

Structural Design

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Structural Design

A Practical Guide for Architects

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SECOND EDITION

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Preface

FMHX

THE THREE ISSUES PROMPTING THE CREATION of this second edition are the adoption or impending adoption of the International Building Code as the standard for the United States and the almost exclusive use of LRFD (Load and Resistance Factor Design) for steel design. As one might expect, there are a number of other material specific code changes that have occurred since the first edition and these changes are also included.

We have also included the relevant shape information and load tables that you will need similar to those provided in the AISC Manual for Steel Construction. This means that you will no longer need to purchase this manual as a companion to "Structural Design". You will need the National Design Specification for Wood Construction, published by American Wood Council, 2005 edition. This specification has been simplified into two basic manuals that are best used in their original form.

Wood and Masonry design procedures are just beginning to make the change to LRFD procedures and the codes are acknowledging these changes by creating adjustment factors in lieu of full fledged revisions. For this reason, we are maintaining working stress or allowable stress (ASD) procedures for these materials. We realize that this switching between to different systems can be confusing, but the alternative is even more confusing, in our opinion.

The IBC procedures for earthquake and wind loadings are "significantly" more complicated than the previous versions. For this reason, we have included the UBC procedures for wind load so that architects will have the option of using the much simpler UBC procedures at least for preliminary design. The IBC equivalents are included on our web site for those of you who have a morbid interest.

Some changes have occurred in Concrete design

to bring it more into agreement with the IBC load criteria, and better understanding of the material in context has created some relevant changes in the Strength Reduction or Capacity Reduction factors.

We have greatly expanded the explanations of example problems so that the logic may be more easily followed.

There has been a significant progress in the introduction of metric tables in the AISC manual for those of you who are interested, however, we do not have complete reference material for all of the tables. To mitigate that deficiency, we have worked the problems in metric and for the final answers converted those answers to SI units so you could use the included tables as much as possible. It is similarly worth noting the many of the metric conversions are not "hard" conversions of actual sizes. We've generally included the soft" conversions i.e. 2 X 4's are internationally know as 38 X 89, and 12, 16, and 24 in. spacings are 300, 400 and 600mm. This may be confusing, but it is generally accepted practice.

The web offers the unique opportunity to present additional information as well as constant updates. We are introducing a web site [www.who knows anything](http://www.who-knows-anything.com) that will have additional instructor and student areas with information relative to teaching, "solved" example problems, as well as more complex information regarding rigid frames and statically indeterminate structures.

We also recommend students refer to building construction manuals, such as the Architectural Graphic Standards, and The Architect's Studio Companion for types of structural systems, construction details, and graphic conventions. Other useful references that can complement this text are the International Building Code and trade organizations listed in the Appendix with their web address.

In the Beginning: The Premise

FMHX

One of the most important aspects of structural design for architects is the selection and configuration of structural systems. After the system is pragmatically conceived, individual members can be designed within that "system". While we would like to be able to give you a set of "rules" associated with this decision, they simply don't exist. We will identify, in each material section, those considerations associated with that particular material. If you consider those issues, consult with a competent structural engineer and remember that the structure and the architecture are inseparable partners, you can arrive at reasonable solutions. Finally, don't be fooled by the 9 decimal point answers your calculator will give you; if you start with faulty assumptions, you end with FAULTY NINE decimal point answers.

Structural design books and manuals for the practical designer are few. They are also frequently written in a manner which leads a designer to believe that approaches to the design of various components of a structure are unique to each component. To help alleviate some of your anxiety, this manual will focus on the idea that there are basically three elements you can design in structures: a bending member, a compression member and a tension member. This is made slightly more complex by combining the options and adding bending + compression and bending + tension. This probably seems too simple, but as we go on, focus on this idea and you'll discover it's a fair representation.

To design these three or five, if you must, members, we'll also focus on the use of the two commandments of structures:

$$\mathbf{F = P/A \quad \text{and} \quad F = M/S \text{ or } M/Z}$$

Most of the infinite formulas presented in structural texts and manuals are an elaboration or reinterpretations of these two simple formulas.

The level of math required is simple algebra and trigonometry. You need to be able to solve a simple

quadratic once in a while, using either the quadratic equation or completion of the square, whichever is easier.

The most important thing to remember is that you've been dealing intuitively with structural principles all of your life; you walk, you carry things, you bend things, you break things. All we are going to do is quantify what's happening. The math is simple and logical; don't let it frighten you.

Our purpose is not to make you a structural engineer, but to give you an appreciation and understanding of the considerations that are necessary for the successful completion of any architectural project.

If you wish to be a complete designer, you must be able to integrate technical issues into your design vocabulary. You will ultimately be obligated to show that understanding and ability through the registration examination. You will also be confronted by situations that will necessitate that you be able to do simple structural calculations either for a small project or preliminary calculations for larger projects. It is assumed the reader knows statics. This understanding is the foundation for understanding structural systems.

When you understand the approaches to structural design for small buildings, you should be able to transfer that knowledge to larger situations. The principles are basically the same; the scale differs. With this knowledge you can talk intelligently with your structural consultants. Remember, you are responsible for the work of your consultants, if you allow them to work in the shadow of an intellectual vacuum; you are placing your future in their hands.

We can classify structural systems into three general categories, each associated with a type of space. You will immediately notice that these systems are basically versions of the original wood and stone systems that are the foundation for what we do today in more sophisticated materials.

Frame

In its simplest form, a frame would be two vertical poles with a horizontal pole spanning between them. The vertical poles are frequently pure compression elements; a pile of stones would work as well. The horizontal pole has several different types of forces acting in it (bending, shear and deflection) requiring that it have both tensile and compressive capacity. Even taking this simple principle, you can understand why the columns on the Parthenon are so closely spaced. Stone has little tensile capacity, so it will only span short distances as a beam. The combination of close column spacing and wide capitals reduced the

span to match the capacity of the stone. (The Golden Section wasn't the only driving force!)

Architecturally, the most forceful use of the ancient wood frame can be found in early Japanese architecture. The best known example of indigenous Japanese architecture is the Ise shrine (Fig. P.2) originally constructed around the third century AD. The wood columns are set in the ground, so they act as cantilevers for earthquake loads. In later times, the columns would rest on a stone base, consequently the columns were not cantilevers and the earthquake resistance was ensured by a very elaborate "rigid" joinery at the connection of columns and beams, as illustrated in the 8th century To shodai-ji temple Fig. P.3.



Fig. P.1 Parthenon



Fig. P.2 Ise Shrine

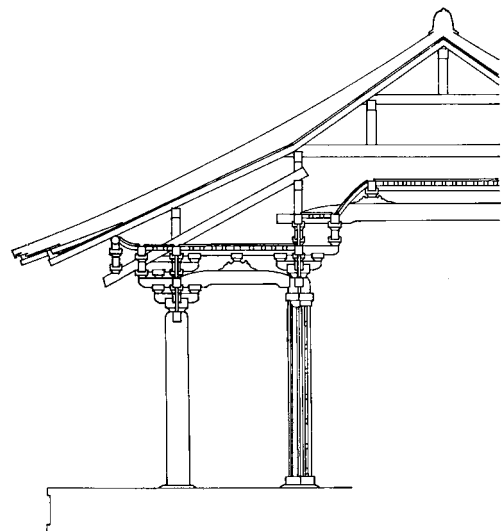


Fig P.3 Toshodai-ji section

Arch and dome

The vault created by a series of arches is an axial space that can theoretically extend indefinitely at the two ends, and at the same time, has a well-defined edge at the spring of the arches. The modern arched structures have a formal analogy with the masonry

frame of the Gothic cathedrals which may well have been inspired by the interlacing of tree canopies, and a construction analogy with timber ships, which are upside down vaults (Fig. P.4).

If you were to take an arch and rotate it 180°, you would create an enclosed volume of space we have come to recognize as a dome. These domes can be

Fig. P.4 United States Frigate Constitution

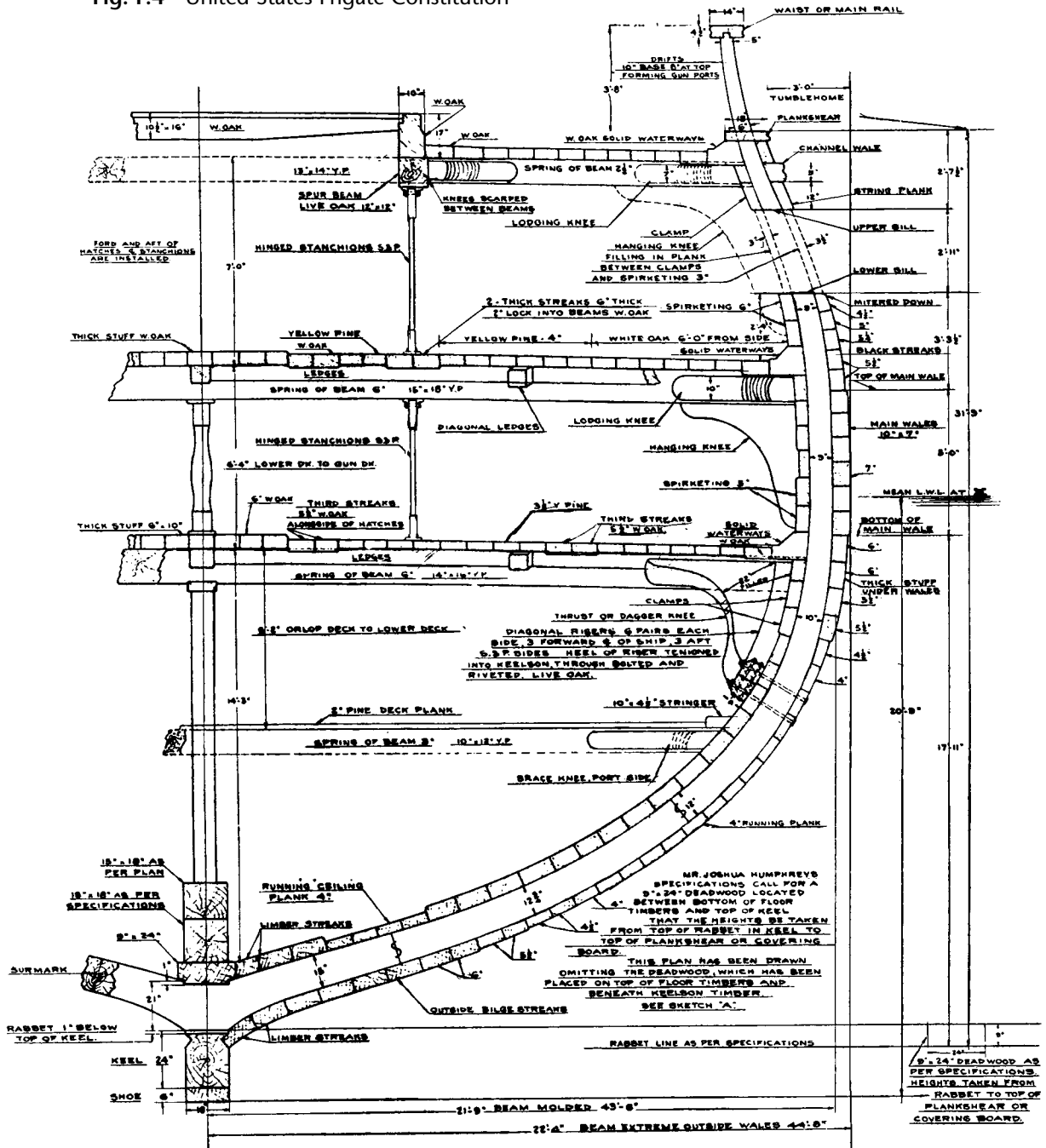




Fig P.5 Roman Coliseum

cut and redefined as the half dome forms we find in many masonry religious structures. From early third century BC. Buddhist tombs and the Pantheon to the exhibition halls of Nervi and the simple igloo of the Eskimo; all are variations of an elementary compression structure.

One of the most commonly recognized masonry structures in the world is the Colosseum in Rome. It combines the arch systems of the Roman aqueducts and the round curves of domes to create a monument that seemingly will live forever. The form may have been both for view lines and to resist the forces of the ropes that once supported the removable roof, like many contemporary sports stadiums.

Wall

Early stone buildings, log cabins, or modern panelized construction are all systems resulting in cellular spaces. These are systems are still widely used in conventional or manufactured housing. A modular house can consist of several sections built in the factory and assembled on site. All walls have the potential of becoming load-bearing, so the space is not as flexible as in the Japanese house, based on the frame or as universal as that defined by the arch or dome. The wall has the advantage over the frame of combining in one panel element load-bearing and enclosure functions, making a rational use of material with modern construction technologies. It can easily be constructed in an area that has lots of stone or, as we learned from plains settlers in American history classes, from great slabs of sod.

These systems can teach us several valuable lessons. Look at indigenous materials; make use of those systems which require minimal manufacturing and



Fig. P.6 Masonry bearing wall housing complex in Venice, Italy

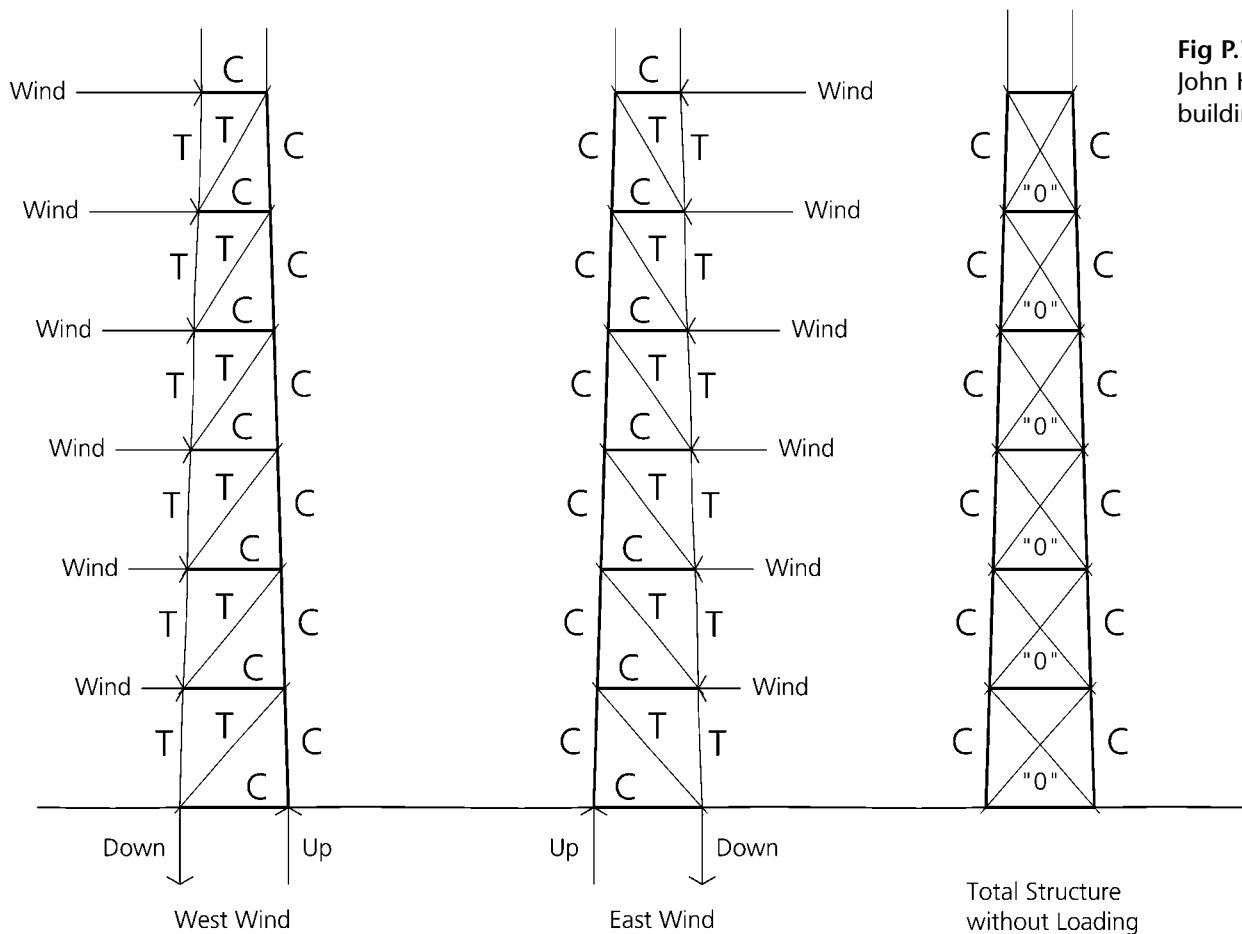


Fig P.7
John Hancock
building

are readily available. Consider the environmental context; structures of great thermal mass are excellent choices in areas of rapid and regular changes in temperature while lighter structures which do not retain heat are better in areas which have consistently high temperatures. It is not an accident that most indigenous architecture is more environmentally sensitive than that which our "technology" can support. Don't be a salmon; work with nature, don't always swim upstream. Although these examples may start with domestic scale structures, their systems are the basis for what we do today at almost every scale. The John Hancock Building in Chicago is partially based on the structural idea of a farm gate: a cantilevered element supported from one end and "trussed" to make it light, yet strong.

Modern sports arenas are frequently large scale examples of dome structures you might build with snow in your backyard in the winter or in other cases

simply big "cold-air" balloons. This book attempts to address structures both on a conceptual level and on a numerical level. We acknowledge that some types of structural design are more likely to be attempted by architects than others. For this reason, we discuss wood and steel in rather laborious detail and present reinforced concrete in a more simplified (believe it or not) version. In most cases, you can do more than you probably should do. A good structural consultant will provide the architect with options based on "structural" considerations. It is your job to coordinate these options with the "architectural" considerations as well as those of the mechanical consultants. Structural consultants will ultimately provide the best solutions to the specifics of your design approaches, but a good architect has to be able to understand the engineer's language. Bring them in at the beginning of the design process and use their conceptual thinking not just their calculators.

PART ONE

STEEL

PN

PT



FG Fig S1 San Francisco International airport.

Introduction: Understanding Loads

H1 1.1 LOADS

TXT1

In statics, loads are forces acting on structural components, and are represented as uniform or varying forces or points. In practice, they represent the weights of the building materials used to construct the building (Fig 1.1), the weights of the people and equipment which will occupy the building, and the forces of nature that the building will be exposed to during its life.

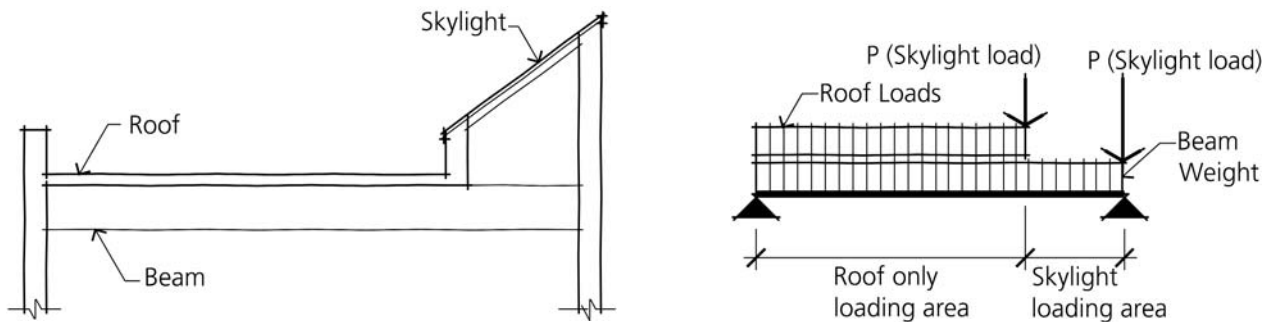
TXT

Material weights are gravity loads which act down, surprise! People and equipment are primarily gravity loads, but in some instances may cause forces which act in some other direction. An example would be a piece of horizontally moving equipment which suddenly comes to a stop causing a horizontal force to be induced in the structure. Highway bridges are constantly subjected to this loading condition.

Wind forces are primarily horizontal, but can induce vertical forces when blowing over surfaces. Note that wind passing over an airplane wing caused the upward lift that keeps the plane in the air. Similar conditions can be induced in the roof structure of a building.

Earthquakes by contrast are wavelike forces which have both horizontal and vertical components; however, the horizontal force component is typically the most destructive of the two since most structures are designed to be primarily vertical load carrying systems.

The effect of these forces is to induce states of stress and deformation or deflection in the structure. Deflections are often the governing factors in the design of a structural system. Obviously, a structure fails when it collapses, however excessive deflection which causes damage to finishes or other building



FG Fig. 1.1a, 1.1b Structural loading diagram of an architectural condition

components without collapse is also defined as a structural failure.

Building codes categorize these loads into two classifications: dead loads and live loads. Dead loads are those permanent loads generated by the construction system. Live loads are those non-permanent loads which are applied to the structure after it is completed. Some loads may be in either category depending upon their time of application. It is essential to understand the construction sequence of the building, and to design for deflection caused by live loads introduced after the construction is complete. For example, a typically permanent (dead) load such as an HVAC (Heating, Ventilating, Air Conditioning) unit should be considered a live load if installed after ceiling finishes are in place, since it would cause deflection of the ceiling/floor components similar to that created by snow on a roof or human occupancy of a level above. This may occur even if a building component is assumed to be in place prior to "finishes". A manufacturing delay, a labor dispute, a delivery problem or even a design change may be responsible for an out-of-sequence installation which could have serious deflection implications.

An objective of the codes is to limit the deflection of structural members to the extent that they would

not damage the connected non-structural components or affect the functionality of the building. In the 2003 International Building Code, as with previous codes, limitations are imposed on deflections due to both dead and live loads.

This shouldn't suggest that dead loads don't cause deflection. The dead load deflection of the structure isn't considered in some cases since it is compensated for during the construction process. For example, the ceiling finish which is (obviously) installed after the horizontal framing is enclosed will be installed "level". Any dead load deflection which exists in the framing will be hidden by adjusting the finished. The possible exception to this is in the roof construction, consequently care must be taken to assure that "flat" roof systems have no water retention areas—ponding—as we mentioned earlier.

The load values to be used depend on the use or occupancy of the structure. Typically loads are floor loads, roof loads, and wind loads acting on walls and roofs, and are given in lb/sqft [or psf] / KN/m^2 . For example, floor loading for offices is 50psf [2.39 KN/m^2], for school classrooms; 40psf [1.92 KN/m^2]. In both cases the buildings will have corridors or circulation spaces on each level which will have a live loading of 80 to 100psf [3.82 to 4.78 KN/m^2]. As this

Table 1.1 Maximum Allowable Deflection for Structural Members^c

Construction	Live Load	Snow or Wind	Live Load + K ^a Dead Load
Roof members supporting: ^b			
rigid ceiling	L/360	L/360	L/240
flexible ceiling	L/240	L/240	L/180
no ceiling	L/180	L/180	L/120
Floor members	L/360		L/240
Exterior/interior walls:			
rigid finishes	—	L/240	—
flexible finishes	—	L/120	—
Farm buildings	—	—	L/180
Greenhouses	—	—	L/120

^aThe value for K varies for each material discussed in this book. In the first section, STEEL, the value for K is 0. This means that the maximum allowable deflection is ONLY a function of LL and is limited to L/360 for most conditions that architects encounter. For WOOD, the combined condition uses K = 0.5

^bNote that a "flat" roof member, one with a slope of ¼ in per foot or less, should be designed or L/360 regardless of ceiling finish to eliminate the possibility of water ponding in the deflected areas.

^cFor cantilevered members, L is taken as twice the actual length of the cantilever.

suggests, structures are subjected to a variety of live loading conditions and the design must work for the worst case scenario; those loading conditions which cause the worst effect on the criteria being investigated.

1.4.2 Earthquake design—equivalent lateral force procedure

The IBC defines the basic structural systems, for the purpose of earthquake design, for application of R factor. Within each of these categories there are a number of different options, so this is not really an architecturally limiting consideration.

1. Bearing wall system;
2. Building frame system;
3. Moment-resisting frame;
4. Dual system with special moment frames
5. Dual systems with intermediate moment frames.
6. Shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls.
7. Inverted pendulum systems
10. Structural steel systems not specifically detailed for seismic resistance.

In arriving at the seismic factor, the code takes into account the degree of probability of a large earthquake, the type of building occupancy, the flexibility of the structure and the physical site characteristics. Using symbols to represent these considerations, the seismic factor in it's most simplified version is given by the following formula. This is known as the equivalent lateral force procedure:

Where:

V_b = The total base shear developed in kips

$$C_s = \frac{S_{ps}}{R/I_E} \text{ (Seismic response coefficient)}$$

S_{DS} = The design elastic response acceleration at short period, $S_{DS} = \frac{2}{3} S_{MS}$

$$S_{MS} = F_A S_S$$

F_A is a function of site classification and location which determine SS and is listed in Table 1.19.

R = Response modification factor from Tables 1.22a thru 1.22h

I_E = The importance factor from Table 1.7

W_S = Effective "seismic" weight of the building in kips, consisting of the dead load and the following proportions of live loads:

1. In storage areas, 25% of the live load with any applicable reduction factors
2. In office structures or others that have a partition load, use the actual partition weight or a minimum of 10 psf [0.48kN/m²], whichever is greater.
3. The total weight of any permanent equipment — mechanical equipment, fire suppression Water storage, etc.
4. 20 percent of the flat roof snow load in areas where the ground snow load exceeds 30 psf [1.44kN/m²]

If we were to substitute some of these factors into the original formula we would get something that is a bit more recognizable in terms of the relevant factors:

$$V_b = \frac{S_{ps}}{R/I_E} W_S$$

This factor acknowledges the location, and expected seismic response (acceleration) of that location, the structural system, the importance of the facility, and the seismic weight of the entire building, all factors that you would expect to have an effect on the base shear.

This may seem pretty complicated and at first glance it is, but with a step by step methodical approach it's not too bad. We'll do that after we discuss a few other items.

$$C_{s(max)} = \frac{S_{D1}}{T(R/I_E)}$$

$$C_{s(max)} = 0.044 S_{DS} I_E$$

S_{D1} = The design elastic response acceleration at a 1.0 second period,

$$S_{M1} = F_V S_1$$

F_V may be found in Table 1.20 where you will see that it is also a function of the site classification and location which determine SS and is listed in Table

S_1 is found on the map figure

T is the fundamental period of the structure and may be established by (the IBC states) $T_A = T$.

H3 1.3.3.1 General building design—UBC Methodology

Common sense tells us, wind velocity is reduced by friction from the irregularities of the terrain. Above certain heights the air can move without being affected by ground conditions. The UBC refers to this height as the "gradient height" and the velocity is called q "wind speed".

The UBS defines three degrees of exposure, almost identical to those of the IBC:

Exposure B has terrain with buildings, forest or surface irregularities 20 ft. [6m] or more in height covering at least 20% of the area extending one mile or more from the site.

Exposure C has terrain which is flat and generally open, extending one-half mile or more from the site in any full quadrant.

Exposure D represents the most severe exposure in areas with basic wind speeds of 80mph [130km/h] or greater and has terrain which is flat and unobstructed facing large bodies of water over one mile [1.6km] or more in width relative to any quadrant of the building site. Exposure D extends inland from the shoreline $\frac{1}{4}$ mile [400m] or 10 times the building height, whichever is greater.

The minimum basic wind speed for determining wind pressure can be found in Fig. 1.10 of the 1997 UBC reproduced following. You will note that this wind pressure map is significantly less "refined" than the IBC maps we have previously reviewed.

Architects have the responsibility to evaluate the best constructional solutions, in terms of cost, health, energy consumption for the building as well as for the community at large. Speed of construction, renew

UL

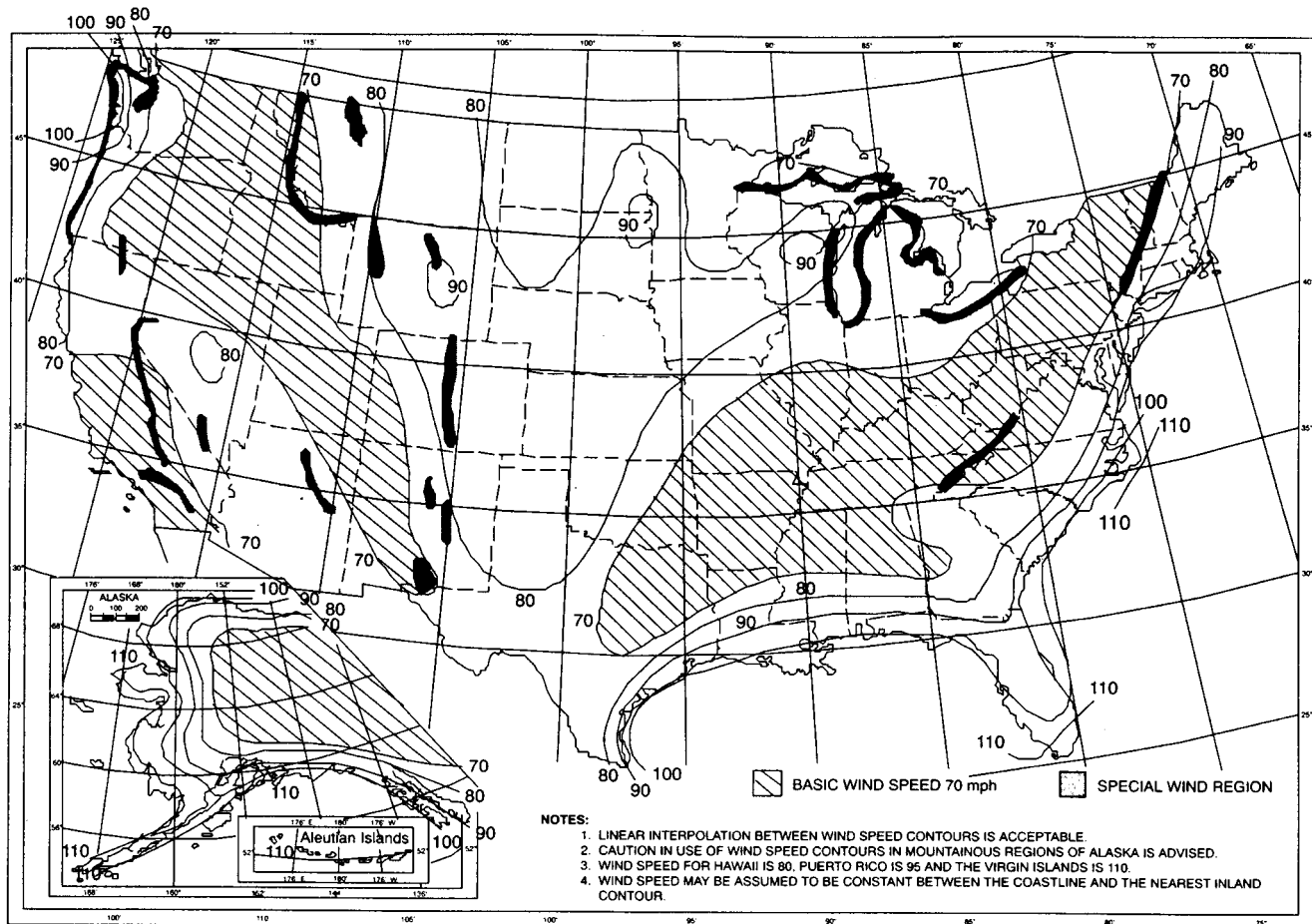


Fig. 1.10 Wind speed map for the United States. Source: adopted from the Uniform Building Code, 1997)

ability, and light weight make wood framing an economic system, which if properly constructed, has sufficient resistance to virtually any loading situation. The unique quality of wood to be able to sustain extreme overloads for short periods of time makes it particularly beneficial in situations that create extremes of loading—earthquakes and hurricanes. .

18.3 SELECTION AND CONFIGURATION OF WOOD SYSTEMS

H1

We told you not to be surprised when you saw this section, as a reference included in each material description..

Selection of a wood system involves a complex set of interrelated issues, so there is no single answer. You need to at least consider the following:

LUHX

SPATIAL REQUIREMENTS

What are the volumetric requirements of the function that the space will house? Is there a planning module or a leasing module that should be column-free or could be easily isolated?

LU

Parking garages, office structures, schools, hospitals, and retail establishments all have identifiable modules that are more desirable.

LUHX

SOIL CONDITIONS

What is the nature of the soil on the site?

LU

Chemical reactions with wood products?

Bearing capacities?

LU1

Wood structures are light structures: This may be an advantage for foundation design and a disadvantage for thermal balancing. Wood product do have a good inherent insulation quality, but you may still have some thermal transfer through structural components. You might look at the dirt collected on painted surfaces over joist or studs.

A wood structure will generally weigh less than its equivalent in steel and significantly less than its concrete counterpart. This could be a major consideration if the building is supported by weak soils, which would require an excessive investment in foundations.

Some common combinations of span/loading/mechanical integration characteristics for simple systems would include the following:

S4S lumber is available today in relatively short dimensions, 20 to 24ft. [6 to 7.2m], and is a common material for small buildings, typically single-family residences and other low-rise structures. However, small lumber pieces can be combined in structural systems such as trusses or lamella vaults to cover large spans, and manufactured components such as laminated wood members allow the production of beams and arches capable of spanning long distances. This ability is achieved by carefully utilizing the strength characteristics of wood in their most advantageous way and by eliminating the “flaws” that are naturally found in solid wood members. Knots, splits, shakes, etc., all reduce wood capacities below their ideal values. The use of short, thin pieces that are selected and edited for flaws is a double benefit in the design of laminated members and in the conservation of our wood resources.

Total System Cost. The same system can be more expensive in one material than another. The optimal construction cost may correspond to different configurations in different materials (e.g., certain structural spans that are economical for steel may not be economical for glulam). The cost economy has to be considered in relation to the total cost, since one should add to the cost of the structure consideration of finishes, building volume, gross floor area, and fire protection. Wood components can require fire or insect treatment or painting. Solid glulam beams do not allow ductwork to easily be run through the web, so additional building height may add to the cost. Heavy timber systems have an intrinsic fire resistance, but sprinkler system could be required by the fire code.

Environmental aspects. These are both global and local. Wood is a renewable resource, but the current rate of consumption is creating supply shortages and is leading to environmental protection measures worldwide. A lot of waste can be generated by design solutions that do not consider accurately timber properties and sizes. The construction of a new family residence can generate two to three tons of waste, part of which is wood cuttings; these do into landfills and are not recycled.

H4

H4

An example of this principle is the child in the swing we previously discussed. If the child leans or shifts their weight properly it will increase the amplitude of the "swing", while if the "timing" is off, this same shifting of mass will cause no additional acceleration and it may even stop the swing.

Several major concerns immediately come to mind when you put these issues all together and the code identifies them as well. Buildings which have either vertical or horizontal discontinuities are subject to serious problems during an earthquake. Some of the most easily understood discontinuities would include:

- A nonuniform distribution of mass: A clear example would be to use a long vertical metal rod and place an apple on it. As you subject to rod to a horizontal force and move the apple to different locations, you'll immediately note that the movement is reduced or amplified depending on the location of the apple relative to the base.
- Sections of a building having different degrees of stiffness: Using the steel rod from the previous example along with two others of different stiffnesses (diameter or cross-sections) held together at the base, subject them to a sharp horizontal force. In this case you'll see or can imagine that they would move at different rates. Imagine what would happen if there were parts of the building connected together. This differential in movement will immediately tear them apart.
- Irregular geometries: As an example you might look at an L shaped plan and imagine a horizontal force applied parallel to either leg. One portion of the building would have length of the leg resisting the horizontal force and the other would have the width of the leg resisting the force. Obviously one is much stronger (stiffer) than the other; consequently the point of connection between the legs would be trying to reconcile two different magnitudes of movement. Result? Crash!

The IBC 2003 and the ASCE 7 elaborate on these basic ideas, but in general, irregularities in form or mass distribution are not good ideas. Think about the Transamerica building in San Francisco. Is there a simple earthquake principle at work there?

1.4.2 Earthquake design—equivalent lateral force procedure

The IBC defines the basic structural systems, for the purpose of earthquake design, for application of R factor. Within each of these categories there are a number of different options, so this is not really an architecturally limiting consideration.

1. Bearing wall system;
2. Building frame system;
3. Moment-resisting frame;
4. Dual system with special moment frames
5. Dual systems with intermediate moment frames.

The problem in modern tall buildings built with a relative light, flexible steel frame is not just strength, but also serviceability; this means, in the case of horizontal loads; drift and vibrations caused by wind, earthquakes or other dynamic loads such as machinery.

1.3.1 Structural systems for lateral loads

Wind loads are transferred from the building envelope to the columns, bracing or shear walls system anchored to the foundations. Earthquake loads similarly are assumed to act at each floor level, and the floor structure and vertical framing must be capable of transferring them to the foundations. Fig. 1.5 illustrates how wind loads are resisted by vertical walls. Note that this diagram does not show all the loads on the building.

Horizontal elements:

- Beams in a rigid frame
- Trussed floor systems (horizontal cross-bracing)
- Floor diaphragms

Vertical elements:

- Columns in a rigid frame
- Trussed walls (vertical cross-bracing)
- Shear walls
- Rigid cores

A trussed floor is designed for horizontal loads essentially in the same way a truss is designed for gravity loads.

H2

DIS

LB

H2

LBHX

LB

EXAMPLE 1.2

We'll now work a different problem with an 80ft. tall slender office building which has a total weight (LL+DL) of 1423kips and a footprint of 40ft. x 100ft. Again using the UBC approach, but using the projected area method. This method is best used when evaluating the overall "overturning" forces on the building, which will likely become an issue in a taller structure with a relatively narrow base.

The building weighs 1423 kips and with a 40ft. base, assuming it will "topple" about the narrow dimension, a pretty clear option, it has an overturning resisting moment (Weight X moment arm) of $1423k (40ft./2) = 28,460ftk$. This is the force which would hold it in place under high wind loads.

The overturning forces can be found by calculating the overturning wind forces acting on the structure.

HORIZONTAL PRESSURE EXH1

Exposure Coefficients EXH2

- $C_o = 0.62$ 0–15ft. Table 1.14
- $C_o = 0.67$ 15–20ft.
- $C_o = 0.72$ 20–25ft. EXL1
- $C_o = 0.76$ 25–30ft.
- $C_o = 0.84$ 30–40ft.
- $C_o = 0.95$ 40–60ft
- $C_o = 1.04$ 60 - 80ft.

Pressure Coefficient EXH2

$C_p = 1.4$

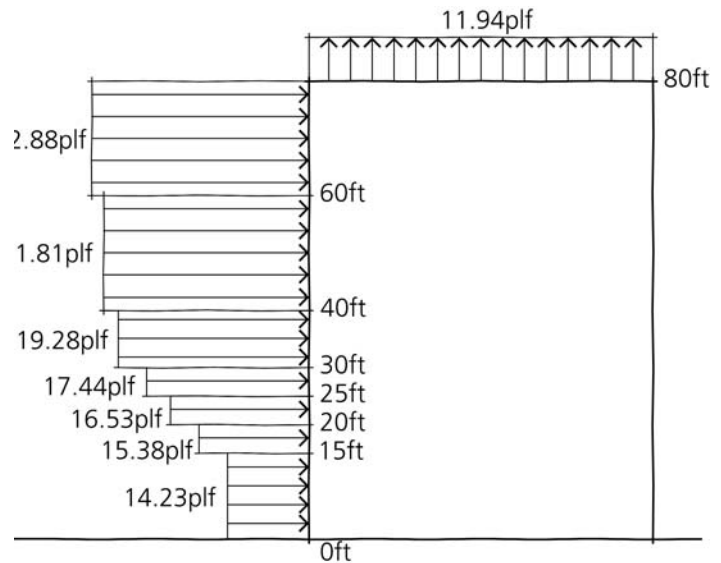


Fig. ex 1.2

TOTAL GROSS PRESSURE (INWARD + OUTWARD) EXH1

Basic wind speed from Table 1.18 = 80mph, therefore $V = 16.4psf$

$I_w = 1.0$ Primary frames and systems Table 1.17

Pressure EXL1HX

- 0–15ft. $P_1 = 0.62 \times 1.4 \times 16.4psf \times 1.0 = 14.23psf$
- 15–20ft. $P_2 = 0.67 \times 1.4 \times 16.4psf \times 1.0 = 15.38psf$ EXL1
- 20–25ft. $P_3 = 0.72 \times 1.4 \times 16.4psf \times 1.0 = 16.53psf$
- 25–30ft. $P_4 = 0.76 \times 1.4 \times 16.4psf \times 1.0 = 17.44psf$
- 30–40ft. $P_5 = 0.84 \times 1.4 \times 16.4psf \times 1.0 = 19.28psf$
- 40–60ft. $P_6 = 0.95 \times 1.4 \times 16.4psf \times 1.0 = 21.81psf$
- 60–80ft. $P_7 = 1.04 \times 1.4 \times 16.4psf \times 1.0 = 23.88psf$

EXAMPLE 6.1 Simple "Fully Laterally Supported" Beam

A-992 Steel, $F_y = 50.000\text{ksi}$

Span 26ft-0in $LL = 7,297\text{N/m}$

$DL = 2,189\text{N/m}$

+ beam weight estimated at 585N/m or about 60kg/m nominal mass.

This indicates the at the total deflection could be as much as 1.5 times the LL deflection indicating that it would only control if the total load was 1.5 time the LL. We can verify whether total load or live load will control by looking at our loading condition. Note that we are now using LRFD methodologies and will be introducing "load factors"

Design loads: $LL\ 7,297\text{N/m}(1.6) = 11,675\text{N/m}$

$DL\ (2,189\text{N/m} + 585\text{N/m})(1.2) = 3,329\text{N/m}$

Total design load = $15,004\text{N/m}$

$TL/LL = 10,071\text{N/m}/7,297\text{N/m} = 1.38 < 1.5$, so live load deflection controls.

Now we can proceed with the determination of the required physical properties of the beam.

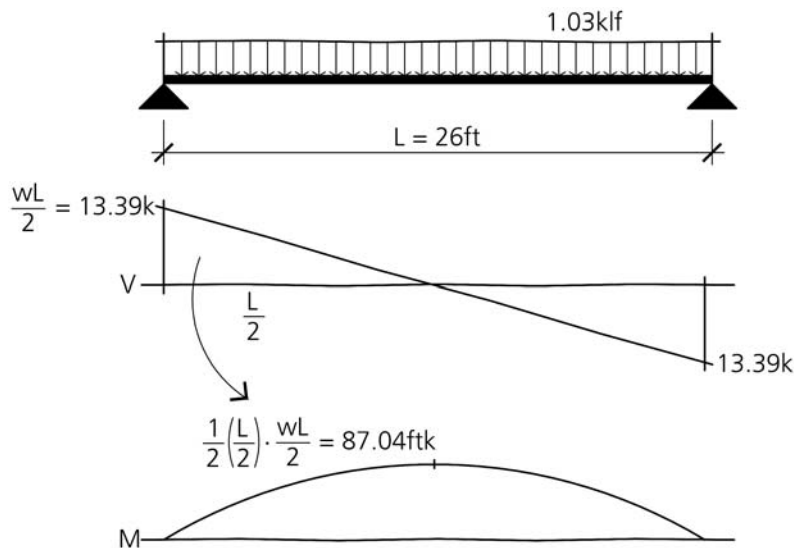


Fig. ex6.1.1

Since it is a simple span with a uniform load the design criteria are relatively straightforward.

Maximum shear = $wL/2 = 15,004\text{N/m}(7.93\text{m})/2 = 59,491\text{N}$

Maximum moment = $wL^2/8 = 15,004\text{N/m}(7.93\text{m})^2/8 = 117,940\text{Nm}$

$$M = \frac{1}{2} \frac{L}{2} \frac{wL}{2} = \frac{wL^2}{8} = \frac{1.03\text{klf}(26\text{ft.})^2}{8} = 87.04\text{ftk}$$

Maximum deflection = 0.87 i

1. Required Web area (A_w): AISC specifies a load factor of 0.9 associated with shear calculations, so,

EXLN

$$F = \frac{V}{A_w} \quad 0.9F_y = \frac{V}{A_w} \quad \text{or} \quad A_w = \frac{V}{0.9F_y}$$

2. Required Plastic Section Modulus (Z_x), based on maximum moment: The AISC specifies that we must use a load factor of 0.9 associated with all bending calculations. The foundation formula for moment therefore becomes:

$$0.9F_y = \frac{M}{Z_x} \quad \text{or} \quad Z_x = \frac{M}{0.9F_y}$$

EXLNEQ

$$Z_x = \frac{87.04\text{ft.k}(12\text{in./ft.})}{0.9(50\text{ksi})} = 23.2\text{in}^3$$

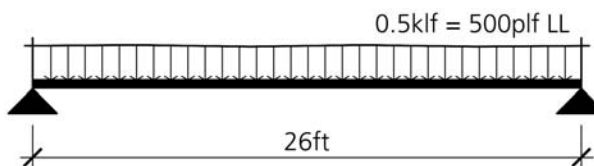


Fig. ex 6.1.2

3. Required Moment of inertia (I_x):

$$\text{Max. All. Deflection } (\Delta) = \frac{L}{360} = \frac{26\text{ft.}(12\text{in./ft.})}{360} = 0.87 \text{ in.}$$

Note that the deflection calculation uses "unfactored" live loads and will in all cases. This deflection formula can be found in Table A-1 (Appendix A). If you always use **pounds and inches**, you will be less prone to making units errors. This is especially true when you move between steel and wood where the load units generally change from kips (steel) to pounds (wood).

EXLNEQ

$$I_x = \frac{5wL^4}{384E\Delta} \quad \text{or} \quad I_x = \frac{5\left(\frac{500\text{plf}}{12\text{in./ft.}}\right)[26\text{ft}(12\text{in./ft.})]^4}{384(29,000,000\text{psi})0.87\text{in.}} = 203.8\text{in.}^4$$

4. First, make a preliminary selection for the Plastic Section Modulus from Table B-1 (Appendix B) The most "economical" sections, by weight, are listed in **bold faced type**.

$$W12X19 : Z_x = 24.7\text{in}^3 > 23.2\text{in}^3 \quad \text{good}$$

The following proportions of live loads:

- In storage areas, 25% of the live load with any applicable reduction factors
- In office structures or others that have a partition load, use the actual partition weight or a minimum of 10 psf [0.48kN/m²], whichever is greater.
- The total weight of any permanent equipment—mechanical equipment, fire suppression Water storage, etc.
- 20 percent of the flat roof snow load in areas where the ground snow load exceeds 30 psf [1.44kN/m²]

EXLN1

EXPOSURE COEFFICIENTS:

EXLNEQHX

$C_o = 0.62$	0–4.6m	Table 1.14
$C_o = 0.67$	4.6m–6.1m	
$C_o = 0.72$	6.1m–7.6m	

EXLN 5. To determine the force distribution throughout the building, we would use

$$F_X = C_{VX}V_B$$

F_X = the force at any level X,

EXLNEQ
$$C_{vx} = \frac{w_x h_x^k}{\sum wh^k} = \text{the vertical distribution factor,}$$

w_X = the weight at level X,

h_X^k = the height of level X above the base,

k = an exponent with the following definitions:

- EXLNEQLN
1. For structures having a period $-T_A$ - of 0.5 sec. or less, $k = 1.0$
 2. For structures having a period $-T_A$ - of 2.5 sec. or more, $k = 2.0$
 3. For structures having a period $-T_A$ - between 0.5 and 2.5 sec. k may be taken as 2.0 or as the linear interpolation between 1.0 and 2.0

$T_A = C_t h_n^x$ Approximate fundamental period of osculation.

V_B = the calculated base shear.

Again, we'll just methodically calculate each term.

This beam does not have full lateral support, consequently our original assumption (hope) was incorrect! We can evaluate other beams and the "bold faced" figures will not necessarily give us the best section if the beam does not have full lateral support.

Our moment capacity needs to be 151.73ftk, so we could look for that value in the Mp or Mr columns, along with Lp and Lr that exceed 7.0ft. It's a little confusing at this point, so let's try some options:

W10X39. Has a moment capacity of 176ftk and Lp of 7.0, it would work, but is fairly heavy.

EXLNLU W14X30 might work, but we would need to proportion its capacity between 176 and 125ftk and unbraced lengths of 5.3ft. and 13.7ft.

This would suggest a rate of change of $176\text{ftk} - 125\text{ftk} = 51\text{ftk}$ occurring over a bracing interval of $13.7\text{ft} - 5.3\text{ft} = 8.4\text{ft}$.

Steel decks are produced in a variety of shapes corresponding to the three basic construction methods: composite floor/roof deck, permanent form, roof deck (Fig. 5.6). Composite floor decks have deformations on the sides of the ribs designed to interlock with the cast-in-place concrete topping. The steel deck works as at least a portion of the reinforcement of the concrete slab; additional reinforcement is gen-

erally required for shrinkage of the concrete at the very least. During construction, the deck provides a form which can carry the full load of the concrete before curing without shoring. In long deck spans, shoring could be required to reduce deflection during construction, before the concrete cures. The manufacturers' tables indicate the maximum unshored span for each type.

Permanent-form decks are primarily used to support the load of concrete during casting, occasionally with some temporary shoring. In this case, the concrete may be designed to work as a reinforced concrete slab without any composite action with the deck, which is simply left in place as a non-removable form.

Roof decks are connected to the roof joists or purlins and support an insulated roofing system, usually of lightweight construction or have adequate internal voids to provide space for additional insulation.

In the first two systems the concrete slab may or may not act in composite action with the steel joists or beams. "Composite" and permanent form decks can both be used to build composite joists and beams: composite deck simply means that the deck works with the concrete topping, but the latter, form deck, may or may not be designed to act with its concrete topping.

Electrical and communication systems can be run under/within the floor using cellular floor deck units. This system provides virtually unlimited flexibility in the planning of office building floor space. Generally, the cellular deck units are intermixed with regular composite metal deck units to provide underfloor duct runs at regular intervals (Fig. 5.7.) Cellular deck units cause some structural complications for fire-

This will obviously add to the overall confusion, but since the open web steel joist tables are NOT created for LFRD, please note that we are NOT using the load factors on any loads, but are depending on the factors of safety built into the tables.

proofing; trench headers require additional filler beams or joists.

Steel deck thickness is given as gage: increasing gage corresponds to decreasing thickness, as shown in Tables 5.2 and 5.3. The tables assume a dead load of 10psf [480N/m²] for the deck and typical built up roof with standard aggregate. Live load deflection does not exceed 1/240th of the span. You should notice that the safe loads are given for one-, two-, or three-span decks. It is more efficient to lay the deck in longer panels to save labor, if a crane is available on site (a 30ft x 3ft [9m x 0.9m] roof deck panel, 16 gage, weighs 314lbs [142 kg] and is obviously too heavy to erect by hand); this is also structurally convenient, since multiple spans reduce the maximum moments and deflections between the supports.

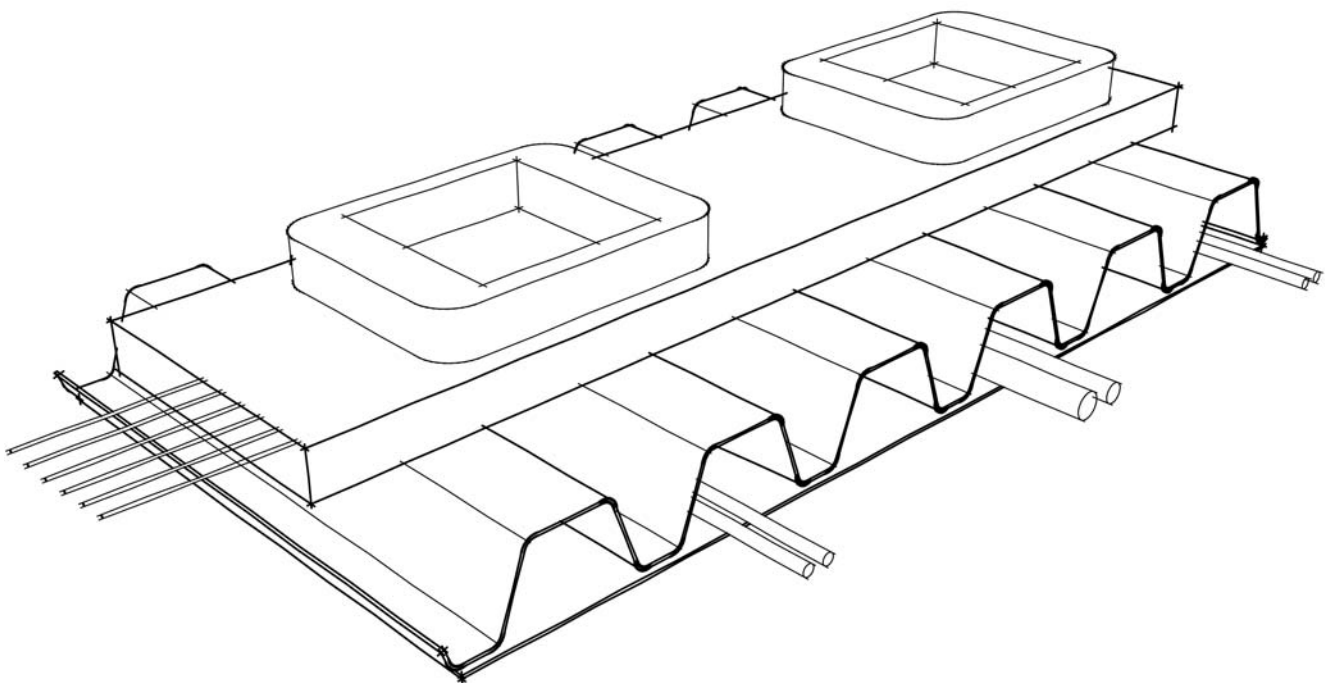


Fig. 5.7 Cellular deck for systems integration

Table 5.6 Typical Roof Deck

TRIPLE SPAN CONDITION											
<i>Safe TOTAL loading in psf (pounds per square foot)</i>											
<i>Span</i>											
Type (ga)	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	8'-6"	10'-0"
22	127psf	105psf	88psf	75psf	65psf	56psf	50psf	44psf	39psf	35psf	31psf
20	157psf	130psf	109psf	93psf	80psf	70psf	61psf	54psf	48psf	43psf	39psf
18	209psf	173psf	145psf	124psf	107psf	93psf	82psf	70psf	61psf	53psf	47psf
16	265psf	219psf	184psf	157psf	135psf	118psf	102psf	87psf	75psf	65psf	57psf

Load and Resistance Factor Design does acknowledge "other" considerations that can't be accounted for in both the manufacture and construction of individual components. This consideration is called a "resistance" factor which is applied to states of yielding and while it remains relatively constant, it does vary with different building components. The combination of these two considerations, one applied to the loads and another applied to the material properties account for the Load Factor and Resistance Factor components of the process.

Load factors cover a wide range of potential critical loading conditions;

Dead Load	D
Occupancy Live load	L
Roof Live load	L _r
Snow Load	S
Rainwater/Ice	R
Wind loading	W
Earthquake loading	E

Using these loading types as initial references, the variety of loading conditions that the designer should consider in determining the most critical situation are:

$$1.2 (D) + 1.6 (L)$$

The bolts listed in Table 15.1 are the most commonly used connectors in steel construction. The A307 bolts were once the standard of all applications, but are currently used predominately for secondary uses that do not require the high strength provided by A325 or A490 bolts.

Bolts types

A307	Unfinished; not for major connections
A325	High-strength; most common sizes: $\frac{3}{4}$ in. and $\frac{7}{8}$ in. [M20 and M22]
A490	High-strength; most common sizes: $\frac{3}{4}$ in. and $\frac{7}{8}$ in. [M20 and M22]

There are several different methods for calculating wind forces on buildings and as you may imagine, the most sophisticated are very complicated. The latest version of the IBC has methodologies that are very sophisticated and quite honestly require the use of a qualified structural engineer. At the same time, architects need to know both the considerations of the IBC code requirements AND they need to have a methodology that will serve them on a daily basis for fundamental calculations. Our approach is to identify those relevant issues that the IBC uses and provide the UBC methodology which, although not as specific, does give very similar force values and can be managed by the typical architect.

The IBC introduces several new considerations into their evaluation of wind loads:

1. Gust factor effect—it is clear that wind does not blow at a steady rate and the impact of wind gusts can have a significant impact in some structural situations. This is one of the factors that lead to the collapse of the Verrazano-Narrows ("Galloping Gertie") bridge in November of 1940. While the causes of the collapse are still argued, it is clear that aerodynamic phenomenon greatly influenced the failure

Table 1.2 Live Load Requirements: Uniform and Concentrated from 2003 International building code

Occupancy or Use	Uniform Load (psf) ^a	Concentrated (lbs) ^b
1. Apartments (see residential)	-----	-----
2. Access floor systems		
Office	50	2,000
Computer use ¹⁰⁰	100	2,000
3. Armories and drill rooms	150	-----
4. Assembly areas and theaters		
Fixed seats (fastened to floor)	60	
Lobbies	100	
Movable seats	100	-----
Stages and platforms	125	
Follow spot, projection, and control rooms	50	
Catwalks	40	
5. Balconies (exterior)	100	
On one-, two-family residences only, and not exceeding 100ft ²	60	
6. Decks	See occupancy	-----
7. Bowling alleys	75	-----
8. Cornices	60	-----
9. Corridors, except as otherwise indicated.	100	
10. Dance halls and ballrooms	100	-----
11. Dining rooms and restaurants	100	-----
12. Dwellings (see residential)	-----	-----
For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm ² , 1 pound per square foot = 0.0479 kN/m ² , 1 pound = 0.004448 kN pound per cubic foot = 16 kg/m ³ Paraphrasing the 2003 International Building Code (consult the IBC in any case)		

^a Floors shall be designed for the specified loading either Uniform or Concentrated which will create the worst loading condition for any criteria; Moment, Shear, Deflection or any localized mode of failure. The specified concentrated load, as set forth above, shall be placed upon any space 2- 1/2ft [762mm] square. It is not necessary to combine the two loading conditions.

^b Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads indicated in Table 1.1 or the following concentrated loads:

1. for garages restricted to 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 which are used for storing passenger vehicles only, 2,250 pounds per wheel.

^c Handrail assemblies and guards shall be designed to resist the following loads; Uniform load of 50 plf [0.73 kN/m] applied in any direction at the top and to transfer this load through the supports to the structure.

Concentrated load—Handrail assemblies and guards shall be able to resist a single concentrated load of 200 pounds [0.89 kN] applied in any direction at any point along the top, and have attachment devices and supporting structure to transfer this loading to appropriate structural elements of the building.

Handrails shall be designed for the specified loading either Uniform or Concentrated which will create the worst loading condition for any criteria; Moment, Shear, Deflection or any localized mode of failure.

Source: from APA—The Engineered Wood Association.

L3 **2. Terrain factors**—if you have ever walked outside or driven down a road on a very windy day, you have noticed that as you round a corner in a city, the wind may dramatically change or as you drive out of a wooded area into a more open area, your car swerves sideways. This is an acknowledgement of the fact that your exposure to the wind is being influenced by localized terrain features. The IBC introduces factors that take into account the local terrain – hills and escarpments and increase or decrease the impact of the wind accordingly.

1.4.3 Base shear force distribution

The base shear V_B that we can calculate with the previous equations is distributed within the structure based on a proportioning system as follows:

$$F_X = C_{VX}V_B$$

EQ F_X = The force at any level X

$$C_{VX} = \frac{w_X h_X^k}{\sum w h^k} = \text{the vertical distribution factor,}$$

w_X = the weight at level X ,

h_X^k = the height of level X above the base,

k = an exponent with the following definitions:

- EQLN**
1. For structures having a period: T_A - of 0.5 sec. or less, $K = 1.0$
 2. For structures having a period: T_A - of 2.5 sec. or more, $K = 2.0$
 3. For structures having a period: T_A - between 0.5 and 2.5 sec. K may

These examples are typical uses of open-web steel joist. A less traditional use would be as extremely tall window wall mullions in the Oakland Alameda Stadium (Fig. 5.8) where they span vertically approximately 70ft along the concourses to support the glass enclosure, yet remain light and visually unobtrusive.

These obviously do not meet the maximum of $\frac{1}{2}$ in./ft. slope. Their load is predominantly lateral, from the wind, which is perpendicular to their axis, the normal loading scenario. In this case they would need to be checked for axial-load capacity as well: a consideration not generally required in other uses.



Fig 5.8 Oakland Alameda Stadium.

FG

PRHX Problem 5.1

PR A floor system is built with open web steel joists at 4ft o.c. spanning 36ft. and carries a live load of 70psf and a dead load of 30psf. Assuming a joist weight of 14lb/ft., find the most economical K-series joist.

Solution **PRSOLHX**

$$\text{Distributed LL} = 70\text{psf}(4\text{ft.}) = 280\text{plf}$$

$$\text{Distributed TL} = 100\text{psf}(4\text{ft.}) = 400\text{plf}$$

$$28\text{K}8 \text{ has TL} = 406\text{plf, LL} = 306\text{plf, weight} = 12.7\text{plf OK}$$

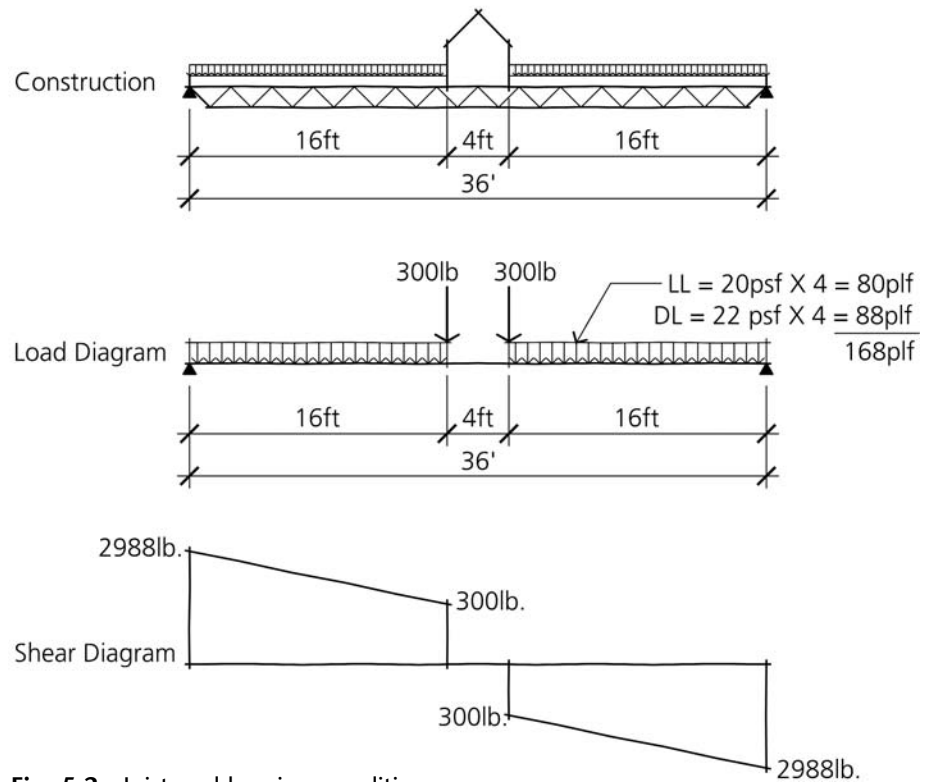
$$30\text{K}7 \text{ has TL} = 395\text{plf} < 400\text{plf, but it is close; weight} = 12.3\text{plf, possible choice.}$$

PRSOL

Problem 5.2

PR A roof system with a 36ft. span carries a 4ft.-wide skylight in the middle, causing two symmetrical 300lb point loads (DL) in addition to the roof distributed load. Roof LL = 20psf, DL = 22psf.

Find the most economical open web joist using a 4 foot spacing. (you may ignore deflection in this problem).



PRFG Fig. 5.2 Joist end bearing conditions

PRSOLHX Solution

TL: Max M = 26,304ftlb., Max V = 2,988lb.

Equivalent loads for V:

$$V = wL/2; w = 2V/L; w = 2(2,988lb)/36ft = 166plf$$

$$w \text{ (shear)} = 166plf,$$

$$M_{max.} = wL^2 / 8; w = 8M/L^2; w = 8(26,304ftlb)/(36ft^2) = 162.4plf$$

$$w \text{ (moment)} = 162.4plf$$

Required design loads:

$$TL = 162.4lb/ft + \text{assumed joist weight } 10plf = \mathbf{172.4plf}$$

18K5 carries TL = 191plf, LL = 92plf (but not a consideration in this case)

and the weight = 7.2plf, the lightest.

PRSOL

You have probably never thought about the differences in the shapes of the Eiffel Tower and the Campanile at Piazza San Marco, in Venice. The Eiffel Tower is a steel, open frame structure, 986ft. [300m] tall, while the Campanile is a masonry solid rectangular structure 325ft. [99m] tall. The Eiffel Tower has a tapered configuration which will reduce the amount of surface area exposed to the higher wind pressures associated

with “height” and this tapered configuration also allows the tower to have a wider stance to provide resistance to overturning. The Campanile in contrast has a uniform configuration for its entire height and has a very narrow footprint. The Eiffel tower, although made of a heavier material (steel weighs 492pcf [7872kg/m³] and stone about 120pcf [1920kg/m³]), is an open truss-work and is actually a very light construction system.

Metric Versions of all Examples and Problems:

MVEX

EXAMPLE M1.1

MVEXHX

MVEX A school in a flat urban area, Indianapolis, Ind., with a footprint of 24.4m x 30.5m with a wall height of 7.3m, flat roof, Exposure B.

Using the UBC approach, determine the total horizontal and vertical forces acting on this building using the "normal force" method which would be most appropriately used for designing individual members or components of the structure.

MVEXLN 1. Positive pressure: inward:

Exposure Coefficients

$$C_o = 0.62 \quad 0-4.6\text{m} \quad \text{Table 1.14}$$

$$C_o = 0.67 \quad 4.6\text{m}-6.1\text{m}$$

$$C_o = 0.72 \quad 6.1\text{m}-7.6\text{m}$$

Pressure Coefficient

$$C_p = 0.8$$

Basic wind speed from Table 1.18 = 129km/hr, therefore = 784N/m²

$$I_w = 1.0 \quad \text{Special occupancy structure} \quad \text{Table 1.17,}$$

Pressure:

$$0-4.6\text{m} \quad P_1 = 0.62 \times 0.8 \times 784\text{N/m}^2 \times 1.0 = 389\text{N/m}^2$$

$$4.6\text{m}-6.1\text{m} \quad P_2 = 0.67 \times 0.8 \times 784\text{N/m}^2 \times 1.0 = 420\text{N/m}^2$$

$$6.1\text{m}-7.6\text{m} \quad P_3 = 0.72 \times 0.8 \times 784\text{N/m}^2 \times 1.0 = 452\text{N/m}^2$$

2. Negative pressure: outward:

Exposure coefficients:

$$C_o = 0.62 \quad 0-4.6\text{m} \quad \text{Table 1.14}$$

$$C_o = 0.67 \quad 4.6\text{m}-6.1\text{m}$$

$$C_o = 0.72 \quad 6.1\text{m}-7.6\text{m}$$

Pressure coefficient:

$$C_p = 0.5$$

Pressure:

$$0-4.6\text{m} \quad P_1 = 0.62 \times 0.5 \times 784\text{N/m}^2 \times 1.0 = 243\text{N/m}^2$$

$$4.6\text{m}-6.1\text{m} \quad P_2 = 0.67 \times 0.5 \times 784\text{N/m}^2 \times 1.0 = 263\text{N/m}^2$$

$$6.1\text{m}-7.6\text{m} \quad P_3 = 0.72 \times 0.5 \times 784\text{N/m}^2 \times 1.0 = 282\text{N/m}^2$$

3. Roof uplift: outward:

$$C_o = 0.72 \quad (\text{based on mean roof height} = 7.3\text{m} \text{ which puts us in the } 6.1\text{m}-7.6\text{m} \text{ range})$$

$$C_p = 0.7$$

$$P_4 = 0.72 \times 0.7 \times 784\text{N/m}^2 \times 1.0 = 395\text{N/m}^2$$

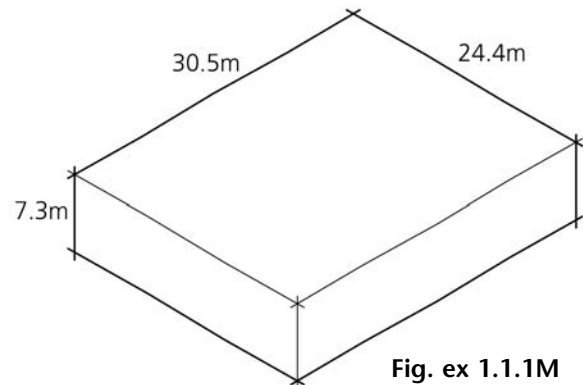


Fig. ex 1.1.1M

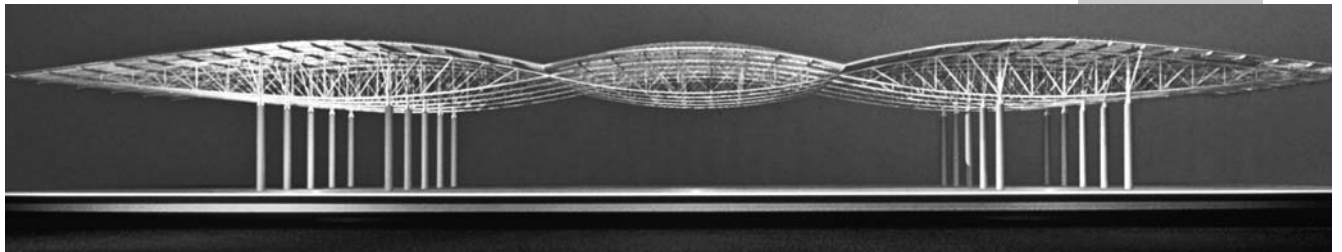
SECTION

1

PSN

Steel Bending Systems

PST



Pre-engineered Systems

STEEL MEMBERS FOR BEAMS AND COLUMNS are produced in standard shapes and in some cases a preliminary design can be carried out with the aid of tables. This is also true for a large number of proprietary structural systems (not just steel, but also wood and concrete). In addition to the ASTM (the American Society for Testing of Materials) shapes for beams and columns, there are other lighter shapes such as the "M" section (a rolled section of the same form as the standard "W" and "S" beam, but thinner). Additionally, lightweight cold-formed shapes are used for secondary framing (steel "C" studs and plates) and, for relatively light, uniform floor and roof loads, open-web joist can be used. One type is shown in Fig. 5.1.

These sections are commonly referred to as "bar joist" because some of the original sections were constructed using bar stock for both the web and flanges. Modern joist use alternative flange configurations consisting of light gage angles or a formed section similar in a configuration similar to angles. The web consists usually of a continuous round bar bent and welded to the flanges yielding a member that looks like a truss, in fact it is a truss. These sections, for all practical purposes are "generic" trusses. These joists are intended to be spaced from 24 to 48in. [610–1220mm] apart depending on the magnitude of load and the capacity of the steel decking or "centering" spanning between them. Special sections are

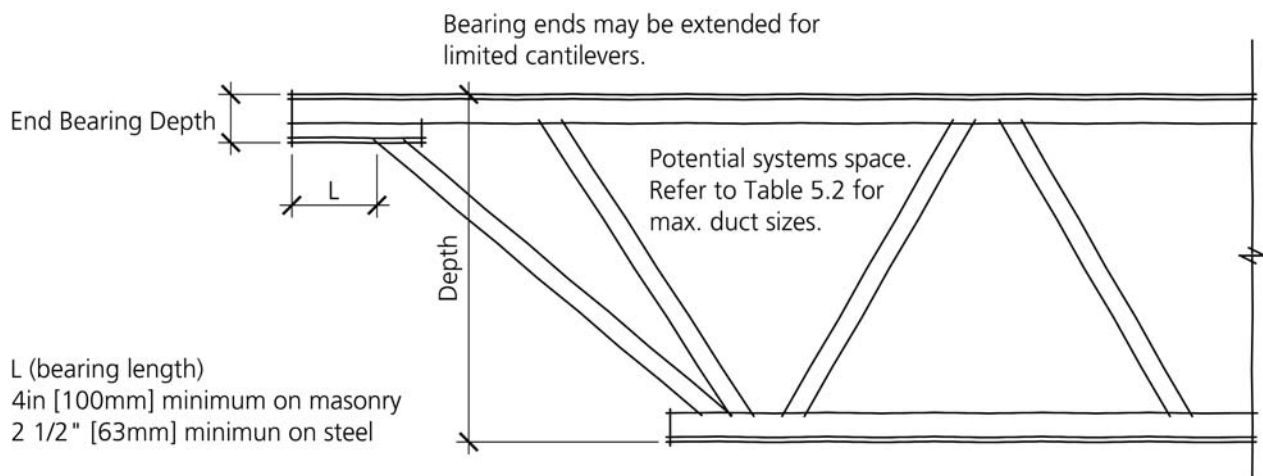


Fig. 5.1 Typical open-web steel joist characteristics

manufactured which have the web members penetrating through and extending above the upper flange elements to allow for encasement and therefore composite structural action with a poured concrete deck.

Open web joists have the convenience as well as the limitations of standardized components, the main limitation being that they are designed to work primarily as simply supported, uniformly

LN

1. Shear: Using or The physical property A (area of web = thickness of web x depth of section) is used to select appropriate members. The actual shear formula is $F_v = VQ/Ib$, however for steel W and I sections the approximation is quite accurate.

F_v is the allowable shear stress based on material/code (AISC).

V is the actual shear load at the point of evaluation, usually the maximum.

A is the web area of the section, typically the value you're attempting to determine.

2. Moment: With the introduction of LRFD methodology, we will be using two versions of $F_b = M/Z$, with Z, the plastic section modulus, being the physical property used for selection, and $F_b = Mc/I$, with S, the elastic section modulus, which equals I/c .

F_b is the allowable bending stress based on material/code (AISC) as a function of F_y for LRFD.

M is the actual moment at the point of evaluation, usually the maximum.

Z is the section modulus of the section typically the value you're attempting to determine.

3. Deflection: using any of a number of "formulas" or combinations of formulas from Appendix A to determine the required moment of inertia to insure that the live load deflection does not exceed code specified criteria. I is the value you will be

seeking when using these formulas, look carefully at the definitions of all of the values used in these formulas and their associated units. The most common mistake in these calculations is a units error. The three design values, required physical properties, A_w , S_x , I, obtained from these calculations are material/building code dependent.

1. Species Groups. Wood is classified in four species groups, in decreasing order of strength: A, B, C and D (NDS). Laminated timber is classified according to compression design values perpendicular to the grain F_c according to NDS.
2. Adjustment Factors. These design values have, as usual, to be adjusted with the following applicable factors:

$$Z' = Z (C_d)(C_m)(C_t)(C_\Delta)(C_{eg})(C_g)$$

All of these factors have already been discussed, with the exception of C_g , the group action correction factor, but to refresh your memory:

- C_d Load duration—adjusts for the period of time that the load must be sustained.
- C_m Wet service—adjusts for the moisture conditions that the component is subjected to.
- C_t Temperature—adjusts for the temperature conditions that the component is subjected to.
- C_Δ Geometry factor—adjusts for the spacing and placement of the connector elements relative to member ends, edges and grain.
- C_d Penetration depth factor—adjusts for connectors attached lag screws
- C_g Group action factor—adjusts for connectors used in a row parallel to the grain of the member.
- C_{st} Steel side plate factor—adjusts for 4in. shear plates that use steel side plates to transfer load..

LNEQ

LNL1

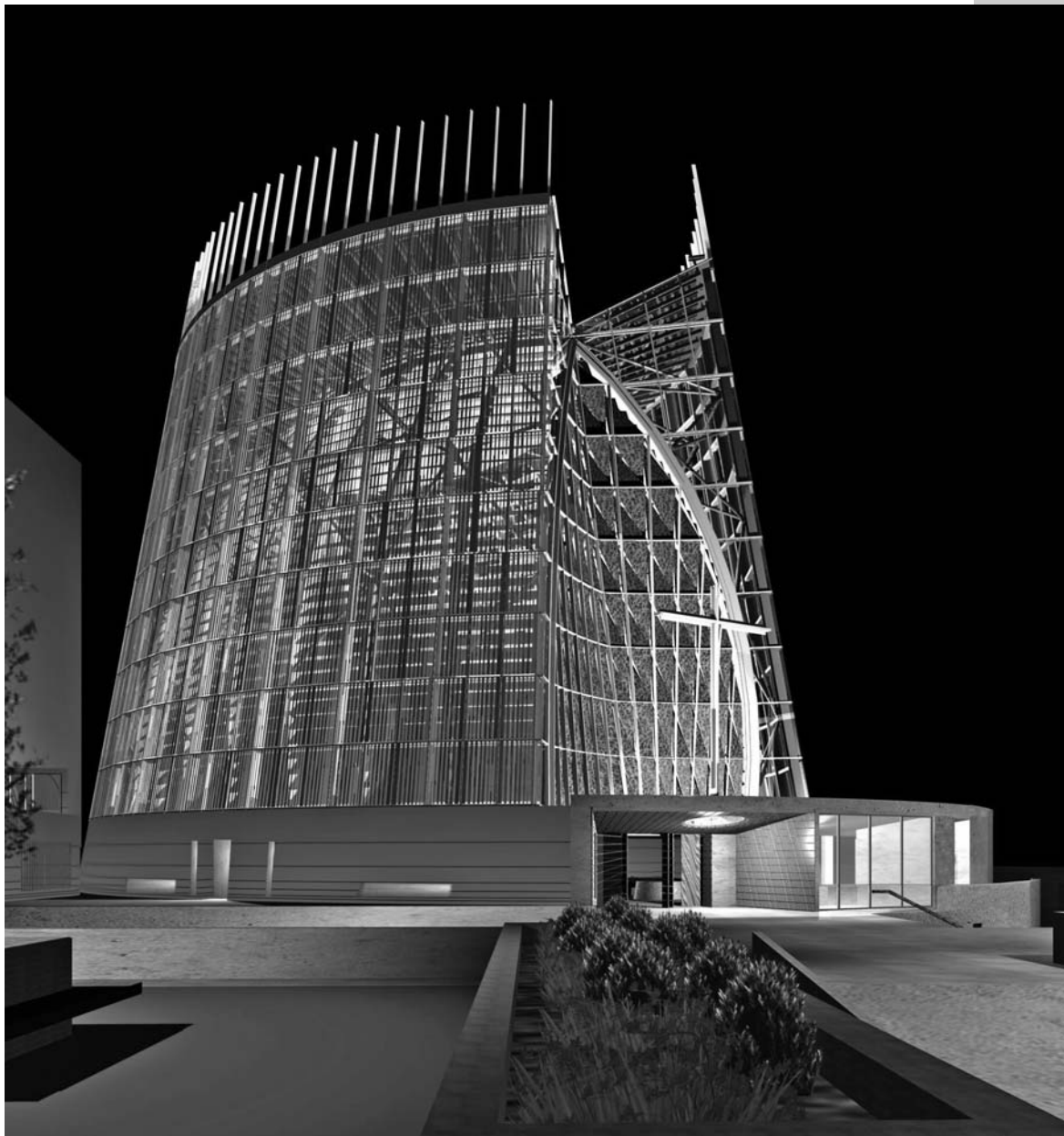
LNLU

PART TWO

WOOD

PN

PT



FG

Fig W Cathedral of Christ of the Light

Structural Systems

CQE

Marco Polo describes a bridge, stone by stone. But which is the stone that supports the bridge?—Kublai Kan asks.

The bridge is not supported by this or that stone—Marco replies,—but by the line of the arch that they form.

Kublai Kan remains silent, meditating. Then he adds:—Why do you tell me about the stones? It's just the arch that matters to me.

Polo replies:—Without stones there is no arch.

CQESN

(Italo Calvino, The Invisible Cities).

40.1 MASONRY CONSTRUCTION

Modern masonry construction consists of bricks, concrete masonry units and stones bonded together by mortar. Stone structures, still common in the first part of the 20th century, are now rare and unfortunately, (nostalgia speaks here) vaulting over large spaces has been replaced by less labor intensive solutions, although it may still be an appropriate technology for countries that have inexpensive labor forces (Fig. 40.1).

In modern masonry construction, a major evolution took place with the introduction of steel reinforcement. Masonry systems are now classified as



Fig. 40.1 Traditional masonry structures in Egypt

Appendix A Title

The qualities that make steel a desirable building material are strength and ductility. Strength is measured relative to the yield strength, corresponding to the stress value where, with virtually no increase in loading, steel deformation continues to increase. This phenomenon will eventually cause the material to actually become stronger due to "strain hardening" and the member will support more load, but at the expense of large deformations. This can be visualized by bending a piece of mild steel wire, a coat hanger, with your fingers: the elastic limit is quickly passed and, once the pressure of the fingers is released, the wire stops bending but the deformation remains. The wire is not broken and the steel has not lost all of its elastic properties.

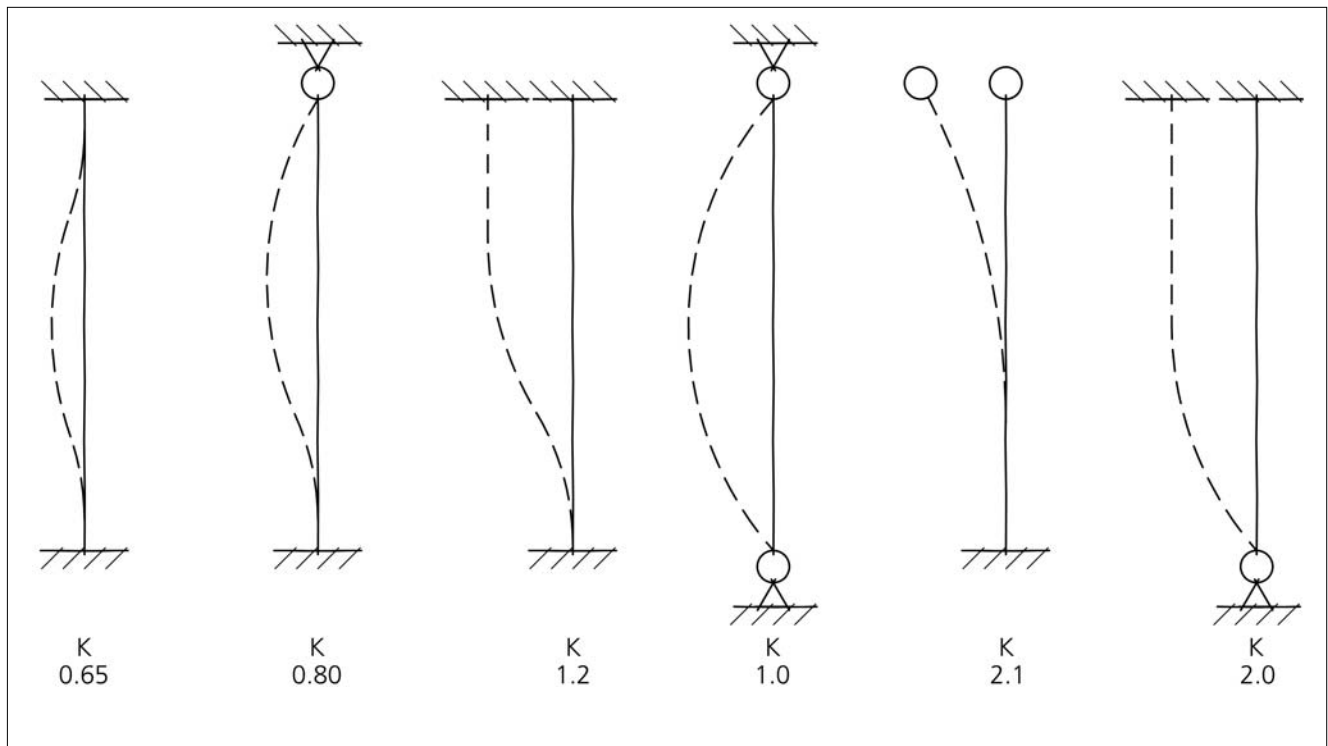
In fact, it can be bent back and straightened, at least theoretically, to its initial shape, with no loss of strength. Under repeated bends however, the material will become hot, evidence that the steel is undergoing physical changes. Eventually the wire will become brittle in the localized area of deformation and will break because of its loss of ductility.

You can see visual evidence of a similar process if you crimp a cheap plastic cup from a vending machine and notice that the material changes from translucent to a milky white. The physical properties have been altered in this area by a process similar to strain hardening. We'll pursue some of the cases when you might use this characteristic to our advantage later.

AXTN

Table A.1 K Values for Columns

AXTT



Metric Conversion Factors from U.S. Customary Units to SI Metric Units

Table A.2 Beam design Criteria Zx, Ix, A

AXSH

AXTH

AXTBH1

AXTB

<i>Basic Section Properties</i>				<i>50 ksi Steel Values</i>				Rate of Change
Shape	Zx	Ix	Aweb	MLp	Lp	MLr	Lr	
W36X798	3,580	62,600	199.9	13,400	14.9	8,940	84.6	63.99
W36X650	2,860	48,900	80.0	10,700	14.5	7,260	69.6	62.43
W40X593	2,760	50,400	77.0	10,400	13.4	7,020	56.5	78.42
W40X503	2,320	41,700	64.8	8,700	34.1	5,940	49.4	76.03
W36X527	2,280	38,300	63.1	8,550	14.2	5,850	58.1	61.50
W40X431	1,960	34,800	55.3	7,350	12.9	5,070	78.0	35.02
W27X539	1,890	25,600	64.0	7,090	12.9	4,710	78.0	36.56
W36X439	1,870	31,000	52.1	7,010	139	4860	50.1	59.39
W14X808	1,830	16,000	85.3	6860	170	4200	268	10.60
W40X397	1,800	32,000	50.0	6,750	12.9	4,680	42.1	70.89
W40X392	1,710	29,900	59.1	6,410	9.33	4,320	14.3	83.70
W40X372	1,680	29,600	47.1	6,300	12.7	4,380	40.1	70.07
W36X393	1,670	27,500	46.1	6,260	13.3	4,350	46.7	58.05
W14X730	1,660	14,300	68.8	6,230	6.6	3,840	241	10.65
W40X362	1,640	28,900	45.5	6,150	12.7	4,260	39.9	69.49
W44X335	1,620	31,100	44.9	6,080	12.4	4,230	35.5	80.09
W33X387	1,560	24,300	45.4	5,850	13.3	4050	47.7	52.33
W36X359	1,510	24,800	41.9	5,660	13.6	3,960	43.8	56.29
W14X665	1,480	12,400	61.1	5,550	16.3	3,450	222.0	10.21
W40X324	1,460	25,600	40.2	5,480	12.6	3,840	37.6	65.60
W30X391	1,450	20,700	45.2	5,440	13.0	3,750	52.1	43.22
W40X331	1,430	24,700	49.8	5,360	9.1	3,630	30.5	80.84
W44X290	1,420	27,100	37.9	5,330	12.3	3720	34.0	74.19
W33X354	1,420	22,000	41.3	5,330	13.2	3,720	44.6	51.27
W40X327	1,410	24,500	48.1	5,290	9.1	3,600	30.3	79.72

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