the mechanical properties of which do not conform to the requirements of one of the listed specifications, the test specimens and procedures shall conform to the applicable requirements of ASTM A370, and the specimens shall exhibit a yield strength equal to or exceeding the design yield strength and an elongation of not less than (a) 20 percent in 2 inches (51 millimeters) for sheet and strip, or (b) 18 percent in 8 inches (203 millimeters) for plates, shapes and bars with adjustments for thickness for plates, shapes and bars as prescribed in ASTM A36/A36M, A242/A242M, A529/A529M, A572/A572M, A588/A588M, whichever specification is applicable on the basis of design yield strength.

The number of tests shall be as prescribed in ASTM A6/A6M for plates, shapes, and bars; and ASTM A606, A1008/A1008M and A1011/A1011M for sheet and strip.

If as-formed strength is utilized, the test reports shall show the results of tests performed on full section specimens in accordance with the provisions of the AISI North American Specifications for the Design of Cold-Formed Steel Structural Members. They shall also indicate compliance with these provisions and with the following additional requirements:

a) The yield strength calculated from the test data shall equal or exceed the design yield strength.

b) Where tension tests are made for acceptance and control purposes, the tensile strength shall be at least 6 percent greater than the yield strength of the section.

c) Where compression tests are used for acceptance and control purposes, the specimen shall withstand a gross shortening of 2 percent of its original length without cracking. The length of the specimen shall be not greater than 20 times the least radius of gyration.

d) If any test specimen fails to pass the requirements of the subparagraphs (a), (b), or (c) above, as applicable, two retests shall be made of specimens from the same lot. Failure of one of the retest specimens to meet such requirements shall be the cause for rejection of the lot represented by the specimens.

3.3 PAINT

The standard shop paint is intended to protect the steel for only a short period of exposure in ordinary atmospheric conditions and shall be considered an impermanent and provisional coating.

When specified, the standard shop paint shall conform to one of the following:

a) Steel Structures Painting Council Specification, SSPC No. 15.

b) Or, shall be a shop paint which meets the minimum performance requirements of the above listed specification.

SECTION 4.
DESIGN AND MANUFACTURE

4.1 METHOD

Joists shall be designed in accordance with these specifications as simply supported, uniformly loaded trusses supporting a floor or roof deck so constructed as to brace the top chord of the joists against lateral buckling. Where any applicable design feature is not specifically covered herein, the design shall be in accordance with the following specifications:

a) Where the steel used consists of hot-rolled shapes, bars or plates, use the American Institute of Steel Construction, *Specification for Structural Steel Buildings*.

b) For members that are cold-formed from sheet or strip steel, use the American Iron and Steel Institute, *North American Specification for the Design of Cold-Formed Steel Structural Members*.

Design Basis:

Designs shall be made according to the provisions in this Specification for either Load and Resistance Factor Design (LRFD) or for Allowable Strength Design (ASD).

Load Combinations:

LRFD:

When load combinations are not specified to the joist manufacturer, the required stress shall be computed for the factored loads based on the factors and load combinations as follows:

$1.4D$

$1.2D + 1.6\ (L, \text{ or } L_r, \text{ or } S, \text{ or } R)$

ASD:

When load combinations are not specified to the joist manufacturer, the required stress shall be computed based on the load combinations as follows:

D

$D + (L, \text{ or } L_r, \text{ or } S, \text{ or } R)$

Where:

D = dead load due to the weight of the structural elements and the permanent features of the structure

L = live load due to occupancy and movable equipment

L_r = roof live load

S = snow load

R = load due to initial rainwater or ice exclusive of the ponding contribution

When special loads are specified and the specifying professional does not provide the load combinations, the provisions of ASCE 7, *"Minimum Design Loads for Buildings and Other Structures"* shall be used for LRFD and ASD load combinations.

4.2 DESIGN AND ALLOWABLE STRESSES

Design Using Load and Resistance Factor Design (LRFD)

Joists shall have their components so proportioned that the required stresses, f_u, shall not exceed ϕF_n where,

f_u	=	required stress	ksi (MPa)
F_n	=	nominal stress	ksi (MPa)
ϕ	=	resistance factor	
ϕF_n	=	design stress	

Design Using Allowable Strength Design (ASD)

Joists shall have their components so proportioned that the required stresses, f, shall not exceed F_n / Ω where,

f	=	required stress	ksi (MPa)
F_n	=	nominal stress	ksi (MPa)
Ω	=	safety factor	
F_n/Ω	=	allowable stress	

Stresses:

(a) Tension: $\phi_t = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

For Chords: $F_y = 50$ ksi (345 MPa)

For Webs: $F_y = 50$ ksi (345 MPa), or $F_y = 36$ ksi (250 MPa)

Design Stress = $0.9F_y$ (LRFD) (4.2-1)

Allowable Stress = $0.6F_y$ (ASD) (4.2-2)

(b) Compression: $\phi_c = 0.90$ (LRFD) $\Omega_c = 1.67$ (ASD)

For members with $\ell/r \le 4.71\sqrt{E/QF_y}$

$$F_{cr} = Q\left[0.658^{\left(\frac{QF_y}{F_e}\right)}\right]F_y \qquad (4.2\text{-}3)$$

For members with $\ell/r > 4.71\sqrt{E/QF_y}$

$$F_{cr} = 0.877F_e \qquad (4.2\text{-}4)$$

Where F_e = Elastic buckling stress determined in accordance with Equation 4.2-5.

$$F_e = \frac{\pi^2 E}{\left(\ell/r\right)^2} \qquad (4.2\text{-}5)$$

For hot-rolled sections, "Q" is the full reduction factor for slender compression elements.

Design Stress = $0.9F_{cr}$ (LRFD) (4.2-6)

Allowable Stress = $0.6F_{cr}$ (ASD) (4.2-7)

In the above equations, ℓ is taken as the distance in inches (millimeters) between panel points for the chord members and the appropriate length for web members, and r is the corresponding least radius of gyration of the member or any component thereof. E is equal to 29,000 ksi (200,000 MPa).

Use $1.2\ \ell/r_x$ for a crimped, first primary compression web member when a moment-resistant weld group is not used for this member; where r_x = member radius of gyration in the plane of the joist.

For cold-formed sections the method of calculating the nominal column strength is given in the AISI, *North American Specification for the Design of Cold-Formed Steel Structural Members.*

(c) Bending: $\phi_b = 0.90$ (LRFD) $\Omega_b = 1.67$ (ASD)

Bending calculations are to be based on using the elastic section modulus.

For chords and web members other than solid rounds: $F_y = 50$ ksi (345 MPa)

Design Stress = $0.9F_y$ (LRFD) (4.2-8)

Allowable Stress = $0.6F_y$ (ASD) (4.2-9)

For web members of solid round cross section: $F_y = 50$ ksi (345 MPa), or $F_y = 36$ ksi (250 MPa)

Design Stress = $1.45F_y$ (LRFD) (4.2-10)

Allowable Stress = $0.95F_y$ (ASD) (4.2-11)

For bearing plates: $F_y = 50$ ksi (345 MPa), or $F_y = 36$ ksi (250 MPa)

Design Stress = $1.35F_y$ (LRFD) (4.2-12)

Allowable Stress = $0.90F_y$ (ASD) (4.2-13)

4.3 MAXIMUM SLENDERNESS RATIOS

The slenderness ratio, ℓ/r, where ℓ is as used in Section 4.2 (b) and r is the corresponding least radius of gyration, shall not exceed the following:

Top chord interior panels 90
Top chord end panels 120
Compression members other than top chord. 200
Tension members 240

4.4 MEMBERS

(a) Chords

The bottom chord shall be designed as an axially loaded tension member.

The radius of gyration of the top chord about its vertical axis shall not be less than $\ell/145$ where ℓ is the spacing in inches (millimeters) between lines of bridging as specified in Section 5.4(c).

The top chord shall be considered as stayed laterally by the floor slab or roof deck when attachments are in accordance with the requirements of Section 5.8(e) of these specifications.

The top chord shall be designed for only axial compressive stress when the panel length, ℓ, does not exceed 24 inches (609 mm). When the panel length exceeds 24 inches (609 mm), the top chord shall be designed as a continuous member subject to combined axial and bending stresses and shall be so proportioned that:

For LRFD:

at the panel point:

$$f_{au} + f_{bu} \le 0.9F_y \qquad (4.4\text{-}1)$$

at the mid panel: for $\frac{f_{au}}{\phi_c F_{cr}} \ge 0.2$,

$$\frac{f_{au}}{\phi_c F_{cr}} + \frac{8}{9}\left[\frac{C_m f_{bu}}{\left[1-\left(\frac{f_{au}}{\phi_c F_e}\right)\right]Q\phi_b F_y}\right] \le 1.0 \qquad (4.4\text{-}2)$$

for $\frac{f_{au}}{\phi_c F_{cr}} < 0.2$,

$$\left(\frac{f_{au}}{2\phi_c F_{cr}}\right) + \left[\frac{C_m f_{bu}}{\left[1-\left(\frac{f_{au}}{\phi_c F_e}\right)\right]Q\phi_b F_y}\right] \le 1.0 \qquad (4.4\text{-}3)$$

$f_{au} = P_u/A$ = Required compressive stress, ksi (MPa)

P_u = Required axial strength using LRFD load combinations, kips (N)

f_{bu} = M_u/S = Required bending stress at the location under consideration, ksi (MPa)

M_u = Required flexural strength using LRFD load combinations, kip-in. (N-mm)

S = Elastic Section Modulus, in.^{3} (mm^3)

F_{cr} = Nominal axial compressive stress in ksi (MPa) based on ℓ/ r as defined in Section 4.2(b),

C_m = 1 - 0.3 $f_{au}/\phi F_e$ for end panels

C_m = 1 - 0.4 $f_{au}/\phi F_e$ for interior panels

F_y = Specified minimum yield strength, ksi (MPa)

$F_e = \dfrac{\pi^2 E}{\left(\ell/r_x\right)^2}$, ksi (MPa)

Where ℓ is the panel length, in inches (millimeters), as defined in Section 4.2(b) and r_x is the radius of gyration about the axis of bending.

Q = Form factor defined in Section 4.2(b)

A = Area of the top chord, in.^{2} (mm^2)

For ASD:

at the panel point:

$$f_a + f_b \le 0.6F_y \qquad (4.4\text{-}4)$$

at the mid panel: for $\frac{f_a}{F_a} \ge 0.2$,

$$\frac{f_a}{F_a} + \frac{8}{9}\left[\frac{C_m f_b}{\left[1-\left(\frac{1.67f_a}{F_e}\right)\right]QF_b}\right] \le 1.0 \qquad (4.4\text{-}5)$$

for $\frac{f_a}{F_a} < 0.2$,

$$\left(\frac{f_a}{2F_a}\right) + \left[\frac{C_m f_b}{\left[1-\left(\frac{1.67f_a}{F_e}\right)\right]QF_b}\right] \le 1.0 \qquad (4.4\text{-}6)$$

f_a = P/A = Required compressive stress, ksi (MPa)

P = Required axial strength using ASD load combinations, kips (N)

f_b = M/S = Required bending stress at the location under consideration, ksi (MPa)

M = Required flexural strength using ASD load combinations, kip-in. (N-mm)

S = Elastic Section Modulus, in.^{3} (mm^3)

F_a = Allowable axial compressive stress based on ℓ/r as defined in Section 4.2(b), ksi (MPa)

F_b = Allowable bending stress; 0.6F_y, ksi (MPa)

C_m = 1 - 0.50 f_a/F_e for end panels

C_m = 1 - 0.67 f_a/F_e for interior panels

(b) Web

The vertical shears to be used in the design of the web members shall be determined from full uniform loading, but such vertical shears shall be not less than 25 percent of the end reaction. Due consideration shall be given to the effect of eccentricity. The effect of combined axial compression and bending may be investigated using the provisions of Section 4.4(a), letting C_m = 0.4 when bending due to eccentricity produces reversed curvature.

Interior vertical web members used in modified Warren type web systems shall be designed to resist the gravity loads supported by the member plus an additional axial load of 1/2 of 1.0 percent of the top chord axial force.

(c) Extended Ends

The magnitude and location of the loads to be supported, deflection requirements, and proper bracing of extended top chords or full depth cantilever ends shall be clearly indicated on the structural drawings.

4.5 CONNECTIONS

(a) Methods

Joist connections and splices shall be made by attaching the members to one another by arc or resistance welding or other accredited methods.

(1) Welded Connections

a) Selected welds shall be inspected visually by the manufacturer. Prior to this inspection, weld slag shall be removed.

b) Cracks are not acceptable and shall be repaired.

c) Thorough fusion shall exist between weld and base metal for the required design length of the weld; such fusion shall be verified by visual inspection.

d) Unfilled weld craters shall not be included in the design length of the weld.

e) Undercut shall not exceed 1/16 inch (2 millimeters) for welds oriented parallel to the principal stress.

f) The sum of surface (piping) porosity diameters shall not exceed 1/16 inch (2 millimeters) in any 1 inch (25 millimeters) of design weld length.

g) Weld spatter that does not interfere with paint coverage is acceptable.

(2) Welding Program

Manufacturers shall have a program for establishing weld procedures and operator qualification, and for weld sampling and testing. (See Technical Digest #8 - Welding of Open Web Steel Joists.)

(3) Weld Inspection by Outside Agencies (See Section 5.12 of these specifications)

The agency shall arrange for visual inspection to determine that welds meet the acceptance standards of Section 4.5(a)(1) above. Ultrasonic, X-Ray, and magnetic particle testing are inappropriate for joists due to the configurations of the components and welds.

(b) Strength

(1) Joint Connections - Joint connections shall be capable of withstanding forces due to an ultimate load equal to at least 1.35 times the LRFD, or 2.0 times the ASD load shown in the applicable Standard Load Table.

(2) Shop Splices – Splices may occur at any point in chord or web members. Members containing a butt weld splice shall develop an ultimate tensile force of at least 57 ksi (393 MPa) times the full design area of the chord or web. The term "member" shall be defined as all component parts comprising the chord or web, at the point of the splice.

(c) Eccentricity

Members connected at a joint shall have their centroidal axes meet at a point if practical. Otherwise, due consideration shall be given to the effect of eccentricity. In no case shall eccentricity of any web member at a joint exceed 3/4 of the over-all dimension, measured in the plane of the web, of the largest member connected. The eccentricity of any web member shall be the perpendicular distance from the centroidal axis of that web member to the point on the centroidal axis of the chord which is vertically above or below the intersection of the centroidal axes of the web members forming the joint. Ends of joists shall be proportioned to resist bending produced by eccentricity at the support.

4.6 CAMBER

Joists shall have approximate camber in accordance with the following:

TABLE 4.6-1

Top Chord Length		Approximate Camber	
20'-0"	(6096 mm)	1/4"	(6 mm)
30'-0"	(9144 mm)	3/8"	(10 mm)
40'-0"	(12192 mm)	5/8"	(16 mm)
50'-0"	(15240 mm)	1"	(25 mm)
60'-0"	(18288 mm)	1 1/2"	(38 mm)

The specifying professional shall give consideration to coordinating joist camber with adjacent framing.

4.7 VERIFICATION OF DESIGN AND MANUFACTURE

(a) Design Calculations

Companies manufacturing **K-**Series Joists shall submit design data to the Steel Joist Institute (or an independent agency approved by the Steel Joist Institute) for verification of compliance with the SJI Specifications. Design data shall be submitted in detail and in the format specified by the Institute.

(b) Tests of Chord and Web Members

Each manufacturer shall, at the time of design review by the Steel Joist Institute or other independent agency, verify by tests that the design, in accordance with Sections 4.1 through 4.5 of this specification, will provide the theoretical strength of critical members. Such tests shall be evaluated considering the actual yield strength of the members of the test joists.

Material tests for determining mechanical properties of component members shall be conducted.

(c) Tests of Joints and Connections

Each manufacturer shall verify by shear tests on representative joints of typical joists that connections will meet the provision of Section 4.5(b). Chord and web members may be reinforced for such tests.

(d) In-Plant Inspections

Each manufacturer shall verify their ability to manufacture **K-**Series Joists through periodic In-Plant Inspections. Inspections shall be performed by an independent agency approved by the Steel Joist Institute. The frequency, manner of inspection, and manner of reporting shall be determined by the Steel Joist Institute. The plant inspections are not a guarantee of the quality of any specific joists; this responsibility lies fully and solely with the individual manufacturer.

SECTION 5. APPLICATION

5.1 USAGE

These specifications shall apply to any type of structure where floors and roofs are to be supported directly by steel joists installed as hereinafter specified. Where joists are used other than on simple spans under uniformly distributed loading as prescribed in Section 4.1, they shall be investigated and modified if necessary to limit the required stresses to those listed in Section 4.2.

CAUTION: If a rigid connection of the bottom chord is to be made to the column or other support, it shall be made only after the application of the dead loads. The joist is then no longer simply supported, and the system must be investigated for continuous frame action by the specifying professional.

The designed detail of a rigid type connection and moment plates shall be shown on the structural drawings by the specifying professional. The moment plates shall be furnished by other than the joist manufacturer.

5.2 SPAN

The span of a joist shall not exceed 24 times its depth.

5.3 END SUPPORTS

(a) Masonry and Concrete

K-Series Joists supported by masonry or concrete are to bear on steel bearing plates and shall be designed as steel bearing. Due consideration of the end reactions and all other vertical or lateral forces shall be taken by the specifying professional in the design of the steel bearing plate and the masonry or concrete. The ends of **K-**Series Joists shall extend a distance of not less than 4 inches (102 millimeters) over the masonry or concrete support and be anchored to the steel bearing plate. The plate shall be located not more than 1/2 inch (13 millimeters) from the face of the wall and shall be not less than 6 inches (152 millimeters) wide perpendicular to the length of the joist. The plate is to be designed by the specifying professional and shall be furnished by other than the joist manufacturer.

Where it is deemed necessary to bear less than 4 inches (102 millimeters) over the masonry or concrete support, special consideration is to be given to the design of the steel bearing plate and the masonry or concrete by the specifying professional. The joists must bear a minimum of 2 1/2 inches (64 millimeters) on the steel bearing plate.

(b) Steel

Due consideration of the end reactions and all other vertical and lateral forces shall be taken by the specifying professional in the design of the steel support. The ends of **K-**Series Joists shall extend a distance of not less than 2 1/2 inches (64 millimeters) over the steel supports.

5.4 BRIDGING

Top and bottom chord bridging is required and shall consist of one or both of the following types.

(a) Horizontal

Horizontal bridging shall consist of continuous horizontal steel members. Attachments to the joist chords shall be made by welding or mechanical means and shall be capable of resisting a nominal (unfactored) horizontal force of not less than 700 pounds (3114 Newtons).

The ratio of unbraced length to least radius of gyration, ℓ/r, of the bridging member shall not exceed 300, where ℓ is the distance in inches (millimeters) between attachments and r is the least radius of gyration of the bridging member.

(b) Diagonal

Diagonal bridging shall consist of cross-bracing with a ℓ/r ratio of not more than 200, where ℓ is the distance in inches (millimeters) between connections and r is the least radius of gyration of the bracing member. Where cross-bracing members are connected at their point of intersection, the ℓ distance shall be taken as the distance in inches (millimeters) between connections at the point of intersection of the bracing members and the connections to the chord of the joists. Connections to the chords of steel joists shall be made by positive mechanical means or by welding.

(c) Quantity and Spacing

The number of rows of top chord bridging shall not be less than as shown in Bridging Tables 5.4-1 and the spacing shall meet the requirements of Section 4.4(a). The number of rows of bottom chord bridging, including bridging required per Section 5.11, shall not be less than the number of top chord rows. Rows of bottom chord bridging are permitted to be spaced independently of rows of top chord bridging. The spacing of rows of bottom chord bridging shall meet the slenderness requirement of Section 4.3 and any specified strength requirements.

(d) Bottom Chord Bearing Joists

Where bottom chord bearing joists are utilized, a row of diagonal bridging shall be provided near the support(s). This bridging shall be installed and anchored before the hoisting cable(s) is released.

TABLE 5.4-1

NUMBER OF ROWS OF TOP CHORD BRIDGING**

Refer to the **K-**Series Load Table and Specification Section 6 for required bolted diagonal bridging.
Distances are Joist Span lengths in feet - See “Definition of Span” preceding Load Table.

*Section Number	One Row	Two Rows	Three Rows	Four Rows	Five Rows
#1	Up thru 16	Over 16 thru 24	Over 24 thru 28		
#2	Up thru 17	Over 17 thru 25	Over 25 thru 32		
#3	Up thru 18	Over 18 thru 28	Over 28 thru 38	Over 38 thru 40	
#4	Up thru 19	Over 19 thru 28	Over 28 thru 38	Over 38 thru 48	
#5	Up thru 19	Over 19 thru 29	Over 29 thru 39	Over 39 thru 50	Over 50 thru 52
#6	Up thru 19	Over 19 thru 29	Over 29 thru 39	Over 39 thru 51	Over 51 thru 56
#7	Up thru 20	Over 20 thru 33	Over 33 thru 45	Over 45 thru 58	Over 58 thru 60
#8	Up thru 20	Over 20 thru 33	Over 33 thru 45	Over 45 thru 58	Over 58 thru 60
#9	Up thru 20	Over 20 thru 33	Over 33 thru 46	Over 46 thru 59	Over 59 thru 60
#10	Up thru 20	Over 20 thru 37	Over 37 thru 51	Over 51 thru 60	
#11	Up thru 20	Over 20 thru 38	Over 38 thru 53	Over 53 thru 60	
#12	Up thru 20	Over 20 thru 39	Over 39 thru 53	Over 53 thru 60	

* Last digit(s) of joist designation shown in Load Table
** See Section 5.11 for additional bridging required for uplift design.

5.5 INSTALLATION OF BRIDGING

Bridging shall support the top and bottom chords against lateral movement during the construction period and shall hold the steel joists in the approximate position as shown on the joist placement plans.

The ends of all bridging lines terminating at walls or beams shall be anchored thereto.

5.6 END ANCHORAGE

(a) Masonry and Concrete

Ends of **K-**Series Joists resting on steel bearing plates on masonry or structural concrete shall be attached thereto with a minimum of two 1/8 inch (3 millimeters) fillet welds 1 inch (25 millimeters) long, or with two 1/2 inch (13 millimeters) ASTM A307 bolts, or the equivalent.

(b) Steel

Ends of **K-**Series Joists resting on steel supports shall be attached thereto with a minimum of two 1/8 inch (3 millimeters) fillet welds 1 inch (25 millimeters) long, or with two 1/2 inch (13 millimeters) ASTM A307 bolts, or the equivalent. When **K-**Series Joists are used to provide lateral stability to the supporting member, the final connection shall be made by welding or as designated by the specifying professional.

(c) Uplift

Where uplift forces are a design consideration, roof joists shall be anchored to resist such forces (Refer to Section 5.11 Uplift).

5.7 JOIST SPACING

Joists shall be spaced so that the loading on each joist does not exceed the design load (LRFD or ASD) for the particular joist designation and span as shown in the applicable load tables.

5.8 FLOOR AND ROOF DECKS

(a) Material

Floor and roof decks may consist of cast-in-place or precast concrete or gypsum, formed steel, wood, or other suitable material capable of supporting the required load at the specified joist spacing.

(b) Thickness

Cast-in-place slabs shall be not less than 2 inches (51 millimeters) thick.

(c) Centering

Centering for cast-in-place slabs may be ribbed metal lath, corrugated steel sheets, paper-backed welded wire fabric, removable centering or any other suitable material capable of supporting the slab at the designated joist spacing. Centering shall not cause lateral displacement or damage to the top chord of joists during installation or removal of the centering or placing of the concrete.

(d) Bearing

Slabs or decks shall bear uniformly along the top chords of the joists.

(e) Attachments

The spacing for slab or deck attachments along the joist top chord shall not exceed 36 inches (914 millimeters), and shall be capable of resisting a nominal (unfactored) lateral force of not less than 300 pounds (1335 Newtons), i.e., 100 plf (1.46 kN/m).

(f) Wood Nailers

Where wood nailers are used, such nailers in conjunction with deck or slab shall be attached to the top chords of the joists in conformance with Section 5.8(e).

(g) Joist With Standing Seam Roofing

The stiffness and strength of standing-seam roof clips varies from one manufacturer to another. Therefore, some roof systems cannot be counted on to provide lateral stability to the joists which support the roof. Sufficient stability must be provided to brace the joists laterally under the full design load. The compression chord must resist the chord axial design force in the plane of the joist (i.e., x-x axis buckling) and out of the plane of the joist (i.e., y-y axis buckling). Out-of-plane strength may be achieved by adjusting the bridging spacing and/or increasing the compression chord area, the joist depth, and the y-axis radius of gyration. The effective slenderness ratio in the y-direction equals $0.94\ L/r_y$; where L is the bridging spacing in inches (millimeters). The maximum bridging spacing may not exceed that specified in Section 5.4(c).

Horizontal bridging members attached to the compression chords and their anchorage's must be designed for a compressive axial force of 0.0025nP, where n is the number of joists between end anchors and P is the chord design force in kips (Newtons). The attachment force between the horizontal bridging member and the compression chord is 0.005P. Horizontal bridging attached to the tension chords shall be proportioned so that the slenderness ratio between attachments does not exceed 300. Diagonal bridging shall be proportioned so that the slenderness ratio between attachments does not exceed 200.

5.9 DEFLECTION

The deflection due to the design nominal live load shall not exceed the following:

Floors: 1/360 of span.

Roofs: 1/360 of span where a plaster ceiling is attached or suspended.
1/240 of span for all other cases.

The specifying professional shall give consideration to the effects of deflection and vibration* in the selection of joists.

* For further reference, refer to Steel Joist Institute Technical Digest #5, "Vibration of Steel Joist-Concrete Slab Floors" and the Institute's Computer Vibration Program.

5.10 PONDING*

The ponding investigation shall be performed by the specifying professional.

* For further reference, refer to Steel Joist Institute Technical Digest #3, "Structural Design of Steel Joist Roofs to Resist Ponding Loads" and AISC Specifications.

5.11 UPLIFT

Where uplift forces due to wind are a design requirement, these forces must be indicated on the contract drawings in terms of NET uplift in pounds per square foot (Pascals). The contract documents shall indicate if the net uplift is based upon LRFD or ASD. When these forces are specified, they must be considered in the design of joists and/or bridging. A single line of **bottom chord** bridging must be provided near the first bottom chord panel points whenever uplift due to wind forces is a design consideration.*

* For further reference, refer to Steel Joist Institute Technical Digest #6, "Structural Design of Steel Joist Roofs to Resist Uplift Loads".

5.12 INSPECTION

Joists shall be inspected by the manufacturer before shipment to verify compliance of materials and workmanship with the requirements of these specifications. If the purchaser wishes an inspection of the steel joists by someone other than the manufacturer's own inspectors, they may reserve the right to do so in their "Invitation to Bid" or the accompanying "Job Specifications".

Arrangements shall be made with the manufacturer for such inspection of the joists at the manufacturing shop by the purchaser's inspectors at purchaser's expense.

5.13 PARALLEL CHORD SLOPED JOISTS

The span of a parallel chord sloped joist shall be defined by the length along the slope. Minimum depth, load-carrying capacity, and bridging requirements shall be determined by the sloped definition of span. The Standard Load Table capacity shall be the component normal to the joist.

SECTION 6.*
ERECTION STABILITY AND HANDLING

When it is necessary for the erector to climb on the joists, extreme caution must be exercised since unbridged joists may exhibit some degree of instability under the erector's weight.

(a) Stability Requirements

1) Before an employee is allowed on the steel joist: BOTH ends of joists at columns (or joists designated as column joists) shall be attached to its supports. For all other joists a minimum of one end shall be attached before the employee is allowed on the joist. The attachment shall be in accordance with Section 5.6 – End Anchorage.

   When a bolted seat connection is used for erection purposes, as a minimum, the bolts must be snug tightened. The snug tight condition is defined as the tightness that exists when all plies of a joint are in firm contact. This may be attained by a few impacts of an impact wrench or the full effort of an employee using an ordinary spud wrench.

2) On steel joists that do not require erection bridging as shown by the unshaded area of the Load Tables, only one employee shall be allowed on the steel joist unless all bridging is installed and anchored.

   * For a thorough coverage of this topic, refer to SJI Technical Digest #9, "Handling and Erection of Steel Joists and Joist Girders".

3) Where the span of the steel joist is within the Red shaded area of the Load Table, the following shall apply:

   a) The row of bridging nearest the mid span of the steel joists shall be bolted diagonal erection bridging; and

   b) Hoisting cables shall not be released until this bolted diagonal erection bridging is installed and anchored, unless an alternate method of stabilizing the joist has been provided; and

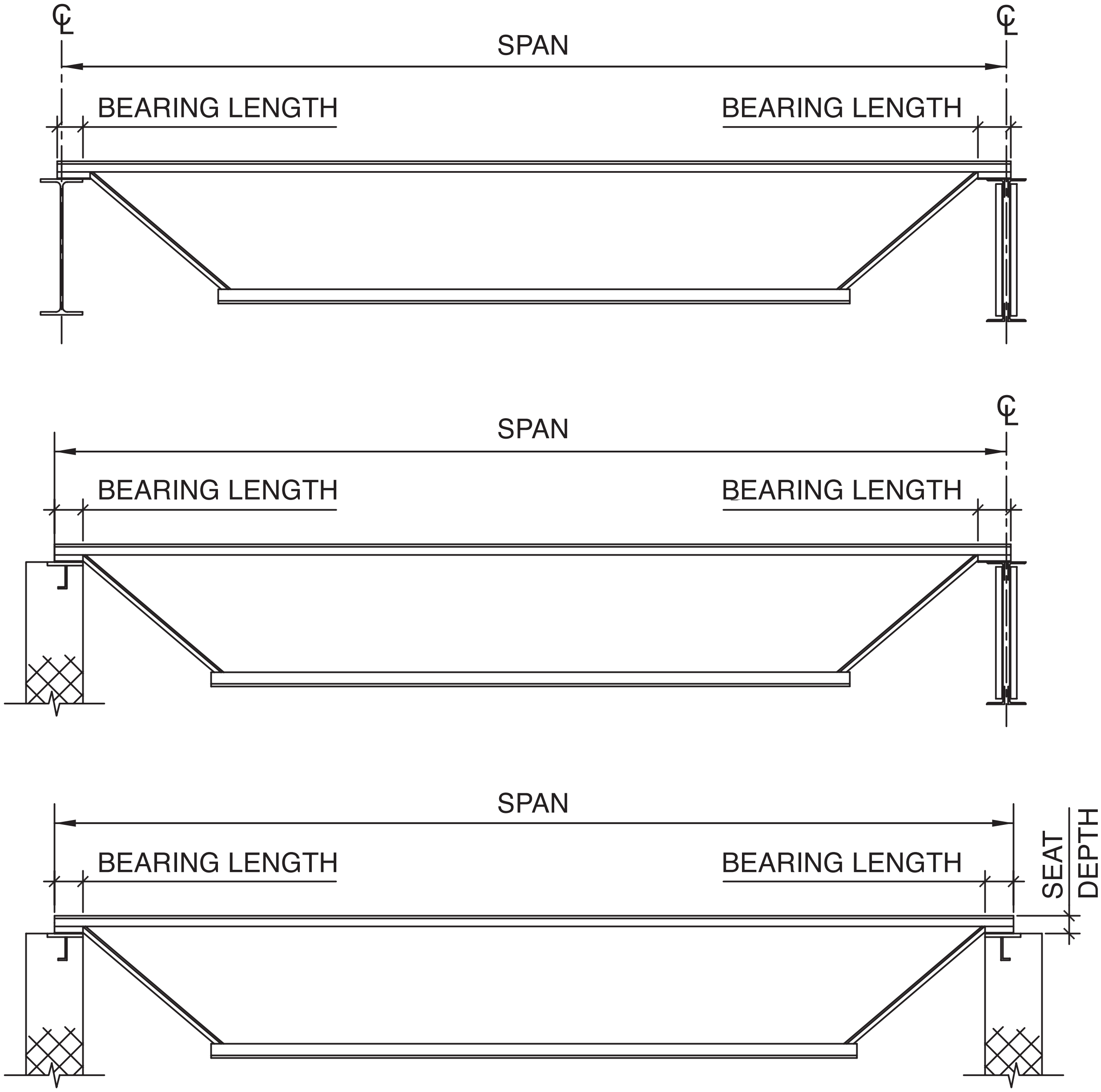
   c) No more than one employee shall be allowed on these spans until all other bridging is installed and anchored.

4) When permanent bridging terminus points cannot be used during erection, additional temporary bridging terminus points are required to provide stability.

5) In the case of bottom chord bearing joists, the ends of the joist must be restrained laterally per Section 5.4(d).

6) After the joist is straightened and plumbed, and all bridging is completely installed and anchored, the ends of the joists shall be fully connected to the supports in accordance with Section 5.6 End Anchorage.

DEFINITION OF SPAN
(U. S. Customary Units)



NOTES:

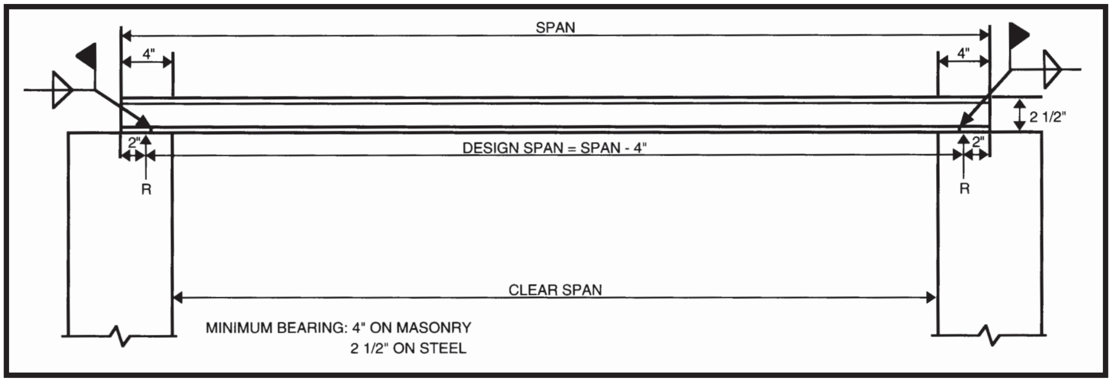
1) **DESIGN LENGTH = SPAN - 0.33 FT.**
2) **BEARING LENGTH FOR STEEL SUPPORTS SHALL NOT BE LESS THAN 2 1/2 INCHES; FOR MASONRY AND CONCRETE NOT LESS THAN 4 INCHES.**
3) **PARALLEL CHORD JOISTS INSTALLED TO A SLOPE GREATER THAN 1/2 INCH PER FOOT SHALL USE SPAN DEFINED BY THE LENGTH ALONG THE SLOPE.**

FABRICATION

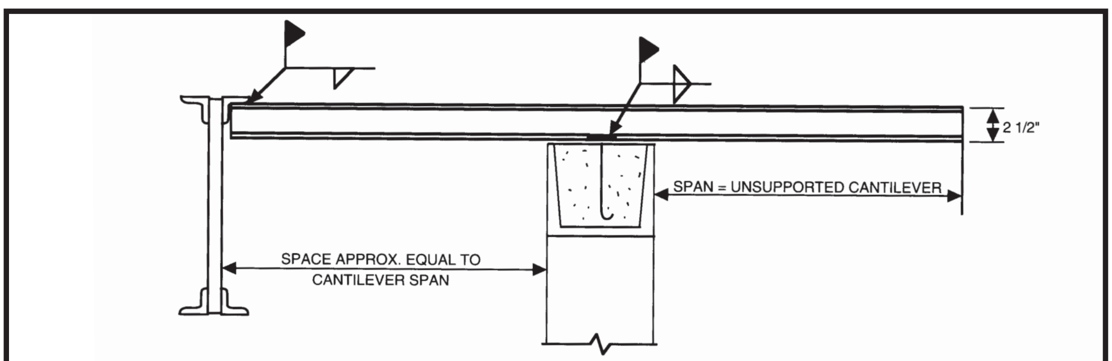
- Depth 2.5 in
- Maximum Length 10 ft
- Minimum Length 3 ft
- Contact your local Vulcraft plant for sloped or pitched seat information.

2.5K SERIES SIMPLE SPAN INFORMATION

2.5K TYPE	2.5K1	2.5K2	2.5K3
S in^3	0.62	0.84	1.2
I in^4	0.78	1.1	1.5
WT lbs/ft	3.0	4.2	6.4



NOTE: 2.5K SERIES NOT U.L. APPROVED.



OUTRIGGER DETAIL

NOTE: 2.5K SERIES NOT U.L. APPROVED.

LRFD

LOAD TABLE FOR LOOSE OUTRIGGERS

OUTRIGGER TYPE	TOTAL ALLOWABLE LOAD FOR UNSUPPORTED CANTILEVER PLF*									
	SPAN ft-in									
	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"
2.5K1	825	749	519	381	293	231	188	155	—	—
2.5K2	825	825	698	512	392	311	251	207	174	—
2.5K3	825	825	825	740	566	447	362	299	252	215

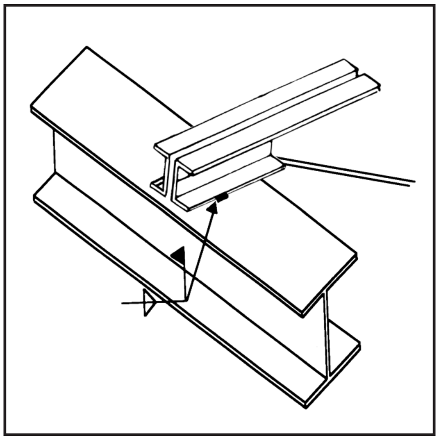
ASD

LOAD TABLE FOR LOOSE OUTRIGGERS

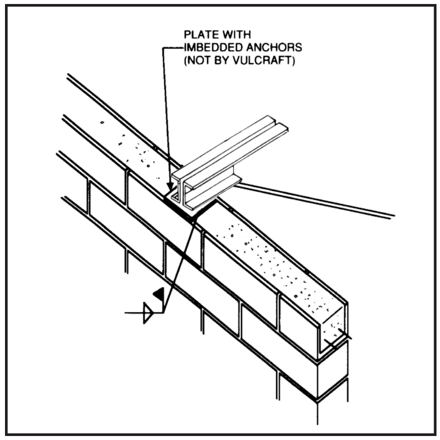
OUTRIGGER TYPE	TOTAL ALLOWABLE LOAD FOR UNSUPPORTED CANTILEVER PLF*									
	SPAN ft-in									
	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"
2.5K1	550	499	346	254	195	154	125	103	—	—
2.5K2	550	550	465	341	261	207	167	138	116	—
2.5K3	550	550	550	493	377	298	241	199	168	143

*Serviceability requirements must be checked by the specifying professional.

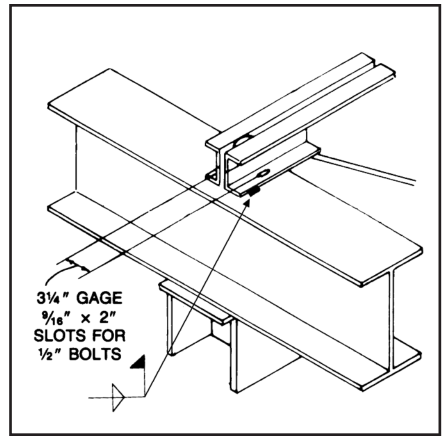
K SERIES OPEN WEB STEEL JOISTS



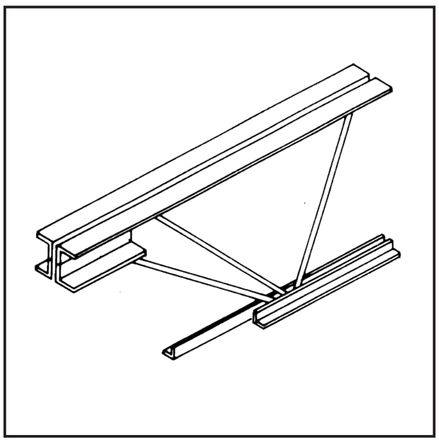
ANCHORAGE TO STEEL
SEE SJI SPECIFICATION 5.3 (b) AND 5.6



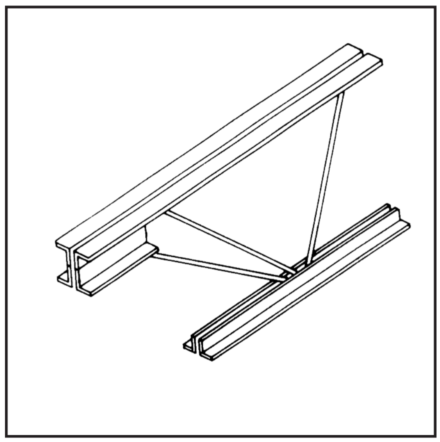
ANCHORAGE TO MASONARY
SEE SJI SPECIFICATION 5.3 (a) AND 5.6



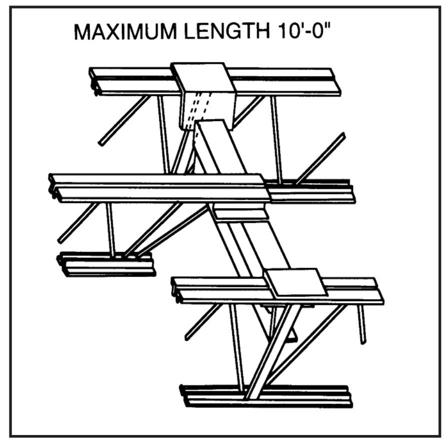
BOLTED CONNECTION*
TYPICALLY REQUIRED AT COLUMNS



CEILING EXTENSION



BOTTOM CHORD STRUT



HEADERS
Note: If header does not bear at a Joist Panel Point add extra web in field as shown.
EW or Panel Point by Vulcraft

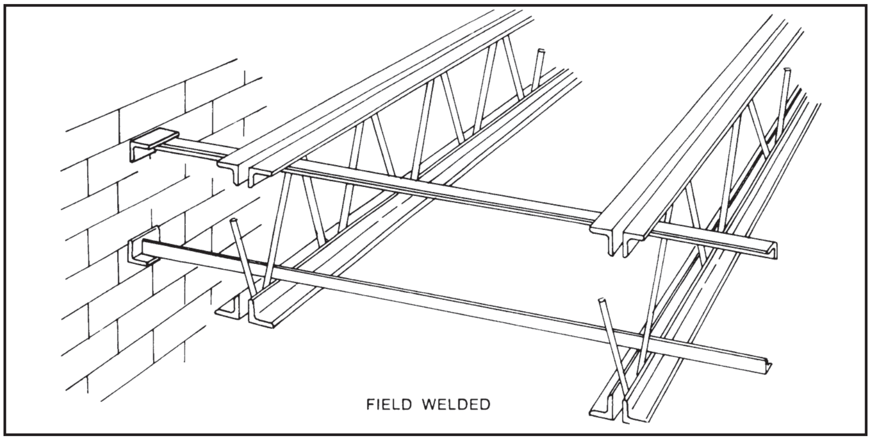
MAXIMUM DUCT OPENING SIZES (K SERIES)*

JOIST DEPTH	ROUND	SQUARE	RECTANGLE
8 inches	5 inches	4x4 inches	3x8 inches
10 inches	6 inches	5x5 inches	3x8 inches
12 inches	7 inches	6x6 inches	4x9 inches
14 inches	8 inches	6x6 inches	5x9 inches
16 inches	9 inches	7 1/2x 71/2 inches	6X10 inches
18 inches	11 inches	8x8 inches	7x11 inches
20 inches	11 inches	9x9 inches	7x12 inches
22 inches	12 inches	9 1/2 x9 1/2 inches	8x12 inches
24 inches	13 inches	10x10 inches	8x13 inches
26 inches	151/2 inches	12x12 inches	9x18 inches
28 inches	16 inches	13x13 inches	9x18 inches
30 inches	17 inches	14x14 inches	10x18 inches
***FOR LH SERIES CONSULT WITH VULCRAFT**			

SEE SJI SPECIFICATION - SECTION 6. FOR HANDLING AND ERECTION OF K-SERIES OPEN WEB STEEL JOISTS AND SJI TECHNICAL DIGEST NO. 9.

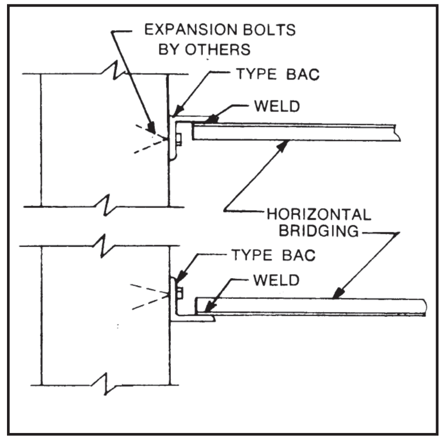
SPECIFYING PROFESSIONAL MUST INDICATE ON STRUCTURAL DRAWINGS SIZE AND LOCATION OF ANY DUCT THAT IS TO PASS THRU JOIST.

K SERIES OPEN WEB STEEL JOISTS

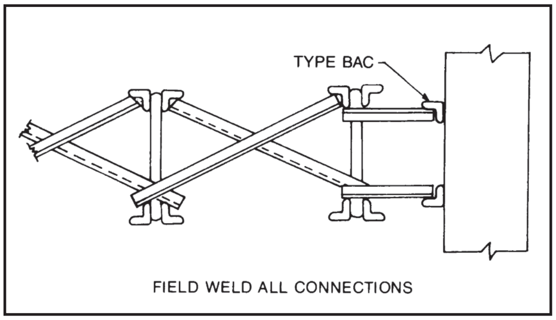


HORIZONTAL BRIDGING
SEE SJI SPECIFICATION 5.5 AND 6.

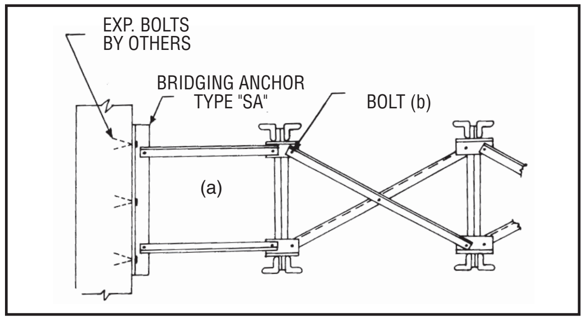
NOTE: DO NOT WELD BRIDGING TO JOIST WEB MEMBERS.
DO NOT HANG ANY MECHANICAL, ELECTRICAL, ETC. FROM BRIDGING.



BRIDGING ANCHORS
SEE SJI SPECIFICATION 5.5 AND 6.



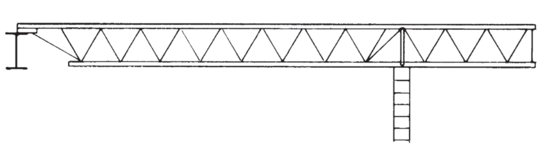
WELDED CROSS BRIDGING
SEE SJI SPECIFICATION 5.5 AND 6.
HORIZONTAL BRIDGING SHALL BE USED IN SPACE ADJACENT TO THE WALL TO ALLOW FOR PROPER DEFLECTION OF THE JOIST NEAREST THE WALL.



BOLTED CROSS BRIDGING
SEE SJI SPECIFICATION 5.5 AND 6.

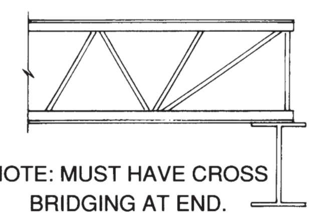
(a) Horizontal Bridging units shall be used in the space adjacent to the wall to allow for proper deflection of the joist nearest the wall.

(b) For required bolt size refer to bridging table on page 136.
NOTE: Clip configuration may vary from that shown.



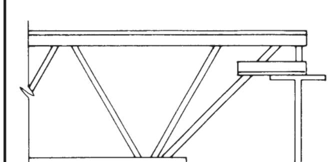
NOTE: MUST HAVE CROSS BRIDGING AT SUPPORT

FULL DEPTH CANTILEVER END
SEE SJI SPECIFICATION 5.4 (d) AND 5.5 FOR BRIDGING REQUIREMENTS.



NOTE: MUST HAVE CROSS BRIDGING AT END.

SQUARE END
SEE SJI SPECIFICATION 5.4 (d) AND 5.5 FOR BRIDGING REQUIREMENTS.



DEEP BEARINGS
CONFIGURATION MAY VARY

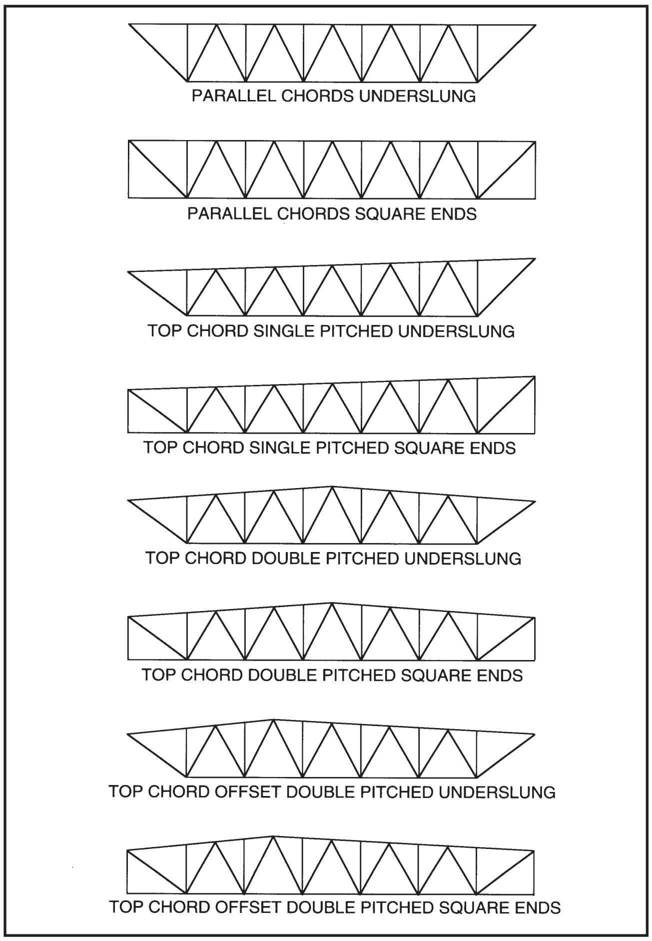
LH & DLH SERIES LONGSPAN STEEL JOISTS

STANDARD TYPES

Longspan steel joists can be furnished with either underslung or square ends, with parallel chords or with single or double pitched top chords to provide sufficient slope for roof drainage.

The Longspan joist designation is determined by its nominal depth at the center of the span, except for offset double pitched joists, where the depth should be given at the ridge. A part of the designation should be either the section number or the total design load over the design live load (TL/LL given in plf).

All pitched joists will be cambered in addition to the pitch unless specified otherwise.



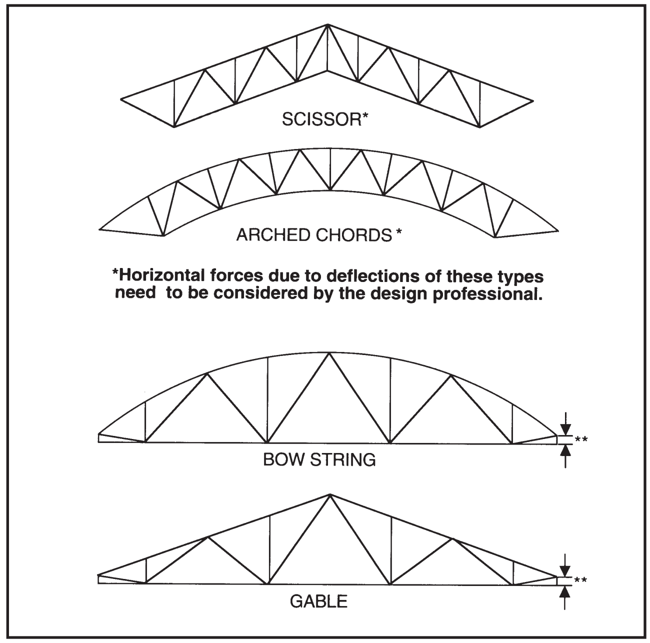
CAMBER

__Non-Standard Types:__ The design professional shall provide on the structural drawings the amount of camber desired in inches. If camber is not specified, Vulcraft will use the camber values for LH and DLH joists based on top chord length.

__Standard Types:__ The camber listed in the table will be fabricated into the joists unless the design professional specifically states otherwise on the structural drawings.

NON-STANDARD TYPES

The following joists can also be supplied by Vulcraft, however, **THE DISTRICT SALES OFFICE OR MANUFACTURING FACILITY NEAREST YOU SHOULD BE CONTACTED FOR ANY LIMITATIONS IN DEPTH OR LENGTH.**



**Contact Vulcraft for minimum depth at ends.

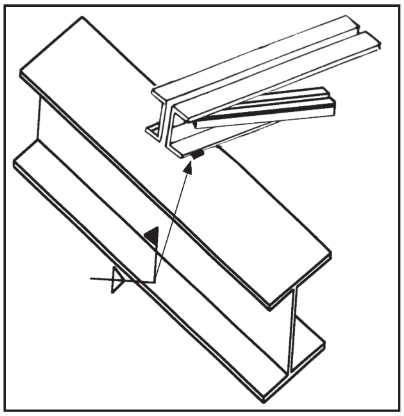
CAMBER FOR STANDARD TYPES

LH &DLH series joists shall have camber in accordance with the following table:***

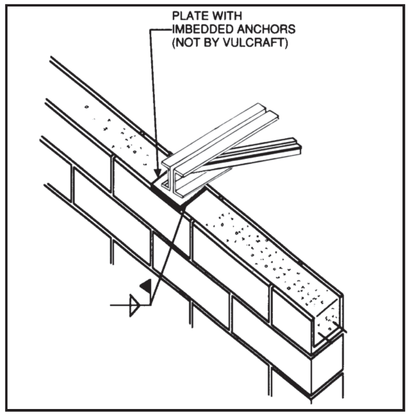
Top Chord Length		Approx. Camber	
20'-0"	(6096 mm)	1/4"	(6 mm)
30'-0"	(9144 mm)	3/8"	(10 mm)
40'-0"	(12192 mm)	5/8"	(16 mm)
50'-0"	(15240 mm)	1"	(25 mm)
60'-0"	(18288 mm)	1 1/2"	(38 mm)
70'-0"	(21336 mm)	2"	(51 mm)
80'-0"	(24384 mm)	2 3/4"	(70 mm)
90'-0"	(27432 mm)	3 1/2"	(89 mm)
100'-0"	(30480 mm)	4 1/4"	(108 mm)
110'-0"	(33528 mm)	5"	(127 mm)
120'-0"	(36576 mm)	6"	(152 mm)
130'-0"	(39621 mm)	7"	(178 mm)
140'-0"	(42672 mm)	8"	(203 mm)
144'-0"	(43890 mm)	8 1/2"	(216 mm)

*** NOTE: If full camber is not desired near walls or other structural members please note on the structural drawings.

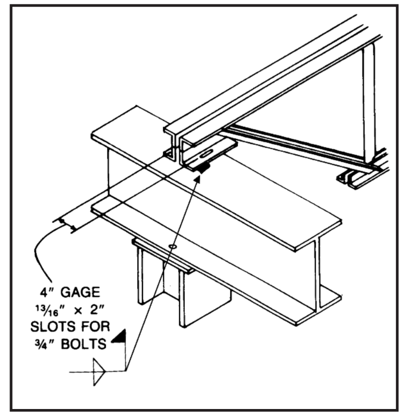
LH & DLH SERIES LONGSPAN STEEL JOISTS



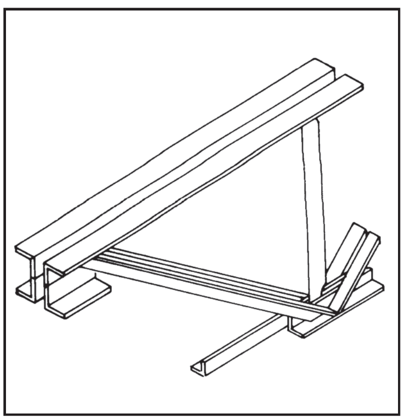
ANCHORAGE TO STEEL
SEE SJI SPECIFICATION
104.4 (b) AND 104.7 (b)



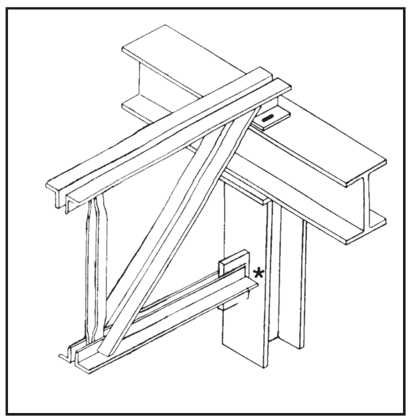
ANCHORAGE TO MASONRY
SEE SJI SPECIFICATION
104.4 (a) AND 104.7 (a)



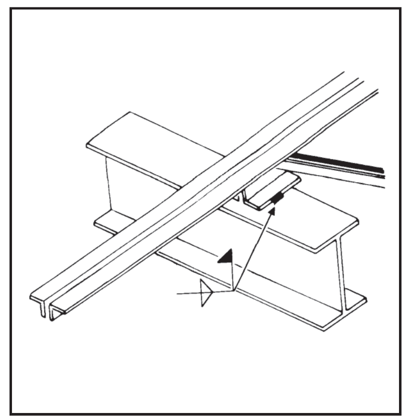
BOLTED CONNECTION
See Note (c)
Typically required at columns



CEILING EXTENSION



BOTTOM CHORD EXTENSION
*If bottom chord extension is to be bolted or welded the specifiying professional must provide axial loads on structural drawings.



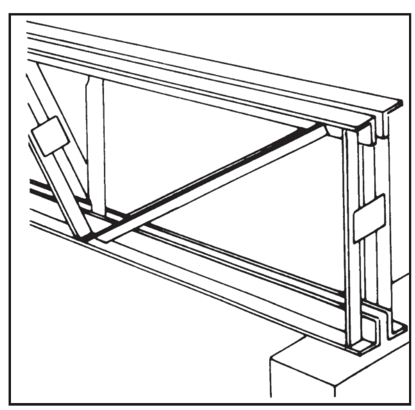
TOP CHORD EXTENSION
See Note (a)

(a) Extended top chords or full depth cantilever ends require the special attention of the specifying professional.
The magnitude and location of the design loads to be supported, the deflection requirements, and the proper bracing shall be clearly indicated on the structural drawings.

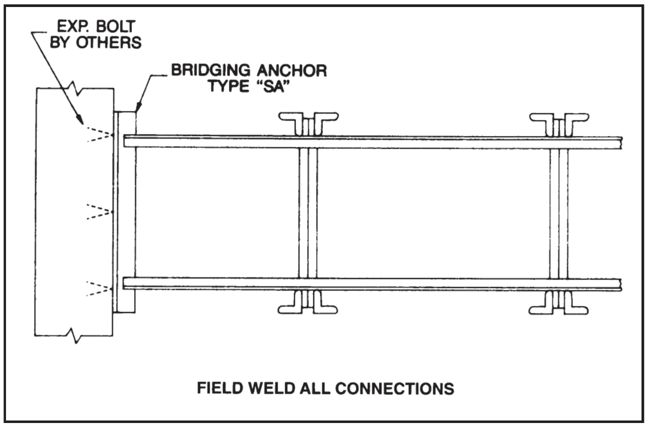
(b) See SJI Specification - Section 105 for Handling and Erection of LH and DLH joists.

(c) The Occupational Safety and Health Administration Standards (OSHA), Paragraph 1910.12 refers to Paragraph 1518.751 of "Construction Standards" which states:
"In steel framing, where bar joists are utilized, and columns are not framed in at least two directions with structural steel members, a bar joist shall be field-bolted at columns to provide lateral stability during construction."

NOTE: Configurations may vary from that shown.



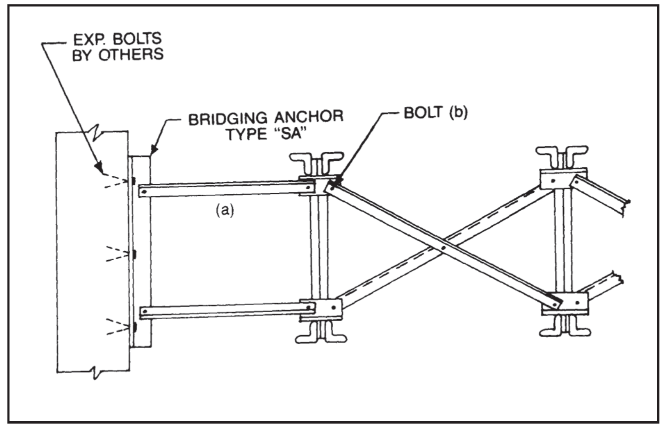
SQUARE END
See SJI Specification 104.5 (f).
Cross bridging required at end of bottom bearing joist.



HORIZONTAL BRIDGING

For the proper use of horizontal bridging refer to sections 104.5(a) and 105.

NOTE: Do not weld bridging to web members. Do not hang <u>any</u> mechanical, electrical, etc. from bridging.



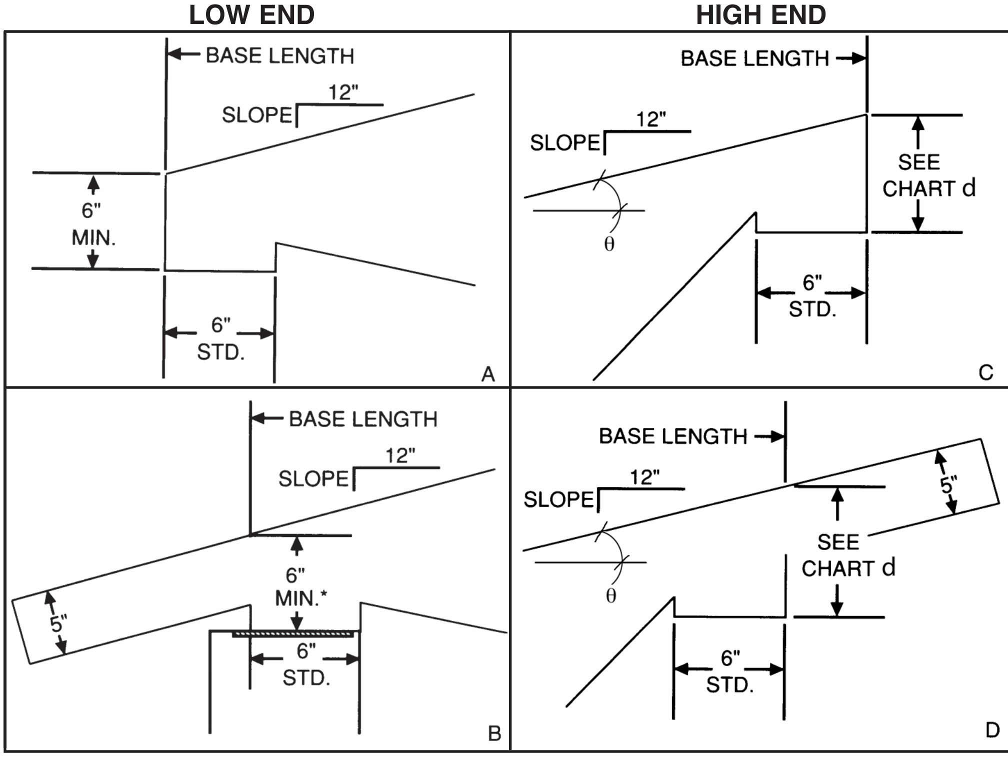
CROSS BRIDGING

(a) Horizontal Bridging units shall be used in the space adjacent to the wall to allow for proper deflection of the joist nearest the wall.

(b) For required bolt size refer to bridging table on page 136. NOTE: Clip configuration may vary from that shown.

LH & DLH SERIES OPEN WEB STEEL JOISTS SLOPED SEAT REQUIREMENTS

LOW END **HIGH END**



SLOPE RATE	HIGH END MINIMUM **SEAT DEPTH d
1/4:12	6 1/2”
3/8:12	6 1/2”
1/2:12	6 1/2”
1:12	6 1/2”
1 1/2:12	7”
2:12	7”
2 1/2:12	7 1/2”
3:12	7 1/2”
3 1/2:12	8”
4:12	8 1/2”
4 1/2:12	8 1/2”
5:12	9 1/2”
6:12 & OVER SEE BELOW	

* 7 1/2" at 18 and 19 chord section numbers. Consult Vulcraft for information when TCX's are present.
** Add 2 1/2" to seat depths at 18 and 19 chord section numbers.

NOTES:
(1) Depths shown are the minimums required for fabrication of sloped bearing seats.
(2) $d = 5/8 + 5 / \cos\theta + 6 \tan\theta$
(3) Clearance must be checked at outer edge of support as shown in detail B. Increase bearing depth as required to permit passage of 5" deep extension.
(4) If extension depth greater than 5" is required (see detail B and D) increase bearing depths accordingly.

HIGH STRENGTH

ECONOMICAL

DESIGN – Vulcraft LH & DLH Series long span steel joists are designed in accordance with the specifications of the Steel Joist Institute.

ACCESSORIES see page 45.

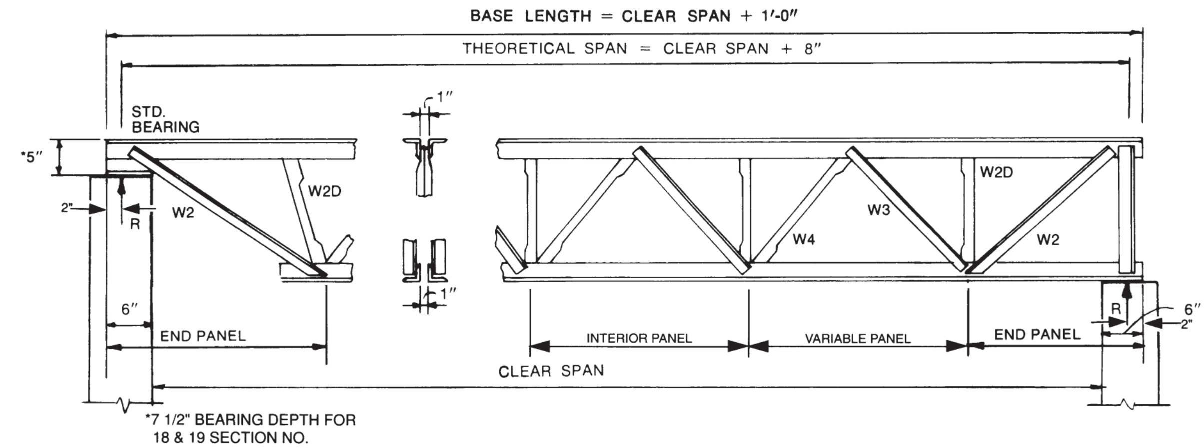
ROOF SPANS TO 144'-0

FLOOR SPANS TO 120'-0

PAINT – Vulcraft joists receive a shop-coat of rust inhibitive primer whose performance characteristics conform to those of the Steel Joist Institute specification 102.4.

SPECIFICATIONS see page 50.

LH & DLH SERIES DETAILS



MAXIMUM JOIST SPACING FOR DIAGONAL BRIDGING					
	BRIDGING ANGLE SIZE-EQUAL LEG ANGLES				
JOIST DEPTH	1x7/64 (25mm x 3mm) r =.20"	1-1/4x7/64 (32mm x 3mm) r =.25"	1-1/2x7/64 (38mm x 3mm) r =.30"	1-3/4x7/64 (45mm x 3mm) r =.35"	2x1/8 (51mm x 3mm) r =.40"
32	6'-1"(1854mm)	7'-10"(2387mm)	9'-7"(2921mm)	11'-4"(3454mm)	13'-0"(3962mm)
36		7'-9"(2362mm)	9'-6"(2895 mm)	11'-3"(3429mm)	12'-11"(3973mm)
40		7'-7"(2311mm)	9'-5"(2870 mm)	11'-2"(3403mm)	12'-10"(3911mm)
44		7'-5"(2260mm)	9'-3"(2819 mm)	11'-0"(3352mm)	12'-9"(3886mm)
48		7'-3"(2209mm)	9'-2"(2794 mm)	10'-11"(3327mm)	12'-8"(3860mm)
52			9'-0"(2743 mm)	10'-9"(3276mm)	12'-7"(3835mm)
56			8'-10"(2692 mm)	10'-8"(3251mm)	12'-5"(3784mm)
60			8'-7"(2616 mm)	10'-6"(3200mm)	12'-4"(3759mm)
64			8'-5"(2565 mm)	10'-4"(3149mm)	12'-2"(3708mm)
68			8'-2"(2489 mm)	10'-2"(3098mm)	12'-0"(3657mm)
72			8'-0"(2438 mm)	10'-0"(3048mm)	11'-10"(3606mm)

LH & DLH TABLE MINIMUM BEARING LENGTHS			
Joist Type	On Masonry	On Concrete	On Steel
LH 02 thru 17 DLH 10 thru 19	6"	6"	4"
MINIMUM BEARING PLATE WIDTHS			
LH 02 thru LH 12 DLH 10 thru DLH 12	9"	9"	
LH 13 thru LH 17 DLH 13 thru DLH 19	12"	12"	

SECTION NUMBER*	MAX. SPACING OF LINES OF BRIDGING	HORIZONTAL BRACING FORCE**	
		lbs.	(N)
02, 03, 04	11'-0" (3352mm)	400	(1779)
05 - 06	12'-0" (3657mm)	500	(2224)
07 - 08	13'-0" (3962mm)	650	(2891)
09 - 10	14'-0" (4267mm)	800	(3558)
11 - 12	16'-0" (4876mm)	1000	(4448)
13 - 14	16'-0" (4876mm)	1200	(5337)
15 - 16	21'-0" (6400mm)	1600	(7117)
17	21'-0' (6400mm)	1800	(8006)
18 - 19	26'-0" (7924mm)	2000	(8896)

NUMBER OF LINES OF BRIDGING BASED ON CLEAR SPAN.
*LAST TWO DIGITS OF JOIST DESIGNATION.
**NOMINAL BRACING FORCE IS UNFACTORED.

MAXIMUM JOIST SPACING FOR HORIZONTAL BRIDGING SPANS OVER 60' REQUIRE BOLTED DIAGONAL BRIDGING						
	BRIDGING ANGLE SIZE-EQUAL LEG ANGLES					
SECTION NUMBER*	1x7/64 (25mm x 3mm) r = .20"	1-1/4x7/64 (32mm x 3mm) r = .25"	1-1/2x7/64 (38mm x 3mm) r = .30"	1-3/4x7/64 (45mm x 3mm) r = .35"	2x1/8 (51mm x 3mm) r = .40"	2-1/2x5/32 (64mm x 4mm) r = .50"
02, 03, 04	4'-7"(1397mm)	6'-3"(1905mm)	7'-6"(2286mm)	8'-9"(2667mm)	10'-0"(3048mm)	12'-4"(3759mm)
05 - 06	4'-1"(1245mm)	5'-9"(1753mm)	7'-6"(2286mm)	8'-9"(2667mm)	10'-0"(3048mm)	12'-4"(3759mm)
07 - 08	3'-9"(1143mm)	5'-1"(1549mm)	6'-8"(2032mm)	8'-6"(2590mm)	10'-0"(3048mm)	12'-4"(3759mm)
09 - 10		4'-6"(1372mm)	6'-0"(1829mm)	7'-8"(2337mm)	10'-0"(3048mm)	12'-4"(3759mm)
11 - 12		4'-1"(1245mm)	5'-5"(1651mm)	6'-10"(2083mm)	8'-11"(2718mm)	12'-4"(3759mm)
13 - 14		3'-9"(1143mm)	4'-11"(1499mm)	6'-3"(1905mm)	8'-2"(2489mm)	12'-4"(3759mm)
15 - 16			4'-3"(1295mm)	5'-5"(1651mm)	7'-1"(2159mm)	11'-0"(3353mm)
17			4'-0"(1219mm)	5'-1"(1549mm)	6'-8"(2032mm)	10'-5"(3175mm)

*REFER TO THE LAST DIGITS OF JOIST DESIGNATION CONNECTION TO JOIST MUST RESIST FORCES LISTED IN TABLE 104.5.1.

MIN. A307 BOLT REQ'D FOR CONNECTION		
SERIES	SECTION NUMBER*	A307 BOLT DIAMETER
LH/DLH	2 - 12	3/8" (9mm)
LH/DLH	13 - 17	1/2" (12mm)
DLH	18 & 19	5/8" (15mm)

*LAST TWO DIGITS OF JOIST DESIGNATION.

NOTES:1. Special designed LH and DLH can be supplied in longer lengths. See SLH Series Page 73.

2. Additional bridging may be required when joists support standing seam roof decks. The specifying professional should require that the joist manufacturer check the system and provide bridging as required to adequately brace the joists against lateral movement. For bridging requirements due to uplift pressures refer to sect. 104.12.

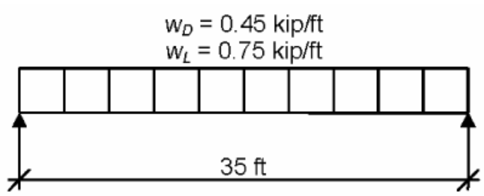
Examples:
Steel

Example 1 (AISC Design Examples vV13.0)

Example F.1-1a W-Shape Flexural Member Design in Strong-Axis Bending, Continuously Braced.

Given:

Select an ASTM A992 W-shape beam with a simple span of 35 feet. Limit the member to a maximum nominal depth of 18 in. Limit the live load deflection to $L/360$. The nominal loads are a uniform dead load of 0.45 kip/ft and a uniform live load of 0.75 kip/ft. Assume the beam is continuously braced.



Beam Loading & Bracing Diagram
(full lateral support)

Solution:

Material Properties:

ASTM A992 $F_y = 50$ ksi $F_u = 65$ ksi

Manual Table 2-3

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.450 \text{ kip/ft}) + 1.6(0.750 \text{ kip/ft})$ $= 1.74$ kip/ft $M_u = \dfrac{1.74 \text{ kip/ft}(35.0 \text{ ft})^2}{8} = 266$ kip-ft	$w_a = 0.450 \text{ kip/ft} + 0.750 \text{ kip/ft}$ $= 1.20$ kip/ft $M_a = \dfrac{1.20 \text{ kip/ft}(35.0 \text{ ft})^2}{8} = 184$ kip-ft

Calculate the required moment of inertia for live-load deflection criterion of L/360

$$\Delta_{max} = \frac{L}{360} = \frac{35.0 \text{ ft}(12 \text{ in./ft})}{360} = 1.17 \text{ in.}$$

Manual Table 3-23 Diagram 1

$$I_{x(reqd)} = \frac{5wl^4}{384E\Delta_{max}} = \frac{5(0.750 \text{ kip/ft})(35.0 \text{ ft})^4(12 \text{ in./ft})^3}{384\,(29{,}000 \text{ ksi})(1.17\text{in.})} = 748 \text{ in.}^4$$

Select a W*18×50 from Table 3-2*

Per the User Note in Section F2, the section is compact. Since the beam is continuously braced and compact, only the yielding limit state applies.

LRFD	ASD
$\phi_b M_n = \phi_b M_{px} = 379 \text{ kip-ft} > 266 \text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = \dfrac{M_{px}}{\Omega_b} = 252 \text{ kip-ft} > 184 \text{ kip-ft}$ **o.k.**

Manual Table 3-2

$I_x = 800 \text{ in.}^4 > 748 \text{ in.}^4$ **o.k.**

Manual Table 3-2

Example 1 (continued)

Table 3–2 (continued)
W Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	*BF*	*BF*	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
	in.^{3}	kip-ft	kip-ft	kip-ft	kip-ft	kips	kips	ft	ft	in.^{4}	kips	kips
		ASD	LRFD	ASD	LRFD	ASD	LRFD				ASD	LRFD
W21×48[f]	**107**	**265**	**398**	**162**	**244**	**9.78**	**14.7**	**6.09**	**16.6**	**959**	**144**	**217**
W16×57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212
W14×61	102	254	383	161	242	4.96	7.46	8.65	27.5	640	104	156
W18×50	101	252	379	155	233	8.69	13.1	5.83	17.0	800	128	192
W10×77	97.6	244	366	150	225	2.59	3.90	9.18	45.2	455	112	169
W12×65[f]	96.8	237	356	154	231	3.60	5.41	11.9	35.1	533	94.5	142
W21×44	**95.4**	**238**	**358**	**143**	**214**	**11.2**	**16.8**	**4.45**	**13.0**	**843**	**145**	**217**
W16×50	92.0	230	345	141	213	7.59	11.4	5.62	17.2	659	124	185
W18×46	90.7	226	340	138	207	9.71	14.6	4.56	13.7	712	130	195
W14×53	87.1	217	327	136	204	5.27	7.93	6.78	22.2	541	103	155
W12×58	86.4	216	324	136	205	3.76	5.66	8.87	29.9	475	87.8	132
W10×68	85.3	213	320	132	199	2.57	3.86	9.15	40.6	394	97.8	147
W16×45	82.3	205	309	127	191	7.16	10.8	5.55	16.5	586	111	167
W18×40	**78.4**	**196**	**294**	**119**	**180**	**8.86**	**13.3**	**4.49**	**13.1**	**612**	**113**	**169**
W14×48	78.4	196	294	123	184	5.10	7.66	6.75	21.1	484	93.8	141
W12×53	77.9	194	292	123	185	3.65	5.48	8.76	28.2	425	83.2	125
W10×60	74.6	186	280	116	175	2.53	3.80	9.08	36.6	341	85.8	129

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_v = 1.50	ϕ_v = 1.00

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.

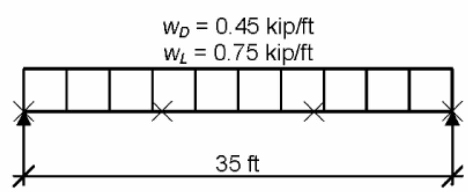
I required is in this grouping, with the W21x44 (bold) the most economical. But this section must be 18 inches maximum, and the W18 x 46 does not have enough (even though it has enough moment capacity of 340 k-ft ($\phi_b M_{px}$))

Look to the next section above for a W18 with I > 748 in^4.

Example F.1-2a W-Shape Flexural Member Design in Strong-Axis Bending, Braced at Third Points

Given:

Verify the strength of the W18×50 beam selected in Example F.1-1a if the beam is braced at the ends and third points rather than continuously braced.



Beam Loading & Bracing Diagram (bracing at ends and third points)

Solution:

Required flexural strength at midspan from Example F.1-1a

LRFD	ASD
M_u = 266 kip-ft	M_a = 184 kip-ft

Example 1 (continued)

$$L_b = \frac{35.0\text{ ft}}{3} = 11.7\text{ ft}$$

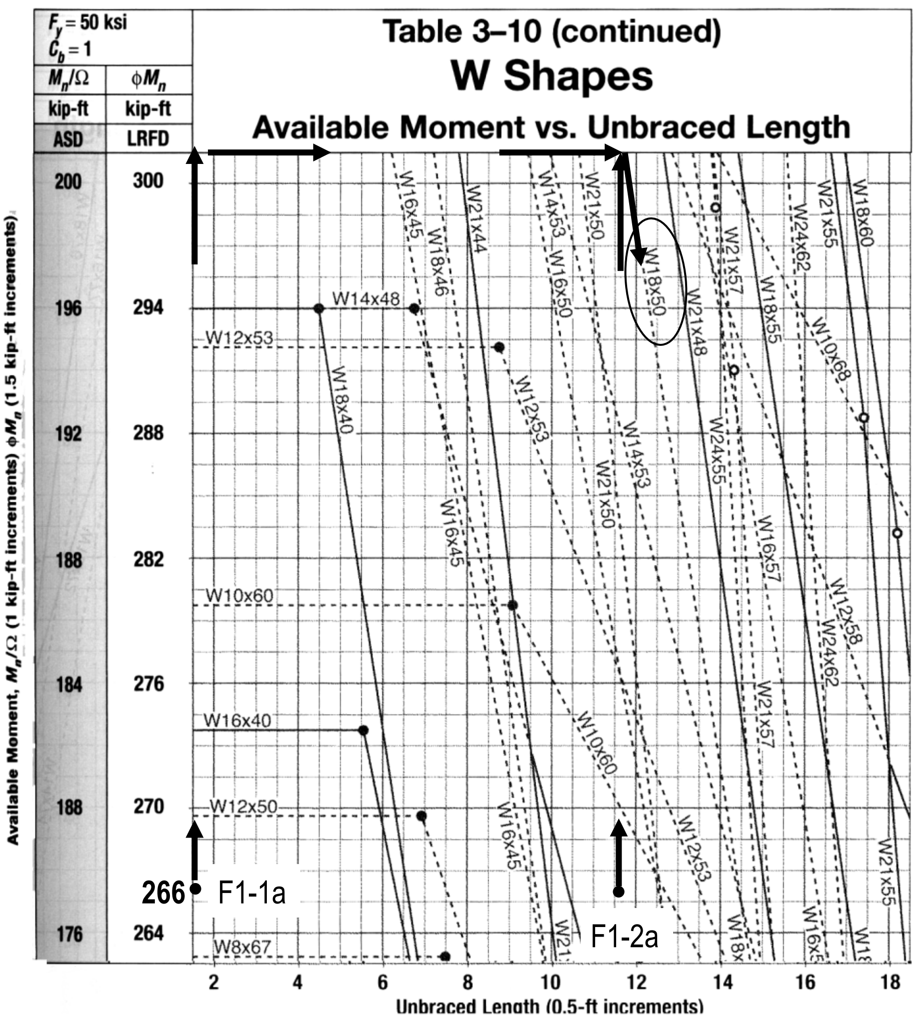
By inspection, the middle segment will govern. For a uniformly loaded beam braced at the ends and third points, C_b = 1.01 in the middle segment. Conservatively neglect this small adjustment in this case. Manual Table 3-1

Obtain the available strength from Table 3-10

Enter Table 3-10 and find the intersection of the curve for the W18×50 with an unbraced length of 11.7 ft. Obtain the available strength from the appropriate vertical scale to the left.

LRFD	ASD
$\phi_b M_n \approx 302$ kip-ft > 266 kip-ft **o.k.**	$\dfrac{M_n}{\Omega_b} \approx 201$ kip-ft > 184 kip-ft **o.k.**

Manual Table 3-10



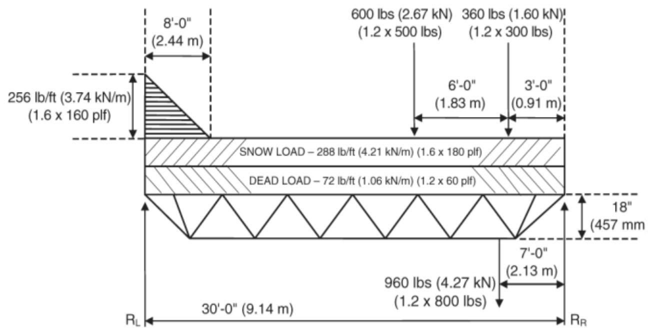
For F1-1a, the unbraced length is zero. There is no zero on the chart, so the far left is used starting at the moment required of 266 k-ft. When a W18 is not encountered with a greater moment capacity going up on the page, going to the right will intersect with a W18 line.

For F1-2a, the unbraced length is 11.7 ft. The same procedure applies, starting at a moment required of 266 k-ft. If no match is close to the

Example 2 (LRFD)

U.S. CUSTOMARY UNITS AND (METRIC UNITS)

Factored Load diagram per ASCE 7 2.3.2(3) 1.2D + 1.6S



18K SP

Joist manufacturer to design joist to support **factored loads** as shown.

Joist Supplier to design joist to support loads as shown above.

$$\text{Total Load} = \frac{256}{2}(8) + (288+72)30 + 600 + 960 + 360 = \mathbf{13{,}744\ lbs.}$$

$$R_L = \frac{256(8)}{2}\left[\frac{30-8/3}{30}\right] + \frac{(288+72)(30)}{2} + 600\left[\frac{9}{30}\right] + 960\left[\frac{7}{30}\right] + 660\left[\frac{3}{30}\right] =$$

$R_L = 6773$ lbs.
$R_R = 6971$ lbs.

$$\text{Assume } R_R = \frac{W_{e1}(L)}{2}, \quad W_{e1} = \frac{2(6971)}{30} = 465 \text{ lbs/ft}$$

Point of Max. Mom. = Point of Zero Shear(V) = L_1
(dist. from rt. end of Jst)
V = Zero = 6971 - (360+600+960) - (288+72)(L_1)
L_1 = **14.03 ft**.

$$M \ @\ L_1 = 6971(14.03) - 360(11.03) - 960(7.03) - 600(5.03) - \frac{(288+72)(14.03)^2}{2}$$

M = 48,634 ft. lbs.

$$\text{Assume } M = \frac{W_{e2}(L)^2}{8}, \quad W_{e2} = \frac{8(48.634)}{(30)^2} = 432.3 \text{ lbs./ft.}$$

Using W_{e1} = 465 LB/ft. @ SPAN = 30',
and D = 18"

Select 18K7 for total load (502) and live load (180) and call it:
18K9SP

(c) Special Considerations

The **specifying professional** shall indicate on the construction documents special considerations including:

a) Profiles for non-standard joist and Joist Girder configurations (Standard joist and Joist Girder configurations are as indicated in the Steel Joist Institute Standard Specifications Load Tables & Weight Tables of latest adoption).
b) Oversized or other non-standard web openings
c) Extended ends
d) Deflection criteria for live and total loads for non-SJI standard joists
e) Non-SJI standard bridging

LRFD

STANDARD LOAD TABLE FOR OPEN WEB STEEL JOISTS, K-SERIES
Based on a 50 ksi Maximum Yield Strength – Loads shown in Pounds per Linear Foot (plf)

Joist Designation	18K3	18K4	18K5	18K6	18K7	18K9	18K10
Depth (In.)	18	18	18	18	18	18	18
Approx. Wt. (lbs./ft.)	6.6	7.2	7.7	8.5	9	10.2	11.7
Span (ft.) ↓							
18	825 550	825 550	825 550	825 550	825 550	825 550	825 550
19	771 494	825 523	825 523	825 523	825 523	825 523	825 523
20	694 423	825 490	825 490	825 490	825 490	825 490	825 490
21	630 364	759 426	825 460	825 460	825 460	825 460	825 460
22	573 316	690 370	777 414	825 438	825 438	825 438	825 438
23	523 276	630 323	709 362	774 393	825 418	825 418	825 418
24	480 242	577 284	651 318	709 345	789 382	825 396	825 396
25	441 214	532 250	600 281	652 305	727 337	825 377	825 377
26	408 190	492 222	553 249	603 271	672 299	807 354	825 361
27	378 169	454 198	513 222	558 241	622 267	747 315	825 347
28	351 151	423 177	477 199	519 216	577 239	694 282	822 331
29	327 136	394 159	444 179	483 194	538 215	646 254	766 298
30	304 123	367 144	414 161	451 175	502 194	603 229	715 269
31	285 111	343 130	387 146	421 158	469 175	564 207	669 243

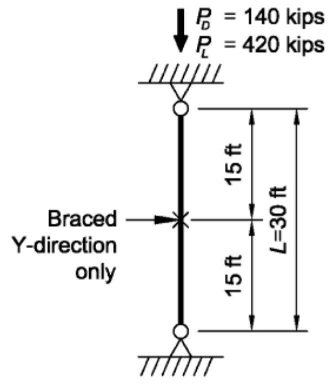
- Top values are total factored distributed load from strength and deflection criteria.
- Values below in gray are for live load deflection limit (unfactored).

Example 3 (AISC Design Examples vV13.0)

Example E.1b W-Shape Column Design with Intermediate Bracing

Given:

Redesign the column from Example E.1a assuming the column is laterally braced about the y-y axis and torsionally braced at the midpoint.



Solution:

Calculate the required strength

LRFD	ASD
P_u = 1.2(140 kips) + 1.6(420 kips) = 840 kips	P_a = 140 kips + 420 kips = 560 kips

Select a column using Manual Table 4-1.

For a pinned-pinned condition, K = 1.0 Commentary Table C-C2.2

Since the unbraced lengths differ in the two axes, select the member using the y-y axis then verify the strength in the x-x axis.

Enter Table 4-1 with a y-y axis effective length, KL_y, of 15 ft and proceed across the table until reaching a shape with an available strength that equals or exceeds the required strength. Try a W14×90. A 15 ft long W14×90 provides an available strength in the y-y direction of

LRFD	ASD
ϕP_n = 1000 kips	P_n/Ω = 667 kips

The r_x/r_y ratio for this column, shown at the bottom of Manual Table 4-1, is 1.66. The equivalent y-y axis effective length for strong axis buckling is computed as

$$KL = \frac{30.0\text{ ft}}{1.66} = 18\text{ ft}$$

From the table, the available strength of a W14×90 with an effective length of 18 ft is

LRFD	ASD
$\phi_c P_n$ = 928 kips > 840 kips **o.k.**	P_n/Ω_c = 618 kips > 560 kips **o.k.**

Manual Table 4-1

The available compression strength is governed by the *x-x* axis flexural buckling limit state.

Example 3 (continued)

F_y = 50 ksi

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

W14

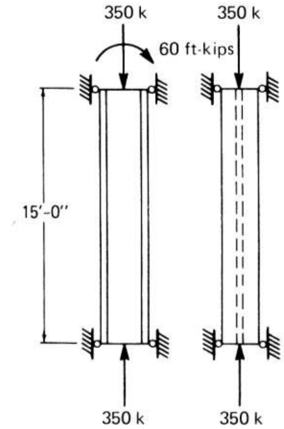
Shape	W14×											
Wt/ft	145		132		120		109		99		90	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y												
0	1280	1920	1160	1740	1060	1590	959	1440	872	1310	792	1190
6	1250	1870	1130	1700	1030	1550	934	1400	849	1280	771	1160
7	1240	1860	1120	1680	1020	1530	924	1390	840	1260	763	1150
8	1220	1840	1110	1660	1010	1510	914	1370	831	1250	754	1130
9	1210	1820	1090	1640	995	1500	902	1360	820	1230	745	1120
10	1200	1800	1080	1620	981	1470	889	1340	808	1210	734	1100
11	1180	1770	1060	1590	965	1450	875	1320	795	1200	722	1090
12	1160	1740	1040	1570	949	1430	860	1290	781	1170	709	1070
13	1140	1720	1020	1540	931	1400	844	1270	767	1150	696	1050
14	1120	1690	1000	1510	912	1370	827	1240	751	1130	682	1020
15	1100	1650	982	1480	893	1340	809	1220	734	1100	667	1000
16	1080	1620	959	1440	872	1310	790	1190	717	1080	651	978
17	1050	1580	936	1410	851	1280	771	1160	699	1050	635	954
18	1030	1550	912	1370	829	1250	751	1130	681	1020	618	928
19	1000	1510	887	1330	806	1210	730	1100	662	995	600	902
20	979	1470	862	1300	783	1180	709	1070	642	966	583	876
22	926	1390	809	1220	735	1100	665	1000	602	906	546	821
24	871	1310	756	1140	685	1030	620	932	562	844	509	765
26	815	1230	702	1050	636	956	575	864	520	782	471	708
28	759	1140	647	973	586	881	530	797	479	720	434	652
30	702	1060	594	892	537	807	485	730	438	659	397	596
32	647	972	541	814	489	735	442	664	399	599	361	542
34	592	890	491	738	443	666	400	601	360	542	326	490
36	540	811	441	663	398	598	359	540	323	486	292	439
38	489	734	396	595	357	537	322	484	290	436	262	394
40	441	663	358	537	322	484	291	437	262	393	236	355
Properties												
P_{wo} (kips)	191	287	175	263	151	227	128	191	111	167	95.9	144
P_{wi} (kips/in.)	22.7	34.0	21.5	32.3	19.7	29.5	17.5	26.3	16.2	24.3	14.7	22.0
P_{wb} (kips)	477	717	407	612	312	468	220	330	173	260	129	194
P_{fb} (kips)	222	334	199	298	165	249	138	208	114	171	94.3	142
L_p (ft)	14.1		13.3		13.2		13.2		13.5		15.2	
L_r (ft)	61.7		56.0		52.0		48.4		45.3		42.6	
A_g (in.^{2})	42.7		38.8		35.3		32.0		29.1		26.5	
I_x (in.^{4})	1710		1530		1380		1240		1110		999	
I_y (in.^{4})	677		548		495		447		402		362	
r_y (in.)	3.98		3.76		3.74		3.73		3.71		3.70	
Ratio r_x/r_y	1.59		1.67		1.67		1.67		1.66		1.66	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})	48900		43800		39500		35500		31800		28600	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})	19400		15700		14200		12800		11500		10400	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

American Institute of Steel Construction, Inc.

Example 4 (LRFD)

Investigate the accepatbility of a W16 x 67 used as a beam-column under the unfactored loading shown in the figure. It is A992 steel (F_y = 50 ksi). Assume 25% of the load is dead load with 75% live load.

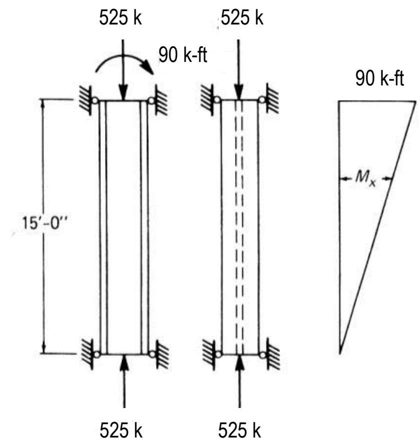


SOLUTION:

DESIGN LOADS (shown on figure):

Axial load = 1.2(0.25)(350k)+1.6(0.75)(350k)=525k

Moment at joint = 1.2(0.25)(60 $^{k\text{-}ft}$) + 1.6(0.75)(60 $^{k\text{-}ft}$) = 90 $^{k\text{-}ft}$



Determine column capacity and fraction to choose the appropriate interaction equation:

$$\frac{kL}{r_x} = \frac{15\,ft(12\,{}^{in}\!/_{ft})}{6.96in} = 25.9 \quad \text{and} \quad \frac{kL}{r_y} = \frac{15\,ft(12\,{}^{in}\!/_{ft})}{2.46in} = 73 \quad \text{(governs)}$$

$$P_c = \phi_c P_n = \phi_c F_{cr} A_g = (30.5ksi)19.7in^2 = 600.85k$$

$$\frac{P_r}{P_c} = \frac{525k}{600.85k} = 0.87 > 0.2 \quad \text{so use} \quad \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

There is no bending about the y axis, so that term will not have any values.

Determine the bending moment capacity in the x direction:

The unbraced length to use the full plastic moment (L_p) is listed as 8.69 ft, and we are over that so of we don't want to determine it from formula, we can find the beam in the Available Moment vs. Unbraced Length tables. The value of ϕM_n at L_b =15 ft is 422 k-ft.

Determine the magnification factor when M_1 = 0, M_2 = 90 k-ft:

$$C_m = 0.6 - 0.4\frac{M_1}{M_2} = 0.6 - \frac{0^{k-ft}}{90^{k-ft}} = 0.6 \le 1.0 \qquad P_{e1} = \frac{\pi^2 EA}{\left(Kl/r\right)^2} = \frac{\pi^2(30x10^3 ksi)19.7in^2}{(25.9)^2} = 8{,}695.4k$$

$$B_1 = \frac{C_m}{1-(P_u/P_{e1})} = \frac{0.6}{1-(525k/8695.4k)} = 0.64 \ge 1.0 \quad \text{USE 1.0} \qquad M_u = (1)90 \text{ k-ft}$$

Finally, determine the interaction value:

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) = 0.87 + \frac{8}{9}\left(\frac{90^{k-ft}}{422^{k-ft}}\right) = 1.06 \not\le 1.0$$

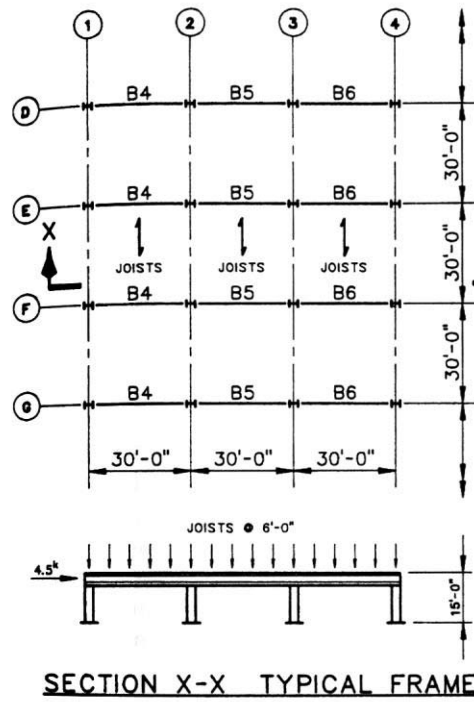
This is **NOT OK.** (and outside error tolerance). The section should be larger.

Case Study in Steel

adapted from Structural Design Guide, Hoffman, Gouwens, Gustafson & Rice., 2nd ed.

Building description

The building is a one-story steel structure, typical of an office building. The figure shows that it has three 30 ft. bays in the short direction and a large number of bays in the long direction. Some options for the structural system include fully restrained with rigid connections and fixed column bases, simple framing with "pinned" connections and column bases requiring bracing against sideway, and simple framing with continuous beams and shear connections, pinned column bases and bracing against sidesway. This last situation is the one we'll evaluate as shown in Figure 2.5(c).

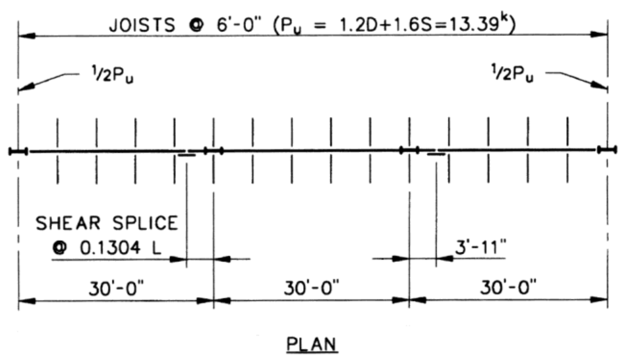


Loads

Live Loads:
Snow on Roof: 30 lb/ft^2 (1.44 kPa)

Wind: 20 lb/ft^2 (0.96 kPa)

Dead Loads:
Roofing: 8 lb/ft^2 (0.38 kPa)
Estimated decking: 3 lb/ft^2 (0.14 k
Ceiling: 7 lb/ft^2 (0.34 kPa)
Total: 18 lb/ft^2 (0.86 kPa)



Materials

A36 steel for the connection angles a
(F_y = 36 ksi, F_u = 58 ksi) and A992 s
for the beams and columns (F_y = 50
K series open web joists and roof de

Decking:

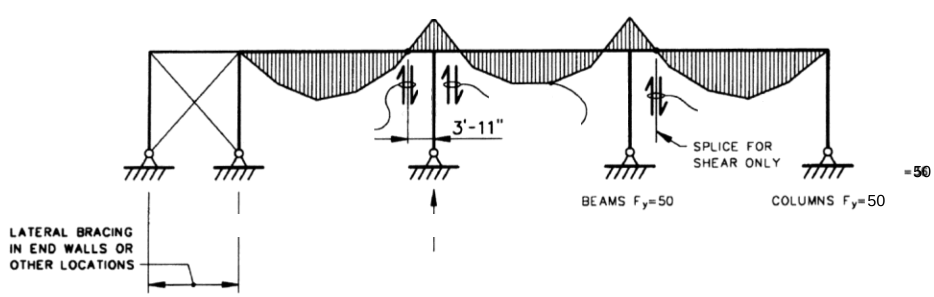


Figure 2.5(c) Type SF — cantilever-suspended span system, braced against sidesway

Decking selection is typically allowable stress design. Tables will give allowable total uniform load (taking self weight into account) based on stresses and deflection criteria for typical spans and how many spans are supported. The table (and description) for a Vulcraft 1.0 E deck is provided.

Areas in gray are governed by live load roof deflection.

The total load with snow and roofing = 30 psf + 8 psf = 38 psf.

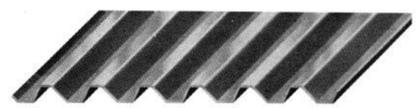
VERTICAL LOADS FOR TYPE 1.0E

No. of Spans	Deck Type	Max. SDI Const. Span	Allowable Total (Dead + Live) Uniform Load (PSF) Span (ft.-in.) C. to C. of Support										
			2'-6	3'-0	3'-6	4'-0	4'-6	5'-0	5'-6	6'-0	6'-6	7'-0	7'-6
1	E26	2'-10	178	107	71	51	39	31	26	22	20	18	16
	E24	3'-5	249	148	97	68	51	40	32	27	24	21	19
	E22	3'-10	316	187	122	85	63	48	39	32	27	24	21
	E20	4'-2	379	224	145	100	73	56	45	37	31	27	24
2	E26	3'-4	273	189	139	107	81	62	49	40	34	29	25
	E24	4'-0	396	275	202	153	111	83	65	52	43	37	32
	E22	4'-6	515	357	263	190	137	102	79	63	52	44	37
	E20	5'-0	634	440	323	227	162	121	94	74	61	51	43
3	E26	3'-4	310	198	128	89	66	51	40	33	28	25	22
	E24	4'-0	469	276	177	122	89	67	53	43	36	31	27
	E22	4'-6	588	344	221	151	109	82	64	52	43	36	31
	E20	5'-0	707	413	264	180	129	97	75	60	50	42	36

Notes: 1. Load tables are calculated using sectional properties based on the steel design thickness shown in the Steel Deck Institute (SDI) Design Manual.
2. Loads shown in the shaded areas are governed by the live load deflection not in excess of 1/240 of the span. A dead load of 10 PSF has been included.

1.0 E

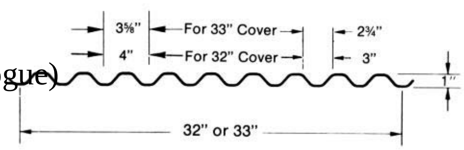
Maximum Sheet Length 42'-0
Extra Charge for Lengths Under 6'-0



Open Web Joists:

Open web joist selection is either based on allowable stress design or LRFD resistance for flexure (*not for deflection*). The total factored distributed load for joists at 6 ft on center will be:

w_{total} = (1.2×18lb/ft^2 + 1.6×30 lb/ft^2)(6 ft) + 1.2*(8 lb/ft estimated)*
= 427.2 lb/ft (with 1.2D + 1.6(L,or L_r, or S, or R) by catalogue)



w_{live} = 30 lb/ft^2(6 ft) = 180 lb/ft

LRFD

STANDARD LOAD TABLE FOR OPEN WEB STEEL JOISTS, K-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown in Pounds per Linear Foot (plf)

Joist Designation	18K3	18K4	18K5	18K6	18K7	18K9	18K10	20K3	20K4	20K5	20K6	20K7	20K9	20K10	22K4	22K5	22K6	22K7	22K9	22K10	22K11
Depth (In.)	18	18	18	18	18	18	18	20	20	20	20	20	20	20	22	22	22	22	22	22	22
Approx. Wt. (lbs./ft.)	6.6	7.2	7.7	8.5	9	10.2	11.7	6.7	7.6	8.2	8.9	9.3	10.8	12.2	8	8.8	9.2	9.7	11.3	12.6	13.8
Span (ft.) ↓																					
18	825 550	825 550	825 550	825 550	825 550	825 550	825 550														
19	771 494	825 523	825 523	825 523	825 523	825 523	825 523														
20	694 423	825 490	825 490	825 490	825 490	825 490	825 490	775 517	825 550	825 550	825 550	825 550	825 550	825 550							
21	630 364	759 426	825 460	825 460	825 460	825 460	825 460	702 453	825 520	825 520	825 520	825 520	825 520	825 520							
22	573 316	690 370	777 414	825 438	825 438	825 438	825 438	639 393	771 461	825 490	825 490	825 490	825 490	825 490	825 548	825 548	825 548	825 548	825 548	825 548	825 548
23	523 276	630 323	709 362	774 393	825 418	825 418	825 418	583 344	703 402	793 451	825 468	825 468	825 468	825 468	777 491	825 518	825 518	825 518	825 518	825 518	825 518
24	480 242	577 284	651 318	709 345	789 382	825 396	825 396	535 302	645 353	727 396	792 430	825 448	825 448	825 448	712 431	804 483	825 495	825 495	825 495	825 495	825 495
25	441 214	532 250	600 281	652 305	727 337	825 377	825 377	493 266	594 312	669 350	729 380	811 421	825 426	825 426	657 381	739 427	805 464	825 474	825 474	825 474	825 474
26	408 190	492 222	553 249	603 271	672 299	807 354	825 361	456 236	549 277	618 310	673 337	750 373	825 405	825 405	606 338	682 379	744 411	825 454	825 454	825 454	825 454
27	378 169	454 198	513 222	558 241	622 267	747 315	825 347	421 211	508 247	573 277	624 301	694 333	825 389	825 389	561 301	633 337	688 367	768 406	825 432	825 432	825 432
28	351 151	423 177	477 199	519 216	577 239	694 282	822 331	391 189	472 221	532 248	579 269	645 298	775 353	825 375	522 270	588 302	640 328	712 364	825 413	825 413	825 413
29	327 136	394 159	444 179	483 194	538 215	646 254	766 298	364 170	439 199	495 223	540 242	601 268	723 317	825 359	486 242	547 272	597 295	664 327	798 387	825 399	825 399
30	304 123	367 144	414 161	451 175	502 194	603 229	715 269	340 153	411 179	462 201	504 218	561 242	675 286	799 336	453 219	511 245	556 266	619 295	745 349	825 385	825 385
31	285 111	343 130	387 146	421 158	469 175	564 207	669 243	318 138	384 162	433 182	471 198	525 219	631 259	748 304	424 198	478 222	520 241	580 267	697 316	825 369	825 369

Deflection will limit the selection, and the most lightweight choice is the 22K4 which weighs approximately 8 lb/ft. Special provisions for bridging are required for the shaded area lengths and sections.

Continuous Beams:

LRFD design is required for the remaining structural steel for the combinations of load involving Dead, Snow and Wind. The bracing must be designed to resist the lateral wind load.

The load values are:

for D: $w_D = 18 \text{ lb/ft}^2 \cdot 30 \text{ ft} + (8 \text{ lb/ft} \cdot 30 \text{ ft})/ 6 \text{ ft} = 580 \text{ lb/ft}$

for S: $w_S = 30 \text{ lb/ft}^2 \cdot 30 \text{ ft} = 900 \text{ lb/ft}$

for W: $w_W = 20 \text{ lb/ft}^2 \cdot 30 \text{ ft} = 600 \text{ lb/ft}$ (up or down)

and laterally V = 600 lb/ft(15ft/2) = 4500 lb

These DO NOT consider self weight of the beam.

The applicable combinations for the tributary width of 30 ft. are:

1.4*D* $w_u = 1.4(580 \text{ lb/ft}) = 812 \text{ lb/ft}$

1.2*D* + 1.6*L* + 0.5(*L*

$1.2D + 1.6L + 0.5(L_r \textit{ or } S \textit{ or } R)$

$w_u = 1.2(580 \text{ lb/ft}) + 0.5(900 \text{ lb/ft}) = 1146 \text{ lb/ft}$

1.2D + 1.6(L_r or S or R) + (L or 0.8W)

$w_u = 1.2(580 \text{ lb/ft}) + 1.6(900 \text{ lb/ft}) + 0.8(600 \text{ lb/ft}) = \underline{2616 \text{ lb/ft}}$

1.2D + 1.6W + L + 0.5(L_r or S or R)

$w_u = 1.2(580 \text{ lb/ft}) + 1.6(600 \text{ lb/ft}) + 0.5(900 \text{ lb/ft}) = 2106 \text{ lb/ft}$

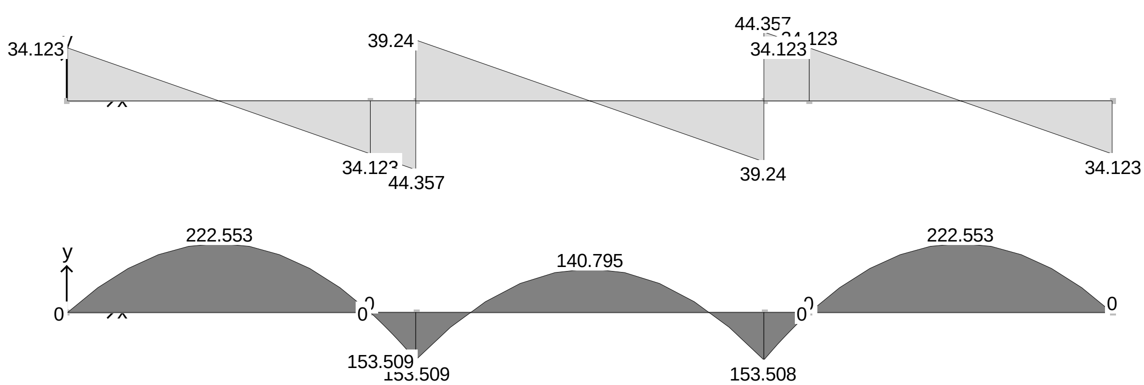
1.2D + 1.0E + L + 0.25S

$w_u = 1.2(580 \text{ lb/ft}) + 0.25(900 \text{ lb/ft}) = 921 \text{ lb/ft}$

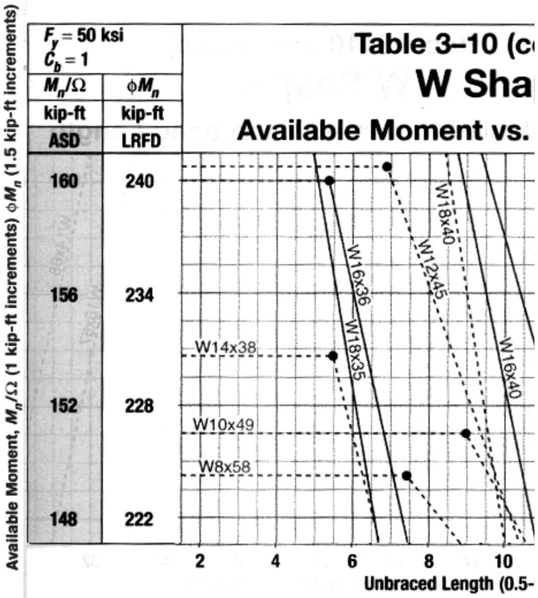
0.9D + 1.6W +1.6H $w_u = 0.9(580 \text{ lb/ft}) + 1.6(-600 \text{ lb/ft})$ *[uplift]* = - 438 lb/ft (up)

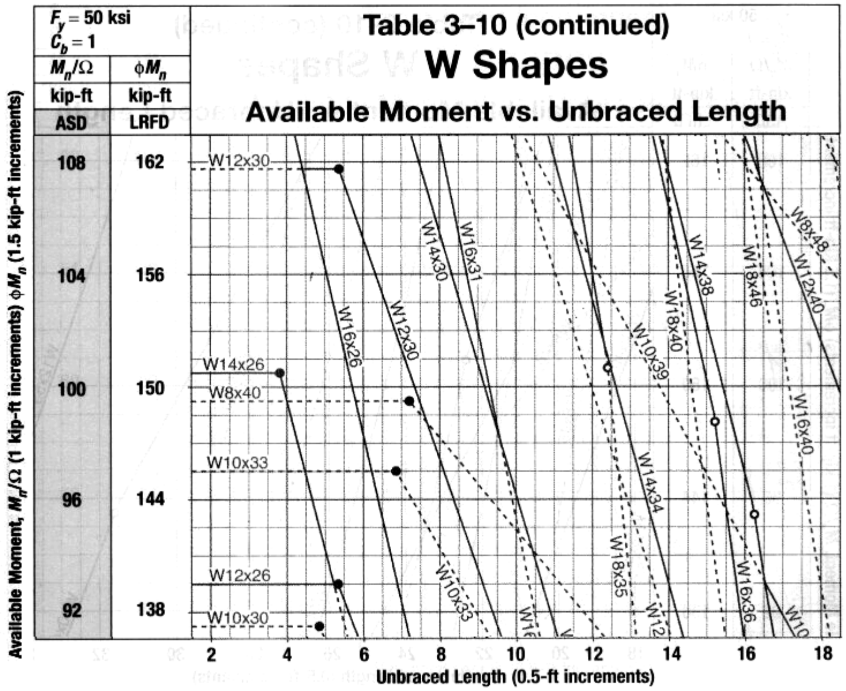
L, R, L_r , E & H don't exist for our case.

For the largest load case, the shear & bending moment diagrams are:

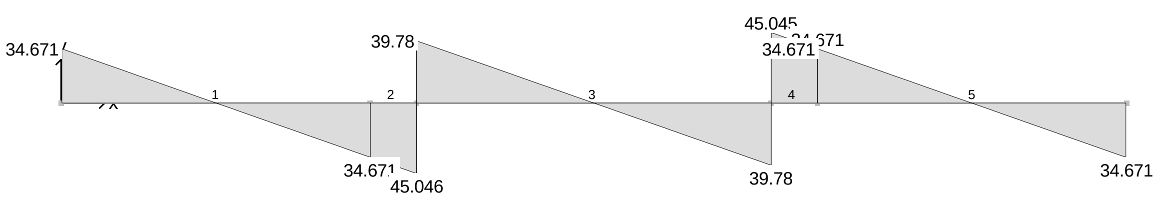


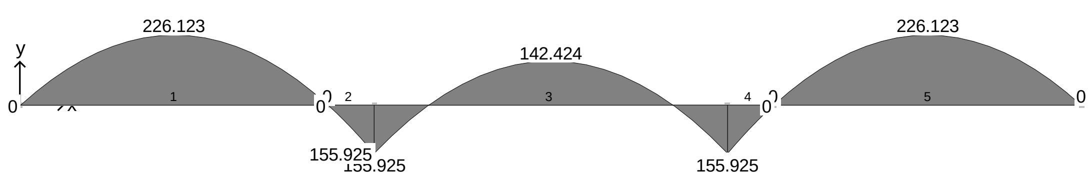
For the beams, we know that the maximum unbraced length is 6 ft. For the middle 6 feet of the end span, the moment is nearly uniform, so $C_b = 1$ is acceptable ($C_b = 1.08$ for constant moment). For the interior span, C_b is nearly 1 as well.





Choosing a W18x35 (M_u = 229 k-ft) for the end beams, and a W12x30 (M_u = 158 k-ft) for the interior beam, the self weight can be included in the total weight. The diagrams change to:





Check beam shear: $V_u \le \phi_v V_n = 1.0(0.6F_{yw}A_w)$

Exterior V_u = 34.67 k $\le$ 1.0(0.6)(50 ksi)(17.1 in.)(0.3 in.) = 153.9 k OK

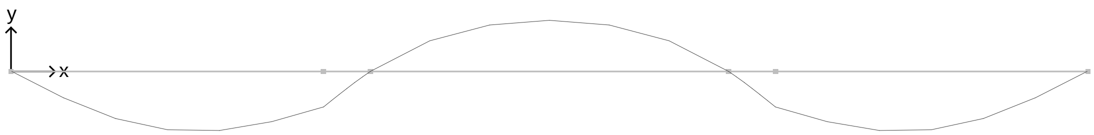
W18x35: d = 17.7 in., t_w = 0.3 in., I_x = 510 in.^{4}

Interior V_u = 45.05 k $\le$ 1.0(0.6)(50 ksi)(12.3 in.)(0.26 in.) = 95.94 k OK

W12x30: d = 12.3 in., t_w = 0.26 in., I_x = 238 in.^{4}

Check deflection (NO LOAD FACTORS) for total and live load (gravity and snow).

<u>Exterior Beam:</u> worst deflection is from no live load on the center span:



Maximum Δ_{total} = 2.20 in.

Is $\Delta_{total} \leq L/240$ = 360 in./240 = 1.5 in.? NO GOOD

We need an I ≥ (2.20in./1.5in.)(510 in.^{4}) = 748 in.^{4}

Maximum Δ_{live} = 1.86 in.

Is $\Delta_{live} \leq L/360$ = 360 in./360 = 1.0 in.? NO GOOD

We need an I ≥ (1.86in./1.0in.)(510 in.^{4}) = 949 in.^{4}

The W21x48 looks promising, but it has a note that it exceeds the compact limit for flexure.

Choose a W21 x 50 (I_x = 984 in.^{4}) (because the W21x48 would require extra work!)

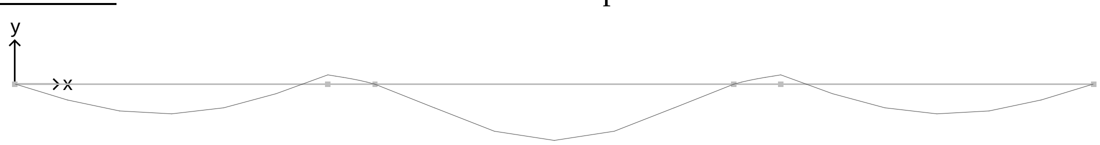
Now, Δ_{live} = 1.07 in., which is reasonable close.

F_y = 50 ksi **Table 3–2 (continued) W Shapes Selection by Z_x** Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF		L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W21×55	**126**	**314**	**473**	**192**	**289**	**10.8**	**16.3**	**6.11**	**17.4**	**1140**	**156**	**234**
W14×74	126	314	473	196	294	5.34	8.03	8.76	31.0	795	128	191
W18×60	123	307	461	189	284	9.64	14.5	5.93	18.2	984	151	227
W12×79	119	297	446	187	281	3.77	5.67	10.8	39.9	662	116	175
W14×68	115	287	431	180	270	5.20	7.81	8.69	29.3	722	117	175
W10×88	113	282	424	172	259	2.63	3.95	9.29	51.1	534	131	197
W18×55	**112**	**279**	**420**	**172**	**258**	**9.26**	**13.9**	**5.90**	**17.5**	**890**	**141**	**212**
W21×50	**110**	**274**	**413**	**165**	**248**	**12.2**	**18.3**	**4.59**	**13.6**	**984**	**158**	**237**
W12×72	108	269	405	170	256	3.72	5.59	10.7	37.4	597	105	158
W21×48[f]	**107**	**265**	**398**	**162**	**244**	**9.78**	**14.7**	**6.09**	**16.6**	**959**	**144**	**217**
W16×57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212
W14×61	102	254	383	161	242	4.96	7.46	8.65	27.5	640	104	156
W18×50	101	252	379	155	233	8.69	13.1	5.83	17.0	800	128	192
W10×77	97.6	244	366	150	225	2.59	3.90	9.18	45.2	455	112	169
W12×65[f]	96.8	237	356	154	231	3.60	5.41	11.9	35.1	533	94.5	142

ASD	LRFD	
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$	[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.

<u>Interior Beam:</u> worst deflection is from load on all spans:



Maximum Δ_{total} (at midspan) = 1.31 in.

Is $\Delta_{total} \leq L/240$ = 360 in./240 = 1.5 in.? OK

Maximum Δ_{live} (at midspan) = 0.94 in.

Is $\Delta_{live} \leq L/360$ = 360 in./360 = 1.0 in.? OK

Columns:

The load in the interior columns: P_u = 85 k (sum of the shears). This column will see minimal eccentricity from the difference in shear and half the column depth as the moment arm.

The load in the exterior columns: P_u = 35 k. These columns will see some eccentricity from the beam shear connections. We can determine this by using half the column depth as the eccentricity distance.

The effective length of the columns is 15 ft (no intermediate bracing). Table 4-1 shows design strength in kips for W8 shapes (the smallest). The lightest section at 15 feet has a capacity of 230 k; much greater than what we need even with eccentricity.

Table 4-1 (continued)
Available Strength in Axial Compression, kips
F_y = 50 ksi W Shapes
W8

Shape	W8×			
Wt/ft	35		31	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length KL (ft) with respect to least radius of gyration r_y: 0	308	463	273	411
6	281	423	249	374
7	272	409	241	362
8	262	394	232	348
9	251	377	222	333
10	239	359	211	317
11	226	340	200	301
12	213	321	189	283
13	200	301	177	266
14	187	281	165	248
15	174	261	153	230
16	160	241	141	212
17	147	221	130	195
18	135	203	118	178
19	123	184	108	162
20	111	166	97.2	146
22	91.5	138	80.3	121
24	76.9	116	67.5	101
26	65.5	98.5	57.5	86.5
28	56.5	84.9	49.6	74.5
30	49.2	74.0	43.2	64.9
32	43.3	65.0	38.0	57.1
34				

The exterior column connection moment (unmagnified) when the W8x31 depth = 8.0in

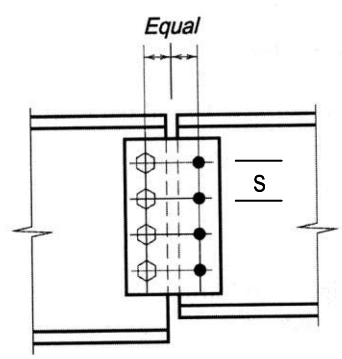
$$(35k)(8.0in/2)(\frac{1ft}{12in}) = = 11.7 \text{ k-ft.}$$

The capacity of a W8x31 with an unbraced length of 15 ft (from another beam chart) = 114 $^{\text{k-ft}}$.

For $\frac{P_r}{P_c} < 0.2$: $\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$

$$\frac{35k}{230k} = 0.15 < 0.2: \qquad \frac{35k}{2(230k)} + \left(\frac{11.7^{k-ft}}{114^{k-ft}}\right) = 0.179 \leq 1.0$$

so OK for eccentric loading of the beam-column (but we knew that).



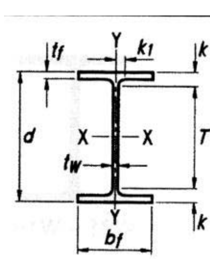
Beam Shear Splice Connection:

For this all-bolted single-plate shear splice, R_u = 35 k

W21x50: d = 20.8 in., t_w = 0.38 in.

W12x30: d = 12.3 in., t_w = 0.26 in.

The plate material is A36 with F_y = 36 ksi and F_u = 58 ksi. We need to check that we can fit a plate within the fillets and provide enough distance from the last holes to the edge.



For the W12x30, T = 10.125 in., which limits the plate height.

For a plate, s (hole spacing) = 3" and minimum edge distance is 1¼".

For ¾ in. diameter A325-N bolts and standard holes without a concern for deformation of the holes, the capacity per bolt is:

shear: $: R_u \le \phi_v R_n \quad \phi = 0.75,\ R_n = F_n A_b$, where F_n = 54 ksi

$$35k \le n(0.75)(54ksi)[\frac{\pi(0.75in)^2}{4}]$$

so n ≥ 1.96. Use 2 bolts (1@3 in. + 2@1.25 ≈ 5.5 in. < 10.125 in.)

bearing for 2 rows of bolts: depends on thickness of thinnest web (t=0.26 in.) and the connected material

$$R_u \le \phi R_n \quad \phi = 0.75,\ R_n = 1.5 L_c t F_u \le 3.0 dt F_u$$

L_c = 1.75 in. from the vertical edge of the beam to the edge of a hole

$$35k \le 2^{bolts}[0.75(1.5)(1.75in)(0.26in)(65ksi) = 38.0 \text{ k}$$
$$\le 2^{bolts}[0.75(3)(0.75in)(0.26in)(65ksi) = 57.0 \text{ k OK}$$

If the spacing between the holes across the splice is 4 in., the eccentricity, e_x, is 2 inches. We need to find C, which represents the number of bolts that are effective in resisting the eccentric shear force.

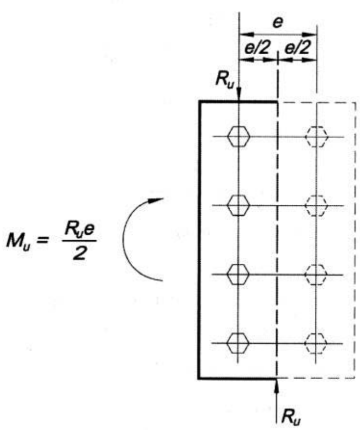


Fig. 10–22. Eccentricity in a symmetrical shear splice.

Table 7-7
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

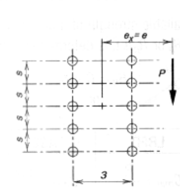
Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$R_n = C \times r_n$

or

LRFD	ASD
$C_{min} = \frac{P_u}{\phi r_n}$	$C_{min} = \frac{\Omega P_a}{r_n}$

where
P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
	2	0.84	2.54	4.48	6.59	8.72	10.8	12.9	15.0	17.0	19.0	21.0	23.0
	3	0.65	2.03	3.68	5.67	7.77	9.91	12.1	14.2	16.3	18.3	20.4	22.5
	4	0.54	1.67	3.06	4.86	6.84	8.93	11.1	13.2	15.4	17.5	19.6	21.7
	5	0.45	1.42	2.59	4.21	6.01	8.00	10.1	12.2	14.4	16.5	18.7	20.8
	6	0.39	1.22	2.25	3.69	5.32	7.17	9.16	11.2	13.4	15.5	17.7	19.8
	7	0.35	1.08	1.99	3.27	4.74	6.46	8.33	10.3	12.4	14.5	16.7	18.8
	8	0.31	0.96	1.78	2.93	4.27	5.86	7.60	9.50	11.5	13.6	15.7	17.8
	9	0.28	0.86	1.60	2.65	3.87	5.34	6.97	8.75	10.7	12.7	14.7	16.8
	10	0.26	0.78	1.46	2.42	3.53	4.90	6.42	8.10	9.91	11.8	13.8	15.9
3	12	0.22	0.66	1.24	2.06	3.01	4.19	5.51	7.01	8.63	10.4	12.2	14.2

r_n is the nominal shear per bolt:

$$C_{min} = \frac{35k}{0.75(54ksi)(0.75in)^2 \pi /4} = 1.95 \quad \text{(which we found as } n\text{)}$$

C off the table is 2.54 bolts which is more than the minimum of 1.95 (which is why we have 2). OK.

If the plate is 3/8 in. thick x 8 in. wide x 9 in. tall, check *bolt bearing on plate:*

$\phi R_n = 2.4 dt F_u$ (per bolt)

2 bolts[2.4(0.75 in.)(0.375 in.)(58 ksi) = 78.3 k > 35 k OK

Check *flexure of the plate:*

design moment: $M_u = \frac{R_u e}{2} = \frac{35k \times 4in}{2}$ = 70.0 k-in

yielding capacity: $\phi M_n = \phi F_y S_x$ $\phi = 0.9$ (5.5 in. tall section, 3/8 in. thick)

$0.9(36ksi)\left[\frac{0.375in(5.5in)^2}{6}\right]$ = 61.25 k-in > 70.0 k-in NOT OK

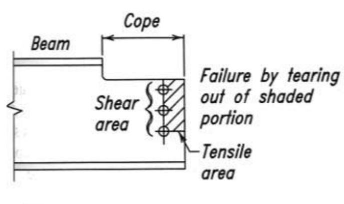
with 6 in. tall, $\phi M_n = 72.9$ k-in

rupture $\phi M_n = \phi F_u S_{net}$ $\phi = 0.75$

$S_{net} = {}^{I_{net}}/_{c}$ and can be looked up or calculated = 1.74 in^3

$0.75(58ksi)(1.74in^3)$ = 75.7 k-in > 70.0 k-in OK

Check *shear yielding of the plate:* $R_u \le \phi R_n$ $\phi = 1.00$ $R_n = 0.6F_y A_g$



(1.00)[0.6(36 ksi)(6 in.)(0.375 in.)] = 48.6 k > 35 k OK

Check *shear rupture of the plate:* $R_u \le \phi R_n$ $\phi = 0.75$ $R_n = 0.6F_u A_{nv}$

for ¾" diameter bolts, the effective hole width is (0.75 + 1/8) = 0.875 in.:

(0.75)[0.6(58 ksi)(6 in. – 2 x 0.875 in.)(0.375 in.)] = 41.6 k > 35 k OK

Check *block shear rupture of the plate:* $R_u \le \phi R_n$ $\phi = 0.75$

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$$

with U_{bs} = 0.5 when the tensile stress is non-uniform. (The tensile stress switches direction across the splice.) *(and assuming 2 in. of width to the center of the bolt hole)*

$$R_n = 0.60(58ksi)(0.375in)[1.5in + 3in - 1.5^{holes}(0.875)] +$$
$$0.5(58ksi)(0.375in)(2in - {}^{0.875in}/_{2}) = 58.6.9k$$
$$\le 0.6(36ksi)(0.375in)(1.5in + 3in) + 0.5(36ksi)(0.375in)(2in - {}^{0.875in}/_{2}) = 47.0k$$

35 k < 0.75(47.0 k) = 35.2 k OK

Column Base Plate:

Column base plates are designed for bearing on the concrete (concrete capacity) and plastic hinge development from flexure because the column "punches" down the plate and it could bend upward near the edges of the column (shown as *0.8b$_f$* and *0.95d*). The plate dimensions are B and N. The concrete has a compressive strength, *f'_c = 3 ksi.*

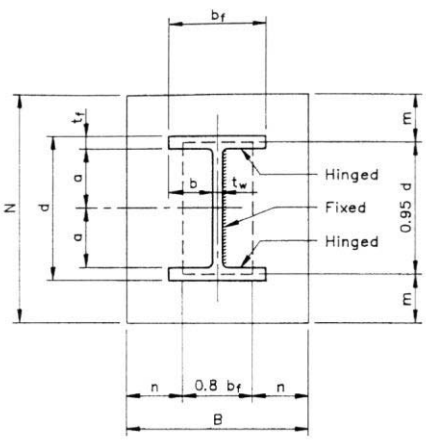


Figure 5.6. Column base plate dimensions

For W8 x 31: d = 8.0 in., b_f = 8.0 in., and if we provide width to put in bolt holes, we could use a 12 in. by12 in. plate (allowing about 2 inches each side). We will look at the interior column load of 85 k.

minimum thickness: $t_{min} = l\sqrt{\dfrac{2P_u}{0.9F_yBN}}$

where l is the larger of m, n and $\lambda n'$

$m = (N - 0.95d)/2$ = (12 in. – 0.95 x 8.0 in.)/2 = 2.2 in.

$n = (B - 0.8b_f)/2$ = (12 in. – 0.8 x 8.0 in.)/2 = 2.8 in.

$$n' = \frac{\sqrt{db_f}}{4} = \frac{\sqrt{8.0in \cdot 8.0in}}{4} = 2.0 \text{ in.}$$

λ is derived from a term X which takes the bounding area of the column, the perimeter, the axial force, and the concrete compressive strength into account:

$$X = \frac{4db_f}{(d+b_f)^2}\cdot\frac{P_u}{\phi_c P_p} = \frac{4db_f}{(d+b_f)^2}\cdot\frac{P_u}{\phi_c(0.85f'_c)BN} = \frac{4\cdot 8.0in\cdot 8.0in}{(8.0in+8.0in)^2}\cdot\frac{85k}{0.6(0.85\cdot 3ksi)12in\cdot 12in}$$

$$= 0.386$$

$$\lambda = \frac{2\sqrt{X}}{(1+\sqrt{1-X})} \le 1 \qquad = \frac{2\sqrt{0.386}}{(1+\sqrt{1-0.386})} = 0.697 \text{ so } \lambda n' = (0.697)(2.0 \text{ in.}) = 1.39 \text{ in.}$$

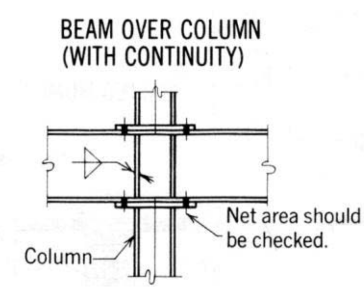
$$t_p = l\sqrt{\frac{2P_u}{0.9F_yBN}} = (2.8in)\sqrt{\frac{2\cdot 85k}{0.9(36ksi)(12in)(12in)}} = 0.534 \text{ in.}$$

Use a 9/16 in. thick plate.

The anchor bolts must also be able to resist lateral shear. There also is friction between the steel and concrete to help. The International Building Code provided specifications for minimum edge distances and anchorage.

Continuous Beam Over Interior Column:

The design for this connection will involve a bearing plate at the top of the column, with a minimum number of bolts through the beam flanges to the plate. Because there will be high local compression, stiffener plates for the web will need to be added (refer to a plate girder design). Flexure with a reduced cross section area of the flanges should be checked.



Masonry Design

Notation:

A = name for area
A_n = net area, equal to the gross area subtracting any reinforcement
A_{nv} = net shear area of masonry
A_s = area of steel reinforcement in masonry design
A_{st} = area of steel reinforcement in masonry column design
A_v = area of concrete shear stirrup reinforcement
ACI = American Concrete Institute
$ASCE$ = American Society of Civil Engineers
b = width, often cross-sectional
= total width of material at a horizontal section
C_m = compression force in the masonry for masonry design
CMU = shorthand for concrete masonry unit
d = effective depth from the top of a reinforced masonry beam to the centroid of the tensile steel
D = shorthand for dead load
e = eccentric distance of application of a force (P) from the centroid of a cross section
E = shorthand for earthquake load
E_m = modulus of elasticity of masonry
E_s = modulus of elasticity of steel
f_a = axial stress
f_b = bending stress
f_m = calculated compressive stress in masonry
f'_m = masonry design compressive stress
f_s = stress in the steel reinforcement for masonry design
f_v = shear stress
F_a = allowable axial stress
F_b = allowable bending stress
F_s = allowable tensile stress in reinforcement for masonry design
F_t = allowable tensile stress
F_v = allowable shear stress
F_{vm} = allowable shear stress of the masonry
F_{vs} = allowable shear stress of the shear reinforcement
h = name for height
= effective height of a wall or column
I_n = moment of inertia of the net section
j = multiplier by effective depth of masonry section for moment arm, jd
k = multiplier by effective depth of masonry section for neutral axis, kd
K = type of masonry mortar
L = shorthand for live load
M = internal bending moment
= type of masonry mortar
M_m = moment capacity of a reinforced masonry beam governed by steel stress
M_s = moment capacity of a reinforced masonry beam governed by masonry stress
$MSJC$ = Masonry Structural Joint Council
n = modulus of elasticity transformation coefficient for steel to masonry
$n.a.$ = shorthand for neutral axis (N.A.)
N = type of masonry mortar
$NCMA$ = National Concrete Masonry Association
O = type of masonry mortar
P = name for axial force vector
P_a = allowable axial load in columns
P_e = critical (Euler) buckling load
Q = first area moment about a neutral axis
r = radius of gyration
s = spacing of stirrups in reinforced masonry
S = type of masonry mortar
= section modulus
t = name for thickness
T_s = tension force in the steel reinforcement for masonry design
TMS = The Masonry Society
V = internal shear force
W = shorthand for wind load

β_1 = coefficient for determining stress block height, *c,* in masonry LRFD design
ε_m = strain in the masonry
ε_s = strain in the steel
ρ = reinforcement ratio in masonry design
ρ_b = balanced reinforcement ratio in masonry design
Σ = summation symbol

Masonry Design

Structural design standards for reinforced masonry are established by the *Masonry Standards Joint Committee* consisting of ACI, ASCE and The Masonry Society (TMS), and presents allowable stress design as well as limit state (strength) design.

Materials

Designation	strength range
M	2500 psi
S	1800 psi
N	750 psi
O	350 psi
K	75 psi

Masonry mortars are mixtures of water, masonry cement, lime, and sand. The strengths are categorized by letter designations (from MaSoNwOrK).

f'_m = masonry prism test compressive strength

Deformed reinforcing bars come in grades 40, 50 & 60 (for 40 ksi, 50 ksi and 60 ksi yield strengths). Sizes are given nominally as # of 1/8".

Clay and concrete masonry units are porous, and their durability with respect to weathering is an important consideration. The amount of water in the mortar is important as well as the absorption capacity of the units for good *bond*; both for strength and for weatherproofing. Because of the moisture and tendency for shrinkage and swelling, it is critical to provide control joints for expansion and contraction.

Masonry Walls

Masonry walls can be reinforced or unreinforced, grouted or ungrouted, single wythe or cavity, prestressed or not. Cavity walls will require ties to force the two walls separated by the cavity to act as one.

From centuries of practice, the height to thickness ratio is limited because of slenderness (h/t < 25 or 35 depending on code). Most walls will see bending from wind or eccentricity along with bearing (combined stresses).

Allowable Stresses

- If tension stresses result, the allowable tensile strength for unreinforced walls must not be exceeded. These are relatively low (40 – 70 psi) and are shown in Table 2.2.3.2.
- If compression stresses result, the allowable strength (in bending) for unreinforced masonry $F_b = 1/3\, f'_m$
- If compression stresses result, the allowable strength (in bending) for reinforced masonry $F_b = 0.45\, f'_m$
- Shear stress in unreinforced masonry cannot exceed $F_v = 1.5\sqrt{f'_m} \leq 120$ psi.
- Shear stress in reinforced masonry for $M/(Vd) \leq 0.25$ cannot exceed $F_v = 3.0\sqrt{f'_m}$
- Shear stress in reinforced masonry for $M/(Vd) \geq 1.0$ cannot exceed $F_v = 2.0\sqrt{f'_m}$
- Allowable tensile stress, F_s, in grades 40 & 50 steel is 20 ksi, grade 60 is 32 ksi, and wire joint reinforcement is 30 ksi..

where f'_m = specified compressive strength of masonry

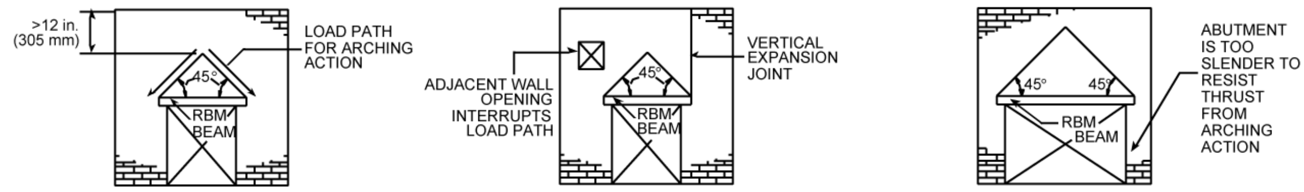
Table 2.2.3.2 — Allowable flexural tensile stresses for clay and concrete masonry, psi (kPa) (F_t)

Direction of flexural tensile stress and masonry type	Mortar types: Portland cement/lime or mortar cement (PCL)		Mortar types: Masonry cement or air entrained portland cement/lime	
	M or S	N	M or S	N
Normal to bed joints				
Solid units	53 (366)	40 (276)	32 (221)	20 (138)
Hollow units[1]				
Ungrouted	33 (228)	25 (172)	20 (138)	12 (83)
Fully grouted	86 (593)	84 (579)	81 (559)	77 (531)
Parallel to bed joints in running bond				
Solid units	106 (731)	80 (552)	64 (441)	40 (276)
Hollow units				
Ungrouted and partially grouted	66 (455)	50 (345)	40 (276)	25 (172)
Fully grouted	106 (731)	80 (552)	64 (441)	40 (276)
Parallel to bed joints in masonry not laid in running bond				
Continuous grout section parallel to bed joints	133 (917)	133 (917)	133 (917)	133 (917)
Other	0 (0)	0 (0)	0 (0)	0 (0)

1 For partially grouted masonry, allowable stresses shall be determined on the basis of linear interpolation between fully grouted hollow units and ungrouted hollow units based on amount (percentage) of grouting.

Loads on Lintels in Masonry Walls

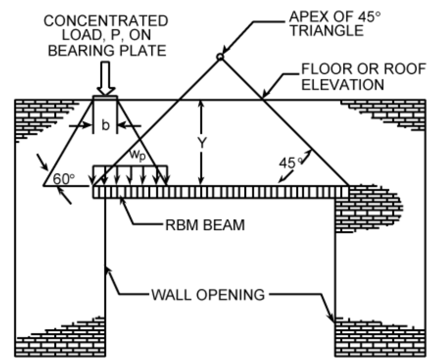
Arching action is present in masonry walls when there is an opening and sufficient wall width on either side of the opening to resist the arch thrust. A lintel is required to support the weight of the wall material above the opening. When arching action is present, the weight that must be supported can be determined from a 45 degree angle. This area may be a triangle, or trapezoid if the wall height above the lintel is less than half the opening width. The distributed load is calculated as height x wall thickness x specific weight of the masonry.



(a) ARCHING ACTION WILL OCCUR.
DESIGN BEAM FOR TRAINGULAR LOADING.

(b) ARCHING ACTION WILL NOT OCCUR.
DESIGN BEAM FOR ENTIRE WALL AREA.

When there are concentrated loads on the wall, the load can be distributed to a width at the lintel height based on a 60 degree angle.



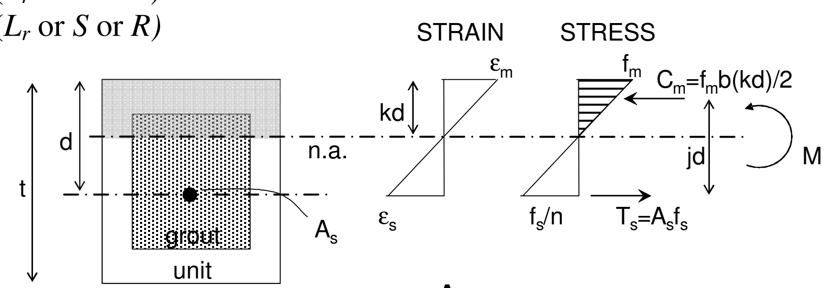
Reinforced Masonry Members

For stress analysis in masonry flexural members

- the strain is linear
- the compressive stress in the masonry is linear
- the tensile stress in the steel is *not at yield*
- any masonry in tension is assumed to have no strength
- the steel can be in tension, and is placed in the bottom of a beam that has positive bending moment

Load Combinations

D
$D+L$
$D + 0.75(L_r \text{ or } S \text{ or } R)$
$D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
$D + (0.6W \text{ or } 0.7E)$
$D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$
$D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$
$0.6D + 0.6W$
$0.6D + 0.7E$



BIA Teknote 17 series

Internal Equilibrium

$$\rho = \frac{A_s}{bd} \qquad \Sigma F = 0: \quad A_s f_s = f_m b \frac{kd}{2}$$

C_m = compression in masonry = stress x area = $f_m \dfrac{b(kd)}{2}$

T_s = tension in steel = stress x area = $A_s f_s$

$C_m = T_s$ and •$M_m = T_s(d\text{-}kd/3) = T_s(jd)$ and $M_s = C_m(jd)$

where f_m = stress in mortar at extreme fiber
kd = height to neutral axis
b = width of section
f_s = stress in steel at d
A_s = area of steel reinforcement
d = depth to n.a. of reinforcement
j = (1 –k/3)

For flexure design:

$M \leq M_m$ *or* M_s
so, $M_m = T(jd) = 0.5 f_m bd^2 jk$ *and* $M_s = C(jd) = \rho bd^2 jf_s$

The design is adequate when $f_b \leq F_b$ in the masonry and $f_s \leq F_s$ in the steel.

Shear Strength

Shear stress is determined by $f_v = V/A_{nv}$ where A_{nv} is net shear area. Shear strength is determined from the shear capacity of the masonry and the stirrups: $F_v = F_{vm} + F_{vs}$. Stirrup spacings are limited to d/2 but not to exceed 48 in.

where:

$$F_{vm} = \frac{1}{2}\left[\left(4.0 - 1.75\left(\frac{M}{Vd}\right)\right)\sqrt{f'_m}\right] + 0.25\frac{P}{A_n}$$ where M/(Vd) is positive and cannot exceed 1.0

$$F_{vs} = 0.5\left(\frac{A_v F_s d}{A_{nv} s}\right)$$

($F_v = 3.0\sqrt{f'_m}$ when M/(Vd) ≥ 0.25)
($F_v = 2.0\sqrt{f'_m}$ when M(Vd) ≥ 1.0.) Values can be linearly interpolated.

Table B.2 BALANCED SECTION PROPERTIES FOR RECTANGULAR MASONRY SECTIONS WITH TENSION REINFORCEMENT

Reinforcement	f_m (psi)	Modular Ratio $n = E_s/E_m$	$F_b = f_m/3$ (psi)	Balanced Section Properties: k	j	K	$p = A_s/bd$
	With Special Inspection—Full Code Values						
Grade 40 F_y = 40 ksi	1350	22	450	0.333	0.889	66.6	0.00375
	1500	20	500	0.333	0.889	74.0	0.00416
	2000	15	667	0.333	0.889	89.7	0.00556
	4000	7.5	1333	0.333	0.889	197.0	0.01111
Grade 60 F_y = 60 ksi	1350	22	450	0.273	0.909	55.8	0.00256
	1500	20	500	0.273	0.909	62.0	0.00284
	2000	15	667	0.273	0.909	82.7	0.00379
	4000	7.5	1333	0.273	0.909	165.4	0.00758

Reinforcement Ratio

The amount of steel reinforcement is *limited*. Too much reinforcement, or *over-reinforced* will not allow the steel to yield before the concrete crushes and there is a sudden failure. A beam with the proper amount of steel to allow it to yield at failure is said to be *under reinforced*.

The reinforcement ratio is a fraction: $\rho = \frac{A_s}{bd}$ and must be less than ρ_b where the balanced reinforcement ratio is a function of steel strength and masonry strength.

Flexure Design of Reinforcement

One method is to choose a reinforcement ratio, find steel area, check stresses and moment:

1. find ρ_b and assume a value of $\rho < \rho_b$
2. find k, j and calculate $bd^2 = \dfrac{M}{\rho j F_s}$ where F_s is allowed stress in steel.

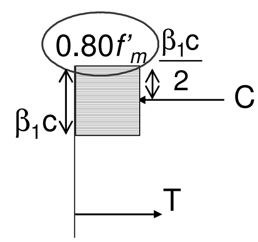
   Choose nice b & d values.
3. find $A_s = \dfrac{M}{F_s jd}$

3. check design for $M < M_s = A_S F_S\ (jd)$

4. check masonry flexural stress against allowable: $f_m = \dfrac{M}{0.5bd^2 jk} < F_b$

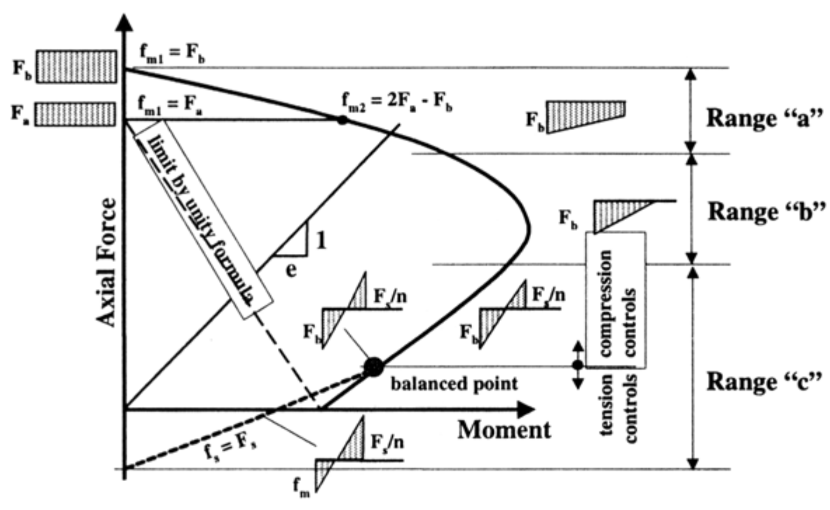
<u>Load and Resistance Factor Design</u>

The design methodology is similar to reinforced concrete ultimate strength design. It is useful with high shear values and for seismic design. The limiting masonry strength is $0.80f'_m$.



Force-Moment Interaction

Combined stresses and the reduction of axial load with moment is similar to that for reinforced concrete column design as shown in the interaction diagram:



Reinforcement is typically placed in the center of walls. Grouting is placed in hollows with reinforcing, while other hollows may be empty. Stirrups are avoided.

Biaxial bending can occur in columns and stresses must satisfy:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1$$

When maximum moment occurs somewhere other than at the end of the column or wall, a "virtual" eccentricity can be determined from $e = M/P$.

Masonry Columns

Columns are classified as having b/t < 3 and h/t > 4. Slender columns have a minimum side dimension of 8" and must have h/t ≤ 25. They must be designed with an eccentricity of 10% of the side dimension, and satisfy the interaction relationship of $\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1$, the tensile stress cannot exceed the allowable: $f_b - f_a \le F_t$ and the compressive stress exceed allowable for reinforced masonry: $f_a + f_b \le F_b$ provided $f_a \le F_a$.

For purely axial loading, the capacity P_a depends on the slenderness ratio of h/r:

unreinforced

$$P_a = [0.25 f'_m A_n]\left[1 - \left(\frac{h}{140r}\right)^2\right] \qquad \text{for } h/r \le 99$$

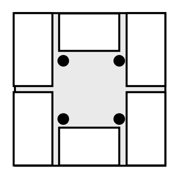
$$P_a = [0.25 f'_m A_n]\left(\frac{70r}{h}\right)^2 \qquad \text{for } h/r > 99$$

reinforced

$$P_a = [0.25 f'_m A_n + 0.65 A_{st} F_s]\left[1 - \left(\frac{h}{140r}\right)^2\right] \qquad \text{for } h/r \le 99$$

$$P_a = [0.25 f'_m A_n + 0.65 A_{st} F_s]\left(\frac{70r}{h}\right)^2 \qquad \text{for } h/r > 99$$

where h = effective length
r = least radius of gyration
A_n = net area of masonry
A_{st} = area of steel reinforcement
f'_m = specified masonry compressive strength
F_s = allowed compressive strength of reinforcement



The least radius of gyration can be found with $\sqrt{\frac{I}{A}}$ for a rectangle with side dimensions of b & d as:

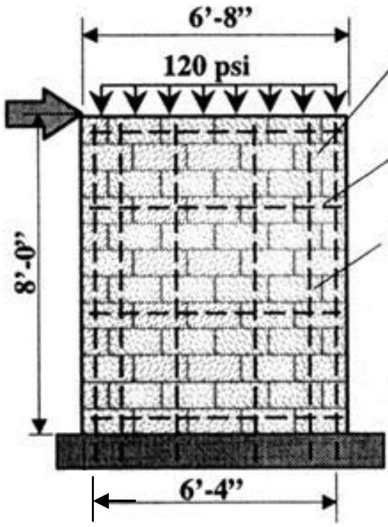
$$r = \sqrt{\frac{\frac{db^3}{12}}{bd}} = \sqrt{\frac{b^2}{12}} = \frac{b}{\sqrt{12}}$$

where b is the smaller of the two side dimensions.

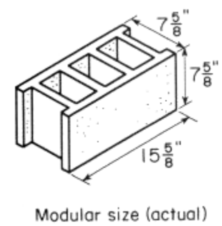
Examples:
Masonry

Example 1

Determine the maximum lateral force, H (by wind), as per MSJC.



Grade 60 steel



Case A: neglect all reinforcement
Case B: consider vertical reinf., neglect horizontal reinf.
Case C: consider vertical and horizontal reinf.
Case D: design horizontal reinforcement for max. shear

Case A: neglect all reinforcement

(M=Hx8' moment arm) $S_g = \frac{7.63 \times 80^2}{6} = 8139\ in^3$

flexure

$- f_a + M / S = F_t \qquad -120 + \frac{96 \times H}{8139} = 0 \qquad H = 10{,}174\ lbs. = 10.2\ kips$

shear

$F_v = 1.5\sqrt{f'_m} = 1.5\sqrt{3000} = 82.2\ psi \qquad f_v = \frac{VQ}{I_n b} = \frac{3V}{2A}$ (solid rectangle)

$V_{max} = \frac{2}{3} F_v bt = \frac{2}{3}(82.2\ psi)(7.63x80) = 33.4\ kips$
(wall area)

Case B: consider only vertical reinforcement

Flexure: neglecting f_a (allowed stress for grade 60 steel)

$M_s = A_s F_s jd = \underbrace{2 \times 0.79\ in^2}_{\text{lumping 2 - \#8's}}\ (32\ ksi)\ (0.9 \times 72.0") = 3276\ k\text{-}in \qquad H_{wind} = 34.0\ kips = \frac{3276 k-in}{8\,ft(12\,in/ft)}$

(j=0.909 from ρ_b table in Note Set 23.1) ave. d for 2 bars

Shear

$M/Vd = \frac{8.0'}{6.0'} = 1.33 > 1$

$for \frac{M}{Vd} > 1 \quad F_{vmax} = 2\sqrt{f'_m} = 2.0\sqrt{3000} = 109.5\ psi \qquad f_v = \frac{V}{A_{nv}}$

$$F_{vm} = \frac{1}{2}\left[\left(4.0 - 1.75\left(\frac{M}{Vd}\right)\right)\sqrt{f'_m}\right] + 0.25\left(\frac{P}{A_n}\right) = \frac{1}{2}\left[(4.0 - 1.75(1.33))\sqrt{3000}\right] + 0.25(120\,psi) = 75.8\,psi$$

$V_{max} = A_{nv}F_v = (7.63")(80")(75.8\ psi)/1000 = 46.3\ kips$
(actual width of 8" nominal CMU block)

Case C: consider all reinforcement

Flexure: same as case B

Shear

$$F_{vm} = 75.8 psi$$

$$F_{vs} = 0.5\left(\frac{A_v F_s d}{A_n s}\right) = 0.5\left(\frac{(0.20in^2)(32ksi)(72in)}{(7.63in)(80in)(32in)}\right)1000lb/k = 11.8 psi$$

F_v = *87.6 psi*

V_{max} =*87.6 psi (7.63in)(80in)/1000 = 53.5 kips*

$$f_v = \frac{V}{A_{nv}}$$

Case D: design horizontal reinforcement for maximum shear strength

$F_{vs} = F_{vmax} - F_{vm}$ =*109.5 psi – 75.8 psi* = 33.7 *psi*

$$s = 0.5\left(\frac{A_v F_s d}{A_n F_{vs}}\right) = 0.5\left(\frac{(0.20in^2)(32ksi)(72in)}{(7.63in)(80in)(33.7psi)}\right)1000lb/k = .11.2in..$$

using #4 rebars (A_v *= 0.20 in*^{2}) *use #4@8 in. horizontal*

Example 2

A 12 in. nominal solid brick column, 16 ft high, is built with brick, M mortar, and Grade 40 reinforcement. There are 4 - #4 bars with #2 ties at 8 in. on center. The column must carry an axial load of 63 kips. Check if the column design is adequate. f'_m = 5,300 psi.

SOLUTION:

Find the allowable axial load, P_a: which depends on h/r

r = $\sqrt{I/A} = \sqrt{db^3/12bd} = b/\sqrt{12}$ =11.5 in x 0.289 = 3.3 in (where b is the smallest dimension)

so b/r = 16ft x 12 in/ft / 3.3 in = 58 < 99

$$P_a = \left[0.25 f'_m A_n + 0.65 A_{st} F_s\right]\left[1 - \left(\frac{h}{140r}\right)^2\right]$$

A_s = 4 (0.20 in^2) = 0.8 in.^{2}

A_n = 11.5 in x 11.5 in – 0.8 in^2 = 131.5 in^2

F_s = 20 ksi,

$$P_a = \left[0.25(5.3ksi)131.5in^2 + 0.65(0.8in^2)20ksi\right]\left[1 - \left(\frac{16\,ft(12in/\ ft)}{140(3.3in)}\right)^2\right] = 152.7\,psi$$

Find the bending stress, f_b:

f_b = M/S, M = Pe, where e = 0.1(11.5 in) = 1.2 in.

f_b = 63k(1000 lb/k)(1.2 in)/(11.5x11.5^2/6)in^3= 298.2 psi

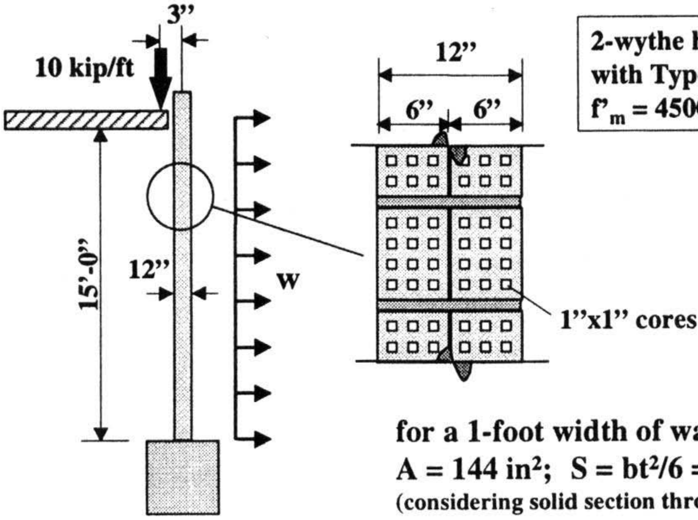
Is $\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1$ or equivalently $\frac{P}{P_a} + \frac{f_b}{F_b} \le 1$

F_b = 0.45f'_m = 0.45(5,300psi) = 2385 psi

$$\frac{63k}{152.7k} + \frac{298.2\,psi}{2387\,psi} = 0.54 < 1 \quad \text{OK}$$

Example 3

Determine the maximum transverse wind load, w, per MSJC.



2-wythe hollow-clay tile
with Type S Portland cement lime mortar
f'_m = 4500 psi, special inspection provided

for a 1-foot width of wall:
A = 144 in²; S = bt²/6 = 288 in³; r = 0.289t = 3.47"
(considering solid section through mortar joint)

$$I = bt^3/12 = 1728 in^4;\ r = \sqrt{I/A} = \sqrt{1728/144} = 3.464 in$$

(b is the 1 ft width of wall and t is the thickness)

Case "A" with wind

Weak section has been assumed to be through mortar bed joint. This assumes that unit strength will be at least twice that of the mortar (ratio of mortar area to clay area).

$$\textit{at midheight of wall}: \ M = \frac{Pe}{2} + \frac{wh^2}{8}$$

$$M = 10\ kip \times \frac{3in.}{2} + \frac{w(15)^2}{8} \times 12\frac{in.}{ft.}$$

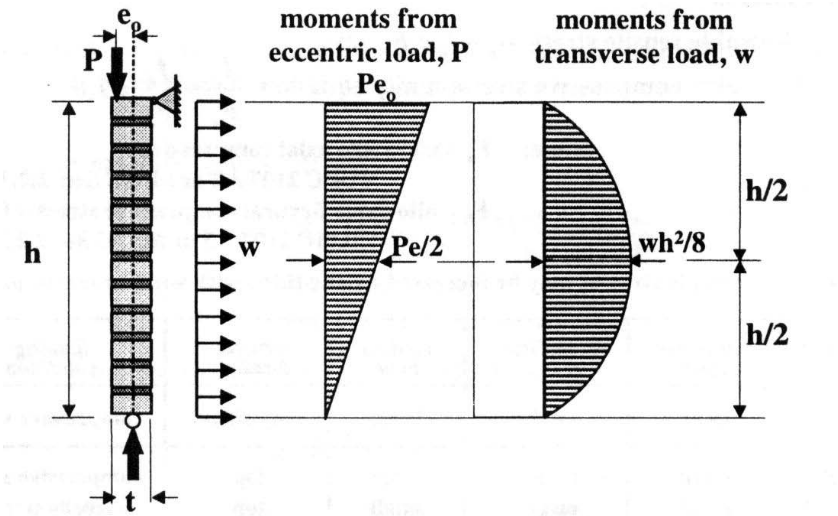
$$M = 338w + 15.0$$

$$\textit{where } w = ksf \textit{ and } M = kip\text{-}in$$

$$\textit{tension criterion}: \ -\frac{P}{A} + \frac{M}{S} = F_t = 53\ psi \quad \text{(Table 2.2.3.2)}$$

$$-\frac{10\ kip}{144\ in^2} + \frac{338w + 15.0}{288\ in^3} = 0.053\ psi \qquad \boxed{w = 60.0\ psf}$$

Note: **assume F_t for solid units since mortar bed is full with respect to tension normal to bed joint.**



for large P and small w: critical location is at *top* of wall: M = Pe
for small P and large w: critical location is *near midheight*: M = Pe/2 + wh²/8

Case "A" with wind

compression criterion :

$$\frac{f_a}{F_a}+\frac{f_b}{F_b}<1.0$$

$$M = 338\times 0.060\ ksf + 15.0 = 35.3\ kip\text{-}in$$

$$f_a=\frac{P}{A}=\frac{10}{144}=0.069\ ksi \qquad f_b=\frac{M}{S}=\frac{35.3}{288}=0.123\ ksi \qquad F_b = 0.33f'_m = 0.33(4500\ psi) = 1500\ psi$$

$$\frac{h'}{r}=\frac{15\times 12}{3.47}=51.8 \qquad F_a = 0.25\ f'_m\ [1-(\frac{h'}{140r})^2\ ] = 0.216\ f'_m = 970\ psi$$

$$= 0.25(4500\ psi)\left[1-\left(\frac{15\cdot 12in}{140\cdot 3.47in}\right)^2\right] = 970\ psi$$

(psi)

$$\frac{69}{970}+\frac{123}{1500}=0.071+0.082=0.153<1.0 \quad ok.$$

Case "B" without wind

at top of wall : $M = Pe = 30\ kip\text{-}in.$

tension criterion : $-\frac{P}{A}+\frac{M}{S}=F_t = 53\ psi$

$$-\frac{10\ kip}{144\ in^2}+\frac{30\ kip\text{-}in}{288\ in^3}\le 0.053\ ksi\ ?$$

$$-0.0694\ ksi + 0.0104\ ksi = 0.0348\ ksi < 0.053\ ksi \quad ok$$

Foundation Design - Soils

from Building Structures, 2nd ed., Ambrose, 1993

CHAPTER THIRTY-NINE

General Considerations

Chapter 39 summarizes the general issues involved in foundation design, the properties and behavioral characteristics of foundation materials of significance for design work, and the problems of establishing useful design data and criteria.

39.1. BASIC PROBLEMS IN FOUNDATION DESIGN

The design of the foundation for a building cannot be separated from the overall problems of the building structure and the building and site designs in general. Nevertheless, it is useful to consider the specific aspects of the foundation design that must be dealt with.

Site Exploration

For purposes of the foundation design, as well as for the building and site development in general, it is necessary to know the actual site conditions. This investigation usually consists of two parts: determination of the ground surface conditions, and of the subsurface conditions. The surface conditions are determined by a site survey that establishes the three-dimensional geometry of the surface and the location of various objects and features on the site. Where they exist, the location of buried objects such as sewer lines, underground power and telephone lines, and so on, may also be shown on the site survey.

Unless they are known from previous explorations, the subsurface conditions must be determined by penetrating the surface to obtain samples of materials at various levels below the surface. Inspection and testing of these samples in the field, and possibly in a testing lab, is used to identify the materials and to establish a general description of the subsurface conditions.

Site Design

Site design consists of positioning the building on the site and the general development, or redevelopment, of the site contours and features. The building must be both horizontally and vertically located. Recontouring the site may involve both taking away existing materials (called *cutting*) and building up to a new surface with materials brought in or borrowed from other locations on the site (called *filling*). Development of controlled site drainage for water runoff is an important part of the site design.

Selection of Foundation Type

The first formal part of the foundation design is the determination of the type of foundation system to be used. This decision cannot normally be made until the surface and subsurface conditions are known in some detail and the general size, shape, and location of the building are determined. In some cases it may be necessary to proceed with an approximate design of several possible foundation schemes so that the results can be compared.

Design of Foundation Elements

With the building and site designs reasonably established, the site conditions known, and the type of foundation determined, work can proceed to the detailed design of individual structural elements of the foundation system.

Construction Planning

In many cases the construction of the foundation requires a lot of careful planning. Some of the possible problems include conditions requiring dewatering the site during construction, bracing the sides of the excavation, underpinning adjacent properties or buildings, excavating difficult objects such as large tree roots or existing constructions, and working with difficult soils such as wet clays, quick sands or silts, soils with many large boulders, and so on. The feasibility of dealing with these problems, primarily in terms of cost and delays, may influence the foundation design as well as the positioning of the building on the site and the general site development.

Inspection and Testing

During the design and construction of the foundation there are several times when it may be necessary to perform inspection or testing. Whether done by the designer or by others, the results of the inspections and tests will be used to influence design decisions or to verify the adequacy of the completed designs or construction. The need for this work will depend on the size of the building, the type of construction, the specific subsurface conditions, the type of foundation system, and the various problems encountered during construction. Some of the ordinary inspections or tests are as follows:

Preliminary Site Investigation. The preliminary investigation usually consists of a site survey and some minimal subsurface investigation prior to the construction and often prior to the final design of the foundation. For major projects or difficult subsurface conditions it is usually necessary to have this information even before the preliminary site design and building design can be done.

Detailed Site Design. In some cases it is necessary to have additional information prior to the final design or the construction of the foundation. In some instances it is possible to incorporate this investigation with the early stages of the foundation work, with any necessary design adjustments made as the work progresses.

Inspection and Testing during Construction. At a bare minimum the completed excavation should be visually inspected prior to any construction to verify that the actual conditions encountered are those assumed for the design. In some cases the site conditions, the type of foundation, or the nature of the building may require extensive and continuous inspection and testing throughout the foundation construction process. Inspections by both the designer and the permit-granting agency may be required.

Inspection and Testing after Construction. In some cases it may be necessary to perform inspection and testing after the foundation construction is complete. This is usually required where progressive soil deformation is anticipated over time or with seasonal changes.

Remedial Alterations

For various reasons it is often necessary to modify the foundation in some way from the original design. This is best done prior to construction, of course, but must sometimes be done as repair or renovation. The remedial measures may be obvious and simple to accomplish, or may require the best efforts of the most-qualified experts. Some of the situations that may require remedial alterations are:

Unanticipated Subsurface Conditions. Where the site conditions are very nonuniform or the preliminary investigations sketchy, or for other reasons, it may be necessary to modify the design due to actual encountered conditions.

Unanticipated Construction Problems. Weather conditions, unusual excavation problems, unavoidable delays, and a host of other possibilities may necessitate expedient change of the design.

Construction Errors. Foundation construction is usually done under the crudest and sloppiest of working conditions. Great accuracy and perfection is not to be expected. Overexcavation, mislocation of elements, errors in dimensions, omission of details, and so on, are common.

Inadequate Performance of the Foundation. During construction, or even at some time after completion of the building, there may be evidence of excessive settlement, uneven settlement, horizontal shifting, tilting, or other forms of foundation failure.

39.2. SOIL CONSIDERATIONS RELATED TO FOUNDATION DESIGN

The principal properties and behavior characteristics of soils that are of direct concern in foundation design are the following:

Strength. For bearing-type foundations the main concern is resistance to vertical compression. Resistance to horizontal pressure and to friction are of concern when foundations must resist the force of wind, earthquakes, or retained earth.

Strain Resistance. Deformation of soil under stress is of concern in designing for limitations of the movements of foundations, such as the vertical settlement of bearing foundations.

Stability. Frost action, fluctuations in water content, seismic shock, organic decomposition, and disturbance during construction are some of the things that may produce changes in physical properties of soils. The degree of sensitivity of the soil to these actions is called its *relative stability*.

Properties Affecting Construction Activity

A number of possible factors may affect construction activity, including the following:

The relative ease of excavation.

Ease of and possible effects of site dewatering during construction.

Feasibility of using excavated materials as fill material.

Ability of the soil to stand on a vertical side of an excavation.

Effects of construction activity—notably the movement of workers and equipment—on unstable soils.

Miscellaneous Conditions

In specific situations various factors may affect the foundation design or the problems to be dealt with during construction. Some examples are the following:

Location of the water table, affecting soil strength or stability, need for waterproofing basements, requirement for dewatering during construction, and so on.

Nonuniform soil conditions on the site, such as soil strata that are not horizontal, strips or pockets of poor soil, and so on.

Local frost conditions, affecting the depth required for bearing foundations and possible heave and settlement of exterior pavements.

Deep excavation or dewatering operations, possibly affecting the stability of adjacent properties, buildings, streets, and so on.

All of these concerns must be anticipated and dealt with in designing buildings and in planning and estimating construction costs. Persons charged with responsibility for design and planning foundation construction must have some understanding of the characteristics of ordinary soils so that they can translate information about site conditions into usable data. The discussions that follow deal with the basic nature of soils of various types, the behavior and design considerations of various foundation elements and systems, and the means for obtaining and using information about specific site conditions.

39.3. FOUNDATION DESIGN CRITERIA

For the design of ordinary bearing-type foundations several structural properties of a soil must be established. The principal values are the following:

Allowable Bearing Pressure. This is the maximum permissible value for vertical compression stress at the contact surface of bearing elements. It is typically quoted in units of pounds or kips per square foot of contact surface.

Compressibility. This is the predicted amount of volumetric consolidation that determines the amount of settlement of the foundation. Quantification is usually done in terms of the actual dimension of vertical settlement predicted for the foundation.

Active Lateral Pressure. This is the horizontal pressure exerted against retaining structures, visualized in its simplest form as an equivalent fluid pressure. Quantification is in terms of a density for the equivalent fluid given in actual unit weight value or as a percentage of the soil unit weight.

Passive Lateral Pressure. This is the horizontal resistance offered by the soil to forces against the soil mass. It is also visualized as varying linearly with depth in the manner of a fluid pressure. Quantification is usually in terms of a specific pressure increase per unit of depth.

Friction Resistance. This is the resistance to sliding along the contact bearing face of a footing. For cohesionless soils it is usually given as a friction coefficient to be multiplied by the compression force. For clays it is given as a specific value in pounds per square foot to be multiplied by the contact area.

Whenever possible, stress limits should be established as the result of a thorough investigation and the recommendations of a qualified soils engineer. Most building codes allow for the use of *presumptive* values for design. These are average values, on the conservative side usually, that may be used for soils identified by groupings used by the codes. Reprints of portions of the *UBC*, 1991 edition, and the *Building Code of the City of Los Angeles*, 1976 edition, are given in Appendix D; both contain presumptive values for design. Soil types are identified only rather broadly in the *UBC*, whereas the Los Angeles code uses what is essentially the unified system in establishing allowable bearing pressures.

from Foundation Analysis and Design, 5th ed., Bowles, 1996

4-14 BEARING CAPACITY BASED ON BUILDING CODES (PRESUMPTIVE PRESSURE)

In many cities the local building code stipulates values of allowable soil pressure to use when designing foundations. These values are usually based on years of experience, although in some cases they are simply used from the building code of another city. Values such as these are also found in engineering and building-construction handbooks. These arbitrary values of soil pressure are often termed *presumptive* pressures. Most building codes now stipulate that other soil pressures may be acceptable if laboratory testing and engineering considerations can justify the use of alternative values. Presumptive pressures are based on a visual soil classification.

Table 4-8 indicates representative values of building code pressures. These values are primarily for illustrative purposes, since it is generally conceded that in all but minor construction projects some soil exploration should be undertaken. Major drawbacks to the use of presumptive soil pressures are that they do not reflect the depth of footing, size of footing, location of water table, or potential settlements.

TABLE 4-8

Presumptive bearing capacities from indicated building codes, kPa

Soil descriptions vary widely between codes. The following represents author's interpretations.

Soil description	Chicago, 1995	Natl. Board of Fire Underwriters, 1976	BOCA,* 1993	Uniform Bldg. Code, 1991†
Clay, very soft	25			
Clay, soft	75	100	100	100
Clay, ordinary	125			
Clay, medium stiff	175	100		100
Clay, stiff	210		140	
Clay, hard	300			
Sand, compact and clean	240		140	200
Sand, compact and silty	100			
Inorganic silt, compact	125			
Sand, loose and fine			140	210
Sand, loose and coarse, or sand-gravel mixture, or compact and fine		140 to 400	240	300
Gravel, loose and compact coarse sand	300		240	300
Sand-gravel, compact			240	300
Hardpan, cemented sand, cemented gravel	600	950	340	
Soft rock				
Sedimentary layered rock (hard shale, sandstone, siltstone)			6000	1400
Bedrock	9600	9600	6000	9600

Note: Values converted from psf to kPa and rounded.

*Building Officials and Code Administrators International, Inc.

†Author interpretation.

Foundation Design - Structure

Notation:

a = equivalent square column size in spread footing design
= depth of the effective compression block in a concrete beam
A = name for area
A_g = gross area, equal to the total area ignoring any reinforcement
A_{req} = area required to satisfy allowable stress
A_s = area of steel reinforcement in concrete design
A_1 = area of column in spread footing design
A_2 = projected bearing area of column load in spread footing design
b = width of retaining wall stem at base
= rectangular column dimension in concrete footing design
= width, often cross-sectional
b_f = width of the flange of a steel or cross section
b_o = perimeter length for two-way shear in concrete footing design
B = spread footing or retaining wall base dimension in concrete design
= dimension of a steel base plate for concrete footing design
B_s = width within the longer dimension of a rectangular spread footing that reinforcement must be concentrated within for concrete design
c = rectangular column dimension in concrete footing design
C = dimension of a steel base plate for concrete footing design
d = effective depth from the top of a reinforced concrete member to the centroid of the tensile steel
= name for diameter
d_b = bar diameter of a reinforcing bar
d_f = depth of a steel column flange (wide flange section)
e = eccentric distance of application of a force (P) from the centroid of a cross section
f = symbol for stress
f'_c = concrete design compressive stress
f_y = yield stress or strength
$F_{horizontal\text{-}resisting}$ = total force resisting horizontal sliding
$F_{sliding}$ = total sliding force
F_x = force in the x direction
h_f = height of a concrete spread footing
H = height of retaining wall
H_A = horizontal force due to active soil pressure
l_d = development length for reinforcing steel
l_{dc} = development length for column
l_s = lap splice length in concrete design
L = name for length or span length
L_m = projected length for bending in concrete footing design
L' = length of the one-way shear area in concrete footing design
M = moment due to a force
M_n = nominal flexure strength with the steel reinforcement at the yield stress and concrete at the concrete design strength for reinforced concrete flexure design
$M_{overturning}$ = total overturning moment
$M_{resisting}$ = total moment resisting overturning about a point
M_u = maximum moment from factored loads for LRFD beam design
N = name for normal force to a surface
o = point of overturning of a retaining wall, commonly at the "toe"
p_A = active soil pressure
P = name for axial force vector
P_{dowels} = nominal capacity of dowels from concrete column to footing in concrete design
P_D = dead load axial force
P_L = live load axial force
P_n = nominal column or bearing load capacity in concrete design
P_u = factored axial force
$q_{allowable}$ = allowable soil bearing stress in allowable stress design
q_{net} = net allowed soil bearing pressure

q_u = factored soil bearing capacity in concrete footing design from load factors
R = name for reaction force vector
SF = shorthand for factor of safety
t = thickness of retaining wall stem at top
T = name for tension force vector
V_n = nominal shear capacity
V_{u1} = maximum one-way shear from factored loads for LRFD beam design
V_{u2} = maximum two-way shear from factored loads for LRFD beam design
W = name for force due to weight
$\bar{y}$ = the distance in the y direction from a reference axis to the centroid of a shape
β_c = ratio of long side to short side of the column in concrete footing design
ϕ = resistance factor
γ_c = density or unit weight of concrete
γ_s = density or unit weight of soil
ρ = reinforcement ratio in concrete beam design = A_s/bd
υ_c = shear strength in concrete design

Foundation Materials

Typical foundation materials include:

- plain concrete
- reinforced concrete
- steel
- wood
- composites, ie. steel tubing filled with concrete

Foundation Design

Generalized Design Steps

Design of foundations with variable conditions and variable types of foundation structures will be different, but there are steps that are typical to every design, including:

1. Calculate loads from structure, surcharge, active & passive pressures, etc.
2. Characterize soil – hire a firm to conduct soil tests and produce a report that includes soil material properties
3. Determine footing location and depth – shallow footings are less expensive, but the variability of the soil from the geotechnical report will drive choices
4. Evaluate soil bearing capacity – the factor of safety is considered here
5. Determine footing size – these calculations are based on working loads and the allowable soil pressure
6. Calculate contact pressure and check stability
7. Estimate settlements
8. Design the footing structure – design for the material based on applicable structural design codes which may use allowable stress design, LRFD or limit state design (concrete).

Shallow Foundation Types

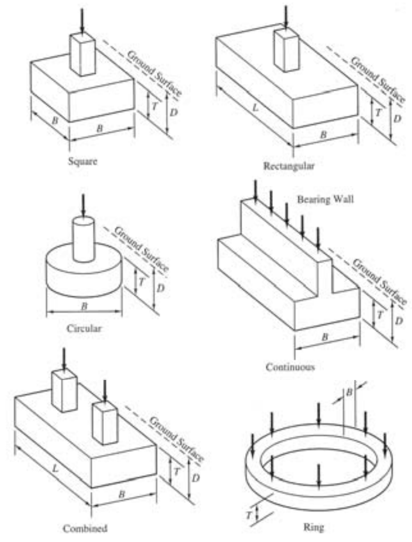


Figure 5.1 Spread footing shapes and dimensions.

Considered simple and cost effective because little soil is removed or disturbed.

Spread footing – A single column bears on a square or rectangular pad to distribute the load over a bigger area.

Wall footing – A continuous wall bears on a wide pad to distribute the load..

Eccentric footing – A spread or wall footing that also must resist a moment in addition to the axial column load.

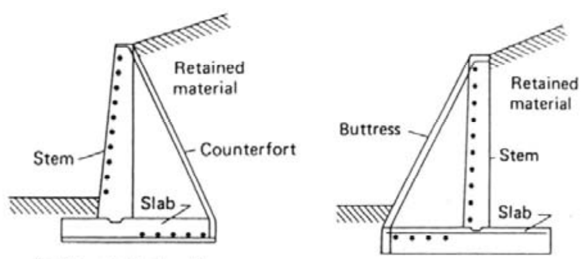
Combined footing – Multiple columns (typically two) bear on a rectangular or trapezoidal shaped footing.

Unsymmetrical footing – A footing with a shape that does not evenly distribute bearing pressure from column loads and moments. It typically involves a hole or a non-rectangular shape influenced by a boundary or property line.

Strap footing – A combined footing consisting of two spread footings with a beam or strap connecting the slabs. The purpose of this is to limit differential settlements.

Mat foundation – A slab that supports multiple columns. The mat can be stiffened with a grid or grade beams. It is typically used when the soil capacity is very low.

Deep Foundation Types



(c) Counterfort wall (d) Buttress wall

Considerable material and excavation is required, increasing cost and effort.

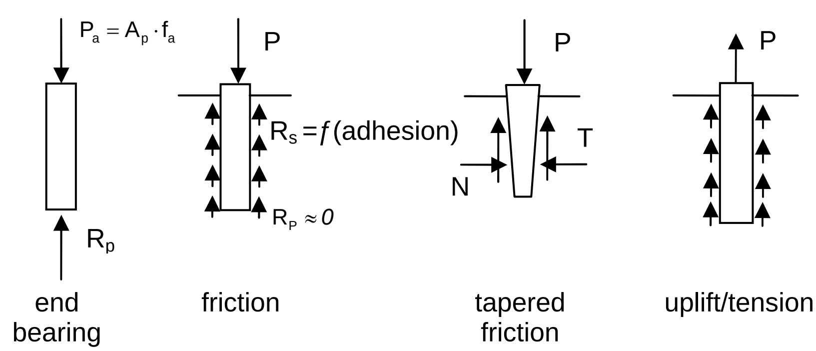
Retaining Walls – A wall that retains soil or other materials, and must resist sliding and overturning. Can have counterforts, buttresses or keys.

Basement Walls – A wall that encloses a basement space, typically next to a floor slab, and that may be restrained at the top by a floor slab.

Piles – Next choice when spread footings or mats won't work, piles are used to distribute loads by end bearing to strong soil or friction to low strength soils. Can be used to resist uplift, a moment causing overturning, or to compact soils. Also useful when used in combination to control settlements of mats or slabs.

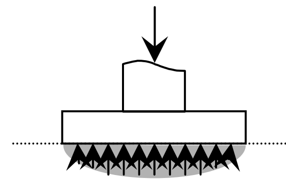
Drilled Piers – Soil is removed to the shape of the pier and concrete is added.

Caissons –Water and possibly wet soil is held back or excavated while the footing is constructed or dropped into place.

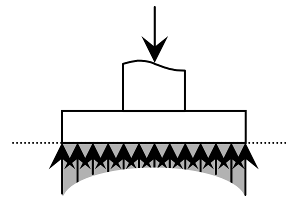


Pile Types

Loads and Stresses


RIGID footing on sand

Bearing loads must be distributed to the soil materials, but because of their variability and the stiffness of the footing pad, the resulting stress, or soil pressure, is not necessarily uniform. But we assume it is for design because dealing with the complexity isn't worth the time or effort.

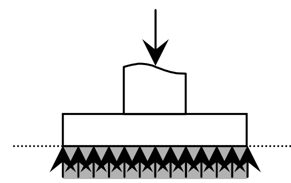

RIGID footing on clay

The increase in weight when replacing soil with concrete is called the overburden. Overburden may also be the result of adding additional soil to the top of the excavation for a retaining wall. It is extra *uniformly distributed load* that is considered by reducing the allowable soil pressure (instead of increasing the loads), resulting in a net allowable soil pressure, q_{net}:

$$q_{net} = q_{allowable} - h_f(\gamma_c - \gamma_s)$$

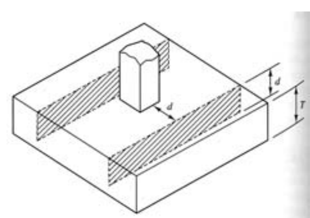
In order to design the footing size, the actual stress P/A must be less than or equal to the allowable pressure:

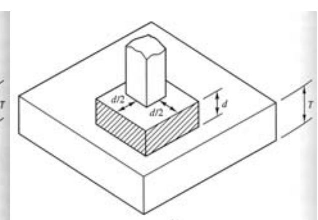
$$\frac{P}{A} \le q_{net}$$


IDEAL

Design Stresses

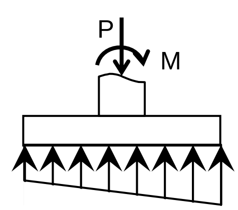
The result of a uniform pressure on the underside of a footing is identical to a distributed load on a slab over a column when looked at *upside down.* The footing slab must resist bending, one-way shear and two-way shear (punching).


one-way shear


two-way shear

Stresses with Eccentric Loading

Combined axial and bending stresses increase the pressure on one edge or corner of a footing. We assume again a linear distribution based on a constant relationship to settling. If the pressure combination is in tension, this effectively means the contact is gone between soil and footing and the pressure is really zero. To avoid zero pressure, the eccentricity must stay within the kern. The maximum pressure must not exceed the net allowable soil pressure.



Overturning is considered in design such that the resisting moment from the soil pressure (equivalent force at load centroid) is greater than the overturning moment, M, by a factor of safety of at least 1.5

$$SF = \frac{M_{resist}}{M_{overturning}} \ge 1.5$$

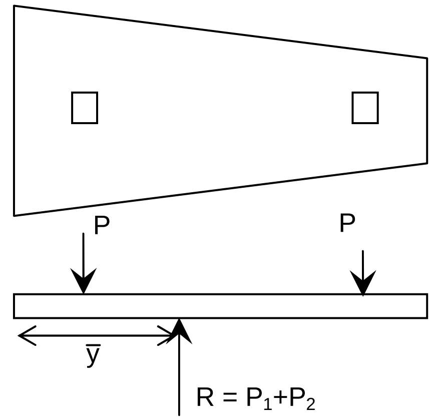
where

M_{resist} = average resultant soil pressure x width x location of load centroid with respect to column centroid

$M_{overturning}$ = P x e

Combined Footings

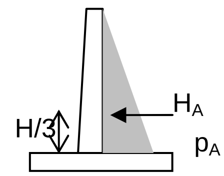
The design of combined footing requires that the centroid of the area be as close as possible to the resultant of the two column loads for uniform pressure and settling.



Retaining Walls

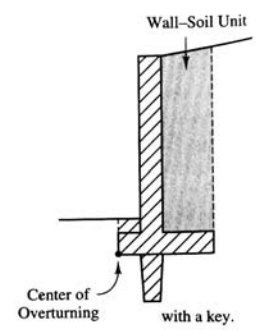
The design of retaining walls must consider overturning, settlement, sliding and bearing pressure. The water in the retained soil can significantly affect the loading and the active pressure of the soil. The lateral force acting at a height of H/3 is determined from the active pressure, p_A, (in force/cubic area) as:

$$H_A = \frac{p_A H^2}{2}$$



Overturning is considered the same as for eccentric footings:

$$SF = \frac{M_{resist}}{M_{overturning}} \geq 1.5 - 2$$

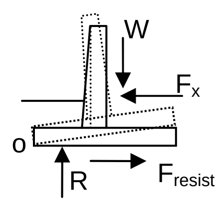


where

M_{resist} = summation of moments about "o" to resist rotation, typically including the moment due to the weight of the stem and base and the moment due to the passive pressure.

$M_{overturning}$ = moment due to the active pressure about "o".

Sliding must also be avoided:



$$SF = \frac{F_{horizontal-resist}}{F_{sliding}} \geq 1.25 - 2$$

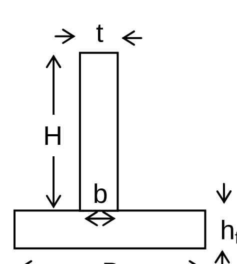
where

$F_{horizontal\text{-}resist}$ = summation of forces to resist sliding, typically including the force from the passive pressure and friction (F=μ·N where . μ is a constant for the materials in contact and N is the normal force to the ground acting down and is shown as R).

$F_{sliding}$ = sliding force as a result of active pressure.

For sizing, some rule of thumbs are:

- footing size, B
- reinforced concrete, B ≈ 2/5 - 2/3 wall height (H)
- footing thickness, h_f ≈ 1/12 - 1/8 footing size (B)
- base of stem, b ≈ 1/10 - 1/12 wall height ($H+h_f$)
- top of stem, t ≥ 12 inches



Design of Isolated Square and Rectangular Footings (ACI 318-02)

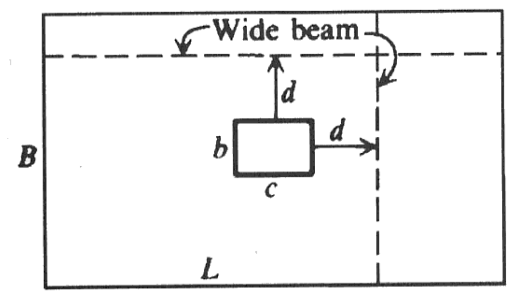
NOTE: This procedure assumes that the footing is concentrically loaded and carries no moment so that the soil pressure may be assumed to be uniformly distributed on the base.

1) Find service dead and live column loads:

P_D = Service dead load from column

P_L = Service live load from column

$P = P_D + P_L$ (typically – see **ACI 9.2**)



2) Find design (factored) column load, Pu:

$P_U = 1.2P_D + 1.6P_L$

3) Find an approximate footing depth, h_f

$h_f = d + 4"$ and is usually in multiples of 2, 4 or 6 inches.

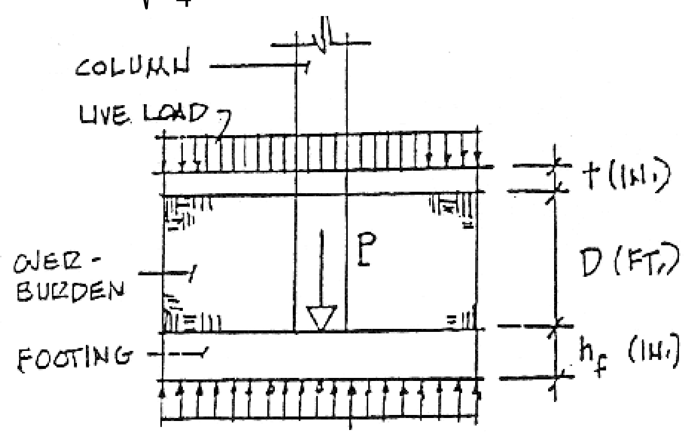
a) For rectangular columns $$4d^2 + 2(b + c)d = \frac{P_u}{\phi \upsilon_c}$$

b) For round columns $$d^2 + ad = \frac{P_u}{\phi \upsilon_c} \qquad a = \sqrt{\frac{\pi d^2}{4}}$$

where: a is the equivalent square column size

$\upsilon_c = 4\sqrt{f'_c}$ for two-way shear

$\phi = 0.75$ for shear



4) Find net allowable soil pressure, q_{net}:

By neglecting the weight of any additional top soil added, the net allowable soil pressure takes into account the change in weight when soil is removed and replaced by concrete: $q_{net} = q_{allowable} - h_f(\gamma_c - \gamma_s)$

where γ_c is the unit weight of concrete (typically 150 lb/ft^3)

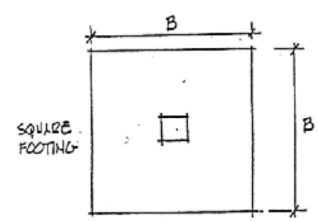
and γ_s is the unit weight of the displaced soil

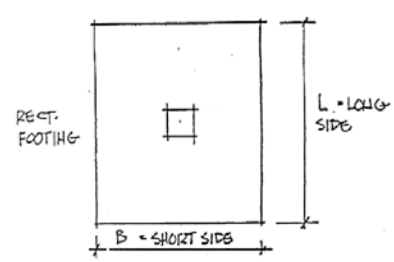
5) Find required area of footing base and establish length and width:

$$A_{req} = \frac{P}{q_{net}}$$

For square footings choose $B \geq \sqrt{A_{req}}$

For rectangular footings choose $B \times L \geq A_{req}$





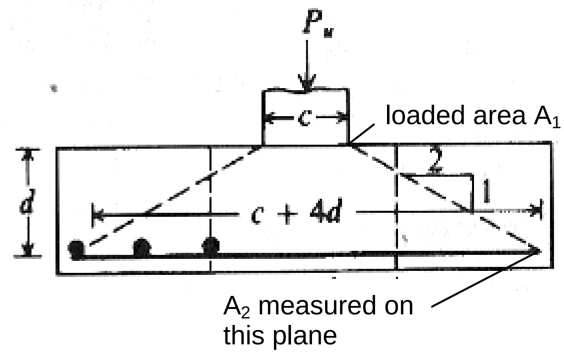
6) <u>Check transfer of load from column to footing:</u> **ACI 15.8**

a) Find load transferred by bearing on concrete in column: **ACI 10.17**

basic: $\phi P_n = \phi 0.85 f'_c A_1$ where $\phi = 0.65$ and A_1 is the area of the column

with confinement: $\phi P_n = \phi 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}$ where $\sqrt{\frac{A_2}{A_1}}$ cannot exceed 2.

IF the column concrete strength is lower than the footing, calculate ϕP_n for the column too.



b) Find load to be transferred by dowels:

$\phi P_{dowels} = P_u - \phi P_n$

IF $\phi P_n \geq P_u$ only nominal dowels are required.

c) Find required area of dowels and choose bars

Req. dowel $A_s = \frac{\phi P_{dowels}}{\phi f_y}$ where $\phi = 0.65$ and f_y is the reinforcement grade

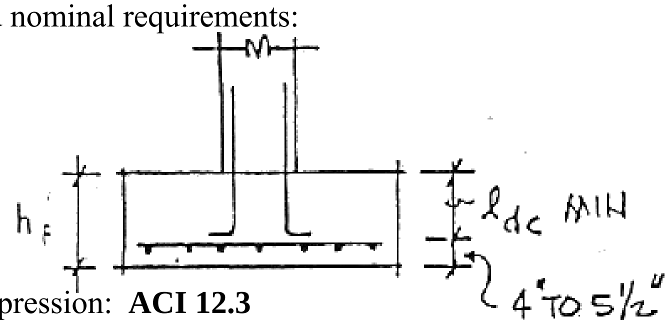
Choose dowels to satisfy the required area and nominal requirements:

i) Minimum of 4 bars

ii) Minimum $A_s = 0.005 A_g$ **ACI 15.8.2.1**

where A_g is the gross column area

iii) 4 - #5 bars



d) Check dowel embedment into footing for compression: **ACI 12.3**

$$l_{dc} = \frac{0.02 f_y d_b}{\sqrt{f'_c}}$$ but not less than $0.0003 f_y d_b$ or 8" where d_b is the bar diameter

NOTE: The footing must be deep enough to accept l_{dc}. Hooks are not considered effective in compression and are only used to support dowels during construction.

e) Find length of lapped splices of dowels with column bars: **ACI 12.16**

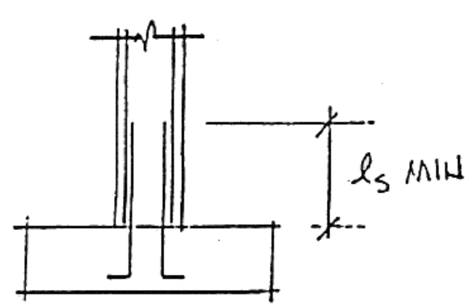
l_s is the largest of:

i) larger of l_{dc} or $0.0005 f_y d_b$ (f_y of grade 60 or less)

of smaller bar $(0.0009 f_y - 24) d_b$ (f_y over grade 60)

ii) l_{dc} of larger bar

iii) not less than 12"



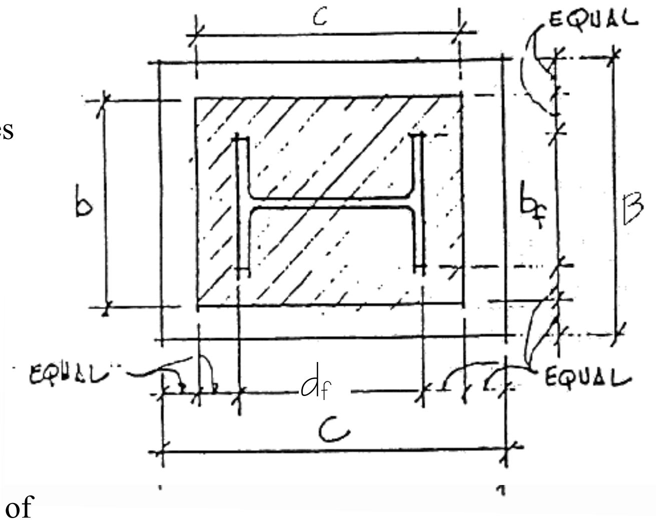
See **ACI 12.17.2** for possible reduction in l_s

7) Check two-way (slab) shear:

a) Find dimensions of loaded area:

i) For concrete columns, the area coincides with the column area, if rectangular, or equivalent square area if circular (see 3)b))

ii) For steel columns an equivalent loaded area whose boundaries are halfway between the faces of the steel column and the edges of the steel base plate is used: **ACI 15.4.2c**.



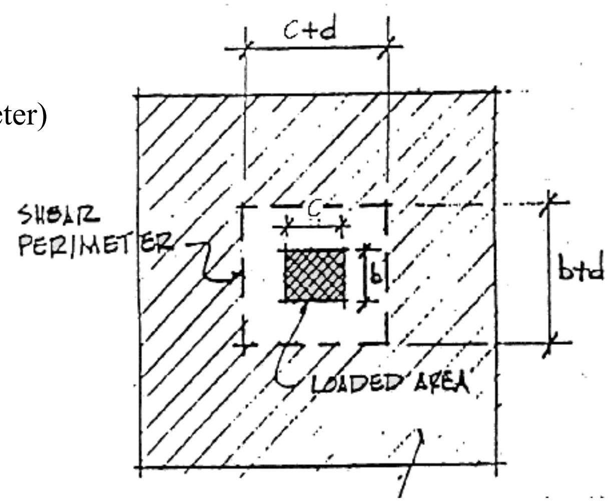
$$b = b_f + \frac{(B - b_f)}{2}$$ where b_f is the width of column flange and B is base plate side

$$c = d_f + \frac{(C - d_f)}{2}$$ where d_f is the depth of column flange and C is base plate side

b) Find shear perimeter: **ACI 11.12.1.2**

Shear perimeter is located at a distance of $d/2$ outside boundaries of loaded area and length is $b_o = 2(c+d) + 2(b+d)$

(average d = h_f – 3 in. cover – 1 assumed bar diameter)



c) Find factored net soil pressure, q_u:

$$q_u = \frac{P_u}{B^2} \text{ or } \frac{P_u}{B \times L}$$

d) Find total shear force for two-way shear, V_{u2}:

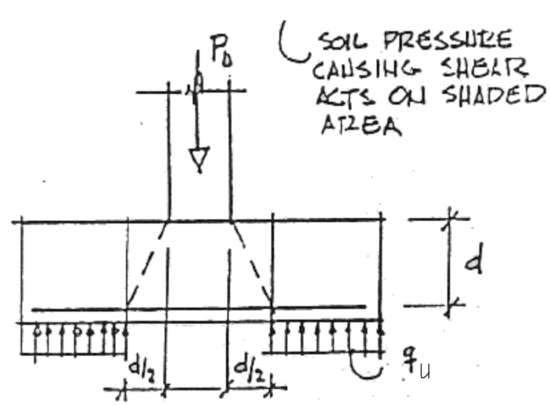
$$V_{u2} = P_u - q_u(c+d)(b+d)$$

e) Compare V_{u2} to two-way capacity, ϕV_n:

$$V_{u2} \le \phi\left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\,b_o d \le \phi 4\sqrt{f'_c}\,b_o d$$ **ACI 11.12.2.1**

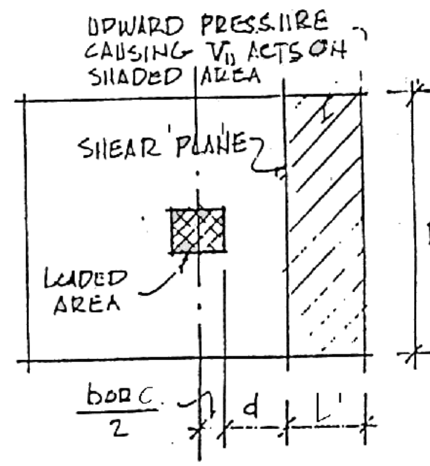
where ϕ = 0.75 and β_c is the ratio of long side to short side of the column

NOTE: This should be acceptable because the initial footing size was chosen on the basis of two-way shear limiting. If it is not acceptable, increase h_f and repeat steps starting at b).



8) <u>Check one-way (beam) shear:</u>

The critical section for one-way shear extends across the width of the footing at a distance *d* from the face of the loaded area (see 7)a) for loaded area). The footing is treated as a cantilevered beam. **ACI 11.12.1.1**



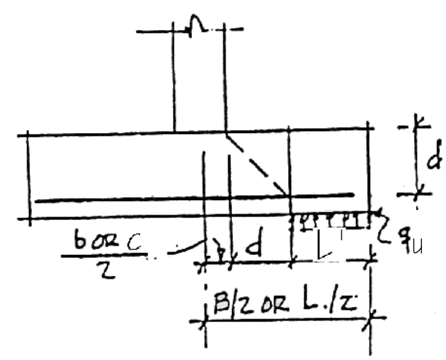
a) Find projection, *L'*:

i) For square footing:

$$L' = \frac{B}{2} - (d + b/2)$$ where b is the smaller dim. of the loaded area

ii) For rectangular footings:

$$L' = \frac{L}{2} - (d + \bullet/2)$$ where • is the dim. parallel to the long side of the footing



b) Find total shear force on critical section, V_{u1}:

$$V_{u1} = BL'q_u$$

c) Compare V_{u1} to one-way capacity, ϕV_n:

$V_{u1} \le \phi 2\sqrt{f'_c}Bd$ **ACI 11.12.3.1** where $\phi = 0.75$

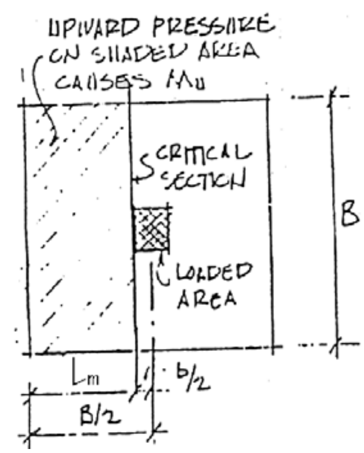
NOTE: If it is not acceptable, increase h_f .

9) <u>Check for bending stress and design reinforcement:</u>

Square footings may be designed for moment in one direction and the same reinforcing used in the other direction. For rectangular footings the moment and reinforcing must be calculated separately in each direction. The critical section for moment extends across the width of the footing at the face of the loaded area. **ACI 15.4.1, 15.4.2**.

a) Find projection, L_m:

$$L_m = \frac{B}{2} - \frac{\bullet}{2}$$ where • is the smaller dim. of column for a square footing. For a rectangular footing, use the value perpendicular to the critical section.

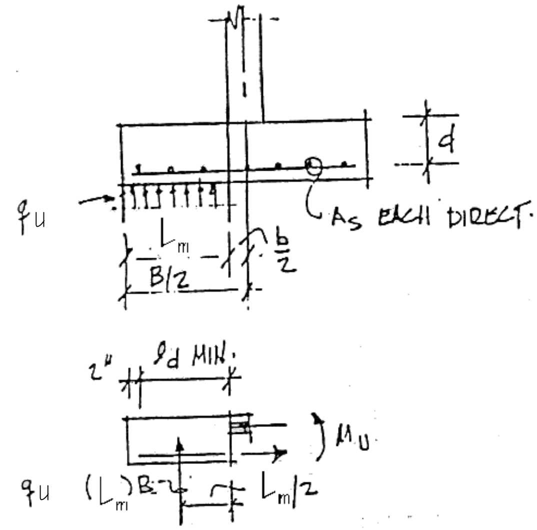


b) Find total moment, M_u, on critical section:

$$M_u = q_u \frac{BL_m^2}{2}$$ (find both ways for a rectangular footing)

c) Find required A_s:

$$R_n = \frac{M_n}{bd^2} = \frac{M_u}{\phi bd^2}$$, where $\phi = 0.9$, and ρ can be found



found from Figure 3.8.1 of Wang & Salmon.

or:

i) guess a

ii) $A_s = \frac{0.85 f'_c ba}{f_y}$

iii) solve for $a = 2\left(d - \frac{M_u}{\phi A_s f_y}\right)$

iv) repeat from ii) until a converges, solve for A_s

Minimum A_s

= 0.0018bh Grade 60 for temperature and shrinkage control

= 0.002bh Grade 40 or 50

ACI 10.5.4 specifies the requirements of **7.12** must be met, and max. spacing of 18"

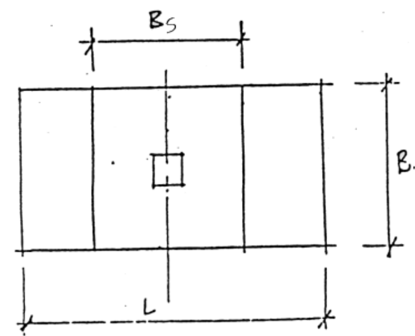
d) Choose bars:

For square footings use the same size and number of bars uniformly spaced in each direction (**ACI 15.4.3**). Note that required A_s must be furnished in each direction.

For rectangular footings bars in long direction should be uniformly spaced. In the short direction bars should be distributed as follows (**ACI 15.4.4**):

i) In a band of width B_s centered on column:

$$\# \text{ bars } = \frac{2}{L/B + 1} \cdot (\#\textit{bars in } B) \quad \text{(integer)}$$



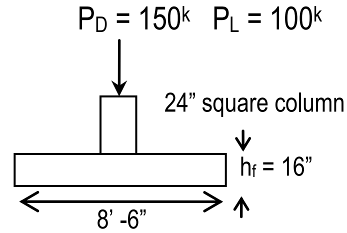
ii) Remaining bars in short direction should be uniformly spaced in outer portions of footing.

e) Check development length:

Find required development length, l_d, in tension from handout or from equations in **ACI 12.2**. l_d must be less than (L_m – 2") (end cover). If not possible, use more bars of smaller diameter.

Examples:
Foundations

Example 1



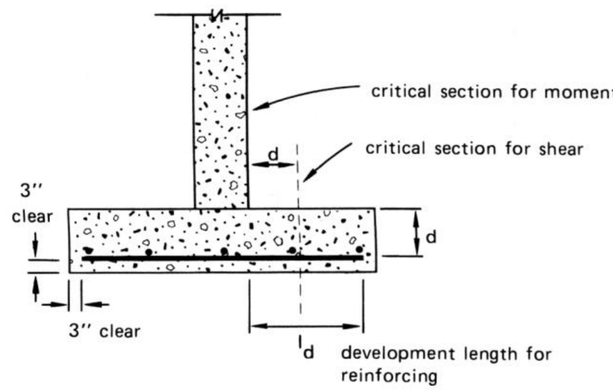
For the 16 in. thick 8.5 ft. square reinforced concrete footing carrying 150 kips dead load and 100 kips live load on a 24 in. square column, determine if the footing thickness is adequate for 4000 psi . A 3 in. cover is required with concrete in contact with soil. Also determine the moment for reinforced concrete design.

SOLUTION:

1. Find design soil pressure: $q_u = \frac{P_u}{A}$

$P_u = 1.2D + 1.6L = 1.2\ (150\ k) + 1.6\ (100\ k) = 340\ k$

$q_u = \frac{340k}{(8.5\,ft)^2}$ = 4.71 k/ft^2



2. Evaluate one-way shear at d away from column face (Is $V_u < \phi V_c$?)

$d = h_f$ – c.c. – distance bar intersection

presuming #8 bars:

d = 16 in. – 3 in. (soil exposure) – 1 in. x (1 layer of #8's) = 12 in.

V_u = total shear = q_u (edge area)

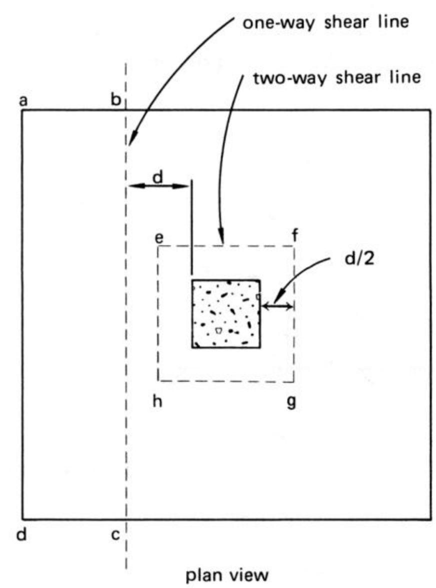
V_u on a 1 ft strip = q_u (edge distance) (1 ft)

V_u = 4.71 k/ft^2 [(8.5 ft – 2 ft)/2 – (12 in.)(1 ft/12 in.)] (1 ft) = 10.6 k

ϕV_c = one-way shear resistance = $\phi 2\sqrt{f'_c}$ bd

for a one foot strip, b = 12 in.

$\phi V_c = 0.75(2\sqrt{4000}$ psi)(12 in.)(12 in.) = 13.7 k > 10.6 k OK



3. Evaluate two-way shear at d/2 away from column face (Is $V_u < \phi V_c$?)

b_o = perimeter = 4 (24 in. + 12 in.) = 4 (36 in.) = 144 in

V_u = total shear on area outside perimeter = $P_u - q_u$ (punch area)

V_u = 340 k – (4.71 k/ft^2)(36 in.)^2(1 ft/12 in.)^2 = 297.6 kips

ϕV_c = two-way shear resistance = $\phi 4\sqrt{f'_c}\, b_o d = 0.75(4\sqrt{4000}$ psi)(144 in.)(12 in.) = 327.9 k > 297.6 k OK

4. Design for bending at column face

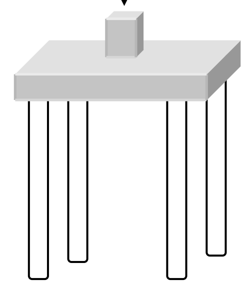
$M_u = w_u L^2/2$ for a cantilever. L = (8.5 ft – 2 ft)/2 = 3.25 ft, and w_u for a 1 ft strip = q_u (1 ft)

M_u = 4.71 ksi(1 ft)(3.25 ft)^2/2 = 24.9 k-ft (per ft of width)

To complete the reinforcement design, use b =12 in. and trial d = 12 in., choose ρ, determine A_s, find if $\phi M_n > M_u$.....

Example 2

Determine the depth required for the group of 4 friction piles having 12 in. diameters if the column load is 100 kips and the frictional resistance is 400 lbs/ft^2.



SOLUTION:

The downward load is resisted by a friction force. Friction is determined by multiplying the friction resistance (a stress) by the area: $F = fA_{SKIN}$

The area of n cylinders is: $A_{SKIN} = n(2\pi \frac{d}{2} L)$

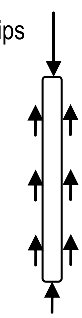
Our solution is to set P ≤ F and solve for length:

$$100k \le 400\ {}^{lb}\!/\!_{ft^2}\,(4^{piles})(2\pi)(\frac{12in}{2})L\cdot(\frac{1ft}{12in})\cdot(\frac{1k}{1000lb})$$

$$L \ge 19.9\ {}^{ft}\!/\!_{pile}$$

Example 3

Determine the depth required for the friction and bearing pile having a 36 in. diameter if the column load is 300 kips, the frictional resistance is 600 lbs/ft^2 and the end bearing pressure allowed is 8000 psf.



SOLUTION:

The downward load is resisted by a friction force and a bearing force, which can be determined from multiplying the bearing pressure by the area in contact: $F = fA_{SKIN} + qA_{TIP}$

The area of n cylinders is: $A_{TIP} = \pi \frac{d^2}{4}$

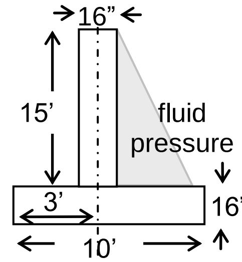
Our solution is to set P ≤ F and solve for length:

$$300k \le 600\ {}^{lb}\!/\!_{ft^2}\ 2\pi(\frac{36in}{2})L\cdot(\frac{1ft}{12in})\cdot(\frac{1k}{1000lb}) + 8000\ {}^{lb}\!/\!_{ft^2}\ \pi\frac{(36in)^2}{4}\cdot(\frac{1ft}{12in})^2\cdot(\frac{1k}{1000lb})$$

$$L \ge 43.1ft$$

Example 4

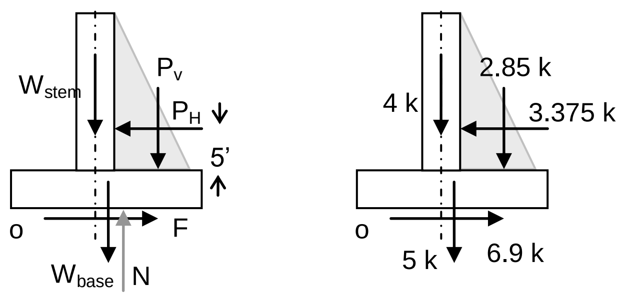
Determine the factor of safety for overturning and sliding on the 15 ft retaining wall, 16 in. wide stem, 10 ft base, 16 in. heigh base, when the equivalent fluid pressure is 30 lb/ft^3, the weight of the stem of the footing is 4 kips, the weight of the pad is 5 kips, the passive pressure is ignored for this design, and the friction coefficient for sliding is 0.58. The center of the stem is located 3' from the toe.



SOLUTION:

This is a statics problem, there is no design of materials involved. Overturning is determined by moments from acting forces and the moment from the resisting force. Sliding is determined by the acting horizontal forces and the resisting sliding force which is determined by mulitplying a friction coefficient based on the materials in contact, μ, by a normal force, N: $F = \mu N$

Find all unknown forces and draw the free body diagram with the weights at the centers of gravity of the stem and base:



The horizontal fluid (equivalent) pressure is a triangularly distributed load with the maximum distributed load equal to the density of water multiplied by the height: $w_h = \gamma H$.

$$w_h = (30\ {}^{lb}\!/\!_{ft^3})(15\,ft)(1\,ft\ strip) = 450 \text{ lb/ft}$$

The horizontal force, P_H = wL/2 acts at a distance of 1/3 the height from the "fat end" of the triangle is

$$P_H = (450\ {}^{lb}\!/\!_{ft})\frac{15\,ft}{2}\cdot\left(\frac{1k}{1000lb}\right) = 3.375 \text{ k}$$

The vertical force from the maximum distributed pressure, P_V = wL over the right side of the base (in the middle of 6.33 ft) is:

$$P_V = (450\ {}^{lb}\!/\!_{ft})\left[10\,ft - 3\,ft - \frac{16in}{2}\cdot\left(\frac{1\,ft}{12in}\right)\right]\cdot\left(\frac{1k}{1000lb}\right) = 2.85 \text{ k}$$

The total downward loads must be resisted by the normal force acting "up":

N = 4 k + 2.85 k + 5 k = 11.85 k

F = (0.58)(11.85 k) = 6.9 k

Overturning requirement:

$$SF = \frac{M_{resist}}{M_{overturning}} \geq 1.5 - 2$$

The total resisting moment will be from those moments counterclockwise about 0:

$M_{resisting}$ = 4 k(3 ft) + 5 k(5 ft) + 2.85 k (10 ft – 6.33 ft/2) = 56.5 k-ft

The overturning moment is only from the horizontal fluid force (clockwise):

$M_{overturning}$ = 3.375 k(5 ft + 16 in.(1 ft/12 in.)) = 21.4 k-ft

$$SF = \frac{56.5^{k-ft}}{21.4^{k-ft}} = 2.64 \geq 1.5 \quad \text{OK}$$

Sliding requirement:

$$SF = \frac{F_{horizontal\text{-}resist}}{F_{sliding}} \geq 1.25 - 2$$

The total resisting force will be from those opposite the hydraulic force (to the right):

$F_{resisting}$ = 6.9 k

The sliding force is only from the horizontal fluid force (to the left):

$F_{sliding}$ = 3.375 k

$$SF = \frac{6.9k}{3.375k} = 2.04 \geq 1.25 \quad \text{OK}$$

Supervision Practices
International Building Code (2003)

TABLE 1704.3
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]	IBC REFERENCE
1. Material verification of high-strength bolts, nuts and washers:				
a. Identification markings to conform to ASTM standards specified in the approved construction documents.	—	X	Applicable ASTM material specifications; AISC 335, Section A3.4; AISC LRFD, Section A3.3	—
b. Manufacturer's certificate of compliance required.	—	X	—	—
2. Inspection of high-strength bolting:				
a. Bearing-type connections.	—	X	AISC LRFD Section M2.5	1704.3.3
b. Slip-critical connections.	X	X		
3. Material verification of structural steel:				
a. Identification markings to conform to ASTM standards specified in the approved construction documents.	—	—	ASTM A 6 orASTM A 568	1708.4
b. Manufacturers' certified mill test reports.	—	—	ASTM A 6 or ASTM A 568	
4. Material verification of weld filler materials:				
a. Identification markings to conform to AWS specification in the approved construction documents.	—	—	AISC, ASD, Section A3.6; AISC LRFD, Section A3.5	—
b. Manufacturer's certificate of compliance required.	—	—	—	—
5. Inspection of welding: a. Structural steel:	—	—		
1) Complete and partial penetration groove welds.	X	—	AWS D1.1	1704.3.1
2) Multipass fillet welds.	X	—		
3) Single-pass fillet welds > $^5/_{16}''$	X	—		
4) Single-pass fillet welds ≤ $^5/_{16}''$	—	X		
5) Floor and deck welds.	—	X	AWS D1.3	—
b. Reinforcing steel:	—	—	AWS D1.4 ACI 318: 3.5.2	1903.5.2
1) Verification of weldability of reinforcing steel other than ASTM A 706.	—	X		
2) Reinforcing steel-resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special reinforced concrete shear walls and shear reinforcement.	X	—		
3) Shear reinforcement.	X	—		
4) Other reinforcing steel.	—	X		
6. Inspection of steel frame joint details for compliance with approved construction documents:		X	—	1704.3.2
a. Details such as bracing and stiffening.	—	—		
b. Member locations.	—	—		
c. Application of joint details at each connection.	—			

For SI: 1 inch = 25.4 mm.
a. Where applicable, see also Section 1707.1, Special inspection for seismic resistance.

TABLE 1704.4
REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]	IBC REFERENCE
1. Inspection of reinforcing steel, including prestressing tendons, and placement.	—	X	ACI 318: 3.5, 7.1-7.7	1903.5, 1907.1, 1907.7, 1914.4
2. Inspection of reinforcing steel welding in accordance with Table 1704.3, Item 5B.	—	—	AWS D1.4 ACI 318: 3.5.2	1903.5.2
3. Inspect bolts to be installed in concrete prior to and during placement of concrete where allowable loads have been increased.	X	—	—	1912.5
4. Verifying use of required design mix.	—	X	ACI 318: Ch. 4, 5.2-5.4	1904, 1905.2-1905.4, 1914.2, 1914.3
5. At the time fresh concrete is sampled to fabricate specimens for strength tests, perform slump and air content tests, and determine the temperature of the concrete.	X	—	ASTM C 172 ASTM C 31 ACI 318: 5.6, 5.8	1905.6, 1914.10
6. Inspection of concrete and shotcrete placement for proper application techniques.	X	—	ACI 318: 5.9, 5.10	1905.9, 1905.10, 1914.6, 1914.7, 1914.8
7. Inspection for maintenance of specified curing temperature and techniques.	—	X	ACI 318: 5.11-5.13	1905.11, 1905.13, 1914.9
8. Inspection of prestressed concrete: a. Application of prestressing forces. b. Grouting of bonded prestressing tendons in the seismic-force-resisting system.	 X X	—	 ACI 318: 18.20 ACI 318: 18.18.4	—
9. Erection of precast concrete members.	—	X	ACI 318: Ch. 16	—
10. Verification of in-situ concrete strength, prior to stressing of tendons in posttensioned concrete and prior to removal of shores and forms from beams and structural slabs.	—	X	ACI 318: 6.2	1906.2

For SI: 1 inch = 25.4 mm.
a. Where applicable, see also Section 1707.1, Special inspection for seismic resistance.

TABLE 1704.5.1
LEVEL 1 SPECIAL INSPECTION

INSPECTION TASK	FREQUENCY OF INSPECTION		REFERENCE FOR CRITERIA		
	Continuous during task listed	Periodically during task listed	IBC section	ACI 530/ASCE 5/TMS 402[a]	ACI 530.1/ASCE 6/TMS 602[a]
1. As masonry construction begins, the following shall be verified to ensure compliance:					
a. Proportions of site-prepared mortar.	—	X	—	—	Art. 2.6A
b. Construction of mortar joints.		X			Art. 3.3B
c. Location of reinforcement and connectors.		X			Art. 3.4, 3.6A
d. Prestressing technique.	—	X	—	—	Art. 3.6B
e. Grade and size of prestressing tendons and anchorages.	—	X	—	—	Art. 2.4B, 2.4H
2. The inspection program shall verify:					
a. Size and location of structural elements.	—	X	—	—	Art. 3.3G
b. Type, size and location of anchors, including other details of anchorage of masonry to structural members, frames or other construction.	—	X	—	Sec. 1.2.2(e), 2.1.4, 3.1.6	—
c. Specified size, grade and type of reinforcement.	—	X	—	Sec. 1.12	Art. 2.4, 3.4
d. Welding of reinforcing bars.	X	—	—	Sec. 2.1.10.6.2, 3.2.3.4(b)	—
e. Protection of masonry during cold weather (temperature below 40°F) or hot weather (temperature above 90°F).	—	X	Sec. 2104.3, 2104.4	—	Art. 1.8C, 1.8D
f. Application and measurement of prestressing force.	—	X	—	—	Art. 3.6B
3. Prior to grouting, the following shall be verified to ensure compliance:					
a. Grout space is clean.	—	X	—	—	Art. 3.2D
b. Placement of reinforcement and connectors and prestressing tendons and anchorages.		X		Sec. 1.12	Art. 3.4
c. Proportions of site-prepared grout and prestressing grout for bonded tendons.		X		—	Art. 2.6B
d. Construction of mortar joints.		X		—	Art. 3.3B
4. Grout placement shall be verified to ensure compliance with code and construction document provisions.	X	—	—	—	Art 3.5
a. Grouting of prestressing bonded tendons.	X	—	—	—	Art. 3.6C
5. Preparation of any required grout specimens, mortar specimens and/or prisms shall be observed.	X	—	Sec. 2105.2.2, 2105.3	—	Art. 1.4
6. Compliance with required inspection provisions of the construction documents and the approved submittals shall be verified.	—	X	—	—	Art. 1.5

For SI: °C = (°F - 32)/1.8.
a. The specific standards referenced are those listed in Chapter 35.

TABLE 1704.5.3
LEVEL 2 SPECIAL INSPECTION

INSPECTION TASK	FREQUENCY OF INSPECTION		REFERENCE FOR CRITERIA		
	Continuous during task listed	Periodically during task listed	IBC section	ACI 530/ ASCE 5/ TMS 402[a]	ACI 530.1/ ASCE 6/ TMS 602[a]
1. From the beginning of masonry construction, the following shall be verified to ensure compliance:					
a. Proportions of site-prepared mortar, grout and prestressing grout for bonded tendons.	—	X	—	—	Art. 2.6A
b. Placement of masonry units and construction of mortar joints.	—	X	—	—	Art. 3.3B
c. Placement of reinforcement, connectors and prestressing tendons and anchorages.	—	X	—	Sec. 1.12	Art. 3.4, 3.6A
d. Grout space prior to grouting.	X	—	—	—	Art. 3.2D
e. Placement of grout.	X	—	—	—	Art. 3.5
f. Placement of prestressing grout.	X	—	—	—	Art. 3.6C
2. The inspection program shall verify:					
a. Size and location of structural elements.	—	X	—	—	Art. 3.3G
b. Type, size and location of anchors, including other details of anchorage of masonry to structural members, frames or other construction.	X	—	—	Sec. 1.2.2(e), 2.1.4, 3.1.6	—
c. Specified size, grade and type of reinforcement.		X	—	Sec. 1.12	Art. 2.4, 3.4
d. Welding of reinforcment.	X	—	—	Sec. 2.1.10.6.2, 3.2.3.4(b)	—
e. Protection of masonry during cold weather (temperature below 40°F) or hot weather (temperature above 90°F).	—	X	Sec. 2104.3, 2104.4	—	Art. 1.8C, 1.8D
f. Application and measurement of prestressing force.	X	—	—	—	Art. 3.6B
3. Preparation of any required grout specimens, mortar specimens and/or prisms shall be observed.	X	—	Sec. 2105.2.2, 2105.3	—	Art. 1.4
4. Compliance with required inspection provisions of the construction documents and the approved submittals shall be verified.	—	X	—	—	Art. 1.5

For SI: °C = (°F - 32)/1.8.
a. The specific standards referenced are those listed in Chapter 35.

TIMBER CONSTRUCTION MANUAL

FOURTH EDITION
1994

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Englewood, Colorado

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1.6 HANDLING, STORAGE, ERECTION, AND SEASONING OF STRUCTURAL TIMBER

The erection of structural timber framing requires experienced erection crews and adequate lifting equipment to protect lives and property and to assure that the framing is not improperly assembled or damaged during handling. The unloading and storage of structural timber framing before erection also demands care and good judgment. It is suggested that a shipment of structural timber framing, on receipt at the job site, be checked for tally and damage. The following general precautions apply.

1.6.1 Precautions During Unloading

Structural timber framing is subject to surface marring and damage when not properly handled and protected. At the erection site, the following precautions are suggested:

1. Lift members or roll them on dollies or rollers out of railroad cars; do not drag or drop them. Unload trucks by lifting from the truck; do not dump or drop members.
2. If unloading with lifting equipment, use wide fabric or plastic belts or other slings that will not mar wood. If chains or cables are used, provide protective blocking or padding to sharp edges or sharp corners.
3. Guard against soiling, dirt, footprints, or abrasions. If members are wrapped, avoid tearing or damaging the protective material.

1.6.2 Precautions During Storage

If structural timber framing is to be stored before erection, it should be placed on blocks well off the ground, and individual members should be separated by strips so that air may circulate around all four sides. The top and all sides of storage piles should be covered with moisture-resistant material. Clear polyethylene films should not be used because wood members are subject to bleaching from sunlight. Individual wrappings should be slit or punctured on the lower side to permit drainage of water that may have accumulated. Water-resistant wrapping used for the in-transit protection of glued laminated members should be left intact until the members are enclosed within the building. If wrapping has to be removed at certain connection points during the erection, it should be replaced after the connection is made. If it is impractical to replace the wrapping, all of it should be removed to avoid the nonuniform appearance caused by sun and weather exposure.

1.6.3 Precautions During Erection

1.6.3.1 Assembly

Trusses are usually shipped partially or completely disassembled and are assembled on the ground at the site before erection. Arches, which are generally shipped in halves, may be assembled on the ground or connections may be made after the half arches are in position. When trusses and arches are assembled on the ground at the site, they should be assembled on level blocking to permit connections to be fitted properly and tightened securely without damage. The end compression joints should be brought into full bearing and compression plates installed where specified.

Before erection, the assembly should be checked for prescribed overall dimensions, prescribed camber, and accuracy of anchorage connections. Erection should be planned and executed in such a way that the close fit and neat appearance of joints and the structure as a whole will not be impaired.

Anchor bolts should be checked prior to the start of erection. Before erection begins, all supports and anchors should be complete, accessible, and free of obstructions. The weights and balance points of the structural timber framing should be determined before lifting begins so that proper equipment and lifting methods may be employed. When long members or timber trusses of long span are raised from a flat to a vertical position preparatory to lifting, stresses entirely different from the normal design stress may be introduced. The magnitude and distribution of these stresses will vary, depending on such factors as the weight, dimensions, and type of member. A competent rigger should consider these factors in determining how much suspension and stiffening, if any, is required and where it could be located.

1.6.3.2 Bracing

All framing must be true and plumbed. Permanent bracing is bracing so designed and installed as to form an integral part of the final structure. Erection bracing is bracing installed to hold the framing in a safe position until sufficient permanent bracing is in place to provide full stability. Proper and adequate temporary erection bracing is introduced whenever necessary to take care of all loads to which the structure may be subjected during erection, including equipment and its operation. This bracing is left in place as long as may be required for safety. Part or all of the permanent bracing may also act as erection bracing. Erection bracing serves to plumb the framing during erection and gives it adequate stability to receive purlins, joists, and roofing materials. It may include sway bracing, guy ropes, tieing off framing nearest to end walls, steel tie rods with turnbuckle take-ups, struts, shoes, and similar items. As erection progresses, bracing is securely fastened in place to take care of all dead load, erection stresses, and normal weather conditions. Excessive concentrated construction loads, such as bundles of sheathing, piles of purlins, roofing, or other materials, should be avoided.

1.6.3.3 Final Alignment

Final tightening of alignment bolts should not be completed until the structure has been properly aligned.

1.6.3.4 Removal of Temporary Bracing

Temporary erection bracing should be removed only after diaphragms and permanent bracing are installed, the structure has been properly aligned, and connections and fastenings have been finally tightened. Retightening of con-

nections prior to final completion or closing in of inaccessible connections is recommended.

1.6.3.5 Field Connections

The joining, holding, and welding of steel connections in the field are performed according to the requirements for shop work of such operations, except where such requirements apply to shop conditions only. Steel connections should comply with the specifications of the American Institute of Steel Construction (18) and the American Welding Society (19).

1.6.3.6 Protection of Field Cuts

All field cuts of timbers should be coated with an approved moisture seal if the member was initially coated unless otherwise specified. All field framing is done in accordance with the requirements of shop practice except where such requirements apply to shop conditions only. If timber framing has been pressure treated, field framing after treatment must be avoided or at least, insofar as possible, held to a minimum. When field cuts in pressure-treated material are unavoidable, additional treatment should be provided in accordance with AWPA Standard M4 (10).

1.6.3.7 Protection Against Moisture

During erection operations, all timber framing that requires moisture content control, whether sawn or glued laminated timbers, should be protected against moisture pickup. Any fabricated structural materials to be stored for an extended period of time before erection should, insofar as is practicable, be assembled into subassemblies for storage purposes.

1.6.3.8 Seasoning Period

Heat should not be fully turned on as soon as the structure is enclosed; otherwise, excessive checking may occur due to rapid lowering of the relative humidity in the building. A gradual seasoning period at moderate temperature should be provided.

US Historical Structure Examples

from Historical Building Construction, Donald Friedman, 1995.

1835 Obadiah Parker House, New York, Parker designer, demolished. House walls were monolithic concrete, probably with natural lime cement.

1841 [Old] Merchants' Exchange, 55 Wall Street at William Street, New York, Isiah Rogers architect, heavily modified 1907, landmarked. Monolithic all-masonry construction.

1853 New York Crystal Palace, George Carstensen and Charles Gildemeister architects, burned 1858. Cast-iron columns, wrought-iron arch ribs and truss girders, arched trusses, and all-glass curtain wall, portal bracing for lateral load.

1854 Harper & Brothers Building, 331 Pearl Street on Franklin Square, New York, John Corlies architect, James Bogardus engineer, James L. Jackson foundry builders, demolished 1925. Brick jack arch floor topped with concrete spanned between wrought-iron floor beams, supported by cast-iron and wrought-iron bowstring truss girders and cast-iron columns. The floor beams were the first lot of wrought-iron beams rolled in the United States, by the Trenton Iron Works. One cast-iron facade, brick side walls and rear.

1860 United States Warehousing Company grain elevator, Brooklyn, George Johnson engineer, Architectural Iron Works builders, demolished. Cast-iron frame supporting brick curtain wall, six stories high, cross-braced in all bays by wrought-iron rods.

1860- Watervliet Arsenal, Watervliet, New York, Daniel Badger designer, Architectural Iron Works building, standing. First all-iron building, 100 feet by 196 feet, cast-iron walls and columns, wrought-iron roof trusses, cast-iron girders with wrought-iron tension rods.

1871 Grand Central Depot, 42nd Street and Park Avenue, New York, John B. Snook architect, Isaac C. Buckhout engineer, Wilhelm Hildenbrand engineer for train shed, Architectural Iron Works builders, altered 1989, demolished 1913. First balloon shed in United States: wrought-iron truss barrel vault for train shed, covered with glass and galvanized iron.

1871 William E. Ward House, Port Chester, Rober Mook architect, Ward designer and builder, standing. First reinforce-concrete building in the United States, entire building monolithic, beams reinforced with wrought-iron I's connected for shear transfer, 3½-inch-thick floors reinforced with rods, 2½-inch-thick partitions reinforced with rods, hollow cylinder columns reinforced with hoops.

1875 Tribune Building, Park Row at Nassau Street, New York, Richard Morris Hunt architect, demolished 1966. Probably highest bearing-wall building in New York at 260 feet high. First tower-type building downtown. Wrought-iron beam floors.

1883 Statue of Liberty, Bedloe's Island, Frederic Auguste Bartholdi architectural designer, Gustave Eiffel engineer, Keystone Bridge Company and D. H. King Contracting builders, landmarked. First full-braced frame in New York, first use of steel columns in the United States, early use of concrete in foundation.

1885 Home Insurance Building[1], Chicago, William LeBarron Jenny, architect and engineer, demolished. Main building 138 feet high, 180 feet with 2 additional floors added later. Construction halted temporarily when building officials were concerned that the building weight was only one-third that of a comparable masonry structure.

1891 Monadnock Building[1] 53 West Jackson Blvd, Chicago, Burnham & Root architects (north), Holabird & Roche architects (south), standing. 197 feet high. The northern half is the last Chicago skyscraper built using load-bearing masonry wall construction with walls of six feet thick at the base.

1892 Manhattan Life Insurance Building, 64-68 Broadway, New York, Kimball & Thompson architects, C. O. Brown engineer, demolished. 67 feet by 119 feet, main building 254 feet high, tower 348 feet high. Tallest building in New York when built, first caisson use on a building anywhere (caissons were used in bridge and tunnel construction as early as 1850s in Europe, 1870s in the United States), fifteen caissons 55 feet below grade, 35 feet below open excavation, cantilevered built-up girders in foundations.

1895 American Surety Building, 96-100 Broadway, New York, Bruce Price architect, standing. First complete skeleton frame in New York, twenty stories and 303 feet high, 85 feet by 85 feet, Z-bar columns, wind braced with rods, caissons to rock 72 feet below curb elevation.

1899 Carson, Pirie, Scott and Company Building[1], 1 South State Street, Chicago, Louis Sullivan, architect, standing. Steel structure allowed for increased window area.

[1] Wickipedia: http://en.wikipedia.org/

1903 Flatiron Building, 175 Fifth Avenue, New York, D. H. Burnham & Company architects, Corydon Purdy engineer, landmarked. Steel frame with portal bracing and knees at all wind girders, tallest building in the city when completed.

1907 Monolith Building, 45 West 34th Street, New York, Howells and Stokes architects, standing. The first tall, reinforce-concrete building in the city, had limestone veneer for three floors at base, but exposed concrete above to full twelve-story height, stone veneer later replaced by stucco.

1909 [Old] New York Times Building, 42nd Street and Broadway, Eidlitz & McKenzie architects, standing altered. First tall building in the country to be designed using live-load reduction on its columns.

1913 Woolworth Building, 233 Broadway, New York, Case Gilbert architect, Gunvald Aus Company structural engineers, landmarked. Fifty-five stories, 760 feet, 6 inches high, tallest in the city when completed, caissons to rock, with moment-resisting portal frame, all-terra-cotta facade, facade rigidly connected to steel structure, no expansion joints provided, facade restoration required in mid-1980s, designed by Ehrnkrantz Group, over 20,000 panels had to be replaced with fiberglass-reinforced polymer concrete, approximately 100,000 reanchored.

1920 Electric Welding Company of America factory, Brooklyn, T. Leonard McBean engineer. Early use of structural welding, Brooklyn Department of Buildings required a full-scale load test before allowing construction.

1930 Chrysler Building[2], 42nd Street and Lexington Avenue, New York, William Van Allen, architect, standing. Briefly, the world's tallest building at 1047 feet to spire prior to the Empire State Building. Steel construction (riveted) with central core and steel-clad roof.

1931 Starrett-Lehigh Building, Eleventh Avenue and 26th Street, New York, R. G. Cory, W. M. Cory, and Yasuo Matsui architects, Purdy and Henderson engineers, landmarked. Flat slab concrete floors, on concrete columns with mushroom capitals at 21 feet on center above third floor, steel columns below; slabs are cantilevered to support curtain wall, nineteen stories high.

1931 Empire State Building, 350 Fifth Avenue, New York, Shreve, Lamb and Harmon architects, H. G. Balcom and Associates engineers, landmarked. Eighty-five stories and 1239 feet high, tallest building in the city when completed, full moment connection wind bracing, early use of aluminum cladding for top tower (dirigible mooring mast), ornament, and spandrel panels, early use of stainless-steel cladding in window edging.

1935 Hayden Planetarium, New York, Trowbridge and Livingston architects, Weiskopf & Pickworth engineers, standing. Early concrete shell dome, 3 inches thick, supporting projection screen.

1950 Secretariat Building of United Nations, near 42nd Street and First Avenue, New York, International Committee and Wallace Harrison architects, standing. First tall, glass curtain wall in New York.

1951 Lake Shore Drive Apartments[3]**,** 860-880 Lake Shore Drive, Chicago, Ludwig Mies van der Rohe, architect, standing. Steel frame with lateral resistance in the exterior (non-curtain) walls from steel plate welded to the frame.

1956 425 Park Avenue, New York, Kahn & Jacobs architects, Charles Meyer engineer, standing. Height 375 feet, "one of the tallest to be built to date with bolted connections," 150,000 field bolts up to 11/8 inches diameter x 7-inch grip in size; 200,000 shop rivets. Early use of two-man bolt crews.

1957 Seagram Building, 375 Park Avenue, New York, Ludwig Mies van der Rohe, Philip Johnson, and Kahn & Jacobs architects, Severud-Elstad-Kreuger engineers, landmarked. At thirty-eight stories and 520 feet high, tallest building using high-strength bolts when built. Shop connections riveted; unfinished bolts used for beam-to-girder connections.

1958 [Former] Union Carbide Building, 270 Park Avenue, New York, standing. At fifty-two stories and more than 700 feet high, tallest bolted frame when built.

1959 Kips Bay Plaza, 30th Street to 33rd Street, First Avenue to Second Avenue, New York, I. M. Pei & Partners and S. J. Kessler architects, August Komendant engineer, standing. Early exposed-concrete apartment houses, using load-bearing exterior walls of Vierendeel truss type.

1960 Western Electric Building, Fulton Street and Broadway, New York, Purdy & Henderson engineers, standing. At thirty-one stories, tallest steel frame with welded connections in the eastern half of the country when built.

[2] Wickipedia: http://en.wikipedia.org/

[3] Emporis Buildings: http://www.emporis.com/

1961 Chase Manhattan Building, Cedar Street and Nassau Street, New York, Skidmore, Owings & Merrill architects, Weiskopf & Pickworth engineers, standing. First glass curtain wall building over 800 feet high, sixty stories, largest building using solely interior bracing, steel rails and mullions mounted to structural frame.

1964 New York State Pavilion, Flushing Meadows Park, Queens, Philip Johnson and Richard Foster architects, Lev Zetlin engineer, standing empty. Early use of slip-forming to create freestanding concrete columns; roof is a bicycle-wheel cable truss.

1964 Marina City Towers[4], 300 North State Street, Chicago, Bertrand Goldberg, architect. Tallest reinforced concrete structures built at the time with 61 floors. Central load-bearing core with column and beam construction.

1965 CBS Building, 51 West 52nd Street, New York, Eero Saarinen architect, standing. Early concrete tube and core structure, thirty-nine stories and 491 feet high.

1968 Madison Square Garden, Seventh Avenue and 33rd Street, New York, Charles Luckman Associates architects, Severud Associates engineers, standing. 425-foot-diameter bicycle-wheel cable truss roof.

1969 John Hancock Building[4], 875 N. Michigan Avenue, Chicago, Fazlur Khah (Skidmore, Owings & Merrill) designer and engineer, standing. 1500 feet tall with external cross bracing to resist lateral loads as the predominant architectural feature for the tubular design.

1972 Transamerica Pyramid[4], 600 Montgomery Street, San Francisco, William Pereira architect, standing. 853 feet tall and was the tallest skyscraper west of the Mississippi River from 1972-1974. Constructed of reinforced concrete, it has a tapering shape from base to tip with two vertical “wings” at the upper stories.

1973 Willis (Sears) Tower[4], 233 South Wacker Drive, Chicago, Bruce Graham architect, Skidmore Owings and Merrill, engineers, standing. 1721 feet high to spire, and the world’s tallest building from 1973-2004. The design incorporates nine steel-unit square tubes in a 3 tube by 3 tube arrangement, with each tube having the footprint of 75 feet by 75 feet. This building was the first with this design.

1974 Avon Building[5], 9 W. 57th Street, New York, Skidmore, Owens and Merrill architects, standing. Lateral forces are resisted by a sloping base in the street direction (setback requirements), and by exposed, inset cross bracing in the narrow direction.

1975 Water Tower Place[4,6] , 845 North Michigan Avenue, Chicago, Loebl, Schlossman, Dart & Hackl and C.F. Murphy Associates architects, standing. Framed tube constructed of high-performance concrete. It was the world’s tallest concrete structure from 1975-1990.

1976 World Trade Center, near Church and Fulton streets, New York, Minoru Yamasaki and Emery Roth architects, Worthington, Skilling, Helle and Jackson engineers, tragically demolished. Early structural tube wind frame, of Vierendeel truss type, early pressure-equalized curtain wall.

1977 Citigroup (Citicorp) Center[4], 601 Lexington Avenue, New York, Stubbins Associates, Emery Roth & Sons architects, William LeMessurier engineer, standing. 72 feet long cantilever spans achieved with 114 feet high stilt-like columns that accommodated an existing church at the corner of the site. The system was not adequately designed for lateral loads when the connections were changed from welds to bolts, and was reinforced in 1978.

[4] Wickipedia: http://en.wikipedia.org/

[5] Skidmore, Owings and Merrill: SOM.com
[6] Emporis Buildings: http://www.emporis.com