A Beginner's Guide to

the Steel Construction Manual

An introduction to designing steel structures using the AISC Steel Construction Manual, 13th edition.

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Preface

The creation of the Beginner's Guide to the Steel Construction Manual (BGSCM) was prompted by the major rewrite of the AISC specification that appeared in the 13th edition of the Steel Construction Manual (SCM). When textbooks were slow to respond to the change, I started thinking that a web based approach would be more responsive. After further thought the site was expanded to include undergraduate topics in structural engineering for civil engineers.

Initially, my thought was to simply put together my 20 years of lecture notes to supplement the same wonderful textbook that I have been using for the past couple of decades. As the work got to be more extensive than a set of notes, I decided to spend the summer trying to morph this into a stand-alone on-line textbook. The next two summers saw the work expanded to include more example and home problems plus refinements recommended by users of the site.

One of the recommendations has been to create a hard copy version of the on-line text. How hard can that be? Well, it turns out that converting the format over has been a very major undertaking. In the process, an in-depth review of the material took place and many unanticipated improvements were made. Unfortunately, however, every time I look at the text I find format errors. I THINK that we have it to a presentable stage, but anticipate more editing will take place as comments come in.

While this hard copy version is nice, the CURRENT, and primary version, will always be available free on the web. All the latest changes will be made there and be instantly available. That is the one of the beauties of the web based text. The other is that the online version can contain media types not available in printed texts.

There are a couple of philosophical bases that have driven the particular approach to this work.

First, it is my intent that the student becomes comfortable referring to the base specifications used in design. As a result, if some information is in an industry standard or specification that the students need to know, I refer them to the particular section and let them read it there. This requires that the student have the relevant industry documents nearby whenever reading this text.

Second, it has always bothered me that graduating structural engineering students are commonly unaccustomed to reading real engineering drawings so I have endeavored to present problems (particularly the homework problems) that are in quasi engineering drawing format. It is hoped that students will quickly grasp the concept of reading engineering drawings and feel more comfortable with their first experiences as a design engineer. To facilitate the goal of enhancing a student's ability to read and use engineering drawings, I have provided drawing sets for homework problems that come from real designs.

Third, and related to item two, is that I have chosen to present problems that are in context of a whole structure. I have seen this done by instructors at other institutions, but it is often difficult to find the time to really develop such problems in a way that students can understand them. As no one structure supplies all types of problems, I have provided several different structures (a building, a tower, and a truss bridge) that cover most of the bases. I have also provided a large set of drawings of random details that are used by the other structures and/or can be used as standalone problems.

Fourth, I wanted the text presented in a way that allows for quick updates and for media types not possible in printed books. This left the Internet website as the logical choice. You will find that many of the figures have hot links that allow for larger views of all or part of the figure. There are also active links to spreadsheets, powerpoint animations, and tutorials that might help a student to better learn how to implement the requirements of the specification. Unfortunately this hard copy version cannot display these other media types. As a result, the text often refers the reader to the web site where these other media types can be utilized.

Fifth, I wanted this to be free to the students. The website version still is. I had toyed with finding a way to put this in a secure website then policing the integrity of the users, but the whole security issue seems tedious and difficult. So, the online version is free for use in teaching structural engineering design. It is copyrighted, however, which means that I'd rather not see my work show up in someone else's book.

The problem with making this free is that I have spent countless hours working on this for free, the site is hosted on a web server in my home which is not always reliable, I don't have access to the services of a good technical editor, and I can't afford a web programmer to further my multimedia ideas. I am currently looking for sponsors who will pay an advertising fee sufficient for me to fix these problems. The proceeds from the sale of this text will help, but will not be enough unless sales are better than I expect. If you know of anyone interested in such a sponsorship, I would be happy to talk with them and share current usage statistics. I can be reached at QandA@alaska.com.

Sixth, I want input concerning the content and pedagogy of this text. I have been fortunate to have received several very good suggestions from people that have viewed the online text. I welcome all input concerning the text.

So there you have it. My thoughts behind this work. Any comments about the text are appreciated.

I also am pleased to thank the people that have (and those that will) send in suggestions for improvement to the text. These have already made a substantial difference in the work and I look forward to receiving more. No text of value is written in a vacuum or an ivory tower.

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analysis performed again. This process iterates until the final calculations use refined loads based on the chosen components.

Design Optimization

Since the objective of design is to find the optimum design, the design problem can be characterized as an optimization problem stated as:

Minimize or Maximize $f_{obj}(dv_1,dv_2,dv_3, \dots ,dv_n)$

subject to:

- $f_{constraint\_1}(dv_1,dv_2,dv_3, \dots ,dv_n)$ compared to allowable value
- $f_{constraint\_2}(dv_1,dv_2,dv_3, \dots ,dv_n)$ compared to allowable value
- $f_{constraint\_3}(dv_1,dv_2,dv_3, \dots ,dv_n)$ compared to allowable value
- .....
- $f_{constraint\_m}(dv_1,dv_2,dv_3, \dots ,dv_n)$ compared to allowable value

Where

- f_{obj} = the objective function
- d_{vi} = the i^{th} design variable
- n = number of design variables
- $f_{constraint\_i}$ =the i^{th} constraint function
- m = number of constraint functions

A feasible design is one where all the constraint functions are satisfied. The optimum design is a feasible solution that represents the design with the best objective function value.

For example, let us say that planet "A" gets a message from planet "B" that "B"ites need a magic potion that "A"ites have a soon as possible. If the potion does not arrive in time then all the "B"ites will all die. With several planets with strong gravity fields along the way, there are several paths by which a rocket could reach planet "B" once it is fired from planet "A". The "A"ites must determine the direction and initial velocity for the rocket that will get there the quickest.

In this problem, the objective function is the time that it takes for the rocket to reach planet "B". The only constraint is that the rocket has to hit planet "B". The two design variables are the initial direction and initial velocity.

Let's see how that problem breaks down mathematically.

The objective function, in this case, would be the function that computes the time it takes for a rocket to reach planet "B".

f_{obj} = TimeInTransit(Velocity, Angle)

The actual computation must take into account the gravitational effect of the other planets in the solar system and is somewhat complicated.

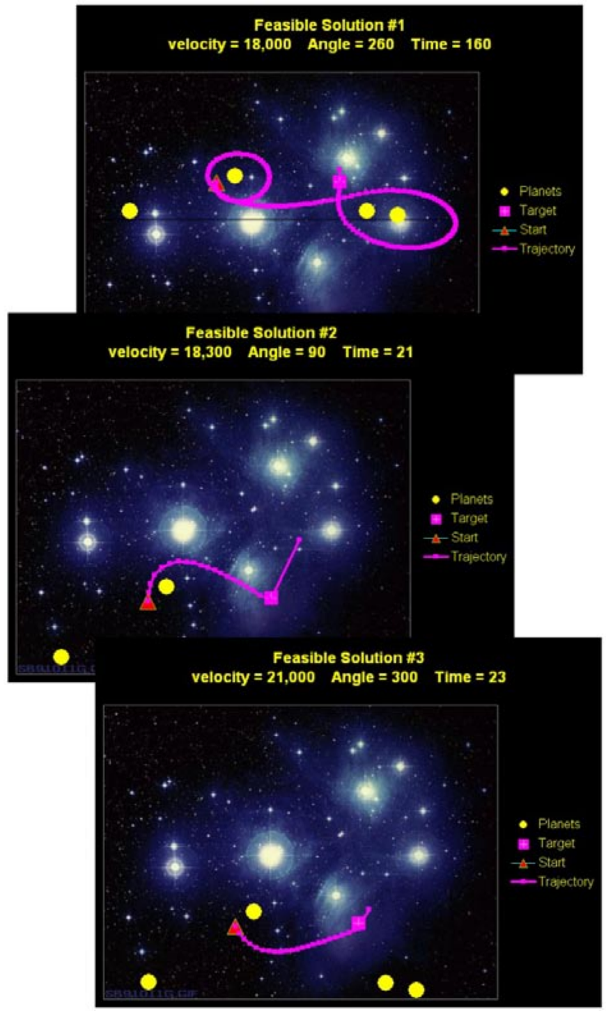
The constraint function, on the other hand, is fairly simplistic. It monitors the progress of the rocket and becomes true if the rocket arrives at planet "B".

$f_{constraint\_1}$ = HitPlanet(Velocity, Angle) must equal "True"

The design variables are the independent variables needed to compute values for both the objective function and the constraint function. In this case the design variables are:

Velocity = the initial velocity of the rocket leaving planet "A"
Angle = the initial direction of the rocket as it leaves planet "A"

Figure DC.1.1
Acceptable Trajectories



To evaluate the alternatives, a computer program was written to analyze the performance of the rocket. The design variables are entered into the program and the computer computes the trajectory of the rocket to determine if the design variables result in a feasible design (i.e. does the rocket hit planet "B"). If the design is feasible, then the objective (time in transit) is computed and saved to compare against other designs.

The analysis returned three feasible solutions to the rocket problem. Many other combinations (not shown) were tried, but rejected because they were not feasible solutions (i.e. they did not hit the planet). Figure DC.1.1 shows the resulting trajectories. The Table DC.2.1 summarizes the results.

Table DC.1.1
Rocket Problem Results

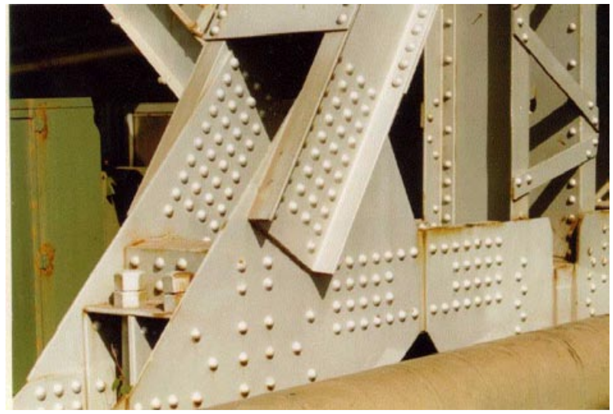
Solution #	Design Variables		Objective
	Velocity	Angle	Time
#1	18,000	260	160
#2	18,300	90	21
#3	21,000	300	23

The preferred alternative is solution #2 because it has the best objective function result.

The preceding example was very simplistic. Most engineering solutions, particularly in steel design, are much more complicated having a multitude of design variables and constraints. The

In the mid 1900s, high strength bolts were introduced and quickly replaced rivets as the preferred method for connecting members together in the field because of their ease of installation and more consistent strengths. High strength is necessary since most bolts are highly tensioned in order to create large clamping forces between the connected elements. They also need lots of bearing and shear strength so as to reduce the number of fasteners needed.

Figure 2.3.1
A Riveted Connection



There are two basic ASTM high strength bolt specifications and one non-high strength ASTM bolt specification that we will be using. The high strength bolts are ASTM A325 and ASTM A490. The non-high strength bolt is ASTM A307.

The ultimate tensile strengths for A325 and A490 bolts are 120 ksi and 150 ksi respectively. These values are rarely needed in applying the equations in the AISC specification, but are useful to know when using theoretical equations for special circumstances. We will also be obtaining bolt strength information for use in the AISC equations from SCM table J3.2 on page 16.1-104. We'll cover that table in more detail later.

The ASTM bolt specifications require that the bolts and their associated nuts and washers be clearly identified with their specification number. Figure 2.3.2 shows the identifiers for A325 and A490 bolts.

Figure 2.3.2
Bolt Identifiers

The Limit State:

The basic limit state follows the standard form. The statement of the limit states and the associated reduction factor and factor of safety are given here:

LRFD	ASD
$P_u \leq \phi_t P_n$	$P_a \leq P_n/\Omega_t$
Req'd $P_n = P_u/\phi_t \leq P_n$	Req'd $P_n = P_a\,\Omega_t \leq P_n$
$P_u / (\phi_t P_n) \leq 1.00$	$P_a / (P_n/\Omega_t) \leq 1.00$
$\phi_t = 0.75$	$\Omega_t = 2.00$

The values of P_u and P_a are the LRFD and ASD factored loads, respectively, applied to the member. In this case Pn is the nominal tensile rupture strength of the member. Note that the values for ϕ_t and Ω_t are different than previously used. Their more restrictive values indicate that the test results for this mode of failure are more highly variable than those tensile yielding.

Nominal Tensile Rupture Strength, P_n:

The nominal tensile rupture strength, P_n, is computed using AISC specification equation D2-2 and is in force units. In this case, we multiply the *effective net cross sectional area*, A_e, by the *tensile stress*, F_u to determine force that would cause the member to rupture in tension near the connections.

Note that the AISC specification generally uses F_u for the stress level whenever rupture or significant deformation is fundamental to the limit state being considered. This is because the failure mode being considered is rupture (failure, or fracture) of the member is the issue. Moderate local yielding to redistribute forces is tolerated.

Having just introducing A_e, we need to take a detailed diversion into section D3 of the specification.

Area Determination, Section D3

AISC Specification section D3 covers the computation of three different areas: gross area A_g, net area A_n, and effective net area A_e. We will now look particularly at A_n and A_e since we've already discussed A_g.

Net Area, A_n

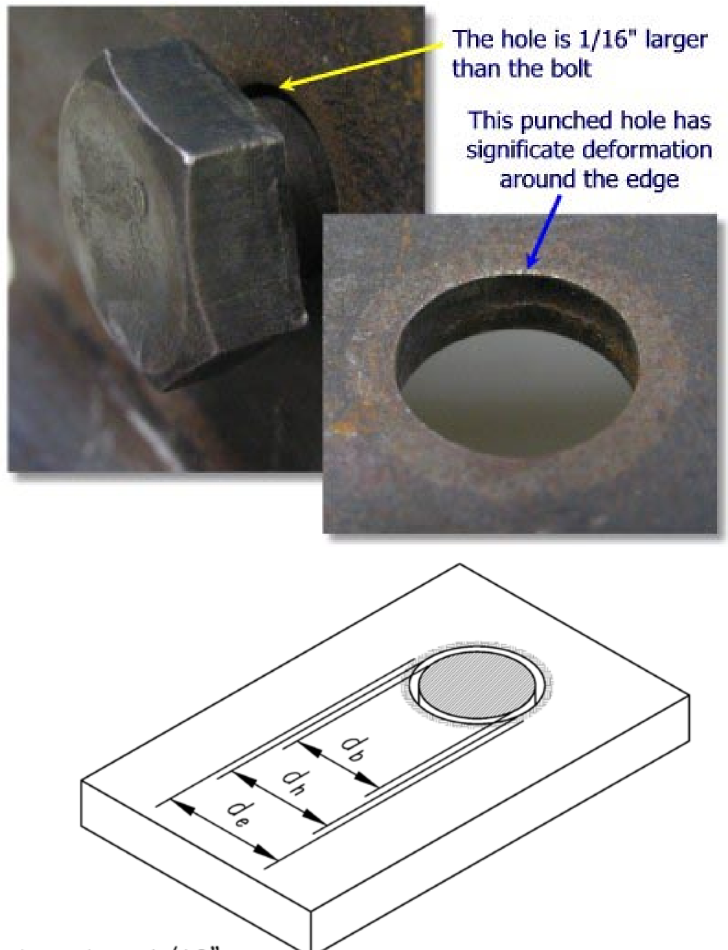
At this point you need to read AISC specification D3.2 on net area. The commentary on this section found on SCM page 16.1-250 is also necessary reading.

The net area computation requires computation of a reduced section due to holes made in the member as well a failure path for the rupture surface.

One way to define the net area is it is the gross cross sectional area less the cross sectional area of any holes (effective hole diameter x plate thickness) plus an increase for any diagonal lines in the failure path. **Effective hole diameter** and **failure paths** are two new concepts to be considered.

First, let's consider the effective hole diameter. Section D3.2 says that "the width of a bolt hole shall be taken as 1/16 in. larger than the *nominal dimension* ***of the hole***." (emphasis added). A standard hole is 1/16 inch larger than the bolt it is to accommodate. This means that the effective hole diameter, for A_n calculations, is to be taken as 1/8" larger than the bolt (i.e. 1/8" = 1/16" for the actual hole diameter plus an additional 1/16" for damage related to punching or drilling.) So, if you specify 3/4" bolts in standard holes, the effective width of the holes is 7/8" (i.e. 3/4" for the bolt diameter + 1/16" for the hole diameter +1/16" damage allowance.)

Figure 3.4.2
Bolt Holes



$d_h = d_b + 1/16"$
$d_e = d_h + 1/16" = d_b + 1/8"$

Figure 3.4.2 shows a typical standard hole and the dimensions that are related to it.

The next concept that needs discussing is the concept of failure paths. Failure paths are the approximate locations where a fracture may occur.

When considering failure paths, you will always start from the side of the member and cut perpendicular to the axis of the member to the center of a hole. The path will then extend from center of hole to center of hole until you take a path to the opposite side which is perpendicular to the axis of the member. The failure path will be located so that it sees the maximum force in the member, in other words all the bolts are on the side of the rupture line that is also connected to another member. Any path that leaves bolts on both sides of the rupture is not feasible unless the member you attach to fails simultaneously along the same path... something that cannot happen for a number of reasons.

Another way to look at this problem is that the failure separates the main member from the connection. All the bolts stay with the connection.

Problem M3.4.2: Determine the axial tensile capacity (both LRFD, ϕT_n, and ASD, T_n/Ω) of the connection based on the capacity of the connection plates.

ϕT_n, = ________________, Controlling Limit State: __________________

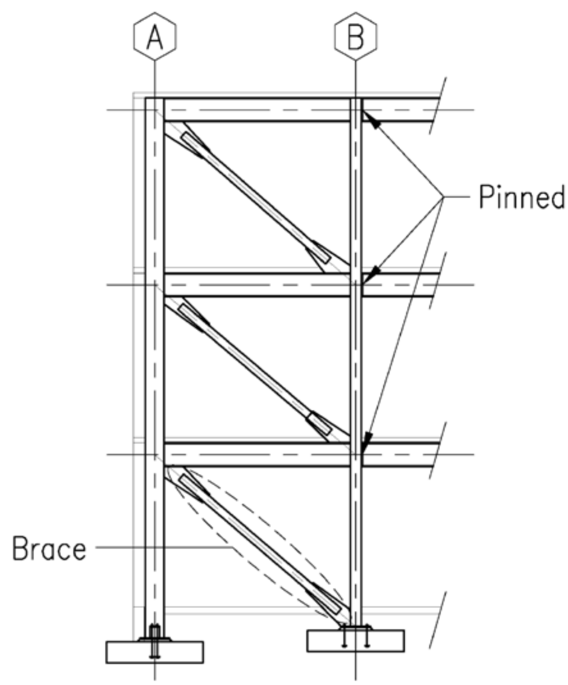
T_n/Ω = __________________, Controlling Limit State: __________________

Dormitory Building Design Problems

The braces for this building are both tension and compression members, depending on the direction of lateral forces. For this chapter, we will design them based on their tensile capacity. Later, we will revisit the design of these members as compression members. See drawings BLDG 4/S3 and BLDG 5/S3 for the following problems.

Problem D3.1: Select a section for the brace on the first floor level on Grid 2 and between Grids A & B (see Figure 10.11.1). For members with bolted connections determine the required number of bolts to satisfy the limit state of bolt bearing on holes. Using the required number of bolts, neatly sketch to scale the layout out of the bolts that you use to compute the tensile rupture and block shear limit states for the member you selected.

Figure 10.11.1
Problem D3.1 Frame



- **D3.1a**: Select the lightest single channel for this member. The connection is similar to MISCDET 1/S2.1 (instead of two channels, use one). In the referenced detail, use A > 1.5" and B > 3" for this problem.
- **D3.1b**: Select the lightest double angle section for this member. If the connected leg is wider than 5", assume that the member has a double gage line of bolts as shown in drawing MISC 3/S2.1, otherwise assume that there is a single line of bolts. Center the bolt group on the connected angle leg. In the referenced detail use A > 1.5", B = 2.0", and C = 3" for this problem.
- **D3.1c**: Select the lightest square HSS section for this member. Assume that the ends of the section are slotted to accommodate a 1/2" thick connection plate similar to that shown on drawing MISCDET 5/S2.1. For now, assume that the connecting welds are each 10 inches long. (i.e. the lap of the member with the gusset plate is 10".)

Problem D3.2: Repeat Problem D3.1 for the other braces in the structure. Complete the following table by adding the lightest member sizes that satisfy the criteria:

On Grid	Between Grids	Level	Problem #					
			D3.2a		D3.2b		D3.2c	
			Square HSS		Dble Angle		Single Channel	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
2	A & B	3						
		2						
		1						
	P & R	3						
		2						
		1						
11	A & B	3						
		2						
		1						
	P & R	3						
		2						
		1						

Tower Design Problems

The tower is made of members that are all axial force members. All the members have one or more load conditions that exert axial tension and compression on the members. For this part of the design, we will only consider the tension forces. In this truss structure, it is possible to analyze the structure so in such a way that axial compression is not allowed in the diagonal members, making them tension only. This is a conservative approach that often allows smaller members to be used. The table of member forces found in the drawing set are based on the diagonals being tension only members.

Problem T3.1: Select a member for a diagonal brace at level "A". For members with bolted connections determine the required number of bolts to satisfy the limit state of bolt bearing on holes. Using the required number of bolts, neatly sketch to scale the end of the member showing the layout out of the bolts that you use to compute the tensile rupture and block shear limit states for the member you selected.

- **T3.1a**: Select the lightest single angle. Assume a single line of bolts along the axis of the member connects the angle to tower leg similar to the detail shown in TOWER 3/S2.
- **T3.1b**: Select the lightest round HSS section. Assume that the member is slotted to accommodate a 3/8" thick gusset plate similar to the connection shown in TOWER 3/S3. Assume that the lap length of the HSS and the gusset plate equals the HSS diameter, D.

For example, an A325-X bolt would be a bolt made from A325 steel and would be installed such that the treads are excluded from any shear plane(s). A A490-SC bolt is a bolt made from A490 steel and would be installing on a slip critical connection where the faying surfaces must meet special requirements.

Limit States

The primary objective of checking all strength based limit states to ensure that the strength of the structural element is strong enough to handle anticipated forces exerted on them. In the case of bolts, this can be expressed as:

The FORCE on the bolt $\leq$ the STRENGTH of the bolt

For bolts the forces applied to the bolts can be resolved into shear (perpendicular to the bolt axis) or tension (parallel to to the bolt axis) components.

Force on the Bolts

The force on any given bolt is the result of the forces being applied to the connection and the geometry of the connection. Principles of Mechanics and Structural Analysis are used to determine the force on any particular bolt in a connection. The next sections discusses several commonly used methods for computing the forces on a bolt.

Strength of a Bolt

Bolts have one tensile capacity and two shear capacities. Typically the SCM denotes the nominal capacities of each as R_n. The capacities are defined by limit states.

Tensile capacity is controlled by the tensile rupture limit state which can be stated as:

The TENSILE FORCE on the bolt $\leq$ The TENSILE RUPTURE STRENGTH of the bolt

Shear capacity is controlled by one of two limit states:

- Slip (The force that overcomes the frictional capacity of the connected surfaces)
- Shear Rupture (The force that causes shear failure of the bolt)

For the case of shear capacity, the limit states can be stated as:

The SHEAR FORCE on the bolt $\leq$ The SLIP STRENGTH of the bolt
or
The SHEAR FORCE on the bolt $\leq$ The SHEAR RUPTURE STRENGTH of the bolt

It is common in connections with pins, rivets, and bolts to connect more than two members together with a given fastener. The force transferred by a fastener then becomes a function of the number of shear planes in the connection. As will be seen in the next section, a more convenient, or maybe more appropriate, way of expressing the shear limit states is:

The SHEAR FORCE on a SHEAR PLANE $\leq$ The SLIP STRENGTH of a SHEAR PLANE

or

The SHEAR FORCE on a SHEAR PLANE $\leq$ The SHEAR RUPTURE STRENGTH of a SHEAR PLANE

The concept of shear planes is presented in the next section.

4.2 Mechanics of Load Transfer

Mechanical Fasteners (i.e. bolts, rivets, and pins) are most frequently used in structural steel connections where the load direction is perpendicular to the bolt axis as shown in Figure 4.2.1. In this situation the principle force in the bolt is shear.

Figure 4.2.1
Bolts in Shear

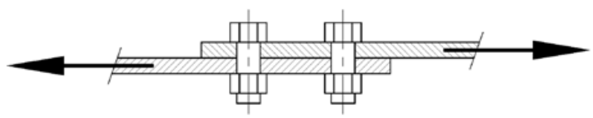


Figure 4.2.2
Bolts in Tension

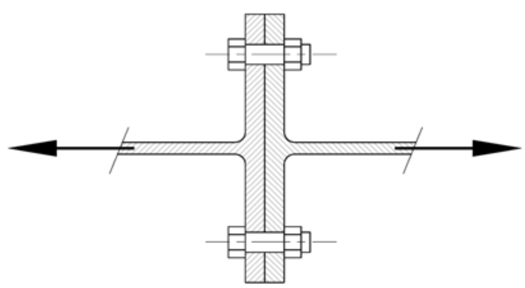
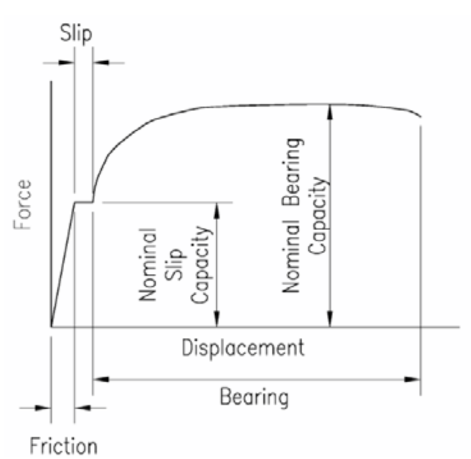


Figure 4.2.3
Connection Load/Deformation Curve



Less frequently, the bolts are placed such that their axis is parallel to the direction of force as shown in Figure 4.2.2. Here the bolts the principle force in the bolts is tensile.

Let's look at the force transfer mechanisms in more detail.

<u>A Bolt in Shear</u>

As mentioned in the prior section, bolts may be installed as snug tight or fully tensioned, the difference being that the snug tight installation does not provide a significant clamping force between the connected members. Let's begin this discussion by considering a fully tensioned bolt.

If you were to place the connection shown in Figure 4.2.1 in a tension test, the force vs. deformation curve would look something like what is shown in Figure 4.2.3.

As the load is progressively applied to the connection, the major force transfer between the connected plates would be by friction. The friction capacity is the result of the normal force (N) between the plates created by the bolt tension and the roughness of the contact surfaces (quantified by the friction coefficient, μ). From your statics course, you will recall that the friction capacity equals μN. Once the applied force exceeds the friction capacity (i.e. the nominal slip capacity), the connected

4.8 Bolt Summary

Strength Limit States:

All strength limit states take the form:

LRFD	ASD
$R_u \leq \phi_t R_n$	$R_a \leq R_n/\Omega_t$
Req'd $R_n = R_u/\phi_t \leq R_n$	Req'd $R_n = R_a\,\Omega_t \leq R_n$
$R_u / (\phi_t R_n) \leq 1.00$	$R_a / (R_n/\Omega_t) \leq 1.00$

Which is: **FORCE on a bolt $\leq$ STRENGTH of a bolt**

The STRENGTH of a bolt is computed by:

Simple Tension or Shear

Limit State	Specification	Nominal Capacity	Typical Design Variables	ϕ	Ω
Tensile Rupture	J3.6	Single Bolt Capacity: $F_{nt}A_b$	Bolt Material, Bolt Size	0.75	2.00
Shear Rupture	J3.6	Single Shear Plane: $F_{nv}A_b$	Bolt Material, Bolt Size	0.75	2.00
Slip Capacity	J3.8	Single Shear Plane: $\mu D_u h_{sc} T_b$	Bolt Material, Bolt Size	0.75	2.00

Combined Shear and Tension:

Bearing Type Fasteners (-X or -N bolts):

- Modify the nominal tensile rupture capacity for the presence of shear (SCM J3.7)
- Apply the shear rupture limit state without modification

Limit State	Specification	Nominal Capacity, R_n	Typical Design Variables	ϕ	Ω
Tensile Rupture	J3.7	Single Bolt Capacity: $F'_{nt}A_b$	Bolt Material, Bolt Size	0.75	2.00
Shear Rupture	J3.6	Single Shear Plane: $F_{nv}A_b$	Bolt Material, Bolt Size	0.75	2.00

Slip Critical Type Fasteners (-SC bolts):

- Modify the nominal slip capacity for the presence of tension (SCM J3.9)
- Apply the tensile rupture limit state without modification

Limit State	Specification	Nominal Capacity, R_n	Typical Design Variables	ϕ	Ω
Tensile Rupture	J3.6	Single Bolt Capacity: $F_{nt}A_b$	Bolt Material, Bolt Size	0.75	2.00
Slip Capacity	J3.9	Single Shear Plane: $\mu D_u h_{sc} T_b k_s$	Bolt Material, Bolt Size	0.75	2.00

<u>The FORCE on a bolt is computed by:</u>

<u>Forces Concentric with the Bolt Group at the Faying Surface:</u>

- All bolts are assumed to be equally stressed in tension.
- All shear planes are assumed to be equally stressed in shear.

<u>Eccentricity in the Plane of the Faying Surface:</u>

- **Elastic Vector Method**: See SCM pg 7-8. Computes shear in the bolts. Direct method that is conservative and has an inconsistent factor of safety.
- **Instantaneous Center of Rotation Method**: See SCM pg 7-6. Computes the relationship between the applied load and the shear load in the worst case bolt. Iterative method that is more consistent with test results and not as conservative as the Elastic Method.

<u>Eccentricity out of the Plane of the Faying Surface:</u>

- **Case I Method**: See SCM pg 7-10. Basic mechanics (Mc/I) using the compression contact area to find the tension in the worst case bolt. Finding Ix may be iterative. If the shear is concentric with the bolt group it is equally divided among the shear planes otherwise use either the elastic vector or IC method to find the bolt shear forces.
- **Case II Method**: See SCM pg 7-12. Uses basic statics (Applied Moment = Pe = rat n' dm= Internal Moment) without considering the contact area to find the tension in the worst case bolt. If the shear is concentric with the bolt group it is equally divided among the shear planes otherwise use either the elastic vector or IC method to find the bolt shear forces.

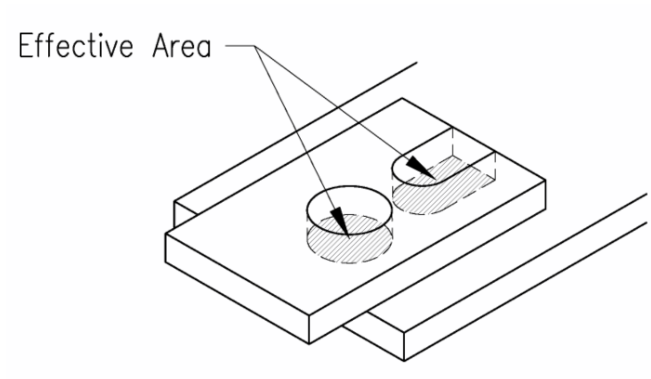
4.9 Example Problems

A spreadsheet based solution for the example problems can be downloaded from

http://www.bgstructuralengineering.com/BGSCM/BGSCM004/BGSCM00409.htm

There are limits on the minimum and maximum sizes of slot welds that can be found in the specification section J2.3b.

Figure 5.3.5
Effective Area of Plug and Slot Welds



5.4 Effective Areas of Base Metal

The capacity of a welded connection is limited by the lesser of the strength of the weld or the strength of the material that the weld is attached to (i.e. the base metal). The strength of the base metal is the force necessary to cause shear rupture or tensile yielding of the base metal where the weld is attached.

A key principle is that the area of rupture sees the full force that the weld sees.

For CJP welds the base metal effective areas are the same as those for the welds themselves. That is the thickness of the plate times the length of the weld.

For PJP welds the base metal effective areas are the same as for the CJP welds. That is, the effective area equals the plate thickness times the length of the weld.

For fillet, plug, and slot welds, the determination of effective base metal area gets more complicated.

The fillet, plug, and slot welds tend to transfer their forces via shear, so the predominate base metal failure is shear rupture. This is similar to the block shear limit state considered for tension member design.

Base Metal Effective Area Example 1

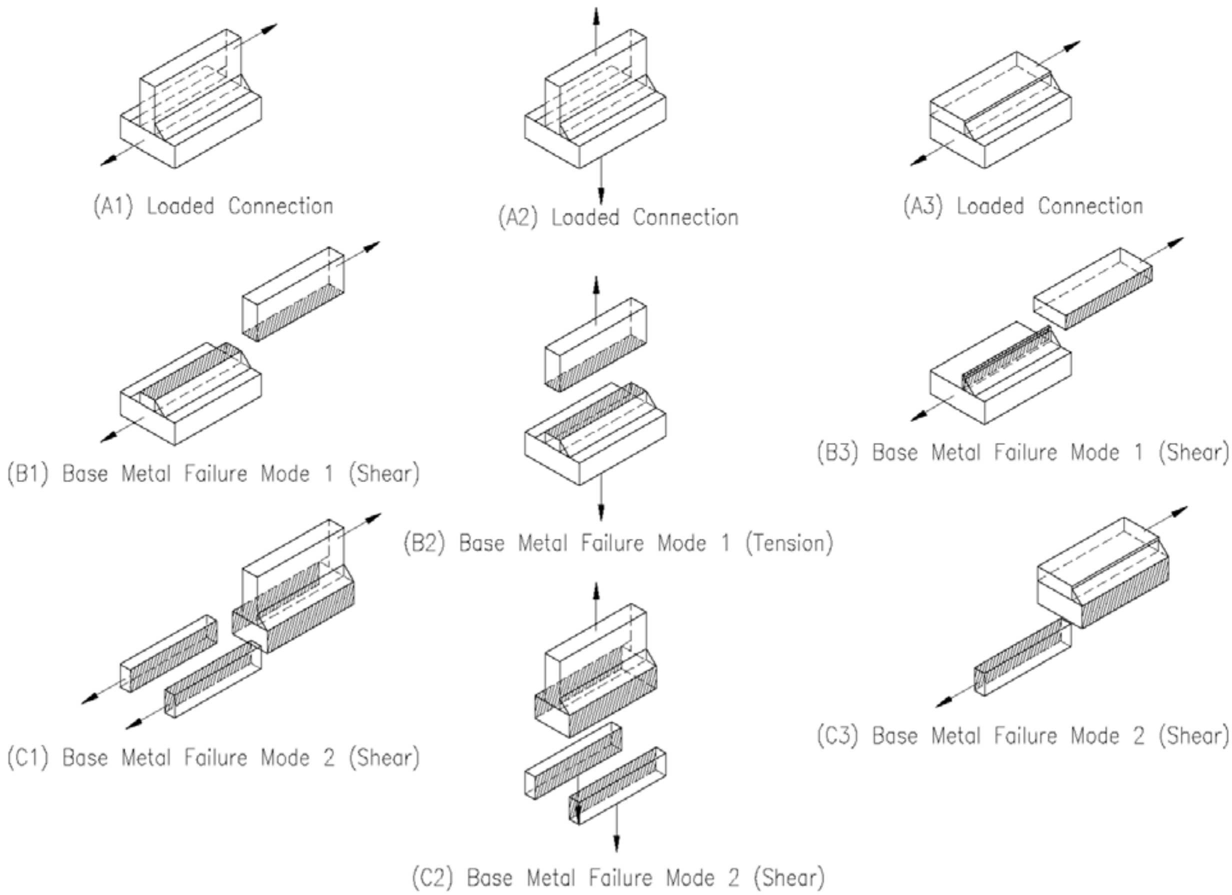
Figure 5.4.1, example 1, shows a T connection made with two fillet welds (one on each side of the stem). The connection is loaded with a shearing force as shown.

As is the case in all connections, there are two potential base metal failure modes to consider. One for each base metal being joined. The effective areas (i.e. the failure surfaces) are shown with a hatch.

Failure mode 1 has an effective area equal to the thickness of the stem times the length of the weld. This failure surface supports the full shearing force. Also notice that this effective area "supports" two fillet welds. That is while the failure surface shown supports the full load, that load is transferred to two equal size fillet welds, each seeing half of the applied force. Consequently, to avoid base metal failure, the base metal strength must be TWICE that of the each weld, or you could say to avoid weld failure, the welds must each be only half as strong as the base metal.

Figure 5.4.1
Base Metal Effective Area

Example 1 **Example 2** **Example 3**



Failure mode 2 has two failure surfaces that, together, support the full shearing force. Another way to look at this, due to symmetry, is that each failure surface supports have the applied force. The effective area in each case equals the thickness of the plate times the length of the weld. In this mode, the effective area sees the same force that the attached weld does. To avoid base metal failure, the base metal needs to be as strong as the weld it is attached to and vice versa.

Note that every weld is connected to two base metals. In the case of example 1, each of the two welds share one shear area on the stem, but each has its own adjacent base metal shear area on the bottom plate.

Base Metal Effective Area Example 2

Figure 5.4.1, example 2, is the same connection as example 1 with the stem loaded in tension. Note that the bottom piece's effective areas are still in shear.

Stiffened vs. Unstiffened Elements

The SCM defines two different types of plate elements in a cross section: Stiffened and Unstiffened. See SCM B4.1 and B4.2 for the definitions. If a plate's edges are restrained against buckling, then plate is stiffer and the force required to buckle the plate increases. If one edge is restrained (i.e. and "unstiffened" plate element) the force to cause out-of-plane buckling is less than that required to buckle a plate with two edges restrained against out-of-plane buckling. An intersecting plate at a plate edge adds a significant moment of inertia out of plane to the edge which prevents deflection at the attached edge.

Figure 6.3.3
Plate Buckling Modes

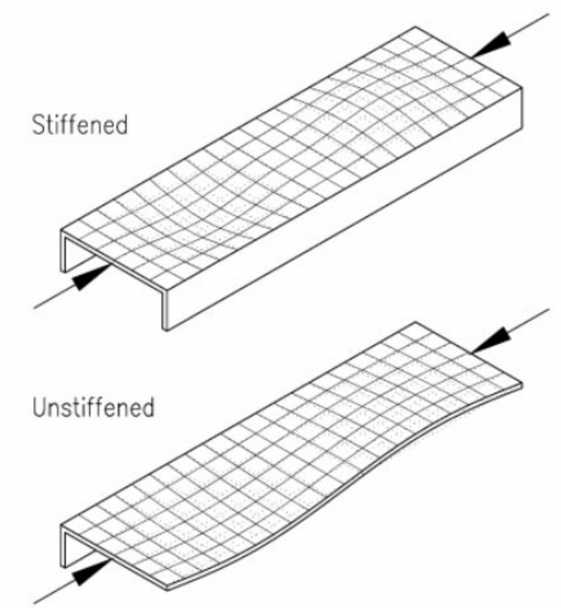


Figure 6.3.3 illustrates the modes of buckling for a stiffened and unstiffened plate elements.

Figure 6.3.4
Buckled Unstiffened Element

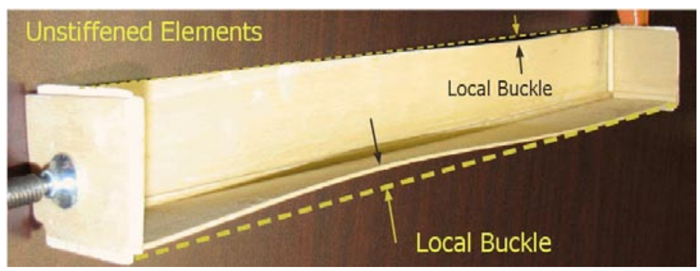


Figure 6.3.4 shows a buckled unstiffened element from an experiment. In this experiment, an "L" shaped cross section was create with thin wood plates. Each of the wood plate elements is "unstiffened" since only one edge is restrained (by the intersecting plate) against out of plane buckling. As a uniform axial compression is added to the member, the initial failure mode is local buckling of the plate elements as shown in the image. As both plates have the same b/t ratio, they both buckled at the same time. Note that the member is not slender (is short with fairly larger "r") so the buckling is not general buckling.

Figure 6.3.5 shows the *unstiffened elements* on some typical steel sections and the measurement of the element width, b, and thickness, t. Note that a "W" section has four unstiffened elements (i.e. each of the outstanding half flanges, all of equal size), a "WT" has three unstiffened elements, a channel has two unstiffened elements, and an angle has two unstiffened elements. When a section has multiple plate elements, the most slender element will control the definition of the member as being compact, non-compact, or slender.

Note that some members have both stiffened and unstiffened elements. The "W" sections and channels are examples of sections with both types of elements in their cross sections.

Figure 6.3.5
Unstiffened Elements

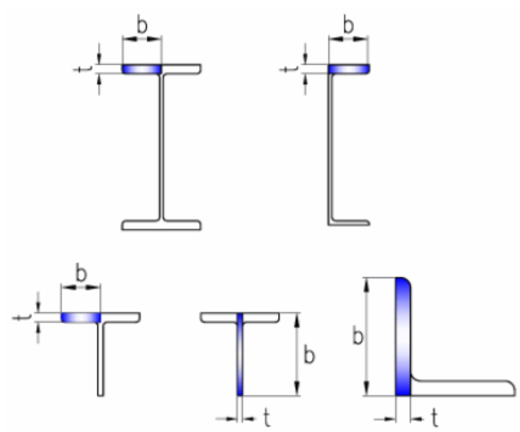


Figure 6.3.6
Stiffened Elements

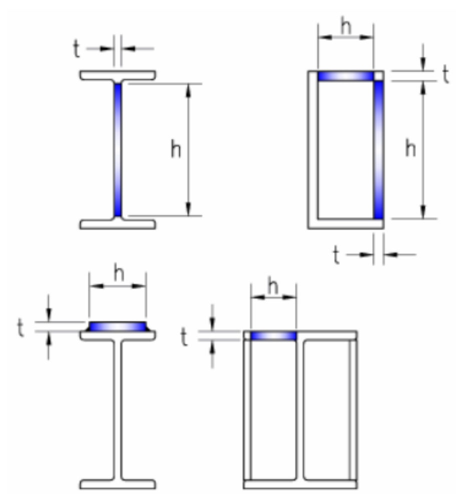
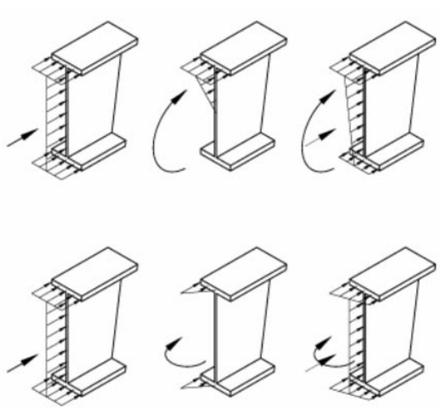


Figure 6.3.7
Stress Distribution



Another factor effecting buckling is the nature of the compressive stress that the element is subjected to. Euler's equation assumes uniform compression. If the compressive stress is not uniform then the onset of buckling is a bit more complicated and methods for determining buckling must account for the stress distribution. Typically any non-uniform stress distribution will require greater maximum compressive stress to initiate buckling than that required to initiate buckling under a uniform stress

As learned in mechanics, normal stress distribution can be characterized as being a function of an applied concentric force and a moment (or moments) about the centroidal axis. Using the basic stress equations for axial force and bending, the force distribution is planar (in 3D) or linear (in 2D). Figure 6.3.7 shows some typical stress distributions on a wide flange section. The top row illustrates combined axial and bending about the strong axis of the member. The lower row illustrates combined axial and bending about the weak axis of the member.

The SCM Section Slenderness Classification

When analyzing a steel section where there is compressive stress on some or all of the cross section, the steel section must be classified as being *compact*, *non-compact*, or *slender* so that the appropriate strength equation can be applied.

SCM B4 (pg 16.1-14) defines the method used for classifying a section. Where a cross section consists of multiple plate elements (both stiffened and unstiffened), the most restrictive case (i.e. the most slender definition) defines classification of the section. The actual classification system is tabulated in SCM Table B4.1 (SCM page 16.1-16). The table describes the various conditions, how to compute the width/thickness ratio and the limits λ_p and λ_r that defined the limits of the slenderness regions.

The first eight cases listed in SCM Table B4.1 refer to unstiffened elements. The remainder refer to stiffened elements. The table includes figures to illustrate the definitions of the various components. What the table does not include is a graphical

6.4.3 Example Problem 6.3

Given: The frame shown Figure 6.4.3.1 (see also BGSE detail MISCDET_STL 3/S5.3).

Wanted: determine the effective lengths for all the column segments in the frame.

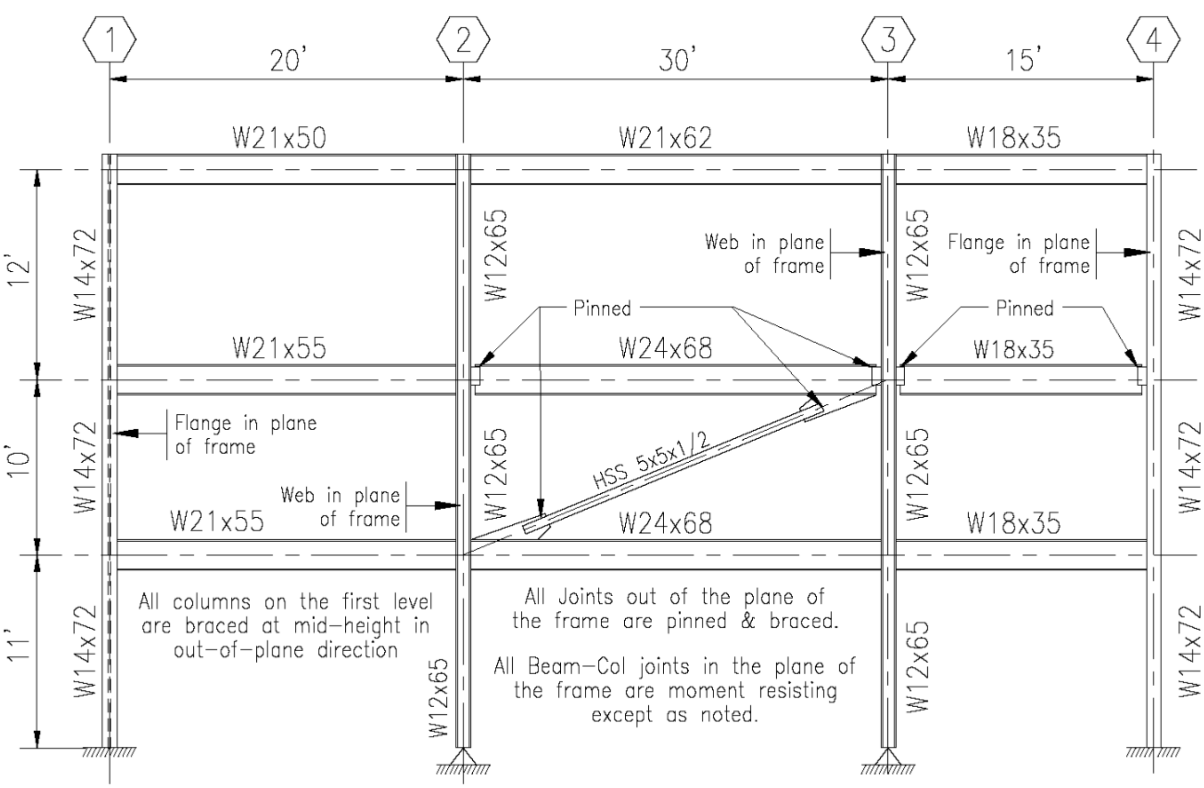
Solution:

In the plane of the frame, there are twelve (12) separate column length computations to do. One for each column at each level on each grid. Out of plane, there are sixteen (16) column lengths, however they are the same for each grid, so there are only five (5) calculations to do. The computations for each presented here. The spreadsheet file for the chapter 6 example problems has the computations as well.

On the web version of Figure 6.4.3.1, click on any column to see the calculation of the effective lengths. In the text version we will go through each column individually. You should look at all of the columns as there are several different scenarios presented. You can use the web version as a tutorial by attempting the problems before clicking on the solutions.

Pay particular note to the drawn and stated support and connection conditions.

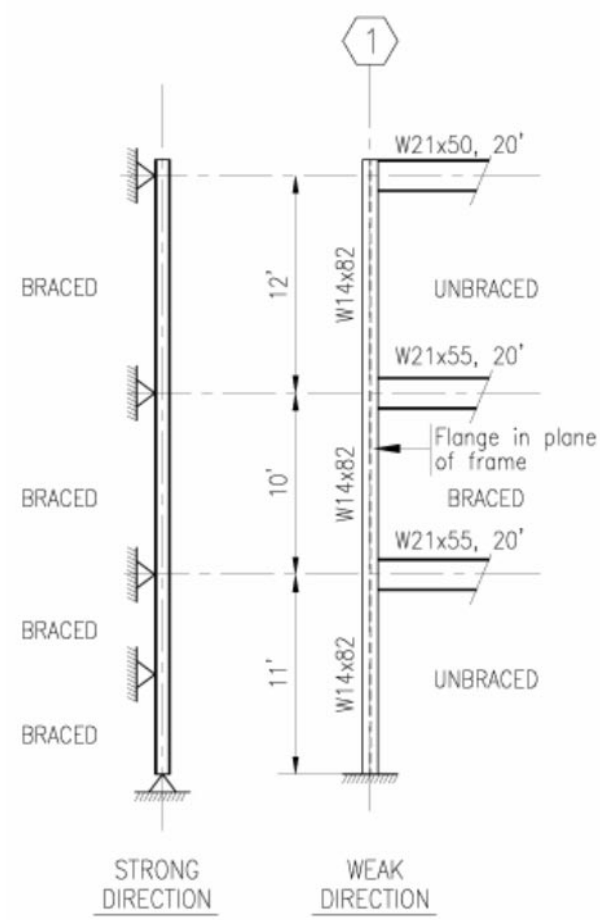
Figure 6.4.3.1
Sample Frame



Grid 1 Columns

Figure 6.4.3.2 shows the support conditions for this column. We will start from the top and work our way down for computing the effective lengths.

Figure 6.4.3.2
Grid 1 Column Diagrams



Third Floor Column Segment

Strong Direction: At this level and direction, there are no beams fixed to the column to create joint rotational restraint and the column is part of a frame that inhibits sidesway (i.e. it is braced) as stated on the drawings.

For these conditions, K = 1.00. This gives an effective length,

$$\mathbf{(KL)_{X,13} = 1.0*12' = 12\ ft.}$$

The subscript used denotes the strong direction, "X", on grid "1" on the third floor, "3".

Weak Direction: At this level, the support conditions are:

- Sidesway is uninhibited. In the frame profile there are no braces on the third floor level so all the columns in this level are unbraced.
- At the top of the column segment, a W21x50 girder and the W14x82 column resist joint translation in plane. Note that joint rotation in the plane of the frame, in this case, causes bending about the strong axis of the girder and bending about the weak axis of the column. So we use I_x for the girder term and I_y for the column term:

  $G_{1R} = \Sigma(I_c/L_c) / \Sigma(I_g/L_g)$

  $G_{1R} = [(148\ in^4)/(12\ ft)] / [(984\ in^4)/(20\ ft)]$

  $G_{1R} = 0.251$

  The subscript used is "1" for the grid line and "R" for the roof level. Note that "G" term need only be computed once for a joint. Joints are often shared by two column segments.
  Also note that we did not change the units so that they are all feet or inches. All the I terms are in^4 and the L units are feet. Even without the unit conversion, all the units cancel, so save yourself the extra computational step! Check it out.

LRFD	**ASD**
$P_u \leq \phi_c P_n$	$P_a \leq P_n / \Omega_c$
Req'd $P_n = P_u / \phi_c \leq P_n$	Req'd $P_n = P_u \Omega_c \leq P_n$
$P_u / (\phi_c P_n) \leq 1.00$	$P_a / (P_n/\Omega_c) \leq 1.00$
$\phi_c = 0.90$	$\Omega_c = 1.67$

The values of P_u and P_a are the LRFD and ASD factored loads, respectively, applied to the column.

In this case P_n is the nominal compressive strength of the member is computed using SCM equation J3-1:

$$P_n = F_{cr}A_g$$

Where:

- F_{cr} is the flexural buckling stress.
- A_g is the gross cross sectional area of the member.

The SCM specification has two formulas for determining the flexural buckling stress. The first equation, E7-2, covers both the plastic and inelastic buckling regions of the typical buckling strength curve as shown in Figure 6.1.3. The second equation, E7-3, covers compression members that are slender.

The criteria for selecting which formula to use is based on either the slenderness ratio for the member or the relationship between the Euler buckling stress and the yield stress of the material. The selection can be stated as:

if ($KL/r \leq 4.71*\text{sqrt}(E/QF_y)$) or ($F_e > 0.44QF_y$) then

use SCM Equation E7-2

else

use SCM Equation E7-3

Figure 7.4.1
F_{cr} for Slender Sections

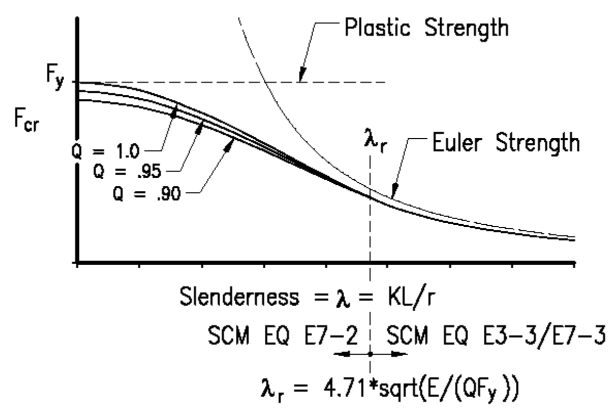


Figure 7.4.1 shows a graph of the F_{cr} equations for various values of Q.

The term Q is a reduction factor that is a function of cross sectional element slenderness and is the product of the reduction factors Q_s and Q_a.

Q_s is a reduction factor for *unstiffened cross sectional elements*. This factor is a function of the largest width/thickness (b/t) ratio of the unstiffened elements in the cross section. It is computed differently based on the compactness of the section. SCM E7.1 gives, for different types of sections, the limiting b/t ratios for the plastic, inelastic, and elastic buckling ranges and the equations for determining Q_s for each region. So, for a given b/t ratio, you can determine which range the element is in and the equation to use for computing Q_s.

Q_a is a reduction factor for *stiffened cross sectional elements*. SCM E7.2 takes a different approach to computing Q_a than was used for Q_s. Q_a is taken as a ratio of effective area, A_{eff}, of the member to its actual area, A_g. As this is a buckling problem, we expect to see equations for the three regions, and this section does not disappoint.

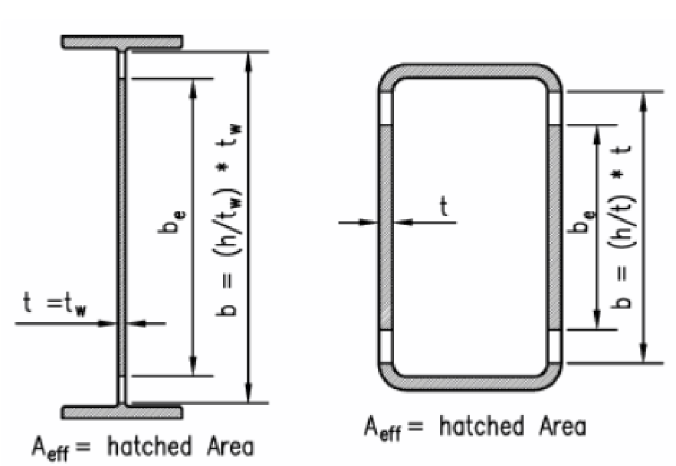
Computing A_{eff}

A_{eff} is computed as the sum of the effective areas of each stiffened cross sectional element ($b_e t$) and the full area of any unstiffened cross sectional elements. b_e is computed using the equations presented in SCM specification E7.2 based on the type of section being considered.

For example in a W section, the stiffened cross sectional element is the web. The web has a "width, b" = $(h/t_w) * t_w$, where both h/t_w and t_w are tabulated values. The effective width, b_e, is found with SCM equation E7-17. The effective width, b_e will always be taken as less than or equal to width b. A_{eff} is the gross area, A_g, less *the area lost* using the reduced web plate width. When b_e is less than b, the loss in cross sectional area becomes $(b - b_e)*t_w$. The resulting A_{eff} equals $A_g - (b - b_e)*t_w$.

From a buckling perspective, the critical buckling stress is achieved when either local buckling capacity or general buckling capacity is reached. If a *slender section* is part of a *slender member* then, as the general slenderness parameter (KL/r) increases, there is a point where general buckling will occur before local buckling will occur. If this happens, then local buckling is a moot point and there is no need to adjust the member capacity for the effects of local buckling. The SCM equations in E7.2a do this. The equations are a function of the general buckling critical stress which is a function of general slenderness (KL/r). The result is that there are many times when a section has slender cross sectional elements, but there is no reduction in capacity as a result and Q_s=1. In other words, cross sectional element slenderness does not always impact member capacity.

Figure 7.4.2
A_{eff} for Slender Sections



For "box" sections, you typically have two slender elements at a time. This means that you have two cross sectional elements that are losing area. Both need to be accounted for in your A_{eff} calculation.

Figure 7.4.2 illustrates A_{eff} on a W section and rectangular HSS.

Note the gray box on SCM page 16-1.43 that allows you to use F_y for f when using SCM E7.2b. This makes the computation much easier, is slightly conservative, and in few cases does the

When the web is not slender and can reach yield strength, then the nominal moment capacity, M_n, can be determined by the flexural yielding limit state.

Figure 8.2.1.8
Tension Flange Yielding

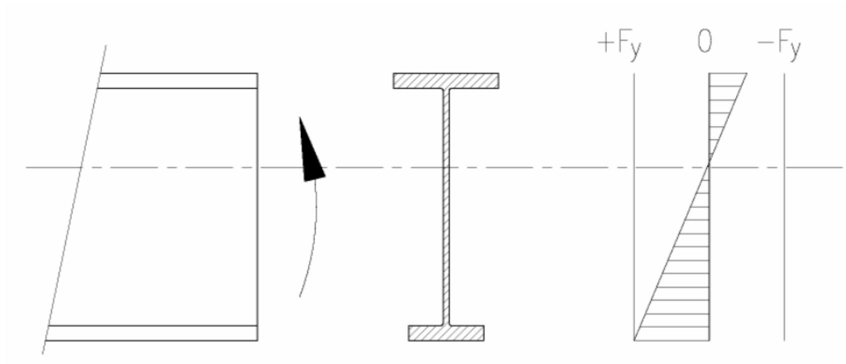
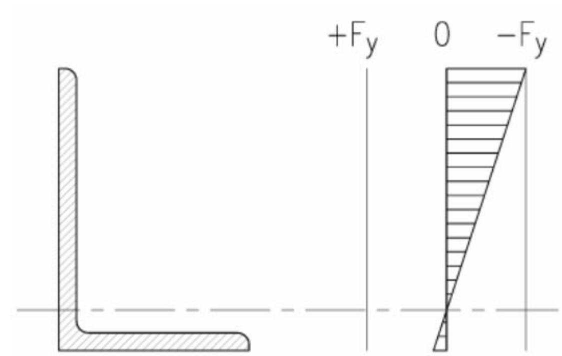


Figure 8.2.1.9
LLB Condition



<u>Local Leg Buckling (LLB)</u>

The limit state of local leg buckling (LLB) is peculiar to single angles in flexure. Similar to FLB and WLB, equations are provided for the three ranges of the buckling curve based on the width thickness ratio of the leg that is in compression.

Note that LLB applies only when the toe of the outstanding leg is in compression as shown in Figure 8.2.1.9.

8.2.2 Determining Applicable Flexural Limit States

The selection of the appropriate limit states, in the appropriate form, can be a bit confusing in the 13th edition of the SCM. To help the process, Table User Note F1.1 has been provided (see SCM page 16.1-45). *This table, however, does not preclude reading of the criteria statements the beginning of each section of chapter F.* It has been found that beginning students are often times confused when selecting the appropriate SCM criteria. To aid the beginning student, additional tables are provided here as another view of the selection for I shaped members.

The choice of Chapter F section depends on the shape of the section and the slenderness of the cross sectional plate elements. While the sections address many of the same limit states, they do so slightly different in each case. The following tables may help to select the correct limit states in the correct sections.

Table 8.2.2.1
Limit States and Applicable Chapter F Sections
For Doubly Symmetric I-Shapes (W, M, S, HP and built-up Sections)
Bent About Their Major Axis

Flange	Web	Limit State				
		Y	CFY	LTB	FLB	TFY
Compact	Compact	F2.1	----	F2.2	----	----
Compact	Non-Compact	----	F4.1	F4.2	F4.3a	F4.4
Compact	Slender	----	F5.1	F5.2	F5.3a	F5.4
Non-Compact	Compact	----	----	F3.1	F3.2a	----
Non-Compact	Non-Compact	----	F4.1	F4.2	F4.3b	F4.4
Non-Compact	Slender	----	F5.1	F5.2	F5.3b	F5.4
Slender	Compact	----	----	F3.1	F3.2b	----
Slender	Non-Compact	----	F4.1	F4.2	F4.3c	F4.4
Slender	Slender	----	F5.1	F5.2	F5.3c	F5.4

Table 8.2.2.2
Limit States and Applicable Chapter F Sections
For Singly Symmetric I-Shapes with Web Attached at Mid Flange
Bent About Their Major Axis

Flange	Web	Limit State			
		CFY	LTB	FLB	TFY
Compact	Compact	F4.1	F4.2	F4.3a	F4.4
Compact	Non-Compact	F4.1	F4.2	F4.3a	F4.4
Compact	Slender	F5.1	F5.2	F5.3a	F5.4
Non-Compact	Compact	F4.1	F4.2	F4.3b	F4.4
Non-Compact	Non-Compact	F4.1	F4.2	F4.3b	F4.4
Non-Compact	Slender	F5.1	F5.2	F5.3b	F5.4
Slender	Compact	F4.1	F4.2	F4.3c	F4.4
Slender	Non-Compact	F4.1	F4.2	F4.3c	F4.4
Slender	Slender	F5.1	F5.2	F5.3c	F5.4

Table 8.2.2.3
Limit States and Applicable Chapter F Sections
For Channels Bent About Their Major Axis

Flange	Web	Limit State	
		Y	LTB
Compact	Compact	F2.1	F2.2
Compact	Non-Compact	SCM Chapter F makes no provisions for these conditions. There is some vague reference in F12 to using principles of mechanics to determine the critical stresses. The goal is to keep the actual stress below the critical stresses determined by principles of mechanics.	
Compact	Slender		
Non-Compact	Compact		
Non-Compact	Non-Compact		
Non-Compact	Slender		
Slender	Compact		
Slender	Non-Compact		
Slender	Slender		

8.2.3 Flexural Yielding Limit State

As described in the section on flexural limit state behavior, the flexural yielding limit state represents that absolute maximum nominal moment that a section can support. At this condition the section is fully yielded. The limit state for strong axis bending in I-shaped member is found in SCM F2.1 and for weak axis bending in SCM F6. Other situations are covered in other sections of SCM Chapter F and are similar to those presented here.

The Limit State

The basic limit state follows the standard form. The statement of the limit states and the associated reduction factor and factor of safety are given here:

LRFD	ASD
$M_u \leq \phi_b M_n$	$M_a \leq M_n / \Omega_b$
Req'd $M_n = M_u / \phi_b \leq M_n$	Req'd $M_n = M_u \, \Omega_b \leq M_n$
$M_u / (\phi_b M_n) \leq 1.00$	$M_a / (M_n / \Omega_b) \leq 1.00$
$\phi_b = 0.90$	$\Omega_b = 1.67$

The values of M_u and M_a are the LRFD and ASD factored loads, respectively, applied to the flexural member.

In this case M_n is the nominal flexural yielding strength of the member. For doubly symmetric compact I-shaped members and channels bent about their major axis:

$$M_{nx} = M_{px} = F_y Z_x \text{ for strong axis bending (SCM equation F2-1)}$$

$$M_{ny} = M_{py} = \min(F_y Z_y,\ 1.6 F_y S_y) \text{ for weak axis bending (SCM equation F6-1)}$$

Where:

- M_p is the plastic flexural strength of the member.
- F_y is the material yield stress.
- Z is the plastic section modulus for the axis of bending being considered.

<u>Sample Spreadsheet Calculation</u>

The following spreadsheet example computes the flexural capacity, about each principle axis, for a typical W section. The input values are in the grey shaded cells and the results in the yellow highlighted cells.

Flexural Yielding

Section:	W10X49			Steel:	A992		
	Z_x	60.4	in^4		F_y	50	ksi
	Z_y	28.3	in^4				
	S_y	18.7	in^3				

M_{nx} =	**251.7**	**ft-k**
M_{ny} =	**117.9**	**ft-k**

8.2.4 Lateral Torsional Buckling Limit State

As mentioned earlier, Lateral Torsional Buckling (LTB) is a strong axis phenomena. It need not be considered for weak axis bending. The equations for each of the cases shown in SCM Table User Note F1.1 are found in the Chapter F sections referenced in the table. The general form used to compute LTB effects is the same for cases F2, F3, F4, and F5. The differences are in the computation of the key quantities (L_p, L_r, M_p, and M_r, see Figure 8.2.1.5) and the details of the equations used in various buckling ranges.

Note also that web slenderness is considered in several of the LTB cases. As a result, Web Local Buckling (WLB) is integrated in the LTB equations, making WLB only a consideration for HSS and other square and rectangular tubes.

<u>The Limit State</u>

The basic limit state follows the standard form. The statement of the limit states and the associated reduction factor and factor of safety are given here:

LRFD	**ASD**
$M_u \leq \phi_b M_n$	$M_a \leq M_n / \Omega_b$
Req'd $M_n = M_u / \phi_b \leq M_n$	Req'd $M_n = M_u \Omega_b \leq M_n$
$M_u / (\phi_b M_n) \leq 1.00$	$M_a / (M_n / \Omega_b) \leq 1.00$
$\phi_b = 0.90$	$\Omega_b = 1.67$

The "Hunt & Peck" Method

The Hunt & Peck method can always be used, regardless of the controlling flexural or shear limit state. This method involves searching through the section table using some means (i.e. algorithm) for determining the next section to try.

Once a selection is chosen, then all the limit states are computed to show that it satisfies all the applicable limit states. If it does not, a new member is selected based on the controlling limit state.

One way of implementing this method, is to search through a given size category (say the W18s) for the lightest section that works, then move to the other size categories. As each size category is investigated, only sections with weights less than the current best choice are considered.

The "Brute Force" Method

This method always works regardless of the controlling limit state and is best done with a spreadsheet or computer program.

In this method, a table is made that has all the sections you wish to consider (such as all the I shapes, all the rectangular HSS shapes, etc) and their section properties needed to compute the limit states. Additional columns are then added to compute the various limit states. Once all the limit states have been computed, the sections with violated limit states are deleted and the remaining table sorted by weight.

Conceivably, you can create a big table for beam design for all the sections of a particular type then just copy this over and do the eliminations and sorts for a given problem.

This method is computationally intensive but very easily done in a spreadsheet. It will always get the best result, after all the programming is debugged!

8.6.2 Cover Plates

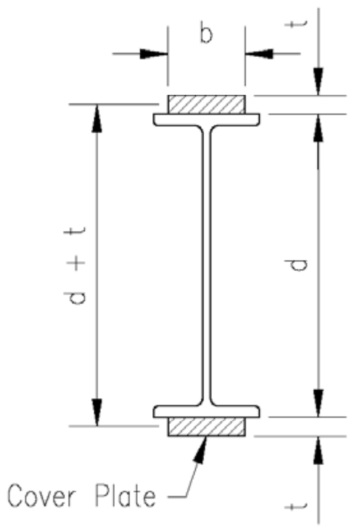
Cover plates are plates added to the flanges of beams to increase the flexural capacity of the beam over some portion of the beam. The use of cover plates in regions of high moment allows the use of a section of lesser weight and lesser flexural capacity to be used as the primary beam. This may result in a cost savings in some cases. Figure 8.6.2.1 shows a typical bridge beam with cover plates. Figure 8.6.2.2 shows a typical drawing of a W section with cover plates.

This technique is useful for compact beams

Figure 8.6.2.1
Beam with Cover Plates

that are not subject to the limit state of lateral torsional buckling (LTB). SCM F13.3 specifies many of the parameters associated with the design of cover plates.

Figure 8.6.2.2 Typical W Section with Cover Plates



Determining Size of Cover Plates

In the case of a compact beam not subject to LTB, the flexural limit state is stated as:

$$\text{Req'd } M_n = (M_u/\phi \text{ or } M_a\Omega) \leq F_y Z_{total}$$

Adding cover plates increases the Z of the section. For symmetrical cross sectionss with symmetrically applied plates, the design inequality becomes:

$$\text{Req'd } M_n \leq F_y\ (Z_{section} + Z_{plates})$$

For design purposes, this equation can be re-written as:

$$Z_{plates} \geq (\text{Req'd } M_n\ /\ F_y) - Z_{section}$$

For symmetrical plates, Z is the area of one plate times the distance between the centers of the two plates, so the strength requirement for symmetrical cover plates becomes:

$$Z_{plates} = bt\ (d+t) \geq (\text{Req'd } M_n\ /\ F_y) - Z_{section}$$

Where

- d is the overall depth of the steel section to which the cover plates are being added and
- t is the thickness of the cover plates.

For unsymmetrical plates (i.e. the cover plates are of different sizes or a cover plate is applied to only one flange), the Z for the whole section must be recomputed using basic concepts. This will involve finding the centriodal axis, locating the center of the areas above and below the centriodal axis, then finding Z by:

$$Z_{total} = (A_g/2)\ (\text{distance between the centroids of the two halves})$$

A restriction on the relative values of b and t is the requirement that the plate be compact. As the plate is generally connected to the flange with welds or bolts on both sides, the cover plated is considered a stiffened element and SCM Table B4.1 case 12 applies:

$$b/t \leq 1.12\ \text{sqrt}(E/Fy)$$

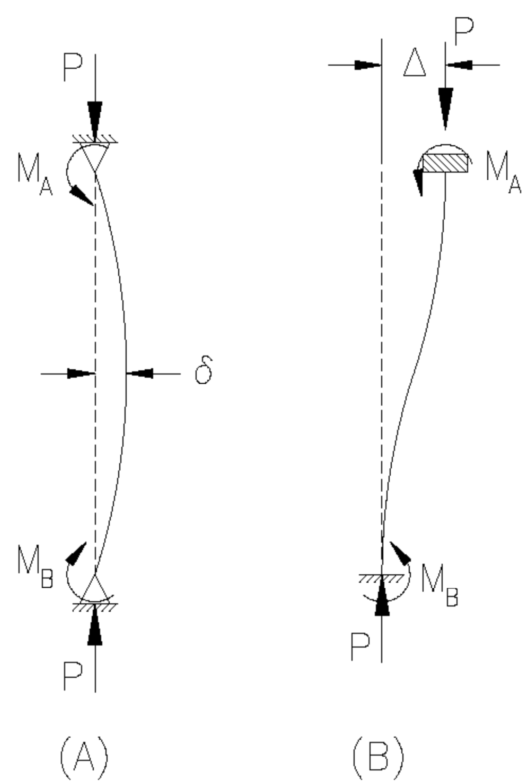
As there are two design variables, b and t, there are an infinite number of combinations of the variables that will result in a Z_{total} that matches $Z_{req'd}$. The best solution is generally the one that yields the smallest area, bt.

The end result of the analysis process is that the internal moment is larger than the moment predicted by normally used first order (i.e. equilibrium on the undeflected shape) analysis. It is important that the increased moment (some times referred to as a "magnified" moment) be used when comparing required strength to actual strength. Failure to do so is non-conservative.

Note that each member will have two principle axes. The second order effects can be considered independently to find the magnified moment about each axis.

Member bending and column stiffness have a major impact on the magnitude of the magnified moments. The stiffness of the member is a function of its section properties (I_x and I_y) and material (E) as well as the member support conditions. The values for I_x, I_y, and E are readily determined from the member selected and the material that it is made from. Determining the support conditions often takes a bit more effort and will require an understanding of how the member fits into the finished structure.

Figure 9.3.2
Braced vs. Unbraced Columns

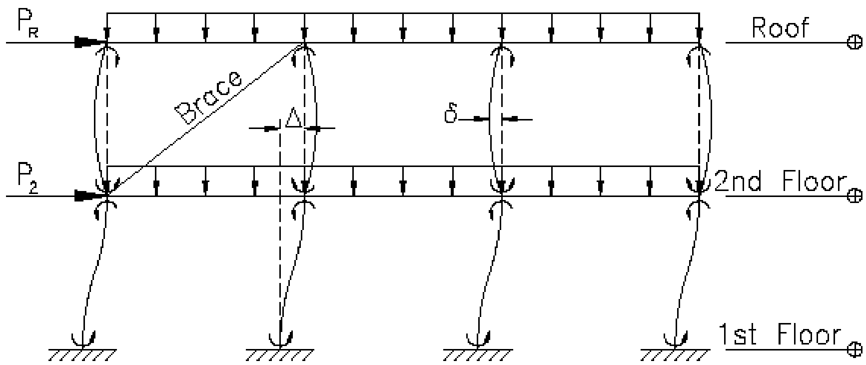


The first major concern when considering member support conditions is to determine if the member is "braced" or "unbraced". The location of maximum deflection, and hence the magnitude of the second order effects, are different in each case. Figure 9.3.2 illustrates typical deflected shapes of (a) a braced frame and (b) an unbraced frame. Note the different locations of maximum deflections. The nature of the end conditions will also be important to determining the buckling slenderness of the member.

Note that the SCM refers to axial force members in braced frames as being "*sidesway inhibited*" while the same members in unbraced frames as being "*sidesway uninhibited*".

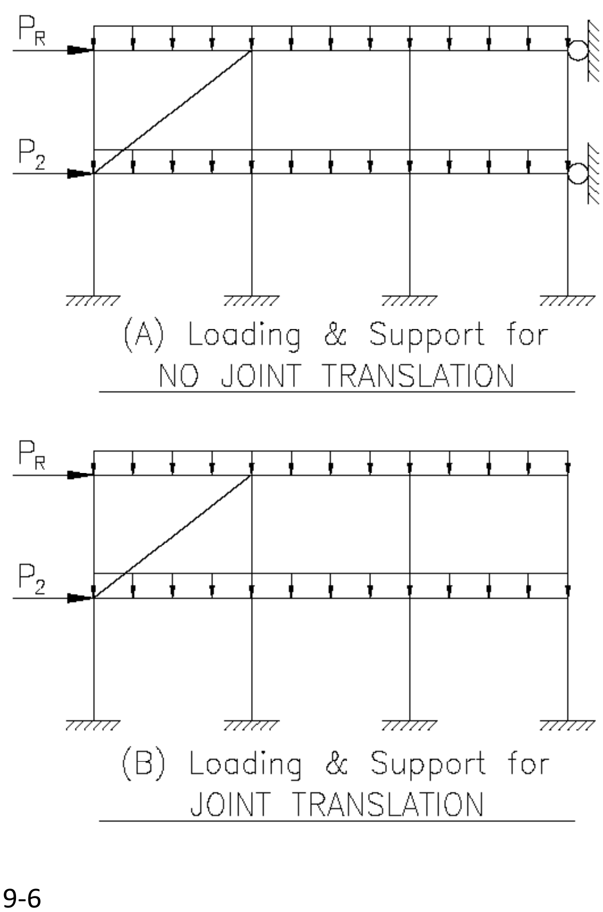
Also, for an axial force member in a braced building frame, the deflection, δ, between the ends of the member is predominate and independent of other members in the same level (i.e. between floor levels where their end points are all connected together by framing and a floor or roof system). In an unbraced building frame the predominate deflection is the relative lateral joint translation of the two ends, Δ. Since each end is connected to a floor or roof system, no column can displace laterally unless all other columns in the same level displace equally. Consequently a lateral deflection, Δ, in an unbraced frame member is a function of all the other members resisting the deflection in the same level.

Figure 9.3.3
Effects of Bracing



In Figure 9.3.3, it can be seen that the columns between the roof and second floor do not have significant lateral joint translation IN THE PLANE SHOWN because of the brace. Consequently there is little or no moment induced in the columns as the result of the lateral loading. The column moments result from eccentrically applied beam reactions, moments from any continuity in the connections and unbalanced loading, or from laterally applied loads to the member itself. The deflections creating the second order effects are along the length of the member and the relative lateral displacement of the ends is near zero. The displacements in a column are predominately the result of the moments that column sees as opposed to moment due to frame lateral translation.

Figure 9.3.4
Two Load Cases



For the columns between the first and second floors, the predominate displacements are caused by the lateral forces applied to the building that cause lateral displacement in the frame and are a function of the stiffness of all the columns in the level that are contributing to the resistance of lateral displacement IN THE PLANE SHOWN. All of the columns in the level have the same lateral joint displacement, Δ.

As a result of these two behaviors, the magnification of moments that result from loads that cause lateral joint translation is different than the magnification of moments resulting from loads that don't cause lateral joint translation. The normal approach to computing the magnified moments is to do two separate analyses: one with the structure restrained against lateral translation and one without lateral restraint. Figure 9.3.4 shows a simplistic version of the loadings for the two different analyses.

The moments and axial forces from the restrained analysis shown Figure 9.3.4a are

In composite steel-concrete beams, the connection between the steel beam and concrete slab is typically accomplished by welding an object to the top flange of the beam that engages the concrete directly. This is most frequently done with steel studs that are quickly and easily welded to the beam. The studs can be seen in Figure 10.2.2.

Other methods of shear connection include small channels welded transverse to the beam or a spiral coil of small diameter rod welded to the top flange.

Since the concrete is generally at an extremity of the beam, it is ineffective in resisting beam shear. As a result, the presence of composite action is ignored when computing the shear capacity of the beam.

Figure 10.2.2
Typical Welded Studs

Computation of the nominal flexural strength, M_n, of the beam is computed either using a strength method or an elastic method. If the steel section's web is sufficiently compact (and all the rolled sections in the inventory are compact for $F_y \leq 50$ ksi) then strength methods are used. For plate girders (i.e. built up members) with slender webs, elastic stress distribution methods are used.

For strength methods, all the steel is assumed to be fully yielded (compression above the plastic neutral axis and tension below the plastic neutral axis) and the concrete is at its ultimate strength in the compression zone. The nominal moment capacity, M_n, is taken as the internal moment that results from the forces on the section components.

For the elastic method, elastic stresses are computed using the bending stress equation (Mc/I) on a transformed section.

This chapter focuses primarily on the strength method.

10.3 Shear Strength

Since the concrete does not contribute to the shear strength of the composite beam, SCM I3.1b states that "The available shear strength ... shall be determined based upon the properties of the steel section alone in accordance with Chapter G."

The computation of shear strength was discussed in this text in section 8.3.2.

10.4 Flexural Strength

The flexural strength requirement is found in SCM I3.2a. Flexural strength is typically computed by either strength or elastic methods. In this course, we will consider the strength methods since they can be applied to all sections in the inventory when $F_y \leq 50$ ksi.

The Limit State

The basic limit state follows the standard form. The statement of the limit states and the associated reduction factor and factor of safety are given here:

LRFD	**ASD**
$M_u \leq \phi M_n$	$M_a \leq M_n/\Omega$
Req'd $M_n = M_u/\phi \leq M_n$	Req'd $M_n = M_a\,\Omega \leq M_n$
$M_u / (\phi M_n) \leq 1.00$	$M_a / (M_n/\Omega) \leq 1.00$
$\phi = 0.90$	$\Omega = 1.67$

The values of M_u and M_a are the LRFD and ASD factored loads, respectively, applied to the beam.

Nominal Moment Capacity, M_n, by Strength Analysis

The nominal moment capacity, M_n, equals the internal couple formed by the tension and compression forces acting on the section below and above the plastic neutral axis.

The plastic neutral axis (PNA) is different than the elastic neutral axis in that it not necessarily located at the center of area of the section. The PNA is found by writing the equilibrium equation for forces in the axial direction in terms of the location of the PNA, then solving for the location of the PNA.

Typically the equation takes the form of:

$$\Sigma F_{longitudinal} = 0 = \Sigma(\text{Tension Forces}) + \Sigma(\text{Compression Forces})$$

The trick is in writing the expressions for the forces. The force calculations are generally in the form of a stress times an area. For the steel contribution to the forces at strength levels, the whole cross section is assumed to have yielded so the stress in the steel equals Fy. The forces in the steel are equal to:

$$T_s = F_y \text{ (area of steel below the PNA)}$$

$$C_s = F_y \text{ (area of steel above the PNA)}$$

Expressions must be written for determining the area as a function of PNA location. The location of these forces is at the center of their respective areas since the stress is uniform.

Computing the concrete compressive force is a bit more involved.

References

The reference list presented here is, admittedly, very brief. The items that made the list are generally those books and articles that were actually used and/or quoted while producing this test. This is not to say that the information presented in the text is base on the author's research. It is far from it. The specifications, theory, and practices used in the profession come from a wealth of research done over the centuries by countless competent engineers, scientists, and mathematicians. While other texts have taken the time to search out the core documents that represent the current practices this one, unfortunately, has not.

With that said, here are the few references directly used and/or quoted in this text.

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2. ASCE 7-05: ***Minimum Design Loads for Buildings and Other Structures***, American Society of Civil Engineers, Reston, VA, 2005.
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4. Geschwindner, L.F., ***Unified Design of Steel Structures***, 1st edition, John Wiley & Sons, Inc., 2008
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7. American Welding Society, ***AWS D1.1: Structural Welding Code - Steel***, current edition
8. ***ASTM Specification A307***, ASTM International, current version
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12. American Institute of Timber Construction, ***Timber Construction Manual***, 5th edition, John Wiley and Sons, Hoboken, NJ, 2005
13. Ruby, David I., ***All About Bolts***, Modern Steel Construction, May 2003
14. ACI 318, ***Building Code Requirements for Structural Concrete***, American Concrete Institute, latest edition.
15. Bjorhovde, R., Engstrom, M.F., Griffis, L.G., Kloiber, L.A., and Malley, J.O., ***Structural Steel Selection Considerations: A Guide for Students, Educators, Designers, and Builders***, SEI, AISC, ASCE Press, January 2001.

Table 2.1
ASCE 7-05 Load Combination Equation Permutations

LRFD

LRFD-LC1	1.4(D+F)
LRFD-LC2a	$1.2(D + F + T) + 1.6(L + H) + 0.5L_r$
LRFD-LC2b	1.2(D + F + T) + 1.6(L + H) + 0.5S
LRFD-LC2c	1.2(D + F + T) + 1.6(L + H) + 0.5R
LRFD-LC3a	$1.2D + 1.6L_r + (0.5 \text{ or } 1)^{*}L$
LRFD-LC3b	$1.2D + 1.6L_r + 0.8W$
LRFD-LC3c	$1.2D + 1.6S + (0.5 \text{ or } 1)^{*}L$
LRFD-LC3d	1.2D + 1.6S + 0.8W
LRFD-LC3e	$1.2D + 1.6R + (0.5 \text{ or } 1)^{*}L$
LRFD-LC3f	1.2D + 1.6R + 0.8W
LRFD-LC4a	$1.2D + 1.6W + (0.5 \text{ or } 1)^{*}L + .5L_r$
LRFD-LC4b	$1.2D + 1.6W + (0.5 \text{ or } 1)^{*}L + .5S$
LRFD-LC4c	$1.2D + 1.6W + (0.5 \text{ or } 1)^{*}L + .5R$
LRFD-LC5a	$1.2D + E + (0.5 \text{ or } 1)^{*}L + 0.2S$
LRFD-LC5b	$1.2D - E + (0.5 \text{ or } 1)^{*}L + 0.2S$
LRFD-LC6a	0.9D + 1.6W + 1.6H
LRFD-LC6b	0.9D - 1.6W + 1.6H
LRFD-LC7a	0.9D + E + 1.6H
LRFD-LC7b	0.9D - E + 1.6H

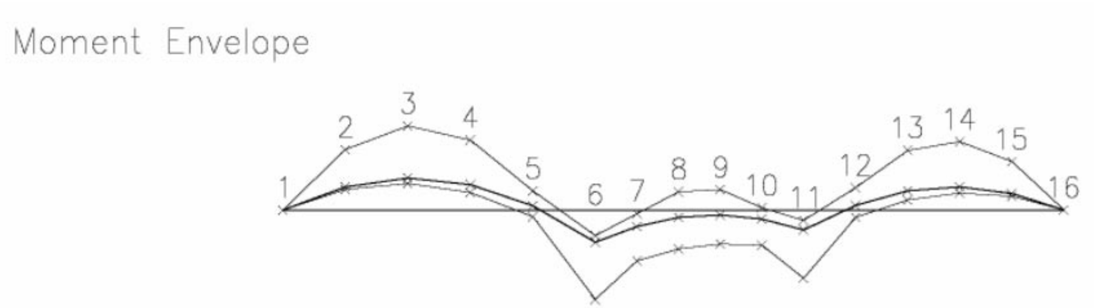
Note that the load factor for L in LRFD equations (3), (4), and (5) is permitted to equal 0.5 for occupancies in which the unit live load is less than or equal to 100 psf, except for garages or areas occupied as places of public assembly. Otherwise the load factor for L equals 1.0.

ASD

ASD-LC1	D + F
ASD-LC2	D + H + F + L + T
ASD-LC3a	$D + H + F + L_r$
ASD-LC3b	D + H + F + S
ASD-LC3c	D + H + F + R
ASD-LC4a	$D + H + F + 0.75(L + T) + 0.75L_r$
ASD-LC4b	D + H + F + 0.75(L + T) + 0.75S
ASD-LC4c	D + H + F + 0.75(L + T) + 0.75R
ASD-LC5a	D + H + F + W
ASD-LC5b	D + H + F - W
ASD-LC5c	D + H + F + 0.7E
ASD-LC5d	D + H + F - 0.7E
ASD-LC6a	$D + H + F + 0.75W + 0.75L + 0.75L_r$
ASD-LC6b	D + H + F + 0.75W + 0.75L + 0.75S
ASD-LC6c	D + H + F + 0.75W + 0.75L + 0.75R
ASD-LC6d	$D + H + F - 0.75W + 0.75L + 0.75L_r$
ASD-LC6e	D + H + F - 0.75W + 0.75L + 0.75S
ASD-LC6f	D + H + F - 0.75W + 0.75L + 0.75R
ASD-LC6g	$D + H + F + 0.75(0.7E) + 0.75L + 0.75L_r$
ASD-LC6h	D + H + F + 0.75(0.7E) + 0.75L + 0.75S
ASD-LC6i	D + H + F + 0.75(0.7E) + 0.75L + 0.75R
ASD-LC6j	$D + H + F - 0.75(0.7E) + 0.75L + 0.75L_r$
ASD-LC6k	D + H + F - 0.75(0.7E) + 0.75L + 0.75S
ASD-LC6l	D + H + F - 0.75(0.7E) + 0.75L + 0.75R
ASD-LC7a	0.6D + W + H
ASD-LC7b	0.6D - W + H
ASD-LC8a	0.6D + 0.7E + H
ASD-LC8b	0.6D - 0.7E + H

upper and lower envelope values come from different live load arrangements. Other methods would require the analysis of all the different load arrangements to get the same results.

Figure CB.4.1.6
Resulting Moment Envelope



Summary

The same approach used here for moment envelope determination could also be used to determine shear or deflection envelopes for the same structural system. The method can also be used with arrangements of point loads and moments as well as distributed loads on more complex structures. It can be used whenever superposition is applicable.

The method of Influential Superposition can be used to reduce the number of analyzes required for the determining envelopes of continuous structures. The method can be easily implemented in numerical analysis programs in order to reduce the number of load cases to be developed by the user and to reduce the computational effort.

The method has the added advantage of reducing the potential for inadvertent neglect of all applicable live load arrangements.

CB.5 Example Problems

A spreadsheet based solution for the example problem can be downloaded from

http://www.bgstructuralengineering.com/BGSMA/ContBeams/BGSMA_CB_05.htm

The following section explains the solution provided in the spreadsheet file.

Example Problem CB.1

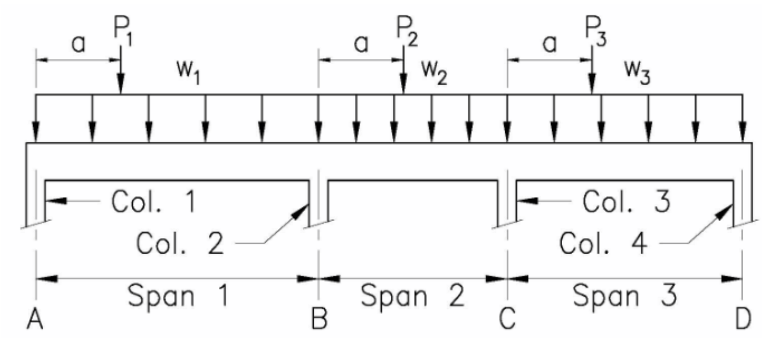
This problem presents a solution to a three span continuous beam with integral supporting columns. The spreadsheet solution will allow the user to change certain variables that will allow the investigation of the sensitivity of the method to various parameters. The end moments and shears are used to develop the shear and moment diagrams.

No example problem is provided here for influential superposition since that was done in some detail in Section CB.4.1.

CB.5.1 Example Problem CB.1

Given: A continuous beam over three spans with integral column supports as shown in Figure CB5.1.1

Figure CB5.1.1
Example Problem CB.1



Wanted: Develop the Shear and Moment diagrams for the beam using moment distribution. Use an electronic spreadsheet.

Solution: A spreadsheet is used for this problem so that it can be used to solve the problem for a wide array of variables. You will need to down load the spreadsheet for this problem to understand this discussion.

We are also making a big simplifying assumption in this problem. That is that the frame will not deflect sideways due to unsymmetrical stiffness and loading. This is only true if the frame can be assumed to be braced.

You will see, in the "given" section of the spreadsheet that input cells (the grey cells) have been provided for all the variables defined in Figure CB5.1.1. These values can be varied to study the sensitivity of the analysis to the various variables. Feel free to experiment with the values!

Determine Relative Rotational Stiffness Values and Distribution Factors

For this step, we need to know the rotational stiffness of the various members framing into each joint. In this problem, we assume that all members are made of the same material. This means that the modulus of Elasticity term is constant, so all we need are the member lengths and moment of inertias.

The moment of inertias are *the moments of inertia resisting bending in the plane of the frame*. For the beams, this is normally I_x. For the columns, you need to know how they are oriented before inputting values for the moment of inertia. If the columns are bent about their weak axis in the plane, the enter I_y else enter I_x.

The relative stiffness term for each structural element then is computed as I/L.