Basic Structural Concepts

(For The Non-Engineer)

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CHAPTER 1
LOADS, FACTOR OF SAFETY, and ENGINEERING LAW

1. General. It is because structures experience loading of some sort or another that I have a job. If there were no loads, there would never be any structural failures, and thus there would be no building departments requiring engineering. Unfortunately, the reality is that every day in the construction industry there are structural failures. What is particularly disturbing about this is that the vast majority of all structural failures are *avoidable* if the builder had a clear understanding of how structures behave; and the designers (architects and engineers) do their job correctly and thoroughly. This book focuses on the former, however, an astute contractor or other non-engineer can go a long way in keeping the architects and engineers honest in the proper performance of their jobs (I welcome it in my practice).

1.1. Causes of Failures. Sometimes structures fail due to poor design by the engineer. In these cases, most often the failure occurs not because a main beam or column breaks in two, but rather a *connection* fails. Engineers have a tendency to pay extraordinary attention to correctly sizing main structural members (beams, columns, shear walls, etc.), but become hurried or lax when it comes to connecting the whole thing together. Sometimes, particularly in residential and light commercial construction, the engineer leaves connection design entirely up to the contractor.

1.1.1. The Contractor. From my own experience in the construction business, I know that the contractor is expected to wear many hats. In the course of assembling a structure, among other things, (s)he has to:

- Determine the best way to do it
- Fill in the design aspects that the engineer / architect left out
- Keep the jobsite safe
- Determine the proper sequence of construction
- Know all about scaffolding, temporary bracing, and shoring, and apply them correctly
- Maintain positive relationships with the Owner, Engineer, Architect, and Building Official, Bank, Insurance Agent, L&I Official, Significant Other, etc.
- Make a buck

Many, many construction failures occur during construction. All too often the engineer designs the finished building, but pays no attention to the conditions that occur during the construction of that building. This is why it is crucial for the contractor to understand basic structural concepts.

Some failures occur because the contractor did not bother to read the plans, or worse, did read them, but then did something different without consulting the architect or engineer. Both of these are 100% avoidable with a little diligence and open, two-way communication.

Most failures occur during vicious acts of nature like wind storms, snow storms, or earthquakes. Some of these failures are due ultimately to poor construction or poor design, which are manifested during the extreme loading event. Other of these failures are due to inadequate code requirements (particularly for older structures that were code compliant at the time of their construction). The poor construction / poor design failures are avoidable; the failures due to insufficient code requirements are typically chalked up to 'acts of God'. All of this is the typical fodder for many a lawsuit.

The bottom line for everyone in the construction industry from the owner to the insurance agent, banker, contractor, and architect is to be as educated as possible concerning the structural concepts involved in safe construction. The more we all know, the more we can help each other recognize potential problems so that they can be remedied before the failure. And that is the true goal – to keep the failure from happening; the preservation of property; the saving of life and limb.

1.2. Loads. All structural analysis or design starts with a determination of *loading*. The basic types of loading are:

1.2.1. Dead Load. This is the weight of the structure itself and any other permanent, fixed-in-place loads. Dead load is considered a *gravity load*, that is, the loading occurs due to the forces of gravity on the mass of load. Gravity loads always act in a downward direction, as opposed to *lateral loads* which act in a sideways (or sometimes upwards) direction.

Some common material weights (dead loads) are as follows:

- Wood – Approximately 40 pounds per cubic foot (pcf) +/-, depending on species and water content.
- Steel – 490 pcf.
- Concrete, normal weight, reinforced or plain – 150 pcf. Reinforced concrete walls or slabs: 4-inch thick = 50 pounds per square foot (psf). 6-inch thick = 75 pounds per square foot (psf). 8-inch thick = 100 psf. 10-inch thick = 125 psf. 12" thick = 150 psf.
- Masonry, poured full of grout – 140 pcf. Masonry walls full of grout: 6-inch wall
- Water – 62.4 pcf.

1.2.2. Live Load. This is the weight of anything that is not dead load, such as: people, cars, furniture, etc. There are two tables (16-A and 16-B) in the 1997 UBC that provide minimum live loads for use in design. Live loads are also gravity loads. Some common live loads follow:

- Residential floors and decks = 40 psf (pounds per square foot)
- Residential balcony = 60 psf
- Office = 50 psf. Also a 2,000 pound point load located anywhere must be analyzed.
- Storage = 125 psf for light storage and 250 psf for heavy storage. If residential storage, 40 psf.
- Garages = 100 psf for general storage or repair. For residential = 50 psf. In both cases point loads of 2,000 pounds must be analyzed as well. See the UBC for complete description of how wheel loads are applied.
- Balcony railings and guardrails = 50 pounds per lineal foot applied horizontally to the top rail.

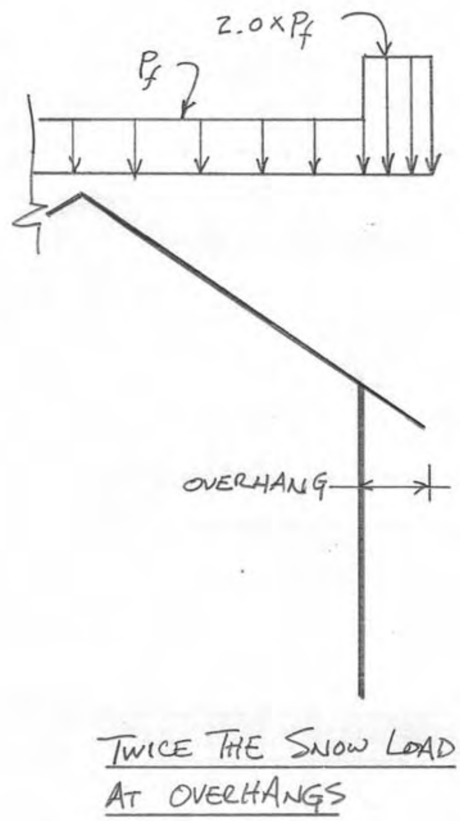
1.2.3. Snow Load. This is actually a live load, but is commonly considered separately in structural calculations. The amount of snow load that is used in design depends on where you are geographically. The UBC and / or most building departments will supply you with the snow loading to be used in their jurisdiction.

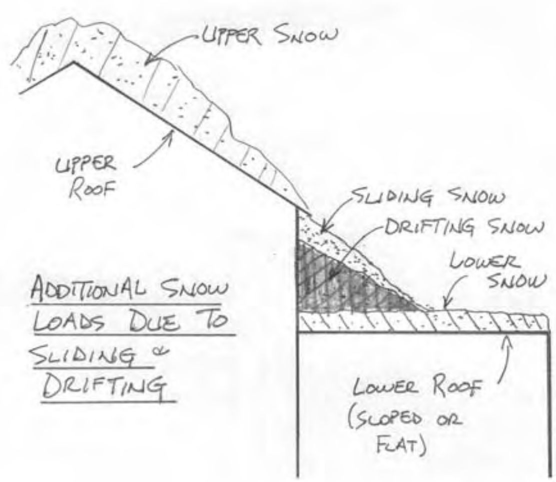
1.2.3.1. Ground Snow Load. The snow load supplied by the UBC and your building department is a ground snow load, i.e. the weight per square foot of snow lying on the ground. This is different than a roof snow load. The UBC allows a modification of ground snow load for application to roofs and balconies. For most residential and commercial facilities this modification is a reduction, as follows: for buildings in open terrain, use 60% of ground snow load; for structures in densely forested or sheltered areas, use 90% of ground snow load; and for all other structures, use 70% of ground snow load. There is an *increase* in ground snow load if your building is an 'essential facility'. Roof snow load may be *decreased* for snow loads in excess of 20 psf and roof slopes greater than 30 degrees (about 7:12) in certain cases.

1.2.3.2. Unbalanced Snow Loads (see 1997 UBC appendix Chapter 16). If you have a gable roof, consideration must be given to different amounts of snow on each side of the ridge (unbalanced condition). The UBC gives elaborate directions for the application of this depending on roof slope, number of gables, and direction of ridge lines.

1.2.3.3. Snow on Eave Overhangs. Where you have eave overhangs, these must be designed for double the normal roof snow load to account for ice dams and snow accumulation.

1.2.3.4. Drift Loads on Lower Roofs, Decks, and Roof Projections. The UBC requires any surface that can collect drifting or falling snow from upper portions of a structure be designed for this increase in snow load. There are elaborate formulas and methods used to determine this.





Also, if you have vertical projections such as parapets, mechanical equipment, or deck railings that can collect blowing or drifting snow, there is an increase that must be applied too.

1.2.3.5. Rain on Snow. If you are in a geographical area where it is likely that rain may fall on and be absorbed by snow, there is a 5 pound per square foot (psf) increase to the snow load that may be applied depending on the steepness of the roof (not necessary for slopes greater than 1:12) or the amount of snow (not necessary for 50 psf ground snow load or more).

1.2.4. Wind Load. This is the force on the structure due to strong winds. It is a lateral load. See chapter 7 for a complete description.

1.2.5. Earthquake Load. This is the force on the structure due to seismic activity. It is a lateral load. See chapter 7 for a complete description.

> ***Question**: The plans examiner at the building department recently told me that I need a lateral analysis on my proposed house, but not a gravity analysis. What was he talking about? **Answer**: He was saying that he felt your house would be susceptible to damage by lateral loads, i.e. wind and earthquake, and an engineer should analyze it for those loads. He was also saying (perhaps indirectly) that all of the beams, columns, and footings, which resist gravity loads appeared okay to him, and no analysis of them was required.*

1.3. Load Combinations. Engineers must apply the worst combination of loads that could occur simultaneously to the structure and it's elements. By worst, I mean the combination that causes the *highest total loads* on the structure, and thus the *highest stresses* in the individual structural members. The Uniform Building Code (UBC) prescribes these load combinations. One of the combinations that must be examined, for example is: Live Load + Dead Load + 1/2 Snow Load + Earthquake Load. You will notice that only half of the snow load is used in this combination. The UBC recognizes that the chances for a maximum wind event happening at the same time as a maximum snow event, happening at the same time as a maximum earthquake are very small. The code therefore does not require full simultaneous loading of these various types of loads.

1.4. Design. Once the loads are determined, structural analysis begins by breaking the structure down into its individual elements and analyzing each under the worst loading combinations. The *loads* on the members are converted into *stresses* within the member. Design of structural members is nothing more than comparing *applied stresses* to *allowable stresses*. Applied stresses are calculated based on the applied loads.

> ***Question**: How do engineers know how much stress a particular wood, steel, or concrete member can take? **Answer**: There exist associations who's purpose it is to test and publish <u>allowable stresses</u> for the particular material that they represent. Predictably, there is a steel association (AISC, American Institute of Steel Construction) a concrete association (ACI, American Concrete Institute), and several wood associations (WWPA, Western Woods Products Association to name one). There are also associations for masonry, plywood, light gauge steel, steel plated wood trusses, welding, and others. All of these groups publish documents that provide <u>allowable stresses</u> for their products. A fair amount of this published*

information winds up in building codes, such as the UBC. So, to answer the question, once the applied stresses are determined through calculations, they are compared to the published allowable stresses. If the applied stresses are higher than the allowable, the structural member in question 'does not work', or 'does not calc', or 'does not meet code'. Regardless of how it is termed, either a stronger member must be selected or some other measure taken (reduce loading, shorten span, etc.) to reduce the applied stresses.

1.5. Factor of Safety.

Question: *If a member does not calc, but it is installed anyway (for shame!) will it fail?* ***Answer***: *It depends on how much overstressed (or undersized) it is. In many cases, it will not fail because the published allowable stresses have Factors of Safety built into them, read on for more.*

When the various associations, as listed above, test their materials for allowable stresses, they find out how much stress is required to actually break a certain size sample. They test many, many members to be sure they get a good representative sampling of the actual stresses that the material will take. After they are sure of how the material behaves under load, from a light load all the way up to rupture, they determine the material's *ultimate strength* (strength at the point of rupture), and it's *allowable stress* (stress just before permanent deformation starts to occur). Factors of safety are built into these strength / stress values, which are then published for engineers to use in design. The factors of safety effectively reduce the amount of load (and thus the amount of stress) that engineers may place on the various structural materials. These factors of safety typically vary from about 1.5 to 2.5. This means that every structural member designed in accordance with code is oversized by at least 50%, and many times by 150% or even more.

Question: *It seems like factors of safety result in much bigger members than are needed, why this huge waste?* ***Answer***: *Did you ever see a builder who built every single thing in exact accordance with the plans? Did you ever know a perfect engineer? In both cases the answer is 'NO'. I can't tell you how many times I've seen one of my designs goofed in the field because the contractor forgot to put in the anchor bolts, or the plumber had to run 3" drain line through the beam, or the framer split the heck out of the rafter heel trying to get the right number of nails into it. Or consider the case of the engineer who celebrated the Seahawks win a little too exuberantly the night before, and was only partially functional the next day when he was designing beams, columns, and shear walls. Let's face it, stuff happens in the design office and in the field that was not anticipated. That is a big reason why factors of safety are there.*

This concept is illustrated further in masonry design. If 'special inspection' during construction is to be provided, the engineer can DOUBLE the allowable stresses used in the design. The masonry association is the only one, to my knowledge who allows this. What is interesting about this is that most times, contractors prefer to use much more steel and masonry and forfeit the lower factor of safety, rather than endure the scrutiny of and pay for special inspection.

***Question**: Does this inherent factor of safety explain why so many of the older buildings that were constructed before codes were invented, and which do not even come close to meeting today's codes are still standing? **Answer**: Yes, most definitely. But there is another reason as well. If you recall, design is performed to the worst possible combination of the worst possible loads that <u>may</u> ever be experienced. Many times, over the life of a structure, such extreme loading never occurs. Right now, this country is filled with buildings that do not meet current codes, but which have never been 'put to the test', i.e. have never experienced a code maximum load combination. If they ever do, and their factors of safety are low or non-existent, they WILL fail, and people will be hurt or killed. This is exemplified every time there is a big earthquake, large snow event, or severe wind storm; there are building failures all over the place. So when a contractor wants to defy the advice of his engineer or building official with the old adage, "I've been building it this way for 50 years and never had one fail yet." now you know why.*

1.6. Duration of Load. The UBC recognizes that factor of safety is built into allowable stresses, and that most construction materials can withstand very high forces if the forces are only applied for a short duration. So, the UBC allows an increase in allowable stresses for short term loading such as wind and earthquake (1997 UBC section 16.12.3.2 and elsewhere.) It is important to note that these allowable stress increases are only valid when the Allowable Stress Design method (ASD) is being used (as opposed to the Load Reduction Factor Design (LRFD) method.) Allowable stress increases of 33% are allowed for concrete, steel, masonry, wood, and foundation soils during a wind or seismic event if the ASD method is used, and certain alternate load combinations (per 16.12.3.2) are used. So for example, let's say an engineer is designing a steel frame that will resist lateral loading from wind and earthquake. The allowable bending stress for most steel under normal loading is 24,000 psi. For the lateral load analysis, he may use 24,000 x 1.33 = <u>32,000</u> psi instead. This will result in smaller frame members.

Other short term load increases are allowed as listed in specific UBC sections for that certain material. For example, there is a table for use with wood in the 1997 UBC, Chapter 23, Table 2.3.2, that allows stress increases for various short term loading including snow (15%), roof construction live loads (25%), impact (100%), wind and earthquake (33%).

This section of code (in conjunction with the factors of safety built into all things designed per code) helps to explain why, for example, when roofers temporarily pile a big stack of plywood sheathing on trusses during construction, overstressing them by 100% or more, they don't break.

1.7. The Law Pertaining to Engineers. Every state has laws governing the professional responsibilities and conduct of licensed engineers. My primary state of licensure, Washington is no exception, in fact, Washington has very strict and copious laws in this area. I could write a separate book on this subject alone, but will refrain here, and only go into a few 'major ticket items' that everyone in the building industry should know about.

This and the following sections of the book pertain only to Washington State. Many other states have similar laws governing engineers, but to be sure of exactly what the law of your land is, you should check with the state licensing board of professional engineers.

1.7.1. Definition of Engineering. Engineering is broadly defined in Washington state law as any activity that requires engineering education and judgement. This includes structural analysis and design. Sizing a beam, or shear wall, or bolted connection IS engineering. Evaluating the strength of an existing structure is also engineering.

1.7.2. Performance of Engineering. Washington State law is very clear that only competent licensed engineers (and in some cases licensed competent Architects) may perform engineering. A corollary to this is that someone who is working under the *direct supervision* of a licensed engineer may also perform engineering. For the remainder of this book, when I say *engineer*, I am referring to all of the above.

> ***Question**: "I'm a builder, and I know how to use span tables to size joists and rafters. If I do so, am I violating the law?" **Answer**: Yes, unless you are also an engineer. In Oregon recently a court case held that only engineers could size structural members regardless of how the analysis was done (span tables, computer programs, etc.). The exception was sizing structural members for the purposes of preparing a bid or cost estimate; anyone could do that. However, when it came time to actually install the member, it's size had to be determined by an engineer.*

> ***Question**: "I was at the City building permit office, and the plans examiner did me a favor of sizing a few beams on my house plans that I was submitting. He is not an engineer, was he breaking the law?" **Answer**: Absolutely, yes. Remember, only an engineer, architect, or someone working under the direct supervision of a licensed engineer may, by law, perform engineering. Plans examiners may check a structure's compliance with code, but when it comes to actually performing the structural design, that must be done by an engineer.*

> ***Question**: "The plans examiner required engineering of a few Glu-Lam beams on my proposed house, but he didn't do it for me. I know a retired Boeing engineer. He's licensed, can he do this work?" **Answer**: It depends on whether or not he is <u>competent</u> to perform the specific tasks required. He is only competent if he has the experience and knowledge to do the work to the standard commonly accepted in the field of structural engineering. If he has never designed a timber beam, chances are that he is not 'competent' to do your specific task, and you will have to find another engineer. It is interesting to note that it is up to the engineer to determine his own competency. He is ethically bound by law to disclose to you whether or not he truly is competent to perform the task at hand.*

1.7.3. Structural Engineer. The last bit of state law that should be understood by all is what a 'structural engineer' really is. Most states have many classifications of types of licensed engineers. Typically (Washington included) structural analysis and design may be performed by qualified *civil engineers* or *structural engineers*. A *civil engineer* is schooled in structural design as well as in water, sewer, roads, and storm drainage. Many civil engineers choose to specialize in the water, sewer, storm drainage, and roads subdisciplines, and do not ever perform structural analysis. Many, however, go the other way, and specialize in structures. Some rare birds, like myself, go both ways.

A *structural engineer* is a civil engineer who has lots of professional level structural experience, and who has passed a supplemental 16 hour state sanctioned structural exam. There are relatively very few structural engineers in this state. Most engineers who practice in the structural arena are civil engineers, like myself, who have not either qualified or bothered to take the structural exam. Civil engineers who do structures for a living are normally just as qualified to do most structural analysis as a structural engineer.

Civil engineers, by law, may not use the term '*structural engineer*' or '*structural engineering*' in their advertising or in talking about their qualifications. Only structural engineers may use those terms.

***Question**: "I was told by my bank that I need a structural engineer to examine the foundation of my modular home. Are you saying that even though you are a licensed engineer, and you do structural analysis and design for a living, you can't do this?" **Answer**: If your bank knows what they are talking about, and understands the difference between civil and structural engineers, and they really are requiring a structural engineer, then no, I can not do this work. If however, (and most likely) your bank doesn't understand this quirk in engineering law, and they want a competent professional engineer, then yes, I can do it. A better way for the bank to word their request could be something like they require the services of a 'competent licensed professional engineer'. This makes it clear that they want an engineer, but do not exclude those subdisciplines of engineering which are qualified to do the work, but are not technically 'structural engineers'.*

CHAPTER 2
BEAMS AND OTHER BENDING MEMBERS

2. Introduction, Definitions. Beams and other bending members are perhaps the most common of structural members. They come in many shapes, sizes, and materials (wood, steel, and reinforced concrete are most common). Their primary function is to carry gravity loads over an opening or open area. They are termed '*bending members*' because they work by resisting primarily *bending forces*, as opposed to say *compression* (posts and columns) or *tension* (cables for instance are pure tension members). We sometimes lump all of the following into the category of 'beams' because they act primarily in *bending*.

2.1. Types of Bending Members. Following are a listing of structural members that function primarily in bending:

- **Joist**. Such as floor joist or ceiling joist. These are typically smaller size members, that are spaced relatively closely together, normally 16" to 24" apart (on-center, or O.C.), and support a horizontal diaphragm such as a floor or ceiling.

> ***Question****: What is the difference between a rafter and a purlin? Between a stud and a girt?*
> ***Answer****: See the following definitions.*

- **Rafters**. These normally refer to smaller size repetitive members used to frame a roof system. They are typically placed at an angle following the slope of the roof, and carry the roofing system.
- **Purlins**. These are also typically smaller size repetitive roof framing members (similar to rafters), but they span *between* roof trusses, frames, or beams, and are normally parallel to the ridge.
- **Girts**. These are typically smaller size repetitive wall framing elements that are placed *horizontally* between building frames. They are similar to studs, except that studs are placed vertically in a wall as opposed to horizontally for girts. Wall siding materials are attached to girts.
- **Beams**. These typically carry the load of other structural members, such as joists, rafters, posts, other beams, etc. They normally act alone (i.e. are not repetitive).
- **Simple Beam**. Any beam that is supported at it's ends, i.e. does not have multiple supports or cantilevers.
- **Continuous Beam**. Any beam that has multiple supports.
- **Cantilever Beam**. Any beam which has one or both of its ends extending beyond the support point(s).
- **Girder**. Generic name for a large beam which carries the load of other beams or other structural members.

2.2. Stresses in Bending Members.

> ***Question****: What are the three basic design elements that must be considered in designing any bending member?* ***Answer****: Shear stress, bending stress, and deflection*.

There are two basic types of *stresses* in all bending members that must be considered in design (deflection is not a stress).

2.2.1. Shear Stress. The best way to think of shear stress is to imagine a heavily loaded beam sitting on two posts. It is the beam's resistance to *shear* that keeps it from splitting or breaking immediately next to the post. To keep shear stresses low within the beam, a large *cross-sectional area* is required.

> ***Question**: Where along the length of any bending member is the shear stress typically the greatest? **Answer**: Directly next to the supports.*

> ***Question**: Is there a place in a bending member where the shear stress is zero? **Answer**: Yes, the shear stress in a simple bending member is zero at the point of maximum bending stress . This is important because this is the location where holes can be drilled or cored with the least effect on the strength of the member, as long as the holes are not cut through the top or bottom of the member. This is true for a simple bending member, but is not necessarily the case for a bending member which is continuous over an intermediate support(s).*

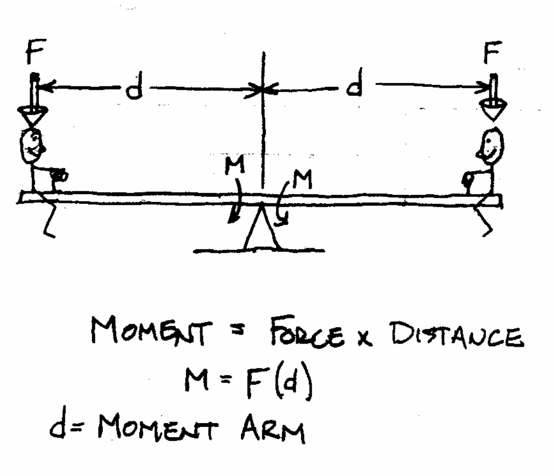
2.2.2. Bending Stress. The best way to think of bending stress is to imagine a long heavily loaded beam spanning between two posts. Consider the sag in the middle of the beam. The beam is experiencing large bending stresses at the sag.

> ***Question**: Where along the length of a simple bending member is the bending stress typically the greatest? **Answer**: The bending stress is usually the greatest at the sagging mid-point. This is always true for simple beams with uniformly distributed loads on them. If the load is not uniform however, (say more load is concentrated toward one end or the other), then the maximum bending stress will shift away from the center of the beam toward the heavily loaded area.*
>
> *Beams which are continuous over an intermediate support (non-simple) will not usually have the maximum bending stress in the middle of the span. Rather there will be three maximums: one at about each third point toward the end supports, and the other directly over the center support (see following section on 'continuous bending members'.*

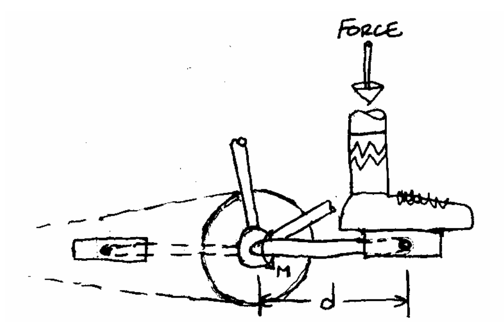
The best way to keep bending stresses low is to use *deep* beams, and / or short spans.

2.2.3. Moments. Engineers are always talking about *bending moments* or just *moments*. What is a moment? I heard a construction worker explain it one day. He said, "A moment is where you are up on a tall scaffold and fall off. You only got a *moment* to think about where you went wrong before you hit the ground." This is not what engineers are talking about when they discuss moments. When you think of *moments*, think of *bending*. Wherever a structural member is being bent due to load, the bend is the member's response to an applied moment.

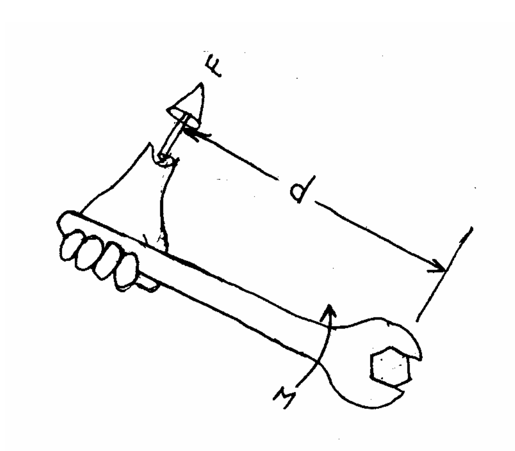
In it's simplest form, an applied moment is nothing more than a force exerted at some distance away from the support along a bending member, i.e. a force times a distance. Consider a teeter-totter that is balanced with two equal sized children. The child at each end exerts a (downward) force. Observing one side of the teeter-totter at a time, the force of the child times the distance from the child to the fulcrum (support) is the applied moment at the fulcrum.



A bicycle peddle is similar. When the peddle is horizontal, there is a moment in the peddle arm

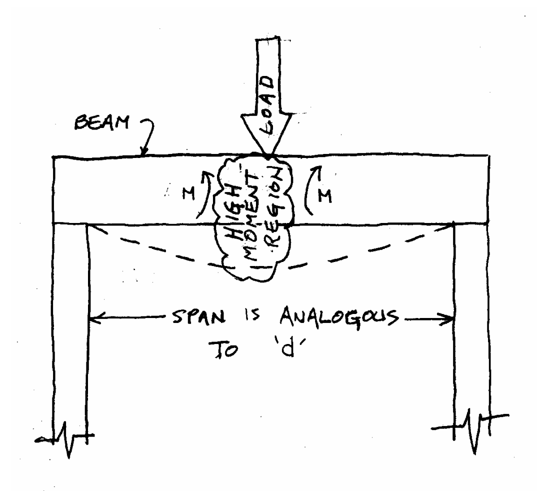


(or crank) that is equal to the force of the foot pushing down times the distance from the peddle to the bend in the crank arm. As another example, when you pull on a wrench, you create a moment in the wrench handle that is equal to the force you are exerting times the distance from your hand to end of the wrench.

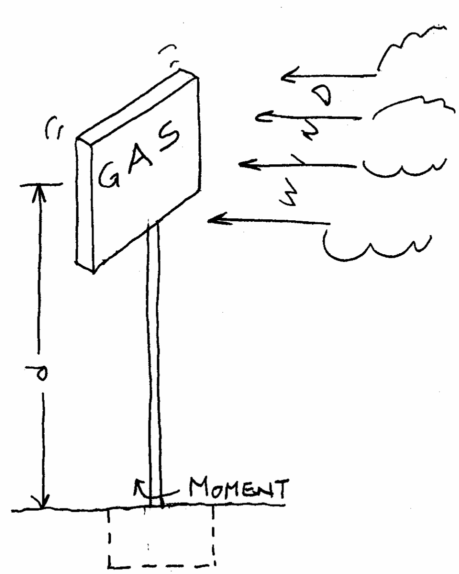


In all of the above examples, the applied forces over a certain distance create applied moments which in turn create bending stresses in the 'beams' (teeter-totter board, bicycle crank, and wrench handle). If these 'beams' are not of sufficient strength, they will break due to overstressing in bending.

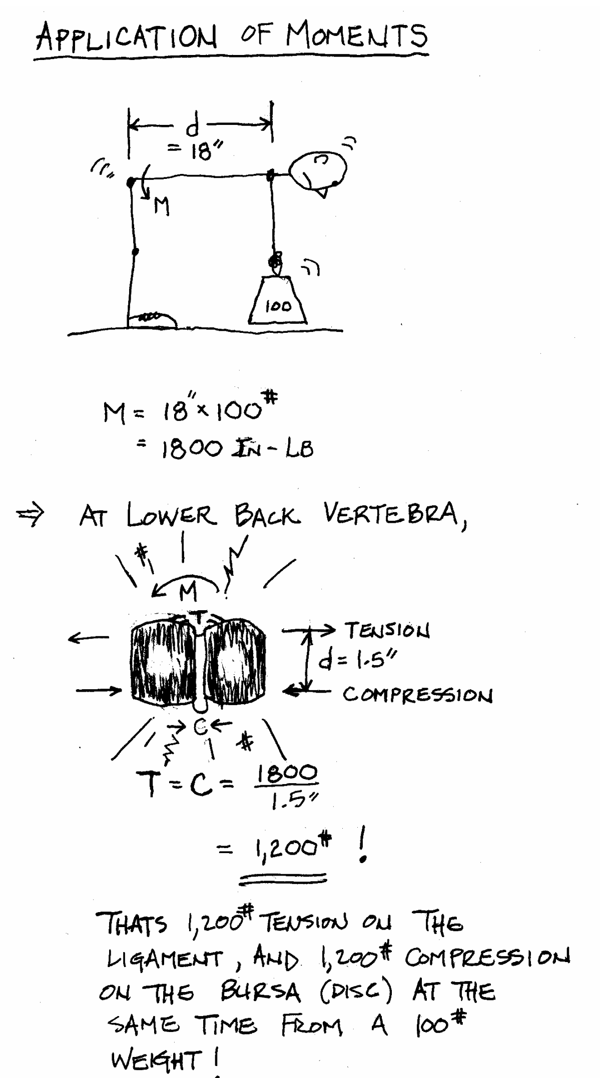
Following are two illustrations of moments in common structural members. The first shows the moment in the middle of a simple beam, and



the second shows the moment at the base of a tall cantilevered pole.

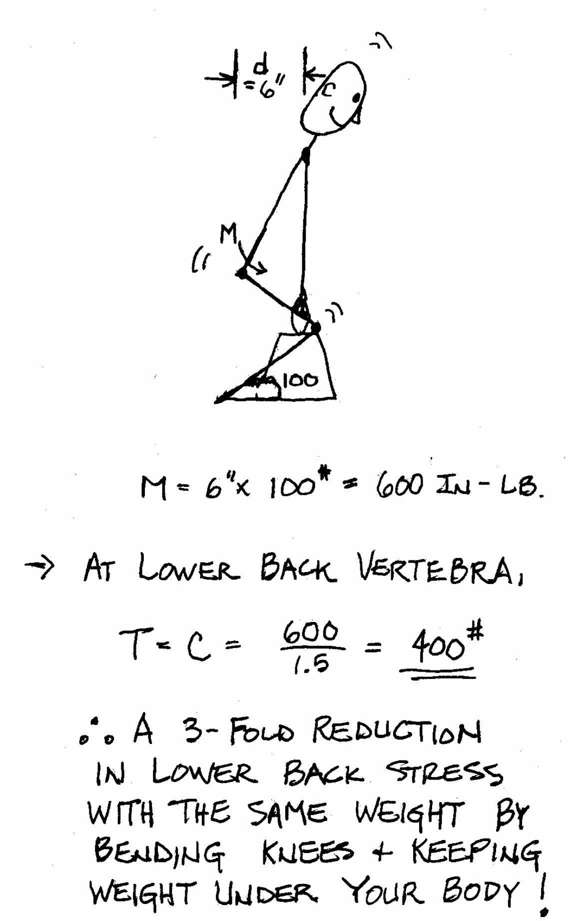


2.2.4. Moment Arm. You will occasionally hear the term *moment arm*. In the above examples, this is simply the distance that the force is away from the support, i.e. the distance that the child is from the fulcrum of the teeter-totter; and the distance from the bicycle peddle to the bend in the crank; and the distance from the person's hand to the bolt at the end of the wrench. In the case of the beam, the moment arm is analogous to the beam span. What is important to understand is this: the longer the moment arm, the greater the moment, and the greater the moment, the greater the bending stress within the beam. Span of a beam is analogous to moment arm, so the longer the distance between the supports, the greater the bending stresses in the beam.



Application of Moments. The previous sketch shows what happens to your lower back when you lift incorrectly, i.e. without bending your legs and forcing your back do all of the work.

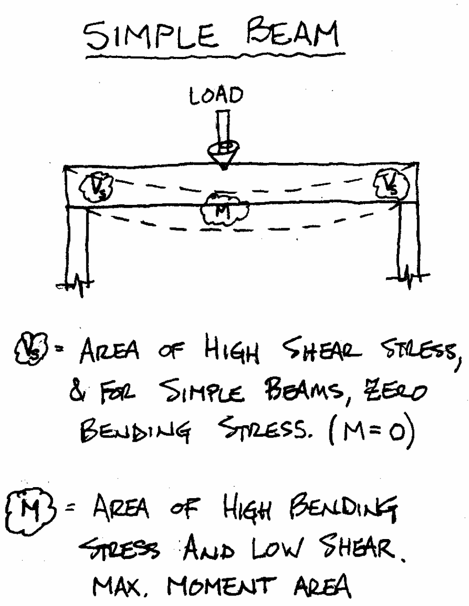
Now check out what happens to the forces in the lower back when lifting is done with the legs, and the weight is kept close in toward the body, thereby effectively reducing the moment arm.



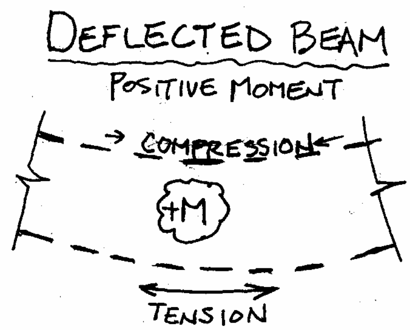
2.2.5. Negative Moments. Applied forces over a distance create applied moments which result in bending stresses in beams, we already know that. But a more subtle part of beam theory holds that *applied* moments result in *internal moments* within the beam from which bending stresses are derived.

These *internal moments* cause *tension* and *compression* within the beam. In simple beams, which sag in their middle, the tension side of the beam is always on the bottom, and the compression side is always the top of the beam.

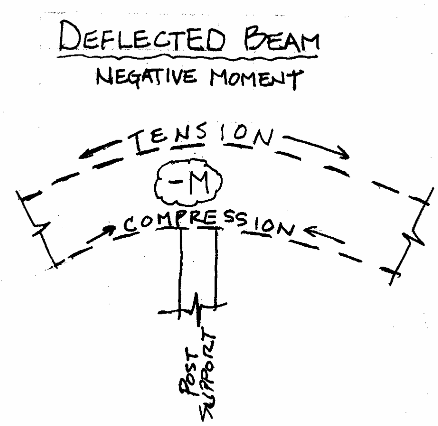
> ***Question**: In a uniformly loaded simple beam, where along the length of the beam is there no shear stress? Where is there no bending stress? **Answer**: There is no <u>shear</u> stress at the point of maximum bending stress, i.e. in the middle section of the beam. There is no <u>bending</u> stress at the supports of a simple beam. There is, however, <u>bending</u> stress at the interior support(s) of a continuous beam – beware of negative moments there.*



> ***Question**: Why is it important to know which sides of the beam are experiencing tension and compression? **Answer**: The compression side of a beam is prone to buckling, and must either be braced, or the beam designed large enough to keep buckling from occurring. Also, you would never cut or notch a bending member where there is a lot of tension or compression happening. This is discussed in detail later.*



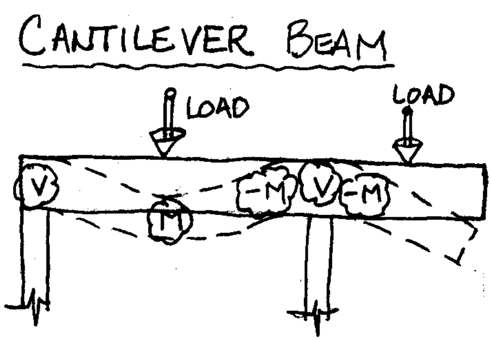
Negative moments occur when the compression side of the beam switches to the bottom and tension goes to the top. This happens in the following common conditions:



- Continuous beams (beams with intermediate support(s)).
- Simple beams which experience uplift, as from wind or seismic forces.
- Beams which are connected rigidly to columns, such as in any pre-manufactured metal building main frame.

- Cantilevered beams.

Negative moments can be extremely dangerous because the lateral bracing required to keep the beam stable, commonly is not provided. It is imperative that beam designers and contractors understand this, and provide adequate lateral bracing to the compression side of the beam.



Cantilever is at right end of beam. Note negative moments on each side of the right support, and positive moment in the span between the supports.

Garrison's First Law of Bending and Deflection. **<u>The most common cause of bending member failures occurs because the compression side of the member is not adequately braced against buckling laterally.</u>**

2.2.6. Cutting or Drilling Bending Members. Why would someone take a perfectly good beam and feel compelled to drill holes - big holes into it? Regardless of the reasons, it happens all the time. Plumbers, electricians, and HVAC contractors are perhaps the most common culprits.

Question*: Is the structural integrity of a bending member always compromised when it is cut or drilled?* ***Answer****: Yes, any time chunks of a bending member are removed, it's internal stresses will be increased. However, if cutting or drilling is done in the right place(s) along the length of the beam and within the lowest stress areas of the beam's cross section, the reduction in strength can be manageable.*

The key to knowing where cutting or drilling is okay, is understanding where shears and moments in beams are high and low.

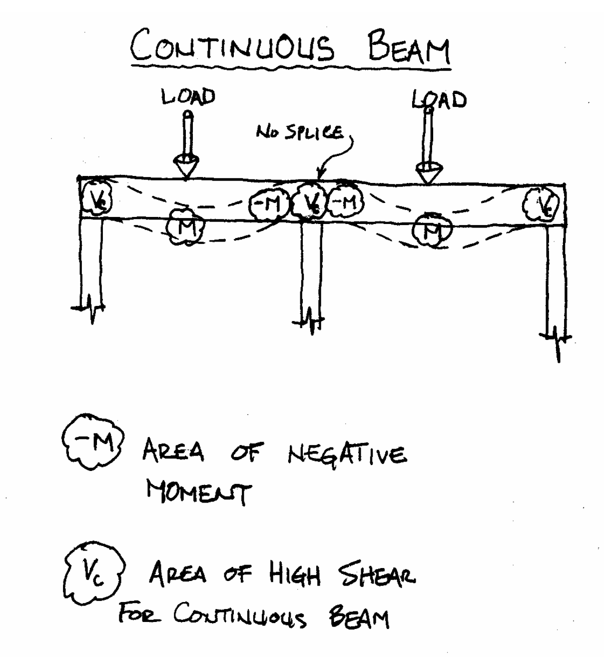
- **Simple Bending Members**. A simple bending member is one with a support at each end of the bending member; with no cantilevered ends and with no other interior supports. With simple bending members, at the location of high internal moments (areas of high bending stress – typically toward the middle of the span), there are large internal tension forces in the bottom of the beam, and large internal compression forces in the top. For internal stresses to go from tension (positive) to compression (negative) over the depth of the beam means that somewhere in between the tension and compressive zones, bending stresses are zero. This place, located at about mid-depth of the beam, is called the *neutral axis.* In zones of high bending stress, the neutral axis (toward mid-depth of the beam) is the place to drill your hole. <u>You would never put a hole or a notch in the bottom or top of a bending member in a zone of high bending stress. To do so would increase the bending stresses in what's left of the member dramatically – very dangerous!</u>

  As you move along the simple beam away from the middle toward one of the supports, away from the point of maximum moment, the bending stresses go down, and the shear stresses increase. In this zone (near the supports of a simple beam), it is not so critical that the hole be placed along the neutral axis of the beam because, with low bending stresses, there is very little tension and compression at the beam's *extreme fibers*. If, however, the beam is maximally stressed in shear, it is using all of it's cross section to resist tearing near the support, and no holes or notches should be allowed at all.

- **Continuous Bending Members**. The previous section applies to simple bending members only (i.e. single spans with no intermediate supports or cantilevers.). If your bending member is continuous over an intermediate support(s), the locations of maximum shear and moment are different than for simple bending members. Beware! Instead of moment going to zero at the intermediate support(s) like it does at the outer supports, it will be quite high. This means that you would never cut or notch the top or bottom of a bending member anywhere near an interior support that the bending member is continuous over.

  Shear stress is also likely to be high near an interior support, so even being able to drill the member at mid-depth may be dangerous. If in doubt, discuss it with your engineer.

  If there is an intermediate support for a bending member, but the bending member is not continuous over the support, i.e. there is a splice in the bending member there, then you really don't have a continuous bending member. By virtue of the splice, you have simple members, which comply with the previous section.



***Question**: How do you know whether or not a bending member is maximally stressed in shear, so that you feel safe putting holes in it?*
***Answer**: If the beam is heavily loaded, and it is minimally sized, it is experiencing high shear stresses near it's supports – don't drill or cut. You can't know for sure exactly what the shear stress is, or how much you are affecting it, however, unless calculations are done.*

Keep in mind that engineers go to great pains to size beams optimally, thereby keeping the cost of the beam as low as possible. In doing this, there is typically not much, if any, extra 'meat' provided to be cutting or drilling away. The bottom line in deciding where or if to drill or notch a bending member is this: Follow the above guidelines, but if in doubt, consult your friendly neighborhood professional engineer.

There is another sneaky structural peril of any continuous bending member, other than the negative moments that will occur in them at the intermediate support(s). Wherever continuous bending members occur, the load they impart to the middle support(s) will be much higher than would be there if simple bending members were used. For example, let's say you are constructing a floor with floor joists that are supported at their ends, and at a beam support somewhere in the middle of the joist. You size that beam for the load that would be there based on the tributary area of the floor that it is carrying. This is the standard practice if the floor joists were simply supported over the beam. *Well, you just undersized the beam by about 25%, a significant error*. The only way to correctly size that intermediate support beam if the joists it is supporting are continuous is to calculate the reactions of the joists at that point, and apply them as the load to the beam. The calculation of the reaction at an intermediate support is quite complicated; I don't even attempt it without the assistance of a computer and some sophisticated software. The bottom line is, 'Beware of continuous bending members!'

***Question**: One time I was using a TJI floor system, and the TJI representative made me actually cut all of the joists about 3/4 of the way through their depth directly over an interior bearing wall. From what this section has said, he made me cut the joist's extreme tension fibers in a location of very high negative moment. What is going on here?* ***Answer**: I'll bet that one of the spans of the floor joist was a short span of say, 4 feet or less. What the TJI guy was doing was converting the joist from one continuous to two simple spans. You can be very sure that he had checked the joists to be sure that they were capable of doing their job even though they were simply supported. In general, a simply supported member must be larger (stronger) than a continuous member. If the joist were sized to be continuous, and just barely calc'd, then cutting them into simple members would have overstressed one or both of the simple span members. Actually what the TJI guy was trying to accomplish was to eliminate uplift at the end of the short span. This uplift would have been caused when there was a heavy load in the long span, and not much load at the end of the joist at the short span. You can see in this case that the short span joist ends would tend to kick upward. The cure is to eliminate the joist continuity over the interior support.*

But wait, there's more to this story! If the interior support happened to be a beam, then presumably it was sized for the continuous joists condition. By cutting the joists, the actual load on the beam would have been reduced, thereby causing the beam (and it's supporting posts and foundations) to be oversized – a conservative situation. The danger to supporting beams / posts / and foundations comes when simple joists are made continuous, and no provision was allowed for the substantial increase in load.

***Question**: I bought my beams and joists at the local lumber store, and the guy there sized them all using a computer. Doesn't this make him the expert? Shouldn't I consult him first as to where I can cut, drill, and notch?* ***Answer**: If I played some sandlot baseball with my 6 year old, am I ready for the Mariners? NO, NO, NO! First of all, the computer program is only as good as the guy inputting the data. Remember, garbage in equals garbage out. Chances are that he has never taken any formal training in beam theory, and does not understand shears, moments, and* *stresses. Secondly, as we learned in chapter 1, only an engineer can perform engineering tasks. Sizing beams for sale and use, or advising contractors how to safely modify them are engineering tasks.*

2.3. Lateral Bracing. Long bending members must have their compression sides braced laterally (i.e. against moving sideways), or their strength is greatly diminished. This restates Garrison's First law of Bending and Deflection (I risk boring you with redundancy here because, as my 6 year old would say, "It's soooper important!)

***Question**: How do I provide lateral bracing to every single bending member in my project – there are hundreds of beams, joists, and rafters in this single job? **Answer**: Fortunately, most bending members are simply supported, i.e. they bear at their ends and their compression sides are up; and most bending members wind up in floor or roof systems which are covered with plywood or some other diaphragm material. Such a covering is all that is required to provide positive lateral bracing to the top (compression side) of the member. A typical example of when you have to worry is the occasional bending member(s) that have point load(s), say from a post above, and no other connection to a diaphragm or other transverse framing members. In these cases, the engineer must have considered that the beam(s) is / are unbraced in the original design. The result will be a larger beam than would otherwise be required, due solely to the fact that there is no lateral bracing. If the unbraced bending members were not engineered, chances are they will be undersized.*

2.4. Temporary Bracing. Perhaps the most critical loading that will ever occur on many bending members happens during construction. This is because during construction, the floor diaphragm, or roof diaphragm, or other intermediate framing members that provide lateral stability are not yet installed. You very frequently see non-braced long slender beams or joists installed, with heavy point loads applied (other building materials stacked on them, a group of men standing there eating donuts, construction equipment piled there, etc.). This is extremely dangerous! The strength of the member(s) greatly depends on the stability provided by lateral bracing. If you pick up any publication on structural failures, you will find that a huge percentage occurs because of this very phenomenon.

***Question**: I am building a two story structure, and don't plan on sheathing off the floor or roof until all of the framing is done. What should I do about this lateral stability issue? **Answer**: First of all, you must be able to recognize which bending members will be most severely affected. In general, any long slender beam, joist, rafter, or truss falls into this category. Second, if these suspect members will experience any significant temporary loading, they must be laterally braced. Always provide full depth blocking or bracing at both ends of each member. If you can't put a diaphragm over them, the next best thing is full depth blocking, or bracing at some regular interval along their length (every 8 feet or so is normally safe). Another option is to provide strongbacks at some regular interval along the bending member. A strongback is a continuous bracing member, 2x4 is common, that is connected to* *the compression side of the member(s), at a right angle, and is firmly connected to a stable structural element a short distance away. Temporary bracing elements such as strongbacks do not have to resist much load; if a normal sized man can not pull or push it, it is probably okay.*

2.5. Deflection. When designing a bending member, not only do you have to consider *shear* and *bending stresses*, you also must consider *deflection*. Deflection is the amount, usually measured in inches or fractions of an inch, that the bending member sags. Engineers typically refer to deflection limits in terms of the bending member's length in inches divided by some constant number, like 240. For example it is common for roof members to keep deflections no greater than L/240. So for a beam that is 15 feet long, or 180 inches long, the maximum deflection it should experience under full loading is 180/240 = 0.75 inches. Maximum deflection for floor members is typically less than for roof members; L/360 or even L/480 is common. The reason for this is that floors with a lot of deflection feel 'bouncy'.

***Question**: If a bending member does not meet the required deflection limit, does that mean it is overstressed? **Answer**: Not necessarily. For example, in the case where a certain size floor joist must comply with an L/480 deflection criteria, it is quite possible that it is well within it's allowable bending and shear stress limits, but it will deflect too much, and thus feel bouncy. It will not, however, fail due to overstress. The remedy to get the deflection under control is to choose a larger (normally deeper) beam, or shorten the span.*

2.6. Bending and Deflection Relative to Span. The most influential factor in determining the bending stress and deflection of a bending member is the span of the member. The bending stress and deflection go up *exponentially* as the span increases. This concept is explained in Garrison's Second Law of Bending and Deflection below.

Garrison's Second Law of Bending and Deflection. <u>If you increase the SPAN of a bending member a little, you will increase it's BENDING STRESS and DEFLECTION a lot</u>.

2.7. Applied and Allowable Stresses. Every bending member can be loaded a certain amount before it fails. The question is, how much? The answer requires two pieces of information: First, a determination of what the *applied stresses* are, and second, a determination of the *allowable stresses* of the member. *Applied stresses* are the actual stresses in the member (shear and bending) due to the particular loading applied. Determining these stresses is a job that must be left to your friendly neighborhood professional engineer. *Allowable stresses* are inherent properties of the member that directly reflect its strength. Strong members have large allowable stresses. To restate a very basic concept presented in Chapter 1: In general, <u>applied stresses must not exceed allowable stresses</u>.

***Question**: I've got a long beam that is okay in shear, but overstressed in bending. Is this possible, and if so, what should I do? **Answer**: Yes, in-fact it is expected. Almost never do the shear and bending strength limits approach their maximums simultaneously – one or the other* *always controls. In this case (bending controls the design), either shorten the span of the beam, add an intermediate support, or increase it's moment of inertia (i.e. use a deeper beam).*

***Question**: I've got a short beam that does not have a bending problem, but overstresses in shear at the support, now what? **Answer**: Add another interior support, or increase the beam's cross-sectional area.*

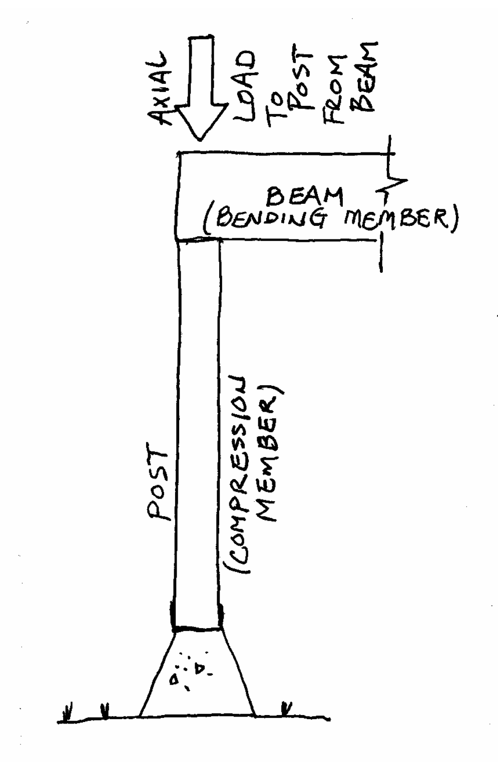
2.8. Moment of Inertia. (MOI). It is this inherent property of bending members that resists bending and deflection. The most critical factor influencing moment of inertia is a bending member's depth. Deep bending members have high moments of inertia, and are strong in bending. Shallow members have low MOI's and are weak in bending. A member's bending strength and resistance to deflection goes up exponentially with its depth; as explained below.

Garrison's Third Law of Bending and Deflection. <u>If you decrease the DEPTH of a bending member a little, you decrease it's bending strength and resistance to deflection a lot.</u>

The other main part that determines a bending member's moment of inertia is how much 'meat' there is at top and botto m edges of the bending member. Recall that there is tension and compression in the bottom and top respectively of a simply supported bending member. It stands to reason that if the top and bottom part of a beam are real meaty, then it would be able to soak up more tension and compression, and would be stronger in bending. A perfect example of this is the wide flange steel beam. It's top and bottom flanges are big and wide, and there is only a thin web in between; the quintessential bending member. Contrast this to a pipe being used as a beam. There is very little meat at the top and bottom to resist the tension and compression forces, thus the moment of inertia is low, with the result being that pipe is a very poor bending member choice. Rectangular wood beams and joists are okay as bending members, but this concept of moment of inertia explains precisely why wood I-joist (sometimes known as 'TJI's') were invented. They have extra meat at the top and bottom (flanges) and only a spindly plywood web in between. Yet they are pound for pound stronger than equivalently graded wood; all because they have a higher moment of inertia.

CHAPTER 3
COMPRESSION MEMBERS and TENSION MEMBERS

3.1. Introduction. Compression members are those which are loaded axially, i.e. in same direction as their length with compressive or 'pushing' forces. Posts and columns are basic examples of compression members. Compression is an axial load, that is it acts along the axis of the member. In contrast, you will recall from Chapter 2 that bending members are loaded perpendicular to their axes.



There are two basic types of axial loads – compression and tension. The first part of this chapter will focus on compression.

> ***Question**: "In the chapter on bending, we learned that bending members experience both tension and compression at the same time. Now you're talking about compression members being different than bending members. What's going on here?" **Answer**: Bending members do experience both tension and compression at the same time due to internal moments brought on by bending stresses. <u>A purely compression member, however, has no tension in it. The entire cross section of the member is in compression.</u>*

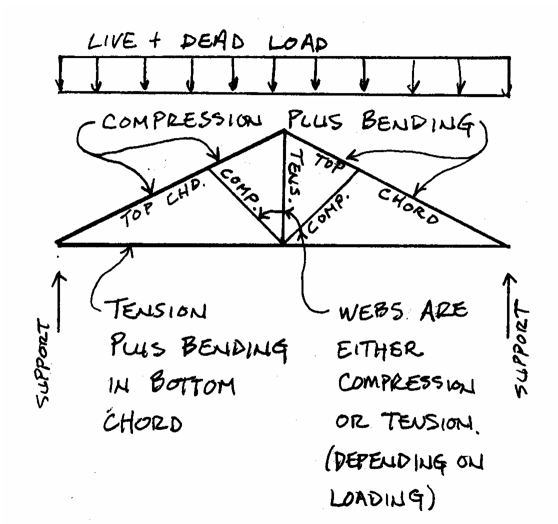
3.2. Types of Compression Members. Posts, columns, and certain truss members are common examples of compression members.

3.3. Stresses in Compression Members.

3.3.1. Applied Stresses. Applied compression stresses come from axial loads normally applied to the top end of a member. These loads typically come from beams or other structural members sitting on top of the compression member. Assuming that the load is applied evenly over the cross sectional area of the end of the member, it spreads evenly throughout the member, creating compressive stress therein.

Typical roof trusses have compression members in them. The webs of a truss are typically either purely compression or purely tension members. Which are which depend on the type of loading. Webs in compression under normal dead and live loading will go to tension in a severe wind event where the net force on the truss is uplift. Truss analysis must be left to your friendly neighborhood engineer (truss manufacturing companies normally have a professional engineer on staff to review and stamp their calculations).

The top chord of a triangular truss (see later section for more on trusses) under gravity load is a compression member as well, but it also has some bending in it too. This case, where a member experiences both compression and bending, is referred to as *combined stresses*. It is important to note that combined stresses are additive. For example, a post loaded in compression only may be able to safely carry, say 5,000 lbs. But if there is bending stress introduced (from a wind load or from an errant forklift for example), the amount of compressive load it can carry will be reduced dramatically.

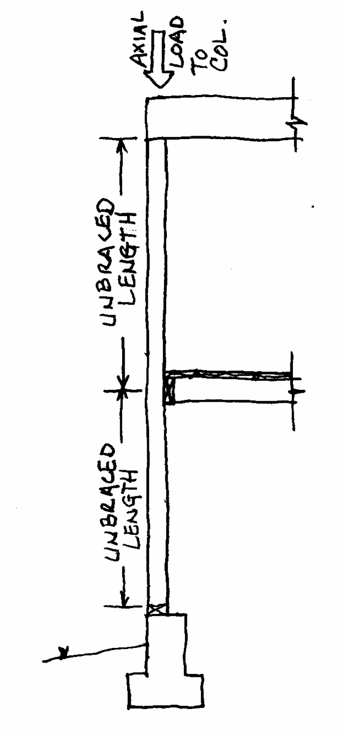


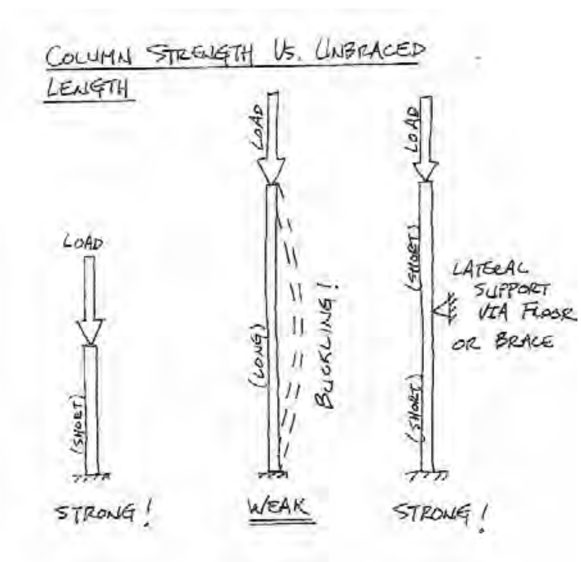
The bottom chord of a triangular truss under gravity load is a tension member. If there is a ceiling or other weight attached to the bottom chord, there will also be some bending in the bottom chord, in which case it will be experiencing combined stresses (tension and bending). These combined stresses, similar to compression and bending are additive.

Question: *If an axial load is applied to the top of a post, isn't the compressive stress greatest there, and diminishes over the length of the member?* ***Answer***: *No, it doesn't work that way. Assuming that the post is supported only at the bottom, and the load is applied at the top, the entire length of the post has the same compressive stress throughout. Where this can vary, however, is if a post is (intentionally or unintentionally) supported at a midpoint, say where it goes through a floor. If the hole where it goes through is tight, or there is some binding between the post and the floor system, part of the axial load will wind up in the floor system, thereby reducing the amount that is in the lower part of the post. So the compressive stress in the top portion of that post will be higher than it will be in the bottom portion.*

3.3.2. Allowable Stresses. As with bending members, there is much published data concerning the allowable compressive stresses of all of the various types of building materials (steel, wood, concrete, masonry, etc.). In design, the applied stresses are compared with these allowable stresses to ascertain that the allowable stresses are not exceeded. It they are, a larger member or less load to the member is required.

3.4. Unbraced Length. The actual strength of any compression member is greatly influenced by its *unbraced length*. <u>The longer the unbraced length of a compression member, the weaker it is</u>. *Unbraced length is the maximum distance between lateral supports along the length of the compression member*. Similar to bending members which require their compression sides to be braced, compression members require bracing to maximize their strength. This is easy to understand if you visualize a stick of uncooked spaghetti. Break off a small piece, say 1 inch long, stand it up, and press on the end. You might sustain an injury pressing before the noodle crushes. Now, take the long piece that is left over, stand it up, and press on the end. With very little effort, the spaghetti will



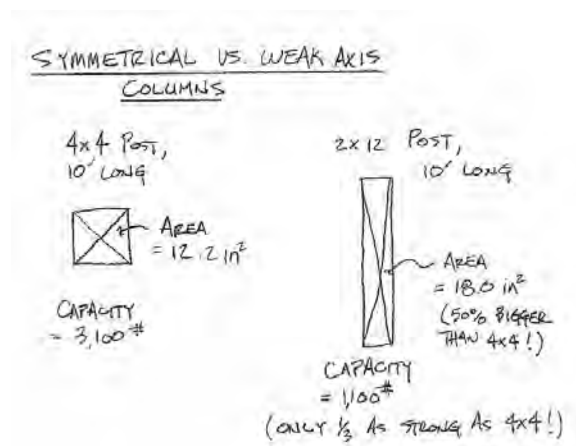


bow in the middle and break. Now, have a couple of friends hold another piece of long spaghetti at the quarter points so it can't bow. You will find that you come close to personal injury again trying to break the long, braced spaghetti. This is the most important concept concerning compression members.

Garrison's First Law of Compression Members. <u>As the length of the unbraced post column, or other compression member increases, its strength decreases dramatically.</u>

Corollary to Garrison's First Law of Compression Members. <u>If long compression members are braced against buckling laterally, their strength increases dramatically.</u>

3.5. Non-Symmetrical (Strong-Axis / Weak Axis) Compression Members. Many times, compression members (particularly temporary support posts) are fashioned from whatever happens to be lying around the building site. This is particularly dangerous if the post selected is non-symmetrical. A non-symmetrical member is any member that has less resistance to bending in one orientation as compared to the other.

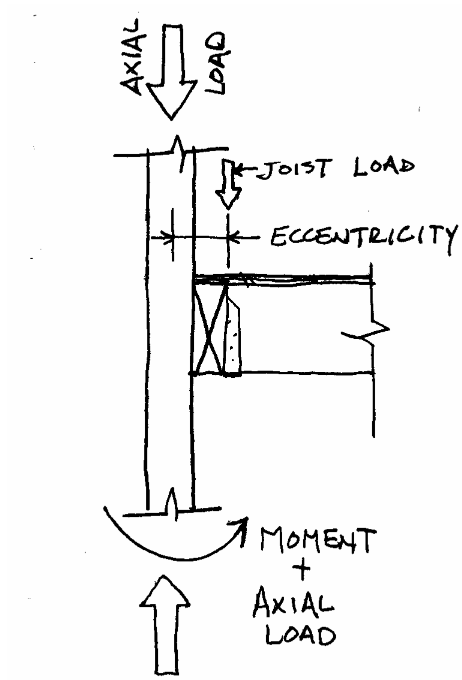


Any wood member that is not square or not circular in cross section is non-symmetrical. So 2x4's, 2x6's, 4x10's, etc. are non-symmetrical. All 4x4's, 6x6's, 10x10's, etc.; and all round members are symmetrical. Nearly all steel wide flanges (or 'I' beams as they are commonly known) are non-symmetrical. They are weaker in one axis of bending than the other. This is easily visualized by imagining taking a long 4x12 beam that is to support a heavy floor load, and laying it flat on the supports instead of tall-wise. It will bend dramatically, or maybe even break under load if installed in this weak-axis manner.

***Question**: "Everyone knows that you install a beam with its strong axis resisting load, but we're talking about posts and columns in this chapter, not beams. What's going on here?*
***Answer**: Remember, any long compression member must be braced against buckling laterally. If, however, the compression member is non-symmetrical, it will require different bracing in its weak axis than in its strong axis. Let's say you use a 2x8 post to temporarily hold up a big beam. Intuitively, you know that it will not likely buckle in the direction of its strong axis, that is, it will not buckle in the 8" dimension. It will however, be very prone to buckling the other direction. It may be that no bracing is required in the 8" direction, but several braces may be required the other way.*

3.5.1. Eccentricity. If the axial load is not applied evenly over the top of the compression member, that is, it is applied to one side or another, it will produce bending stresses within the compression member. The distance that the load is applied from the center of the cross section is called *eccentricity*. Sometimes eccentricities are created on purpose and sometimes by accident.

3.5.1.1. Intentional Eccentricity. An example of an intentional eccentricity is the use of a ledger connected to wall studs. Prior to attachment of the ledger, the studs were supporting the weight of the roof or upper floor, and were pure compression members, i.e. no bending stresses in them. [Actually, exterior studs will experience bending stresses under wind load. This example pertains to interior load bearing studs, or exterior studs without wind load.] When a ledger is installed, and rafters or joists are hung from it, this introduces additional compressive forces in the studs, however, these are applied not at the stud's cross-section middle, but out at the edge of the stud. The



eccentricity is the distance from the outside edge of the ledger to the center of the cross-section of the stud. This may remind you of something we covered in chapter 1; moments. Recall that a moment is a force times a distance. In the case of a ledger, a moment is induced in the stud that is equal to the downward load on the ledger times the eccentricity (force times distance). The bottom line is that the vertical load carrying capacity of the stud is diminished due to the introduction a bending stress via a moment.

> ***Question**: "I have to use a ledger at the second floor level in my balloon framed house, how can I minimize the bending stresses in the studs?" **Answer**: There are two ways, either reduce the eccentricity, or reduce the load on the ledger by adding additional supports to the floor system. To reduce the eccentricity, you could bear the floor joists on top of the ledger instead of hanging them from the ledger. This will reduce the eccentricity by half the width of the ledger.*
>
> *You could also, of-course, use whatever load and eccentricity you want, and just be sure that the studs are sized to take the combined stresses.*

> ***Question**: What if I use no ledger at all? Instead, I run the floor joists into the wall, and nail them directly to the studs. I'll install blocking below each joist to adequately transfer the vertical load into the stud. Would that be okay?" **Answer**: It might, but you are still introducing a moment into the stud. The induced moment is, however, in the weak axis of bending, rather than the strong axis. In other words, connecting the joist to the side of the stud tends to bend it in the plane of the wall, as opposed to out of the plane of the wall as when using a ledger. In both cases, the compressive capacity of the stud is diminished. Only your engineer can tell you which alternative yields the lowest overall combined (compression plus bending) stress.*

3.5.1.2. Accidental Eccentricity. Accidental eccentricity occurs when axial load is intended to be applied evenly over the top of a compression member, but something unanticipated happens, and the load winds up being applied toward the edge or side of the member. A classic example is where a post is not cut level at its top or bottom. When a beam is placed on top, it sits on the high side (edge) rather than on the entire cross section. The distance between the bearing point and the center of the post is the (accidental) eccentricity, and of-course, this induces a moment and bending stress in the post, thereby reducing its axial load carrying capacity.

3.5.2. Other Induced Moments in Compression Members – End Fixity. There are ways, other than through eccentricities, that moments and bending stresses get into compression members. This occurs mainly where the end or ends of the post or column are fixed rigidly, rather than pinned. Most times when engineers design simple columns, they assume that the ends are free to rotate a little. This is known as a pinned condition. If, however, the column ends are fixed, and may not rotate at all, the connection is termed *fixed* or a *moment connection*. Such a connection is said to resist any internal moments that happen to occur at the column end. Consider a telephone pole. Its bottom end is buried deeply. It is a fixed end, or moment resisting end. If it were pinned, the first time the wind blew, the pole would simply lay over, rotating about the 'pin' at its base. Engineers use pins and fixity to control the stresses in structural members. A column with fixed end(s) is stronger in compression than a column with pinned end(s). This is because the fixed ends tend to make the column behave as if it were shorter than it really is – and short columns can take more axial load than long ones.

***Question**: "I always use Simpson framing hardware for my post bases and post caps. Are these pinned or fixed?" **Answer**: Excellent question! The most accurate answer is, neither. They are not purely pinned nor truly fixed – they are somewhere in between. That is, they will resist some moment, but not a whole lot. When engineers use these in design, however, they are assumed to be pinned.*

***Question**: Okay, this concept of pinned vs. fixed is easy enough to understand, but so what? Am I creating a risky situation in the field every time I install a post or column, and don't know whether it is pinned or fixed or somewhere in between? **Answer**: An even better question! The short answer is no, you are not creating a dangerous situation in using framing hardware, as long as you use it per the engineer's and manufacturer's instructions. Where it really matters is in the case where the engineer has specifically designed a column base or top to be <u>fixed</u>. In such a case, even the most unobservant builder will notice something unusual about the joint. It will be hell-for-stout. For example, if a timber post and beam are to be connected as a fixed connection, there will be gusset plates (steel or heavy plywood) on both sides of the connection with lots of big bolts or nails. Or if the base of a wood post is to be connected to a concrete foundation in a 'fixed' manner, there will be heavy duty holdowns on at least two opposite sides of the post with lots of bolts or nails into the post. The danger occurs when the builder does not construct the joint as designed by the engineer. THERE IS A REASON FOR FIXED JOINTS! They almost always occur to resist lateral loading (wind and earthquake). This may not seem obvious to even the most seasoned contractor. He may see such a joint detailed on a plan set, and look at the gravity loads that are being supported, and think, "What kind of kook engineer designed this! A beam and column half this size would work here, I just know it. And what's with this ridiculous steel gusset plate connection! Those darned engineers are always over-engineering, costing us thousands of dollars. I'll leave the beam and column sized like he wanted, but as for the gusset connection, I'm just going to slap a Simpson post cap on this baby, and we'll be fine." My response is, "You'll be fine alright, fine until we have an earthquake or severe wind storm, then WATCH OUT!" It is just like the telephone pole. What if the telephone pole installer didn't want to bore a 10' deep hole for the base, and instead buried it 3' deep, thinking even that was over-kill. In the first good wind, the pole would be on the ground, taking the power lines and anything else in its path with it.*

3.5.3. Trusses. Common roof trusses are constructed with slender wood members that are designed to act in either tension or compression. These days, trusses are designed by very sophisticated computer software. The software applies all of the code required loads; snow, dead, live, wind, and seismic in all of the required combinations. All of the members and connector plates are sized per code requirements. Depending on what the load is, any of the members can go from tension to compression and back again. This seemingly freakish phenomenon happens largely due to wind. Strong winds over a roof act somewhat like an airplane wing, and tend to lift the roof off the house. We'll go into this further in a later chapter, but where it applies here is the case of the long slender truss members that experience compression. Truss manufacturers are careful to note in their designs which members must be braced laterally or diagonally. These are the members that are either normally in compression, or will go to compression in a wind event, and are long and slender – prone to buckling. If the builder chooses not to install these braces, he is looking for buckling trouble when the wind or snow storm comes. Remember the spaghetti!

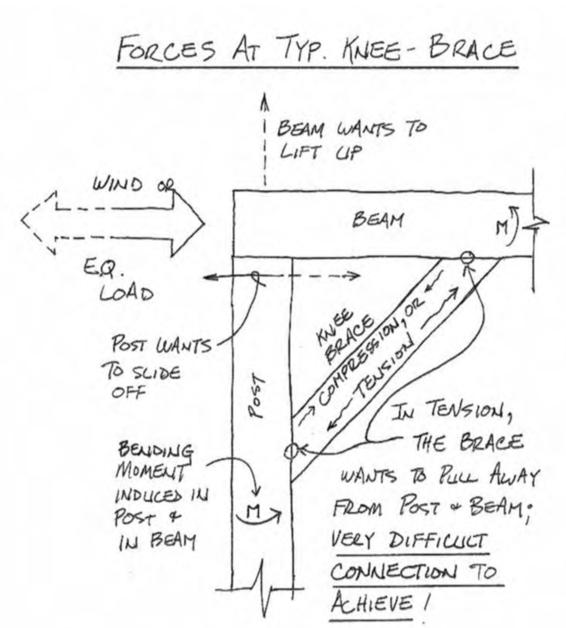
3.6. Tension Members. Like compression members, tension members are loaded axially. Unlike compression members, however, the axial load is a tensile (pulling) load rather than compressive (pushing) load. Pure tension members are comparatively rare on the normal construction site. Some common places you will find tension members follows:

- Cables
- Tension rods
- Rope
- Certain members of a truss (can be tension or compression)
- X-braces (can be tension or compression)
- Knee braces (can be tension or compression)

Some tension members, such as cables, rope, and tension rods can take tension only, i.e. they will take zero compressive load. Some (but not all) tension members such as truss webs, knee and X-braces are designed to take both tension and compression, the occurrence of which depends typically on which way the wind is blowing or the ground is moving in an earthquake. Tension members in building construction almost never fail. This is because they do not need to be laterally braced (like bending or compression members) to achieve full strength. In general, their length has nearly no bearing on their strength, i.e. a long cable is just as strong as a short one of the same diameter and material. There are exceptions to this, however, in most residential and light commercial applications they won't apply.

3.7. Tension Connections. If there is an area where tension members chronically experience problems, it is at their ends; where they connect to other structural members. These connections come in all varieties: welded, bolted, gang-nail plated, nailed, eye-hooked, etc. <u>What is important here is that the strength of the tension member is only as good as the strength of the connector</u>. I've seen big diameter steel x-bracing rods that were designed to resist wind and seismic loads in a metal building welded to the steel frames with tiny puny sheet metal gussets. Guess where the failure will occur? This could have been a case where the engineer designed the rod, but left the connection design to the contractor – big mistake for the engineer. Or, the engineer may have designed the whole thing, but the contractor ran out of proper gusset plates, and grabbed the aluminum foil wrapper from his Twinkie and used it instead. Either way, a serious hazard was built into the structure. What's worse is that the connection occurred at the top of a steel frame, some 20 feet in the air, and no-one would ever notice it way up there to correct it.

One other common place where tension member connections are chronically undersized is at the ends of a knee brace. **Knee braces** are used commonly in barns or on decks where the construction is post and beam, and few, if any shear walls exist. It is the knee braces that provide the resistance to wind and earthquake lateral forces. I can honestly say that of the dozens of knee braces I've seen built, NOT ONE HAS EVER BEEN CONNECTED TOGETHER ADEQUATELY, except of-course for the ones that were designed by a competent engineer. The problems with knee braces are that the forces in the members are typically very high (thousands of pounds is common), and the forces switch from tension to compression as the lateral loads change direction. It is very difficult to design a good tension joint in a wood to wood connection, such as a knee brace, unless big bolts or gusset plates are used. Most non-engineered wood knee braces do not have gussets, and instead depend on nails to withstand the very high tensile forces – a bad practice. A good rule of thumb for any non-engineered tension joint is to make it so stout that the tension member will have to rip in two before the joint comes apart.



CHAPTER 4
WOOD AS A STRUCTURAL MATERIAL

4.1. Introduction. Structural timber is the most common construction material used in residential and light commercial construction in the United States. Interestingly, it is perhaps the most complex from a design point-of-view. The reason for this is that wood's allowable stresses depend on so many things. In other words, an engineer can't just use the same allowable bending, shear, or compressive stress for every piece of wood on a job; virtually every piece of wood must be looked at separately, and the correct allowable stress applied.

4.2. Allowable Stresses. The basic allowable stresses that are applicable to wood design are: bending 'Fb'; shear 'Fv'; compression parallel to grain 'Fc-parallel'; compression perpendicular to grain 'Fc-perpendicular'; and tension 'Ft'. Each of these allowable stresses can vary depending on the below conditions.

Also involved in timber design is the wood's stiffness coefficient, called *modulus of elasticity*, 'E'. Note that 'E', which is not technically an allowable stress, depends on the species of wood, and is not affected by the following (with the exception of wet use conditions).

- **Repetitive Member Use**. Certain (but not all) allowable stresses can be increased if more than one member is used to support the same load, for example where floor joists are spaced 24" apart or less, they 'share' the load, and higher allowable bending stresses may be used.
- **Flat member use**. If a wood bending member is used in *weak axis bending*, like a girt on a pole building that is nailed flat to the post, it gets a higher allowable bending stress.
- **Short term loading**. Certain allowable stresses can increase for wood members experiencing short term loads such as wind, earthquake, certain roof construction live loads, and snow load.
- **Wet conditions**. Allowable stresses and stiffness, 'E' decrease if the member will be wet much of the time.
- **Heat Conditions**. Allowable stresses go down for members installed in hot conditions.
- **Size**. As the size of the member gets *larger*, it's allowable stresses get *smaller*. For sawn beams and posts, the allowable bending stresses can be *half* of those of small joists and studs of the same species. This is one reason you don't see many large sawn timber beams on jobs any more. Their allowable stresses are typically only 35% or so of glue-laminated or PSL type wood beams. This means that the sawn member would have to be about three times larger than a Glu-Lam or PSL for the same application!

4.3. Direction of Load. Wood's allowable compressive strength varies depending on whether the load is applied parallel or perpendicular to the grain of the wood. Compressive load applied *parallel* to grain is what posts and columns experience under normal loading. An example of compressive load applied *perpendicular* to grain is the bearing surface where one wood member bears on another, like the under-side of a floor joist bearing on a top plate, or a beam bearing on a post. The horizontal member (joist or beam) experiences compression perpendicular to grain at the bearing point.

4.4. Species of Wood. Another complexity of wood is that it is available in different groups of species. In the northwest, we commonly use Douglas Fir- Larch, or Hemlock (Hem Fir is a generic name for Hemlock) for framing. These species have similar bending and shear strengths (allowable stresses), but with Doug Fir being the stronger of the two. One area where there is a dramatic difference is in the strength of bolted connections; Doug Fir values can be almost double those of Hem Fir, depending on bolt size, direction of grain, and thickness of bolted members.

If Cedar, Spruce Pine Fir, or White Fir, or some other softwood is used, beware, their strength values are considerably lower than Doug Fir.

4.5. Grade of Wood. To make matters even more complicated, each species comes in different grades of quality. These grades can be seen for joists and rafters in the 1997 UBC, Table 23-IV-V-1. Allowable bending stresses for a few of the more common grades for 2x6 (remember, different sizes have different allowable stresses) with the repetitive member stress increase added are shown below:

- Doug Fir- Larch, Stud Grade: Fb = 775 psi.
- Doug Fir- Larch, No. 2: Fb = 1,310 psi.
- Doug Fir-Larch, No. 1 and better: Fb = 1,985 psi.
- Doug Fir-Larch Select Structural: Fb = 2,500 psi.
- Hem Fir, Stud Grade: Fb = 775 psi.
- Hem Fir, No. 2: Fb = 1,270 psi.
- Hem Fir, No. 1 and better: Fb = 1,570 psi.
- Hem Fir, Select Structural: Fb = 2,095 psi.
- Western Cedars, No. 2: Fb = 1,045 psi.
- Western Cedars, No. 1: Fb = 1,085 psi.
- Western Cedars Select Structural: Fb = 1,495 psi.

Note the dramatic decrease in allowable bending stresses between Stud Grade and No. 2 (about half!).

Question: *"Now that I see this these allowable stresses, why in the world don't engineers specify Select Structural or No1 and better grades instead of the No. 2 grades that always seem to wind up on my jobs? You don't have to be a genius to see that the strength values are much higher for those grades, which would result in smaller, cheaper, members. Right?"* ***Answer***: *Partially right. It is correct that if engineers could design every sawn structural member using the strength values from the Select Structural grade, this would result in considerably smaller members than Contractors are used to seeing on their jobs. However, the problem arises when you go to buy Select Structural or the other high grades of sawn lumber. Many times, they are not even available; or must be special ordered. And if you can get them, the cost is quite high. So in the final analysis, most times, the most efficient sawn lumber to use are No. 1 and No 2 grades.*

4.6. Moisture Resistance. All wood is prone to rot if it experiences wet / dry cycles (though some species like redwood and cedar take much longer to rot than others like the firs). A common remedy for this is to specify *pressure treated* lumber for exterior applications where wet / dry cycles cannot be avoided. (This discussion is intended for structural wood only. Other materials like certain roofing, siding, and decking have proprietary alternatives, like Hardiplank or Trex, that are not 100% wood products, but rather are made with other ingredients such as cement and plastic and are manufactured to look like wood. These proprietary materials do not need the same level of water protection that natural wood does.) Pressure treatment consists of the application of certain rot resistant chemicals in a giant vacuum, so that the chemicals are actually 'sucked' into the wood. The wood is slightly perforated first to help with chemical absorption. It is interesting to note that *Doug Fir* is very difficult to pressure treat. So when you buy pressure treated wood, it is almost always *Hem Fir*. This is important to know, because remember, Hem Fir has lower strength properties than Doug Fir.

Another noteworthy item concerning pressure treated woods is that certain constituents of the treatment chemicals can react with the galvanized coating on nails and framing hardware to create corrosion, and premature failure of these metal connectors. Certain pressure treatment chemicals can also react negatively with aluminum building materials. The bottom line is to make sure you know the limitations of your pressure treatment system, and follow the manufacturer's recommendations.

Pressure treatment is available in different strengths. This 'strength' is called retention, and is measured in pounds of treatment chemical retained in the wood per cubic foot of wood. Typical retentions vary from about 0.25 pounds per cubic foot (pcf) to 1.0 pounds per cubic foot. Following are some minimum recommended retention values for common applications:

- Decking, fence boards, hand rails, use 0.25 – 0.4 pcf retention.
- Fence posts, mud sills, landscaping, piers and docks (not in direct contact with salt water) and other ground contact with fresh water; use 0.4 pcf retention.
- Wood foundations in direct contact with earth, poles for barns or transmission lines; use 0.6 pcf retention.
- Piling; use 0.8 or 1.0 pcf retention.

Fasteners to Pressure Treated Wood. The 1997 UBC (2304.3) requires fasteners into pressure treated (preservative or fire treatments) be hot dipped galvanized, (not electroplated galvanized), stainless steel, silicon bronze or copper. This means, for example, that the row of shear wall nailing that penetrates a pressure treated mud sill must be hot dipped galvanized or one of the other allowed types listed above.

4.7. Old Wood.

***Question**: "I've got an old barn that I want to scavenge the wood from. It is well over 100 years old. Is the wood still good? Has it gotten weaker? **Answer**: As long as the wood has not been chronically overstressed, or has not been wet, it is stronger than it was when cut. Wood actually gains strength for the first 200 – 300 years, then slowly, slowly loses strength. Wood that up to 5,000 years old, and has not been wetted or otherwise distressed is nearly as strong as it was the day it was cut.*

4.8. Engineered Wood Products: Over the years, strength values for sawn woods have decreased. Apparently the reason for this is that younger, faster grown trees are being used these days, instead of the larger, slower growing, older growth trees of the past. These smaller, fast growing trees have lower allowable stresses. One alternative to this weaker sawn lumber is the advent of engineered wood products, such as Glue Laminated Timbers (Glu Lams), Manufactured wood I-Joists, and PSL's (PSL is a generic name for Parallel Strand Lumber).

4.8.1. Glu Lams. Glue laminated timber beams and columns (called 'Glu Lams') are made from selected dimensional sawn lumber (commonly 2x material), which are joined, pressed, and glued together in a proprietary method to create larger structural members. Because only the best dimensional lumber pieces are used, the allowable stress values for Glu Lams are very high – on the order of, or even higher than those for Select Structural sawn grades. Glu Lams come in different grades, and thus different allowable stresses.

Glu Lam Beams; Designation / Allowable Stresses. You have to be careful to purchase the correct type of Glu Lam, or you may well be installing a beam that does not meet the engineer's design intention. A typical grade that I specify is 24F-V4, DF/DF. What the letters and numbers mean is this: The DF/DF means that the top and bottom laminations are of Douglas Fir (as opposed to say HF for Hem Fir); The 24F means that the allowable *tension* portion of bending stress (i.e. the tension at the bottom of the beam at mid-span of a simply supported beam) is 2,400 psi. The V4 indicates that the allowable *compression* portion of bending stress (i.e. the compression at the top of the beam at mid-span of a simply supported beam) is 1,850 psi (recently upgraded from 1,200 psi). A V4 Glu Lam will have a top and bottom face, so be careful to install them correctly or they will only have about 75% of the intended bending strength.

Note that if you had a Glu Lam that was continuous over an intermediate support (which would have a high negative moment at the intermediate support) you may well need a beam with the same allowable tension and compression portions of bending stress. This very attribute is available with a 'V8' designation.

Allowable *shear stresses* vary in Glu Lams depending on their designation. For example, a 24F- V4, DF/DF has an allowable shear stress of 190 psi, whereas a 24F- V5, DF/HF has an allowable shear stress of 155 psi.

<u>So beware! Not all Glu Lams are created equal</u>.

Camber. Glu Lams can be special ordered with pre-manufactured bends, crowns, or camber. Some Glu Lams are used for arches, or other heavily bent structural members. You've probably seen them in a church at sometime in your life.

Question: *"My engineer sometimes specifies a Glu Lam with a certain camber, usually some fraction of an inch, like 1/2" of camber. What is camber, and why does he do this?"* ***Answer***: *Camber is the amount of crown (or to say it another way, 'unloaded deflection') that is built into the beam. Sawn beams, joists, and rafters frequently have a naturally occurring crown, and good carpenters know to install them with the crown up; so that when gravity loads are* *applied, they flatten out some. What is nice about Glu Lams is that you can special order them with a specified amount of camber (or crown). If, for example your architect wants the beam perfectly flat with the dead load only applied, the engineer can calculate the dead load deflection, which becomes the amount of camber desired. In this case there will be no sag in the beam until the live load is applied. Other manufactured beam products, such as PSL's and I-Joists cannot be ordered with camber; they come absolutely flat and level. So when any load is applied, they have no choice but to sag; albeit within code limits assuming they were designed properly.*

Glu Lam beams usually come with a certain amount of camber. Some examples of factory built-in camber follow (these assume a camber radius of 3,500 feet; if the camber radius is less, say 2,000 feet, then the camber amounts will be greater).

- Camber for 16' span = 1/8"
- Camber for 24' span = 1/4"
- Camber for 32' span = 7/16"

Preservative Treatment. Glu Lams can be treated for exterior applications. A word of caution here, though. Only certain types (and strengths) of Glu Lams can be treated, so be sure that this is taken into account in the design of any exterior beam applications.

Glu Lam Columns. There are special Glu Lams that are manufactured specifically for columns. These do not have any camber built in to them, and their strengths differ depending on their designation and the number of laminations. For example, a 5 DF L1 with 3 laminations has a modulus of elasticity of 2.0 (million psi), Fb of 2,100 psi, and Fv of 160. But if 4 laminations are used, the Fb goes up to 2,400.

Weak Axis Use. Another variable with Glu Lams (beams and columns) is that their allowable stresses can vary considerably depending whether they are loaded parallel or perpendicular to the laminations. So, make sure you know (especially with columns) which way the engineer intended them to be oriented relative to the applied load.

Appearance Grades. There are three basic appearance grades available in Glu Lams. These may vary from manufacturer to manufacturer. Interestingly, the appearance grade has nothing to do with the strength. A typical listing of the three grades, in order of quality (and cost) follow (check the manufacturer for more specific information):

- **Framing Appearance**. For concealed use only. Irregularities are not filled, no surfacing is performed.
- **Industrial Appearance**. For concealed or non-concealed use where appearance is not of primary importance.
- **Architectural Appearance**. For use when members are exposed to view. Voids larger than 3/4" are filled and all exposed faces are surfaced.

4.8.2. PSL's, LVL's, LSL's. These are manufactured wood products that are extremely strong and dimensionally stable. They are sometimes specified by their stiffness value, 'E' (modulus of elasticity). For example you can commonly find a 2.0E PSL. This is a Parallel Strand Lumber member with a modulus of elasticity of 2.0 million psi. (recall that Doug Fir has an 'E' value of about 1.6 million psi, some 20% less).

When using any of these types of members, you must also pay attention to the allowable stresses, because they vary widely depending on the manufacturer and type of product being used. For example a 2.0E PSL may have an allowable bending stress, Fy = 2,900 psi, but a 1.5E LSL could have an Fy = 2,250 psi; about 22% less.

- **PSL's**. PSL is a generic acronym for Parallel Strand Lumber. There are several manufacturers of PSL's each having their own proprietary name for their product. These are manufactured wood products that use wood chips or strands and epoxy resins, then apply heat and pressure to form the finished structural member. They are incredibly strong and stiff, with allowable bending stresses of up to 2,900 psi (approximately three times stronger than sawn Doug Fir); and stiffness (resistance to deflection) approximately 25% more than Doug Fir. Their allowable shear strength is approximately triple that of sawn Doug Fir.

  Because they are a manufactured product, they come in exact dimensions (sawn lumber frequently shrinks and warps such that all of the 'same sizes' are not really the same).

  Some manufacturers are able to provide camber to their PSL's, if special ordered. Typically, camber is only available in spans over 20'.

  Pressure Treatment. Some manufacturers are able to provide pressure treated PSL's for exterior use. The Wolmanizing process is one type of pressure treatment available. There are typically three levels of protection available:

  - **Level 1**. For dry use only. Maximum moisture content of 19%
  - **Level 2**. Above ground service with maximum moisture content of 28%.
  - **Level 3**. For ground contact or saturated use. Moisture content greater than 28%.

  Beware when using pressure treated PSL's. Allowable stresses and stiffness are seriously reduced with any of these products; and as the service level increases, the allowable stresses decrease even more. However, even at service level 3, the allowable stresses of a Wolmanized PSL beam is about double that of a Hem Fir beam, with the stiffness being about the same. Also, check with the manufacturer for dimensional stability (swelling) when used in wet conditions.

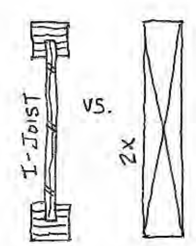
  Cost. PSL's appear expensive compared to sawn lumber, however, because you can use smaller members, in the final analysis, they actually may be cheaper.

- **LSL's**. LSL is an acronym for Laminated Strand Lumber. These are manufactured similarly to PSL's, with an exception that they are made with different types of wood for the strands. Trus Joist MacMillan's brand of LSL, called TimberStrand uses Aspen or Yellow Pine. LSL's are considerably weaker and more flexible than PSL's and are recommended for use as rim boards and studs rather than beams or columns.

- **LVL's**. LVL is an acronym for Laminated Veneer Lumber. These are made from thin sheets of wood (similar to the sheets used to make plywood) that are bonded together with epoxy resin under tremendous pressure. The result is a structural member that is similar to thick plywood, and has very high strength values. Typical strength and stiffness values approach those of PSL's, but are slightly less.

4.8.3. Manufactured Wood I-Joists. These are another manufactured wood product. You sometimes hear them called 'TJI's'. TJI is a proprietary name for Trus Joist MacMillan's brand of I-Joist (there are other readily available brands which are of similar strength and cost).

COMPARASON OF SAWN LUMBER VS. I-JOIST



I-JOIST	VS.	DOUG FIR
9.5", 150	44% STRONGER	2x10 #2
9.5", 150	3% STIFFER	2x10 #2
9.5", 150	10% STRONGER	2x10 #1 & BTR.
9.5", 150	8% LESS STIFF	2x10 #1 & BTR
11 7/8", 150	44% STRONGER	2x12 #2
11 7/8", 150	SAME STIFFNESS	2x12 #2
11 7/8", 150	10% STRONGER	2x12 #1 & BTR.
11 7/8", 150	12% LESS STIFF	2x12 #1 & BTR.
11 7/8", 250	29% STRONGER	2x12 #1 & BTR.
11 7/8", 250	SAME STIFFNESS	2x12 #1 & BTR
9.5", 150	8% STRONGER	2x12 #2
9.5", 150	42% LESS STIFF	2x12 #2

All of these are essentially the same; they typically have an LVL or PSL top and bottom flange, and a plywood or LVL web. They look a little like a wood 'I' beam, and are commonly used for joists and rafters. They come in many depths, and are available in any length that can be shipped. They typically are used in lieu of 2x sawn material. Like PSL's they have superior allowable stresses compared to sawn lumber, but are more expensive when compared board-foot to board-foot. They are dimensionally uniform, another advantage over sawn members.

Question: *"I've been building with TJI's for years and really like them. The only problem is a lot of the floors I construct with them turn out to be bouncy. If they are so much stronger than sawn, why do the floors feel so bouncy?"*
Answer: *This is a very good question because the answer is subtle. Yes, they are much stronger than sawn, but remember, strength and resistance to deflection are two different things. Recall that in bending member design, three things had to be considered: shear, bending, and deflection. Shear and bending are strength properties. Deflection is a function of stiffness. So it is possible to have very strong members which happen to be somewhat flexible (i.e. low stiffness). With any wood product, while strength properties can go up and down dramatically with the quality of the wood, stiffness does not vary that much – even with manufactured wood products. For example, look at the first two rows in the table to the left and you will see a similar sized I-Joist are 44% stronger than sawn, but approximately the same stiffness. So the answer to the question is that with a long span I-Joist floor, it is the deflection that almost always controls, not bending or shear strength. As a result, you can get away with lighter, flimsier I-Joist members, whose*

strength is greater than a sawn member, but whose resistance to deflection (bouncyness) may not be any better. If you would have used sawn members, it is likely that bending strength would have controlled, and you would have had to use a beefier section or closer joist spacing to cover the strength issue, which would have resulted in a stiffer floor by default.

If you are bent on using I-joists, make sure to tell your engineer that you want a stiff floor. He will then size the joist using a more stringent deflection criteria, which will result in deeper or beefier I-joists than you would have otherwise expected.

4.9. Connections with Wood. The most common way of connecting wood together, or to other structural materials is with bolts and nails. Probably the most important thing to remember about nailed and bolted connections is that the code allowed strength of these is pathetically low. For example, one 1/2" anchor bolt in a 2x6 mud sill is good for a measly 250 pounds of shear perpendicular to grain and 400 pounds of shear parallel to grain. This is shocking when you consider that the amount of shear in a typical mud sill, due to wind and seismic forces is commonly well into the thousands of pounds, sometimes tens of thousands of pounds. So for example, say you have a 20' long shear wall, with 7,500 pounds of earthquake induced shear. The number of 1/2" anchor bolts required would be 12 anchor bolts, or one anchor bolt every 20 inches (and this calculation includes a 60% allowable stress increase due to the short term loading).

***Question**: "Hey, wait a minute. I thought the UBC says that I can put one 1/2" anchor bolt at the corners of my walls and at 6' spacing in between. For a 20' foot wall, this results in about 5 anchor bolts – less than half of what you just said. What's going on here?" **Answer**: The UBC gives lots of MINIMUM prescriptive design information, such as standard anchor bolt spacing. But if you actually calculate the number of anchor bolts needed in a wall to resist wind and earthquake, using the published bolt values in the UBC, you will find that a lot more are needed than the same UBC's prescriptive minimums. This quirk in the UBC occurs in various places throughout the code. It is very maddening to engineers who have to answer this question to contractors who know their code well. To make some sense of this, the UBC has lots of weasel language that says such things as: "In the case of a conflict, use the more stringent value"; or "…this section of code may be superceded by the building official"; or "…. Non-standard construction must meet the requirements of section- BLAH.2.(a).(iii).IV.7.(f)"; etc. My interpretation here is this: the UBC is a set of MINIMUM rules and design suggestions that are supposed to cover every single type of construction in the world. If you build by the prescriptive sections of code, your structure will be pretty competent. It may however (and likely so) not meet the requirements of an engineered structure (using the very same code!). So in the cases where the owner, contractor, or building official wants a higher level of constructed strength and lower level of vulnerability to failure, apply engineering and observe as many aspects go beyond the UBC's minimum prescriptive requirements.*

4.10. Nails. Published shear strength values for nails are, in my opinion, surprisingly low. This is not because the <u>nail</u> will break if loaded beyond the published maximum value, rather it is because the wood around the nail will start to crush. But even if it starts to crush at a high load, does that mean the connection has failed? Not necessarily; and that is what really bugs me about published nail strength values.

Typical strength values for standard **shear type** nailed connections in Doug Fir with full nail penetration using box nails are: 16-d is good for about 100 pounds and 8-d is good for about 70 pounds. If common wire nails are used, the values increase to 140 lbs for 16-d and 90 lbs for 8-d. (Shear connections are where the wood members being connected would slide apart if not for the nailing. This is different than pullout loading, where nails are in tension and hold two members from pulling apart.) So what this means is that if you have, say 1,000 lbs of shear between members, you would need at least 7, 16'd commons to hold them together. If the members were 2x4's and they overlapped 3-1/2", you'd likely split the heck out of them trying to get this many nails in the connection. The UBC handles this by saying that if you can't get the prescribed number of nails into a connection, you have to *pre-drill* the nail holes. My response is, "Yeah, right - how many framers actually pre-drill their nail holes? None that I've ever seen."

For common nails being used to resist pull-out, the code allows 40 lbs for a 16-d and 32 lbs for an 8-d (in Doug Fir). I, however would never count on a nail to resist pull-out. When this situation arises in my office, I will always specify the use of screws.

Question: *"One time I had a truss designed by an engineer, that I wanted to field construct from 2x4's and plywood gussets. I could not believe the number of nails required in the heel gussets. There must have been 50, 10-d's at each heel, which took gussets on each side of the truss 3-1/2 feet long to accommodate. Was the engineer wacko or what? I know I could have used half as many nails, and a bulldozer still would not have been able to tear the heel apart.* ***Answer***: *The engineer was probably me, and I was just following the code. The shear force (derived from compression in the top chord and tension in the bottom chord, trying to slide them past each other) at the heel of any truss spanning 25 or more feet is several thousand pounds. Lets say you are connecting the truss heel together with 8-d's and the heel force is 3,000 lbs. Now remember, that's 3,000 lbs in each the top and bottom chord, and each has to be nailed to the gusset to take that load. So, you'd need 34, 8-d's in the top chord and 34 in the bottom chord to make this connection; that's 68 total per heel, 17 per member per side. To avoid splitting of the members you'd have to space the nails say 1-1/2" apart, so your gusset winds up being about 30" long, allowing a little edge distance. This is absolutely crazy, in my opinion. I think that if you actually did a load test on this connection, you'd probably break the test equipment or snap the top or bottom chord way before the joint started to slide apart. So what's going on here is Factor of Safety. Nail values have a tremendous Factor of Safety (I've heard on the order of 5!) built into them to allow for such things as splitting, hitting knots, bent nails, and framers who can't count to know how many they've installed. But as much as I may disagree with published nail values, I still have to go by the code, and so do all other engineers.*

Question: *"With this last example in mind, how in the world do truss manufacturers get away* *with those puny little metal gang nail plates?" **Answer**: They have gone to great expense to actually perform tests on those gang nail plates, and have published data, accepted by ICBO (the people who publish the UBC) on the allowable strength of each one they use. And the Factor of Safety for the plates is probably in the 2.5 range, not 5.*

4.10.1. Toenails. In the old days, before the advent of framing hardware and angle clips, lots of wood to wood connections were made with toenails (angling of nails through one member into another). The UBC still allows this, but the strength values are 5/6 of a standard nailed connection. Actually, in seismic zones 3 and 4 (Western Washington) toenails are not allowed to transfer shear loads of greater than 150 pounds per lineal foot (UBC 2318.3.1).

I personally do not like any toenailed connection that has any substantial or critical load in it. The problem is that framers very often split the wood, or get poor nail penetration with these connections. I always specify an angle clip, say a Simpson A34 or L50 instead.

4.11. Bolts. To design a bolted connection, you must know the allowable strength of the bolt(s) in wood, and the applied load. There are tables in the UBC (23-III-3-1 and 23-III-3-2) which give you allowable strengths of bolts in single and double shear applications respectively (see below for description of single and double shear). <u>Beware!, this table has a footnote that is very important which says that the tabulated values shall be adjusted per Division III, Part I</u>. Division III, Part I says almost nothing other than the UBC adopts the National Design Specification (NDS) for Wood Construction. If you have a copy of the NDS, you will find that there are many very important modification factors to be used for bolt design! Some of them follow:

- **Duration of Load**. Bolt values are increased or decreased depending on how long the applied load is expected to last. The UBC actually has a duration of load adjustment table too, which supercedes the NDS duration of load values.
- **Bolt Group Action**. Bolt values are adjusted depending on how many bolts are used in a group and on how the bolts in the group are arranged.
- **Lag Bolts**. Shear values for lag bolts are adjusted depending on the amount of penetration of the screw portion.
- **Edge, End, and Bolt-to-Bolt Distance**. Bolt values are reduced if inadequate edge, end, or bolt-to-bolt distances are provided.

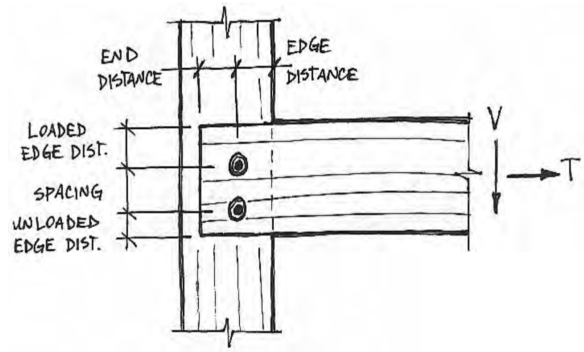
Also, the UBC bolt tables only list a few thickness of bolted members. If you are bolting members of different thickness than shown, there are complicated design equations that must be used to determine the allowable bolt strengths.

4.11.1. Parallel vs. Perpendicular to Grain. Bolt values vary hugely depending on how the load is applied. If the load is parallel to grain, the bolt value is about double that of perpendicular. If it is applied at some angle to the grain, the value is somewhere in between. The UBC bolt tables show this as $Z_{parallel}$ and $Z_{perpendicular}$.

4.11.2. Single vs. Double Shear. The allowable strength of a bolted connection acting in shear (i.e. the boards loaded such that they tend to slide apart) depends on the number of boards being joined. It is common to see published bolt values in *single shear* or *double shear*. Double shear means that a bolted member is sandwiched between two other members, with a bolt(s) going through all three. In the real world, this happens only a certain percentage of the time. In most of the other cases, two members are simply bolted together, side by side. This is called *single shear*; and the strength of the bolts in the connection is about half of the published double shear value. The case of an anchor bolt into a mud sill or ledger is an example of single shear; and remember, the strength of each bolt is about half that shown in a double shear table.

4.11.3. Species of Wood being Bolted. The strength of a bolted connection depends on the species of wood being connected. Doug Fir has some of the highest bolt strength values for softwoods. Beware if you use Hem Fir or other species – the values can go down dramatically. This has to do with the density of the wood; the denser the wood, the higher the bolt value. The UBC bolt tables do not list Hem Fir, but Spruce-Pine Fir is listed which is similar in density and bolt strength. Hardwoods (maple, alder, birch, oak, etc.) have very high bolt strength values, but hardwoods are used more for trim and finish work rather than structural timber applications.

4.11.4. Edge, End, and Bolt-to-Bolt Distances. One of the most important aspects of a bolted connection is the distance between the bolt center and the edge of the member being bolted. In order to develop the full strength of the connection, adequate *edge distance* must be provided. There are many rules for this which apply not only to edge distance, but also to *end distance* (the distance between the bolt center and the end of the member), and *bolt -to-bolt distance*.



Edge, end, and bolt-to-bolt distances are referred to in multiples of the diameter of the bolt being used. For example, standard edge distance is 5 bolt diameters, so if you have a 3/4" diameter bolt, you must provide 3/4 x 5 = 3-3/4" between the center of the bolt and the edge of the member. Edge, end, and bolt-to-bolt distances vary, but if you want to be safe, provide 7 bolt diameters for everything and never less than 4 diameters. The **exception** to this is unloaded edge distance. For unloaded edge distance, more is not better – it is worse. Typically, 1.5 bolt diameters for unloaded edge distance is recommended.

4.12. Lag Bolts. Lag bolts are actually big screws, 1/2" diameter is common, that are used to connect structural wood. They normally have a hexagonal head for application with a wrench or socket. Lags are allowable to resist both shear forces and pullout.

Other than small diameter lags (i.e. ¼" diameter and less) I personally don't much care for these, but there are cases where will I allow them in my designs. The reason I'm leery of lags is that there is a high degree of care and expertise required to install one correctly, and most framers are in too much of a hurry to do it right. Every lag bolt greater than ¼" diameter requires a pilot hole to be drilled before the bolt is installed. Now I know there are contractors out there who would disagree with me on this point – they would say, "Heck no, Garrison, you can install lags in wet wood just fine with no splitting at all. I've been doing it that way for 127 years and never had one fall out yet." My answer is, "Have you been back to look at every one to be sure the wood isn't split or the lag hasn't loosened in the hole?" At any rate, A pilot hole must be drilled, and it should be about the same diameter as the shaft of the bolt; a little smaller is okay so long as the wood doesn't split when the bolt is screwed in. So how do you know what the shaft diameter is? Most times you eyeball it, and this is where mistakes are made. It is easy to oversize the pilot hole by misjudging the bolt's shaft diameter, or by putting the hole in the wrong place, then having to ream it out some, or just by sloppy drilling. In any of these cases (which happen frequently, and which are seldom reported or corrected by the perpetrator) the holding power of the lag goes way down. Sometimes, if the lag is intended to resist pull-out (tension), these types of mistakes can render the lag absolutely useless, yet it is frequently installed anyway and not corrected.

4.13. Washers. Washers under bolt heads and nuts are always a good idea. What do washers do anyway? A couple of things. First, they help to keep the bolt head and nut from gouging into the wood upon tightening.

Secondly, if you have a shear type connection (as opposed to tension), the bolt value in shear typically does not take into account the friction of the members being connected. If, however you snug up the connection with a washered nut and bolt head, you will develop a fair amount of friction in the connection to assist in the shear holding power; which is good, particularly in a wind or seismic event.

Another use for washers is in a bolted connection which must resist tension in the direction of the axis of the bolt, like an anchor bolt through a mud sill experiencing uplift during a strong wind or earthquake. If there are no holdowns installed (like in many older homes), the only thing to keep the walls down are the anchor bolts. If there are no washers, it is quite easy for the mud sill to be pulled right through the nut (particularly in the cases where the contractor did a poor job in getting the bolt holes in exactly the right place and had to ream them out to get the mud sill installed). The inclusion of a washer more than triples the connection's resistance to pull-through. In fact, when I design shear walls, if the uplift is 1,000 lbs or less, I sometimes count on the anchor bolts, with 2"x2"x3/16" plate washers (these washers are *required* on all anchor bolts in seismic zones 3 (western WA) and 4, per UBC 1806.6.1), to take the uplift rather than specifying the installation of a holdown(s).

4.14. Plywood Shear Walls. There is considerable language in the UBC concerning plywood (and other materials for that matter) shear walls. Shear walls are discussed in detail in chapter 7 herein.

CHAPTER 5
STEEL AS A STRUCTURAL MATERIAL

5. Steel. Similar to wood, steel comes in a vast array of sizes and strengths. If you compare the strength and stiffness of standard A36 structural steel to Doug Fir #2, you find the following:

- Steel is approximately 25 times stronger in bending than wood.
- Steel is approximately 18 times stiffer (resistant to deflection) than wood.
- Steel is approximately 200 times stronger in shear than wood.
- Steel is approximately 85 times stronger in compression than wood.
- Steel weighs approximately 10 times more than wood.

Because steel is so much stronger than wood, it is much more prevalent in larger construction projects.

5.1. Nomenclature. There are several trade organizations, who's mission in life it is to develop, promote, and / or test steel. Of course each has it's own way of classifying the various types of steels. The American Iron and Steel Institute (AISI) has their system, which consists of a 4 digit code indicating the chemical composition and the steel making process. For example with an AISI Grade 1035, the 10 indicates carbon steel, nonresulfurized with manganese content of 1.00% maximum; and the 35 indicates a carbon content of between .32% to .38%. The American Society of Testing and Materials (ASTM) has their own nomenclature. They use an 'A' in front of a two or three digit number which indicates chemical composition and / or minimum strength and ductility. In the construction industry, it is the ASTM specification that is most commonly used.

Question: *"I always hear people referring to 'mild steel'. What does this mean, and is the steel that is delivered to my jobsite 'mild'?"*
Answer: *Sometimes (particularly in the past) steel is referred to by it's hardness. The hardness of steel is a function chiefly of the amount of carbon in its chemical makeup. Soft steel has .20% carbon maximum; mild steel has .15% to .25%; medium steel has .25% to .45%; hard steel has .45% to .85%; and spring steel has .85% to 1.15% carbon. In general, mild steel refers to non high-strength carbon steel, such as A36, which is the most common type of steel used in the construction industry.*

5.2. Yield Stress and Ultimate Strength. *Yield Stress* is a measure of a steel's useable strength without *permanent or plastic* deformation. It represents the maximum amount of load that can be applied *before the member deforms plastically*. In other words, you can load steel up to it's yield point, and not cause any permanent damage or permanent deflection; it would spring back to its original shape if unloaded. This is called *elastic* behavior. Yield strength and elastic behavior differs from *ultimate* strength and *plastic* behavior in that *ultimate* strength is a measure of total reserve strength that is achieved only after the steel has deformed permanently or plastically. Ultimate strength represents the maximum amount of load you can apply just before the member breaks or ruptures. If you load steel to it's ultimate strength, you load it beyond it's yield strength and into the range of plastic (permanent) deformation. If you were to unload it, it would not resume it's original shape, it would have a permanent bend or deflection in it.

When engineers design steel members, they can use one of two methods; the *allowable stress method* or *ultimate strength design*. These methods use steel's allowable yield stress or it's ultimate strength respectively in the design. The allowable stress method is usually more conservative and typically results in larger members. The differences between the two are as follows:

- The allowable stress design (ASD) method applies a factor of safety to the <u>strength</u> of the steel, and it uses allowable yield stress. The actual calculated loads are used, unadjusted, to determine internal stresses in the steel members.
- The ultimate strength design method, also known as Load Reduction Factor Design (LRFD) applies a factor of safety to the <u>loads</u> rather than to the allowable stresses of the steel. In doing this, the ultimate strength of steel may be used rather than the more conservative yield stress.

5.3. Grade of Steel. Sometimes steel is referred to by it's yield stress or 'grade'. You may hear '36 ksi' or '50 ksi', or grade 36, or grade 42 when referring to the strength of steel. All of these numbers indicate the maximum allowable *yield stress* of the steel in ksi. Ksi means kips per square inch (a kip is nothing more than a term for 1,000 lbs). When someone talks about a 36 or 50 (or 60 or 75, etc.) ksi steel; or grade 36, grade 42 or grade 50, etc. they are talking about the maximum allowable yield stress of that steel. Of course, the higher the grade, the higher the allowable yield stress and the stronger the steel.

5.4. Stiffness (Modulus of Elasticity 'E') A very important, but subtle point concerning the various strengths of different steels is that the allowable stresses do vary considerably, but the modulus of elasticity, 'E' stays the same (29,000 ksi) for all structural steels. Recall from Chapter 2 that 'E' is a measure of the material's relative stiffness, or its resistance to deflection. So, you can obtain a very high strength steel, which may result in a smaller size member than with a lower strength steel, but its resistance to deflection will be the same. For long members, deflection typically controls the design (rather than shear or bending), so there may not be any advantage in using the more expensive higher strength steel.

5.5. Types of Steel. There are many different types of steels. Most never find their way to a construction site because they are used primarily in the world of heavy industry for such applications as automotive parts, electric motors, generators, tools, machinery, heavy equipment, etc. There are a few basic types of steel that occur commonly on the construction site.

5.5.1. Carbon Steel. This is the basic structural steel that has been around for many years and is still the most common today. There are two commonly available types of carbon steel. A36 is the all-purpose version widely used in building and bridge construction. As structural steels go, A36 is not a 'high strength steel', but it is cheap and readily available in nearly any desired structural shape. Carbon steels are great for all around use; they are easily weldable and drillable, but are prone to corrosion. The other common carbon steel is A529. It is only available in grade 42, which makes it higher strength than A36; and it is available in the smaller size shapes only.

5.5.2. High Strength Low Alloy Steel. These structural steels are specially formulated to impart greater strength and / or resistance to atmospheric conditions than regular carbon steel. The following bulleted items are the commonly available high strength low alloy steels. You will note that there are several grades available for each. In general, the smaller size shapes are available in the highest strength grades, and the very large sizes are only available in the lower strength grades.

- **A441**. High strength low alloy structural manganese vanadium steel. Grades available in structural shapes: 50, 46, 42 depending on the size. Grade 40 is available only in plates and bars.
- **A572**. High strength low alloy columbium – vanadium steel. Grades available for shapes, plates and bars: 65, 60, 50, 42 depending on the size.
- **A242**. Corrosion resistant high strength low alloy structural steel. Grades available in shapes, plates and bars: 50, 46, 42 depending on the size.
- **A588**. Corrosion resistant high strength low alloy structural steel. Grades available for shapes: 50. Grades available for plates and bars: 42, 46, and 50 depending on thickness.

***Question**: "Why are the very large size shapes only available in the lower strength steels? It seems to me that it would be more efficient to use higher strength steels for the larger sizes – you could have longer spans with smaller size members. Isn't that the goal of engineering anyway?" **Answer**: The first goal of engineering is always safety. A secondary goal is efficiency. The use of high strength steels in very large sizes and long spans surely satisfies the efficiency goal, however it puts the first goal of safety in jeopardy. The basic reason for this is that large sizes used for long spans must be extra stable against lateral buckling. Smaller, stronger members may be able to take more load or span farther, but they do not have enough 'meat' to provide the lateral stability of a larger member.*

Another reason for using only lower strength steels in the very large sizes was alluded to previously in the section on stiffness. To restate, long span members are very frequently controlled in their design by deflection, not bending or shear. Deflection is a function of <u>stiffness</u> (modulus of elasticity, 'E'), not <u>strength</u>. So, in these cases, it does no good to use high strength steel because it's modulus of elasticity, 'E' is the same as for lower strength steel (all structural steels, regardless of strength have the same 'E').

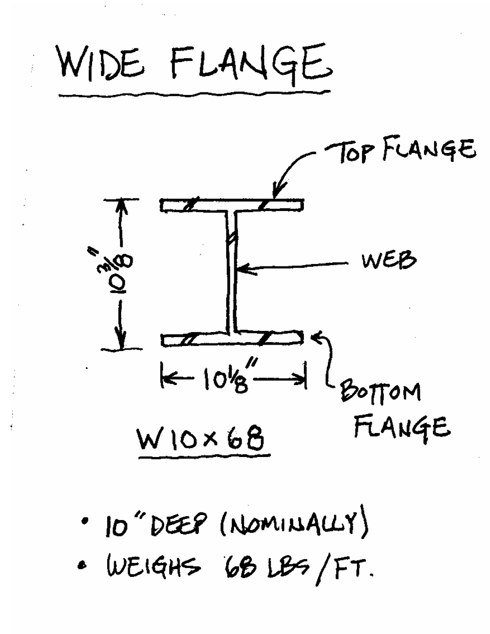
5.5.3. Improved Atmospheric Corrosion Resistance Steels. These are high strength steels with formulations that resist corrosion. A242 and A588 listed above are the two most common varieties. These form a natural rust colored self healing oxide coating on themselves which inhibit further corrosion when left unpainted. In order for these steels to behave as intended, it is necessary that design, detailing, fabrication, erection, and maintenance be properly performed. If these are coated (painted), the coating usually lasts longer than on other steels. These steels are more expensive than other steels, but their reduced maintenance can make them more cost effective in the long run.

5.5.4. Stainless Steel. There are three basic type of stainless steels, but only the Austenitic Nickel – Chromium Steels are common in construction. They are sometimes referred to as '18-8' alloys, because they contain 18% chromium and 8% nickel. Common types are 301, 302, 303, and 304, with 302 being the most common. Type 316 is the most resistant to corrosion and attack by salt spray, and thus is most common in marine environments and applications. All of these steels have excellent corrosion resistance, with high strength and ductility, but are expensive compared to carbon steels.

5.6. Shapes. In the construction industry, there are certain shapes of steel members that are common. Following are a few of these.

5.6.1. 'W' Shapes - Wide Flange Sections. These are perhaps the most common. Some (uninformed) people may call them 'I' beams. Strictly speaking, there is no such thing as an 'I' beam or section. There is such a thing as an 'S' section, however, that looks just like an 'I', and is not a 'W' section. See below for more on 'S' shapes. Wide flange shapes have a parallel top and bottom flange, and the perpendicular part that joins them is called the web. This nomenclature (flanges and web) is generic to all steel sections that look like an 'I' or 'H' in cross section.

Wide flange beams are referred to by their depth and weight per foot. For example, a W12x50 is a wide flange section that is 12 inches deep and weighs 50 lbs per foot.



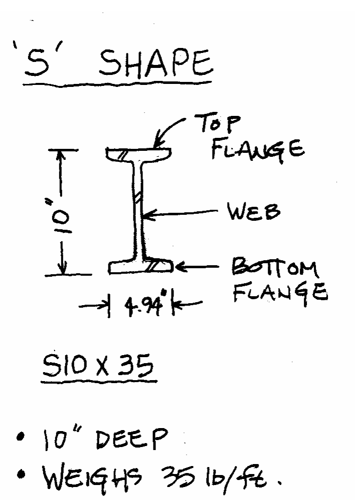
***Question**: "I think you're wrong about that Garrison; when I talk about wide flange sections, I talk about the depth and the <u>width</u> of the beam. For your W12x50, I'd call that a 12 x 8 because the depth is 12 inches and the flange width is 8 inches. Who's right?" **Answer**: The proper way to specify wide flange sections is by the depth and weight, like I explained above. I know though that certain people insist on using the other approach. Be assured that this is a 'construction faux pas'.*

***Question**: "Okay, so I'll start doing it the right way – I sure don't want to be embarrassed in front of my steel worker buddies. But, when I'm trying to fit a wide flange beam into a masonry pocket that is only so wide, how will I know if it will fit if I don't know the width?" **Answer**: Do what the rest of the world does and buy yourself a steel handbook that has all of the dimensions* *and properties of all the various sections that are available. Not only does it list wide flanges, but it also lists pipe, tube sections, angles, channels, bars, etc. If you don't want to buy one, and you have to know about the beam pocket problem at hand, you could call your local steel yard.*

Wide flange sections are available in carbon and high strength grades. Because they are the most common type of structural members used, they are relatively cheap and readily available.

A word of caution about wide flange sections. They are excellent bending members when loaded in their strong axis AND are laterally braced. If they are installed to resist bending by their weak axis, their strength is much lower, as much as half. Also, they are extremely prone to buckling if the compression flange is not braced against lateral movement. Remember your negative moments!

5.6.2. 'S' Shapes – American Standard Shapes. These could be considered the classic 'I' beams (shapes) because they look like a capital 'I' when viewed in cross section. *But don't call them 'I' beams – call them 'S' beams.* They are similar to wide flange shapes, but are



not as strong in bending because their flanges (tops and bottoms) are not as wide. They are also more unstable to lateral buckling. These are not nearly as commonly used as wide flange sections, and are not available in as many sizes.

5.6.3. Tube Sections. These are square or rectangular tubular structural sections. They are normally only available in 46 ksi (high strength) grades. The typical specification for tube sections is Cold Formed A500 Grade B. They are excellent for use in low rise and certain residential construction, however, their cost is high compared to wide flanges, and they are sometimes not as readily available. They typically have good weak axis bending properties, and are not as prone to buckling as wide flanges. The nomenclature for specifying tube sections is with a 'TS' (for Tube Section). A common small size is TS 6x4x1/4. The 6 and 4 are the outside dimensions and the ¼ is the wall thickness. The corners are rounded (not square) with corner radius equal to approximately two times the wall thickness. So the thicker the walls, the rounder the corners.

5.6.4. Structural Steel Pipe. Pipe is readily available in many diameters, and three wall thickness. It is used primarily for columns and posts, but can be used for bending or tension members as well.

***Question**: "Why not use pipe for a bending member? It is quite stable against lateral buckling, no?" **Answer**: True, pipe is relatively stable against lateral buckling, but it does not even come close in resistance to bending as compared to say a wide flange section. For example, a 6" diameter pipe which weighs about the same as a 6" wide flange is only about <u>half</u> as strong in bending. This has to do with the* *wide flange having about twice the moment of inertia compared to the pipe. See the section on moment of inertia in chapter 2 for an explanation of this.*

Pipe is normally 35 or 36 ksi steel. Interestingly, although 35 ksi pipe is a little weaker than 36, AISC allows 35 ksi pipe to be designed to the 36 ksi allowable stress. The common types of pipe are cold worked A53 Grade B, 35 ksi; and hot formed A501, 36 ksi. Pipe is specified as Standard Weight, Extra Strong, or Double-Extra Strong.

- **Standard Weight** is standard weight pipe, also referred to as schedule 40.
- **Extra Strong** has thicker walls and is also known as schedule 80.
- **Double-Extra Strong** has thicker walls yet and is known as schedule 120.

Standard Weight pipe is specified by it's *inside* diameter. Interestingly however, as you move into extra and double-extra strong, for each nominal pipe size, the inside diameter gets smaller, but the outside diameter remains the same. For example, 6" standard pipe has an inside diameter of 6.065" and an outside diameter of 6.625". That outside diameter is exactly the same for 6" extra strong and 6" double-extra strong, but the inside diameter reduces to 5.761" and 4.897" respectively.

Question. *"I recently was working on a job where I had to fit a pipe column through a gap in some machinery that was exactly 8" across. I thought, no problem, just get some schedule 40, 8" diameter pipe. When I delivered it to the jobsite, and went to place it, it didn't fit through the gap. Did the steel yard screw up or what?"* ***Answer***. *It was you who goofed, not the steel yard. Schedule 40 (standard weight) pipe is specified by it's inside diameter. If you would have checked your steel handbook, you would have seen that the outside diameter of 8" schedule 40 is 8.625", 5/8 of an inch over what would fit through your gap. It is interesting to note that regardless of whether you bought schedule 40, 80, or 120 8" pipe, you would have missed your fit by 0.625", because the outside diameter of all three is exactly the same (but the inside diameter gets smaller with the heavier grade pipe).*

Pipe is unbelievably strong when used as a post or column, as long as the unbraced length is kept short. For example if you have a short 4' long post of 3" diameter standard weight pipe, it will safely hold 60 kips of compression force (assuming no additional moment loads), which equals thirty tons! That same 3" post if lengthened to 20 feet, and with no lateral bracing will only hold 12 kips or 6 tons. If you wanted to use that same 3" pipe for an unbraced column of 25 feet, the steel code would not allow it at all.

Question: *"You keep talking about short columns and posts. How is 'short' defined exactly? How would I know, when looking at a post or column whether it is short or long?"* ***Answer***: *I wish the answer was simple, but it is not. The calculation involved in determining whether a column is short, intermediate, or long is not straightforward. Consider an uncooked piece of spaghetti; if it were a column, in order to be short and not buckle it would have to be 3, maybe 4 inches at the most. Now consider an 8 inch diameter pipe column. It could be 10 feet long and still be considered 'short'. The non-engineer should know that any compression* *member's strength has to do with three things basically:*

- *the unbraced length of the member*
- *the member's inherent resistance to bending or buckling in any direction (i.e. its stiffness)*
- *the fixity of the member's ends*

As we've discussed previously, the longer the unbraced length, the weaker the compression member. Regarding resistance to buckling or bending, this has to do with how stiff the member is, which is based on it's cross sectional shape. In general, the deeper the cross section, the stiffer and more resistant to buckling in that direction. But, you have to look at <u>*both axes*</u> *of the member; the member does not care which way it buckles and will seek out the weakest direction. Lastly, if the ends of the member are fixed, it is less likely to buckle than if the ends are pinned (free to rotate). In summary, there are too many variables involved to give you a rule of thumb for determining whether a column is short or not. If you are in doubt, I suggest you contact your friendly neighborhood engineer to make the call for you*.

One great thing about pipe when used for a post or column is that because it is symmetrical, it's resistance to buckling is the same in all directions (this is true for square tubular members as well, of-course). So once you calculate the allowable compressive load, you don't have to worry about weak axis bending or buckling, because there is no weak axis.

5.6.5. Angles. Structural angles are shaped like an 'L' in cross section. They come in many sizes and thickness. Some sizes have the same length legs, and some have different. Angles are commonly available in 36 and 50 ksi steels. A commonly available angle is L 3 x 2-1/2 x ¼. This is an angle with one leg of 3 inches, the other of 2-1/2 inches, and 0.25" thickness. Angles are commonly used as tension or compression members, for example webs in a fabricated steel truss. Many times angles are used back-to-back as 'double angles'. Angles are not typically used to carry heavy bending loads, that job is usually reserved for wide flange sections.

Question*: "I love working with angles, use them all the time. I can easily get big L6x6x3/8 angles at the steel supplier. Why shouldn't I use them to carry heavy bending loads?"* ***Answer****: You certainly can use them for bending members, but before you go out and buy a bunch more, consider this. If you orient an angle as a beam with the bottom leg horizontal and the other leg vertical sticking up, the vertical leg becomes the compression part of the beam in bending (assuming a simply supported beam). This vertical leg has almost no resistance to buckling laterally, and can easily flop out of plane causing a catastrophic and sudden failure. If you understand lateral bracing principles and orient the flat leg up and the vertical leg pointing downward, you greatly help this compression flange buckling problem because the flat leg becomes the compression flange and is wide and resistant to moving laterally. But your tension 'flange' is only the vertical leg. In this case, your allowable bending load is quite small before you overstress the vertical (tension part) leg. You'd be much better off to use a wide flange or tube section for your bending member.*

5.6.6. Channel Shapes. These are shaped like a square cornered 'C' in cross section (outside corners are squared, inside are rounded). They are commonly available in 36 and 50 ksi steels. An example of a common channel section is a C6x8.2, which means it is 6 inches tall and weighs 8.2 pounds per foot. You'll notice that these are specified like wide flanges, with the height and weight per foot. A C6x8.2 happens to be 1.92" wide (flange width), but you wouldn't specify it by it's width. Like wide flanges, channels are strong in bending in their strong axis, but can be very weak if loaded in bending the other way. If used as bending or compression members, they should be braced against lateral buckling because of their relatively low weak axis bending resistance (propensity to buckle out of plane).

5.6.7. HP Shapes. These are extra wide, wide flange sections. They are only available in sizes ranging from 8" to 14" deep, and are normally used for heavy construction and piling. Their available grades are 36, 42, 50, and 60, but your local steel yard will likely only have one or two grades. They have good resistance to bending in both axes. Any HP section is approximately as wide as it is deep. An example of a typical HP section is an HP 12x63, which is 12 inches deep and weighs 63 pounds per foot. It happens to be 12.125" wide.

5.6.8. M Shapes. These are similar to wide flanges, but are very limited in their available sizes and weights. In general if you want a certain depth 'W' shape, but the lightest section is still too heavy for your application, look into an 'M' shape. They are generally narrower and lighter than a 'W' of the same depth. Your local steel yard may or may not even stock 'M' shapes.

5.6.9. Structural Tees. These are tee shaped sections that are cut (and then re-straightened) from standard 'W' shapes. A common tee is a WT 8x20, which is 8 inches deep and weighs 20 pounds per foot; and has a flange width of 7 inches.

5.6.10. Rods and Cables. There are times when a design calls for a 'tension only' member. This is a perfect application for a rod or cable, because these have good tensile capacities, but zero compression capacity. You couldn't ask for a more efficient design when dealing only in tension.

In the AISC manual, rods are a subset of the general category called 'bars'. Bars include square, round, and rectangular solid sections. Bars come in all of the steel grades listed previously. They can be custom cut to the length you specify, and the ends threaded prior to delivery to the jobsite.

Cables (sometimes referred to as 'bridge wire') are typically made of very high strength steel. Minimum breaking strength is about 200 ksi (kips per square inch) and minimum yield strength (at 0.7% elongation) is about 150 ksi. Recall that minimum yield stress of A36 steel is 36 ksi, only 25% as strong as cable. The actual strength of cable depends on the coating type and thickness (galvanizing typically), so check with your supplier for actual yield and breaking strengths. Cables are connected to other structural members with either factory installed ends (open sockets, button sockets, threaded stud sockets, etc.) or compression / friction type fittings that can be installed in the field. Regardless of the connection type, the rod or cable is only as strong as the weakest connection, so care must be exercised to ensure that every connection or splice has at least that same ultimate tensile capacity as the rod or cable itself.

A common application for rods and cables is in X-braced steel frames. Whenever rods or cables are used for X-bracing or other lateral load (wind or seismic) resisting purposes it is important that they be installed tightly, but not too tight. If left slack, they will not begin to function until substantial movement / deflection has taken place – which may be too late to save the structure. On the other hand, if they are overtightened, they can pull and distort the structure out of plumb.

***Question**: "I install mobile homes, and use cables for earthquake tie-downs. I usually leave them a little slack. Is this wrong?" **Answer**: Yes. There should not be any slack in a tension cable. It should be installed to a very snug condition. If it is left slack, when the earthquake hits, the mobile home will move some before the cable starts to do it's job. It might move enough to tip over a pier or come off of a foundation; then what good is the cable? It may be that the only way to properly snug up a cable is to use a turnbuckle or other hardware that can be adjusted after the cable is in place. Of course the hardware must have the same ultimate tensile capacity of the cable or it becomes the weak link.*

5.7. Hot Rolled and Cold Rolled Sections. When you go down to the steel yard and buy a wide flange beam, you are buying a steel product that has been formed by hot rolling. Hot rolling means that the shape was formed by rolling the heated raw steel through shaping rollers at a high temperature. Nearly all heavy W, S, and HP sections, (wide flange sections), angles, channels, and tees, are hot rolled. This is different than cold rolled or cold drawn products. Cold rolling is accomplished when steel sheets are shaped in rollers at room temperature. The shaping bends or draws the metal beyond it's yield point so that a permanent set is achieved. Most light gage steel framing products used for residential and light commercial construction, and structural tube sections are cold drawn shapes.

***Question**: "Okay, so some steel is shaped while it's hot and some is worked while it's cold. Big deal. Why should I care?" **Answer**: Most of the time it makes no difference to the Contractor. You should just be aware that there is a difference, and know that someone is talking about a certain type of steel manufacture, rather than a luncheon meat product when they mention cold rolled. The two most important aspects of cold working of steel are; first that cold working hardens and strengthens it, and secondly, cold working also makes steel more brittle.*

5.8. Ductility. One of the most important properties of any structural steel is its ductility or its ability to bend without breaking. Think of ductility as the opposite of brittleness. Brittle steels are very dangerous, particularly in earthquakes because as the structure sways and deflects, a brittle steel can break suddenly and completely causing a catastrophic failure. A ductile steel has the ability to bend plastically but not break; or if it does break, it may rip or crack a little, but not all the way. In this case, the member has, technically speaking, 'experienced a failure', but it did not result in collapse, thereby saving lives and property. This issue of ductility helps to explain why cast iron, which is very brittle, is not a common structural material. In general, the stronger the steel, the more brittle it is. However, all of the aforementioned structural steels are relatively ductile when used and installed correctly. There are several key factors, listed below, which can dramatically increase any steel member's tendency to fail in brittle fracture. Because brittle fracture is normally sudden and catastrophic, care must be exercised whenever any of the following are encountered.

5.8.1. FACTORS WHICH CAN CONTRIBUTE TO BRITTLE FRACTURE:

- **Low Temperature**. As the air temperature decreases, the strength of steel generally *increases*. But, conversely the ductility goes down with a drop in temperature. Very low temperatures in conjunction with some of the following can cause brittle failure.
- **Notches or Geometrical Discontinuities**. These create stress concentrations in the steel which can contribute to brittle failure. These can be created by such things as: rough handling, errant drilling or machining, improper welding, or they can be inadvertently cast into a member by the manufacturer.
- **Rapid or Impact Loading**. The faster the loading rate, the more likely a brittle failure will occur.
- **Cold Working**. Cold working enhances the strength of steel, but like low temperature, it decreases the ductility. If cold working has to occur, avoid sharp radius bends or kinks.
- **Welding**. During welding, the heat generated imparts residual tensile stresses in the steel that are permanent and add to all other externally applied tensile stresses. Consequently there is a greater likelihood of overstressing and / or brittle behavior of heavily welded members.

> ***Question**: "I've got some used high-strength tube steel beams that were salvaged from an old building. I'm going to weld myself up a bridge with them. What do you think about that?" **Answer**. Read this previous section and you will see that you are begging for disaster on some cold winter day when a heavy truck goes flying across. You have every single ingredient for a brittle failure, particularly if the beams are only marginally sized to begin with.*

5.9. Connection of Steel Members. There are two primary methods of connecting structural steel: bolting and welding.

5.10. Bolts and Bolting. Bolted connections are probably the most common way to connect steel. Bolts, like structural steel members come in a wide array of sizes and strengths. The two most common types of bolts are 'common' and 'high strength'.

5.10.1. A307 (Common) Bolts. These are the standard mild steel bolts found on most residential and low rise commercial jobsites. They are commonly referred to as: unfinished, rough, common, ordinary, and machine. Two grades are available: Grade A, which are intended for general applications and will be furnished automatically unless otherwise specified. Grade B are used exclusively in piping system joints; and as such will not be discussed further herein. A307 Grade A bolts are cheap and readily available. Their allowable tensile stress is 20 ksi, and the allowable shear stress is 10 ksi. These bolts do not have a rigorous tightening specification like high strength bolts do. They are normally tightened to a very snug-tight condition with long handled wrenches. Mild or hardened steel washers are recommended, but not required in many cases.

5.10.2. High-Strength Bolts. There are two common types of high strength bolts, A325 and A490. A325 come in three types.

- **A325, Type 1** are of medium carbon steel, in sizes ½ to 1-1/2 inches in diameter. These are identified by the manufacturer's symbol and the legend A325 on the head. They may be hot galvanized. If not specified otherwise, Type 1 are usually supplied by default.
- **A325, Type 2** are of low carbon-martensite steel, in sizes 0.5 to 1 inch in diameter. These may not be hot galvanized. They are identified by three radial lines on the bolt head.
- **A325, Type 3** have atmospheric corrosion resistance and weathering characteristics comparable with those of A588 and A242 steels. They come in sizes 0.5 to 1-1/2 inch in diameter. Type 3 bolts are identified by the legend A325 being underlined.

The allowable tensile stress of all types of A325 bolts is 44 ksi and the allowable shear stress ranges between 12 and 30 ksi, depending on the type of joint.

5.10.3. A490 Bolts. The strongest commonly available structural bolts are A490. These are available in only one type and are identified by the manufacturer's symbol and the legend A490 on the head. Their allowable tensile stress is 54 ksi, and the allowable shear stress ranges between 15 and 40 ksi, depending on the type of connection.

Question: *"Every time I use a high strength bolt on my job, the building inspector requires a special inspection of the bolted connection; which causes me nothing but trouble and costs me money. So, I outsmarted him, and am using A307 bolts instead so that I don't have to get special inspections any more. Aren't I smart?"*
Answer: *No, overriding the engineer's bolt specification is not a smart choice. There is a good reason for using high strength bolts, and it has to do with why they are called 'high strength'. Generally, a connection made with high strength bolts is designed to resist a tremendous amount of stress. Hopefully you noticed previously in this section, the allowable stresses of high strength bolts are more than double those of A307 bolts, i.e. they are more than twice as strong.*

5.10.4. Nuts. Nuts should be of adequate strength and thread length to ensure that the bolt fails before the nut strips off. When using A325 bolts, acceptable nuts will have three circumferential marks on at least one face, or by the inscription 2, 2H, D, DH, or 3 and the manufacturer's symbol. When using A490 bolts, only nuts with the 2H or DH and the manufacturer's symbol may be used.

5.10.5. Washers. There are two basic types of structural washers: hardened and mild steel. Hardened washers are required for bolts in oversize and slotted holes. The washer size depends on the hole type and size. Hardened washers must also be used under either the head or the nut, whichever is turned in tightening, with A325 bolts in standard holes tightened by the calibrated wrench method (the washer is not needed if the turn-of-the-nut method is used). When using A490 bolts with mild steel (Fy<=40 ksi), a hardened washer must be used under both the head and the nut (washers are not required for higher strength steels). In general, it is a good idea to use washers (hardened, if using high strength bolts, mild steel washers okay for A307 bolts) under the element being turned to avoid galling of the substrate steel and to avoid faulty torque readings.

5.10.6. Types of Bolted Connections. In general, there are two types of bolted steel connections: tension and shear. Tension connections occur whenever the bolt experiences a tensile (pulling) load. An example of this is an anchor bolt on a holdown in a shear wall.

The other type of bolted connection is a shear connection. A typical shear connection is where two or more plates or other members are connected side-by-side, and their load causes them to try and slide apart. The bolts(s) holding them together penetrate through the members and keep them from sliding. These bolts are being subjected to shear loading.

There are two types of shear connections: *slip critical* and *bearing*. A slip critical connection is designed so that there will not be any movement of the members being connected, even in a wind or earthquake event. These connections make use of the friction between the connected steel members. In order to utilize that friction, the bolts joining the metal must be tightened sufficiently to squeeze the metal hard enough that it will not ever slip. The only way (per AISC) to do this is with high strength bolts that have adequate tensile strength to resist the extreme tightening. You may see a bolted connection specified as A325-SC, or A490-SC. This tells the type of bolt to use and that the connection is <u>S</u>lip <u>C</u>ritical. The mating surfaces of the metal must also be prepared; cleaned, chips and burrs removed, de-scaled (tight mill scale is an exception and may be left in place), etc. to achieve adequate friction. The amount of tightness that is applied is called torque, and it is measured in foot-pounds. One foot-pound of torque is equivalent to applying one pound of pulling or pushing force to the end of a wrench that is one foot long.

Question*: "Wait a minute, we covered this topic of force times distance in chapter 2. You showed us that crude sketch of a hand on a wrench producing a moment equal to the same force times distance you are talking about here. You never said anything about torque then. Why are you trying to confuse the issue now?"*
Answer*. How observant of you! Torque is analogous to moments, which we covered in chapter 2. The difference is that torque acts along the axis of the member and moments act perpendicular to the axis of the member. So, going back to my poorly drawn wrench sketch; there is a moment in the <u>shaft of the wrench</u> equal to the force of the hand times the distance to the bolt, and there is a torque (or torsion) in the <u>bolt</u> equal to that same force times the same moment arm distance. Fascinating!*

The other type of shear connection is a *bearing connection*. This type does not depend on friction between the connected members; shear is transferred through the bolt and the bearing surfaces of the bolt hole. Bearing connections can be either *standard* or *slot type*. *Slot type* allows for limited movement of the connected members along the axis of the slotted bolt hole.

Standard bolt holes are 1/16" larger than the bolt diameter (this applies to both slip critical and bearing type connections).

The best way to achieve a good bearing connection is to ensure that no threads exist in the body of the pieces being joined (i.e. no threads in the shear plane). If threads must be included, the allowable stresses of the connection are substantially (approximately 30%) reduced. You may encounter a bolted specification that looks like A325-N or A490-X. The 325 or 490 describe the type of high strength bolt, and the N or X means threads may be included or excluded in the shear plane, respectively.

Sometimes a bolted connection must resist both shear and tension; as in an anchor bolt for a steel frame of a pre-manufactured metal building. In this case, the allowable *tensile* stress of the bolt(s) is reduced to account for the summing of applied shear and tension.

5.10.7. Edge Distance. Edge distance is the minimum distance from the center of the bolt to the edge or end of the steel piece(s) being joined. A good rule of thumb is to provide 1.5x the bolt diameter at the minimum. For example with a 0.75" bolt, the minimum edge distance would be 1.5x.75" = 1.125" or 1-1/8". For a 1" bolt it would be 1.5x1" = 1.5". Less edge distance can be used, however, the allowable bearing strength of the connection is diminished.

5.10.8. Tightening Methods. There are three acceptable methods for tightening structural bolts.

- The first is called the **calibrated-wrench method**, and it is accomplished with the use of a torque wrench. Either power impact wrenches or manual torque wrenches may be used. In either case, the wrench must be calibrated to apply bolt tension of 5% in excess of the bolt proof load. (Proof load is the tension in the bolt in kips that is 70% of the bolt's allowable tensile strength. For example, the proof load for a 0.75" A325 bolt is 28 kips.) A torque wrench senses the amount of torque that is being applied to the bolt, and clicks or has some other type of indicator to let the installer know that he's tightened it the specified amount.
- The second tightening method is called the **turn of the nut** method. With this bolt tightening method, the installer snugs the nut, then turns it an additional amount as specified, say 1/2 of a turn, which results in the desired torque. *Snug-tight* is defined as the point at which an impact wrench begins to impact; or as the full effort of a worker using an ordinary spud wrench. It is important that all of the bolts in a bolt group be brought to a snug-tight condition before the final turn-of-the-nut is applied. The amount of final tightening depends on the bolt length, bolt diameter, and whether or not the pieces being bolted are perpendicular or skewed to the axis of the bolt(s).
- The third tightening method allowed by AISC for use with high-strength bolts is called **direct tension tightening**. This method involves the use of direct tension indicator devices such as load-indicating washers or bolts with elements that will shear off or twist off at a predetermined tension.

5.11. Welding. Welding is the second common way of joining structural steel. There are three basic methods of welding; gas, arc, and resistance. With structural steel, arc welding is far and away the most common, so will be the only method discussed herein.

There are three types of arc welding commonly used in construction: *shielded-metal arc*, *submerged arc*, and *gas shielded arc*. Discussions of each follow. In general, all arc welding is accomplished by applying a high electrical current through two pieces of metal such that an electrical arc is produced. At the arc, the temperature is hot enough to melt the metal pieces, thus joining them. Typically, the electrical current is applied through an electrode which is a small diameter metal rod that is inserted or fed from the electrode holder lead of a welder. When the welder operator touches the electrode to the piece or pieces of metal to be joined (which are connected to the other lead of the welder), the arc is produced thus melting the electrode and the metal pieces. The electrode provides filler material as it melts.

5.11.1. Types of Welding. Oxygen and nitrogen in the air have a detrimental effect on the hot metal in a weld area. Several methods have been devised to 'shield' the weld area from these naturally occurring air elements. The three most common types of arc welding in the construction industry are known by their shielding methods.

5.11.1.1. Shielded Metal Arc (SMA). The most widely used arc welding method is '*Shielded Metal Arc*', also known as the '*Coated Stick Electrode'* method. With this method, the electrode is factory coated with a flux material that releases an inert gas as it is melted which repels oxygen and nitrogen. The flux can also contain alloying elements which affect tensile strength, hardness, corrosion resistance, and other physical properties of the weld metal. The stick is consumable, and must be manually fed into the weld by the welder operator as the weld progresses.

Electrode sticks are available in an array of sizes, strengths, and fluxes. Each is identified by a code number EXXXX, where E stands for electrode, and each X represents a number. The first two (or three) numbers indicate the minimum tensile strength, in ksi of the deposited metal in the as-welded condition. For example E70XX is a common electrode, with an as-welded strength of 70 ksi (we'll get to the other X's in a minute). This 70 ksi will be higher than the tensile strength of nearly any steels being welded.

The next number refers to the position in which the electrode will make a satisfactory weld: 1 means all positions (flat, vertical, overhead, and horizontal); 2 means flat and horizontal fillet welds; and 3 means flat only.

The last number indicates the electrical current to be used and the type of coating on the electrode. For example, there are high-cellulose sodium coatings for use with direct-current reverse polarity (electrode positive); or iron powder, titania coatings for use with direct current - either polarity, or alternating current; and others. So if you should encounter an E6018 electrode, you will know that it has a minimum tensile strength of 60 ksi; may be used in all positions; and has an iron-powder, low hydrogen coating, suitable for alternating or direct current – reverse polarity.

The decision of which stick electrode to use is generally left up to the welder, with the exception that the engineer will usually specify the as-welded strength. Structural welding is an elaborate science, and must be performed only by highly trained, certified individuals (see following section on welder certification).

One of the most important, yet most commonly violated aspects of shielded metal arc welding is that the electrode sticks be stored and maintained so as to keep them out of the atmosphere until the time they are to be used. This is because the flux coating will absorb water vapor and other detrimental constituents of air only to be released directly into the weld area, thereby defeating the purpose of the flux and compromising the weld.

> ***Question***: *"The building inspector told me I have to bake my electrodes in an oven before I can use them on the jobsite. First of all, I don't bake; and even if I did, this is an incredible hassle. Why can't I just grab an electrode out of the package, fire up the welder and go?"*
> ***Answer***. *The building inspector is doing his best to be sure that your welds are top quality. If your electrodes are not in a factory hermetically sealed container, they must be oven heated to drive out the water vapor that the flux has absorbed. There are other rules about storing electrodes in ovens, maximum number of 'reheats', maximum time between oven and use, and so on.*

5.11.1.2. Submerged Arc Welding. This is a process similar to shielded metal arc, with the exception that the flux is not pre-adhered to the electrode stick; it is supplied in granular form, and is applied separately as the weld progresses. Because the flux blankets the weld zone, the arc is not visible, hence the term 'submerged' arc. The welding electrode is usually fed (and consumed as the weld progresses) automatically from a coiled reel. This process is well adapted to flat or horizontal applications where long straight welds are called for. With this process, high heat - high penetration welds are easily obtained, and the speed of the weld is much faster than with shielded metal arc.

5.11.1.3. Gas Metal Arc (or gas shielded arc) Welding. This is the third type of arc welding common in the construction industry. A continuous spooled consumable electrode is fed into the weld zone through a hand held 'stinger'. Shielding of the weld zone is accomplished in either of two ways. An external gas supply floods the weld zone with an inert gas or gas mixture (which usually contains an inert gas). With this method, the electrode can be bare metal (no flux), but welding must commence in a windless (usually shop) environment so the gas is not blown from the weld zone. This type of welding is sometimes called *Metal Inert Gas or MIG* welding.

The other way the weld zone is shielded is through the use of an electrode which has flux in the core of it, thus the name *flux cored arc welding*.

When external gas is used, it is stored in a steel 'bottle' and is an integral part of the welding rig. You will occasionally hear the term *wire feed* welding; this is *gas metal arc* welding.

5.11.2. Welding Position. This refers to the position the welded metal is in relative to the welder. When a welder operator wishes to become certified, there are four basic welding positions that he must be proficient in.

- ***Flat*** position is where the weld is made from the top of a nearly horizontal surface, and the top of the weld itself is horizontal. This is the fastest and easiest type of welding.

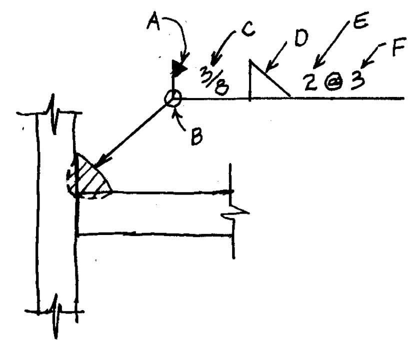
- The flat position is slightly different from the ***horizontal*** position. This is where one of the members being welded is in a vertical position and the other is horizontal. In this case, the top of the weld is not flat, but at a 45 degree angle.
- The third position is ***vertical***. This is where the weld is applied to members standing vertically. This is more difficult and time consuming than flat or horizontal.
- The fourth, and most difficult position is ***overhead***; where the weld is applied upside down (overhead).

5.11.3. Types of Welds. There are four basic types of welds: *fillet, grove, plug, and slot*. Note that these are types of <u>welds</u>, not types of welding processes (shielded metal arc, gas metal arc, etc.) nor types of welding positions (horizontal, flat, vertical, etc.).

5.11.3.1. Fillet Welds. These are perhaps the most common, wherein two pieces of metal at 90 degrees (or close to 90 degrees) to each other are joined by a weld at the intersection. With a fillet weld, the pieces of metal are not grooved or otherwise modified at the location of the weld. Weld material is simply laid down at the intersection of the pieces of metal.

The weld is triangular in cross section. The size of a fillet weld is the length of the longest leg of the weld in cross section. The throat is the distance from the deepest part of the weld to the surface. If the weld face is convex, then the throat distance is more critical than the leg distance, because it represents the weakest potential fracture plane of the weld. If the weld is flat or concave, the leg distance is the critical distance.

<u>FILLET WELD</u>

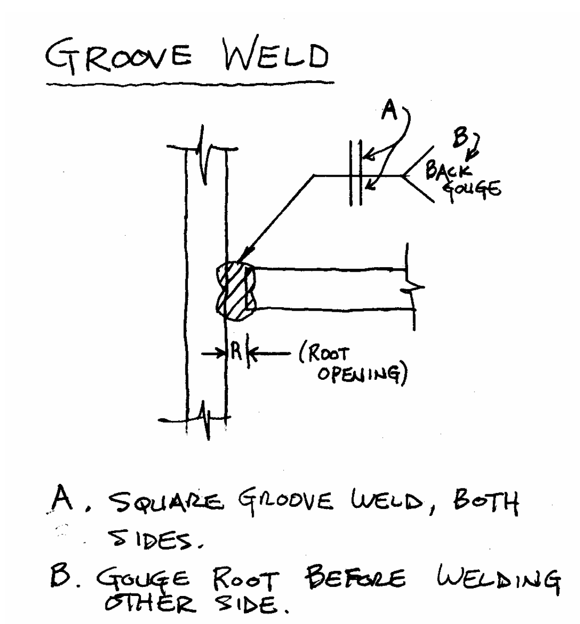


A = FIELD WELD SYMBOL
B = WELD ALL AROUND SYMBOL
C = WELD SIZE, INCHES
D = FILLET WELD SYMBOL
E = LENGTH OF WELD, INCHES
F = SPACING OF WELDS, CENTER TO CENTER, INCHES.

Fillet welds most commonly occur in the size range 3/16" to 5/16". The *minimum* allowable size of fillet welds depend on the thickness of metal being joined. For example, the minimum size fillet for any material 0.25" and smaller is 1/8". For material between 0.25" and 0.5" thick, minimum fillet size is 3/16". Fillet welds may be larger than 5/16", but if so, they must be made by multiple passes. After each pass, the slag (slag is a waste by-product of burnt flux that adheres to the weld) must be thoroughly chipped from the weld before the next pass is applied. This is inefficient and expensive. If a fillet weld is made along the edge of a member, it may not be larger than the thickness of the 'edge member'.

5.11.3.2. Groove Weld. This is a weld that is made in a gap or groove between the pieces of metal being joined. There are nine types of grooves:

- square
- single-V
- double-V
- single-bevel
- double-bevel
- single-U
- double-U
- single-J
- double-J



Groove welds are classified by the amount of penetration through the thickness of the members being joined as follows:

- L = limited base metal thickness, complete joint penetration
- U = unlimited thickness, complete joint penetration
- P = partial joint penetration.

Whenever you hear '*full penetration weld'*, a groove weld is being referred to in which the weld extends the full depth of the members being joined. Most of the high strength welded joints used in the construction industry are full penetration or partial penetration groove welds. These are stronger than fillet welds because there is much more penetration of the weld into the joint; and thus greater bonding of the weld to the members being joined is achieved.

There are many *pre-qualified* groove weld joints shown in the literature. *Pre-qualified* means that the weld is standardized; it has been designed and tested and is reliable in it's capacity so long as the listed procedure is followed. If non-pre-qualified welds are desired, they must be tested and approved prior to actual use (a procedure that is expensive, time consuming and very seldom done). Strictly speaking, even when pre-qualified welds are used, there must be a written procedure created by the welder operator available on-site which lists: amperage; wire feed speed; voltage; travel speed; and shielding gas flow rate.

5.11.3.3. Plug and Slot Welds. Plug welds are created by depositing weld material into a circular hole cut in one of two lapped members. Slot welds are similar, except that the hole is elongated, not circular. In both cases, the hole can be partially or completely filled. These are used to supplement fillet welds on lapped members when there is not enough fillet weld space. They are also used to prevent buckling of lapped parts.

5.11.4. Types of Welded Joints. There are five basic types of welded joints: butt, corner, tee, lap, and edge.

- ***Butt joints*** are made to members lying in approximately the same plane.
- ***Corner joints*** are made between members at approximately 90 degrees to each other.
- ***Tee joints*** are similar to corner joints, except that the intersection forms a tee instead of a corner.
- An ***edge joint*** joins the edges of two or more parallel (or nearly parallel) parts.
- A ***lap joint*** is formed at the intersection of two overlapping parts.

5.11.5. Primary and Secondary Welded Joints. If the welded joint is intended to transfer all of the load through the connected members it is termed *primary*. If only part of the load is intended to be transferred through the joint, it is called *secondary*.

5.11.6. Weldability. The weldability of any steel is controlled by it's carbon content. As the carbon content goes up, it's weldability goes down. A steel with a high carbon content combined with the intense heat of welding can cause brittle zones that are prone to brittle fracture under certain conditions. The upper limit of carbon content for good welding is about 0.25%. All of the commonly available structural steels (A36, A529, A242, A441, A572, A588, and A852), are weldable. Some of these have carbon contents at the upper limit of good weldability and some have other alloying metals added; both of which mean that certain proper welding techniques must be used.

5.11.7. Preheat and Interpass Temperature. When welding very thick steel (generally greater than ¾" thick), it is necessary to pre-heat the steel in the vicinity of the weld, and keep it at that temperature during welding. This is required for two reasons: first, thick material accelerates the rate of cooling of the weld, thereby making it brittle and prone to cracking; and second, the thick material does not expand sufficiently to allow the weld to contract as it cools – thus large residual tensile stresses are induced.

5.11.8. Welder Certification. Any welded structural steel connection should be performed only by a welder certified by a recognized regulatory agency. In Washington state, such certification is provided by WABO (Washington Association of Building Officials). Welders are certified for the various types of joints and positions as listed earlier.

> ***Question***: *"Most times engineers specify all welding by a WABO certified welder. Does this mean that any WABO certified welder can do all of the welding on the job?"* ***Answer***: *Not necessarily. There are various certifications, and not all welders have all of the available certifications. So if a welder is certified for flat and horizontal fillet welds, he may not perform vertical, overhead, or groove welds. Also, a welder must be certified for the type of welding process being used (shielded metal arc, gas shielded arc, etc.).*

5.11.9. Inspections. All structural welding should be inspected by a qualified inspector. The UBC requires this, unless the welding is done in a shop that has been previously approved by the building department. Further, the inspector must be *continuously present* during the welding operations, except in the following cases.

Exceptions to Continuous Welding Inspection:

- Inspector has pre-approved the materials, qualifications of welders, procedures, and
- Inspector makes periodic inspections of the welding operations, and
- Inspector visually inspects all welds prior to being placed in service, and
- Valid only for:
  - Single pass fillet welds
  - Floor and roof deck welding
  - Welded studs when used for structural diaphragm or composite systems
  - Welded sheet steel for cold-formed steel framing members such as studs and joists
  - Welding of stairs and railing systems

The author agrees it is always prudent to have any load bearing welded connection inspected. There are two reasons for this: first, structural failures almost always occur at connections (welded connections in particular) - so it is a good idea to have those connections inspected; and secondly, welding like any other trade has lots of room for 'corner cutting'. Inspection helps keep everyone honest. The welding code allows only certified inspectors or competent engineers (or technicians) to provide inspection. Competent is defined as "… *an engineer or technician who by training or experience or both in metals fabrication inspection and testing is competent to perform inspection of the work*." There is a lot more to inspection than just a visual look at the finished weld. At a minimum an inspector should look at and approve the following:

- Appropriate welding procedure
- Welding equipment
- Welder operator's certification
- Proper electrode use
- Correct size, location, and length of welds
- Joint preparation
- Quality of the weld

There are several methods for checking the quality of a weld. Many imperfections are internal to the weld and / or base metal and are not visible to the naked eye. However, visual inspection is always essential, and is particularly beneficial with the aid of a good 10x magnifying glass. There are other methods for non-destructively testing welds, such as: magnetic-particle inspection, penetrant inspection, radiographic (x-ray) inspection, and ultrasonic inspection. All of these involve specialized equipment, materials or both and should be performed by one trained and experienced in that specific method.

CHAPTER 6
CONCRETE AS A STRUCTURAL MATERIAL

6. Concrete. Concrete is a mixture of cement, aggregate and water. You will notice that cement is a constituent of concrete. Cement is the powder that binds the constituents together in the presence of water. People who do not understand this and refer to their ‘cement driveway’ or a ‘cement column’ are committing an egregious construction faux pas. There is no such thing as a cement slab, cement beam, or cement column. Remember, cement is a powder.

6.1. Portland Cement. Portland Cement. In 1824 a brick mason from Leeds England named Joe Aspdin took out a patent on a material he called portland cement. It was called ‘portland’ because the concrete made with it was supposed to resemble the limestone quarried near Portland, England. Though it has evolved over the years, today the vast majority of cement used is this same type. Portland cement is manufactured with quarried limestone, mixing in some clay or shale and then burning at about 2,700 degrees F. The burned product is called ‘clinker’. The clinker is then cooled and crushed, then a small amount of gypsum is added to regulate setting time. After additional processing, the finished product is an extremely fine powder called portland cement. You can buy portland cement at most hardware or concrete supply stores in 94 pound bags. Also available in similar bags is pre-mixed cement and aggregate which is ready to make concrete by just adding water.

6.2. Types of Portland Cement. The ASTM C150 designation provides for eight types of portland cement.

- **Type I, Normal**. This is the general purpose cement used in most residential and commercial jobs where other specialized properties are not required.
- **Type IA, Normal, Air-entraining**. Same as Type I, Normal, except that air-entrainment is added (air-entrainment is discussed in detail later in this chapter under admixtures).
- **Type II Moderate**. This cement provides a moderate amount of protection against sulfate environments. Alkaline soils contain sulfate salts which actively destroy the cement constituent in concrete. If the sulfate levels are relatively low, type II cement is recommended. If high concentrations of sulfate are present, a type V cement is recommended. Type II cements also have lower heat of hydration (see following section for more on heat of hydration) than Type I, and thus are recommended for massive concrete members, particularly in hot climates.
- **Type IIA, Moderate, Air-entraining**. Same as Type II, but with air-entraining added.
- **Type III, High Early Strength**. Type III cements are specified when it is necessary for the concrete to exhibit high strength properties within a couple or few days of placing; rather than a week or more after placing. Three specific examples of when to use Type III are: when concrete is to be placed in service quickly; when protection against low temperatures during curing is desired to be kept at a minimum; and when it is otherwise uneconomical to provide high

early strength via a richer mixture of Type I cement.

Question: *"I want to strip my forms soon after pouring, should I use a Type III?"* **Answer**: *Probably a good idea, but not necessarily. Type III indicates high early strength, not necessarily high early set. Check with your concrete supplier to see if their brand of Type III is also a high early set. Another option is to use a rich Type I mix. This will give you both high early set and higher early strength than a standard Type I mix. Another option could be to use an accelerator admixture with a Type I cement (see following section on admixtures). In any case, talk it over with your ready-mix supplier.*

- **Type IIIA, High Early Strength, Air-entraining**. Same as Type III, except that air-entrainment is added.
- **Type IV, Low Heat of Hydration**. This type of cement is used in applications where the heat of hydration must be kept at a minimum, such as for massive structures like dams. The time required to achieve design strength is correspondingly longer than with Type I.
- **Type V, Sulfate Resisting**. This type of cement is recommended for applications where soil alkalinity and sulfate salts are very high. It is also recommended for saltwater marine applications. The rate of strength gain is slower than with Type I.

6.3. Definitions: Concrete, Grout, Mortar. Concrete is created when cement, sand, gravel, and water are combined. Fresh concrete refers to the plastic – liquid state that occurs just after mixing, but before any set has occurred. Fresh concrete will conform to any shape that it's forms dictate. Hardened concrete is concrete that has undergone hydration and has become hard. Green concrete is hardened, but not cured; i.e. it has undergone some hydration and is firm to the touch, but it has not reached any appreciable strength.

- **Mortar**. Mortar is concrete with aggregate that is no larger than sand-sized (less than about 1/4" in diameter). Admixtures may be added as necessary.
- **Grout**. Grout is a mixture of cement and water, either with or without fine aggregate, containing enough water to allow pouring but without segregation of the constituents. Grout has a wetter consistency than mortar. *Neat cement* refers to grout without any aggregate. Admixtures may be added as necessary.

6.4. Hydration. This refers to the chemical reaction between water and cement to form a new compound (concrete, mortar, or grout). Hydration begins the instant that water comes into contact with cement. The reaction occurs rapidly at first, then continues (albeit slowly) for years. It is for this reason that concrete actually gets stronger over time, assuming that no external factors act to destroy or weaken it. The actual rate of hydration depends on the following:

- **Composition of cement**. The greater the percentage of cement in the mix, the faster the rate of hydration
- **Fineness of cement**. Coarser cement hydrates more slowly than fine
- **Temperature**. The hotter the temperature, the faster the hydration rate

- **Presence of admixtures**. Some retard and some accelerate hydration

Water must be present for concrete to cure or hydrate. No water – no hydration.

Question: *"I once saw a contractor who had just poured a slab, inundate the slab with water and keep it inundated for days. What in the heck was he doing? How could the concrete dry and set up if it is soaked with water?"* **Answer**: *Never speak of 'drying concrete'. Concrete gets hard and strong by curing or 'hydration', not by drying. Hydration requires the presence of water. Once concrete has set, if water is provided in infinite supply (as by inundation) you are guaranteeing that there will be enough water for the hydration process to commence; this is a good thing. If, on the other hand you pour a slab in the middle of the desert in summer, and do nothing to provide extra water for hydration, you are guaranteeing that your concrete will have lots of problems. It may well dry out, in which case the hydration process will stop, and the concrete will get no harder. Actually, if the dry concrete is re-wetted, it will again commence hydration, but at a reduced rate. See the following section on curing for more on this topic.*

6.5. Heat of Hydration. Hydration is an exothermic reaction, that is, heat is given off by the chemical reaction of the water and cement. In most residential and commercial construction, pours are relatively small, heat of hydration is not a problem, and may be ignored. In massive concrete projects, like dams and large bridges, the heat of hydration can cause severe problems, and must not be ignored. Generally, if the pour is so massive that the heat of hydration is trapped within the concrete mass, while the concrete outer layers cool, a severe temperature differential is created. This temperature difference causes tension stresses to develop in the face of the concrete. Cracks are the result. If your structure is a dam or concrete water tank, you don't want a lot of cracks. The following items are sometimes used to mitigate this problem in large pours:

- Use of Type IV, low heat cement
- Use of minimal amount of cement
- Use of pozzolanic admixtures
- Place the concrete strategically to minimize heat problems
- Place concrete during cold weather
- Add ice to the concrete mix
- Place pipes within the concrete to circulate cold water during curing
- Use steel forms that dissipate heat
- Remove forms as quickly as possible.

Hydration heat can sometimes be used to an advantage. In cold weather if insulated forms are used, the heat of hydration can be employed to keep the concrete at acceptably warm temperatures to ensure good hydration (hydration does not occur at very low temperatures, see later section on hot and cold weather concreting).

6.6. Aggregates. Aggregates are the 'filler' portion of concrete. They generally constitute approximately 60 – 70% of the volume of concrete. All aggregates should meet the specifications of ASTM C33; in other words, poor quality or dirty aggregates (like sand or gravel from unreliable sources) which would not meet the strict requirements of ASTM C33 should not be used, unless you don't care about the strength or durability of your concrete. Remember, your concrete will only be as good as your aggregates.

Good concrete depends on the cement paste coating completely every single particle of aggregate therein. All aggregates must be clean and free of loam, clay, or any organic materials. It is also desirable that the aggregates are well-graded; that is they vary in size rather than all being the same size. Well-graded is the opposite of uniformly graded (all particles approximately the same size). There are three basic types of aggregates: coarse, fine, and light-weight.

6.6.1. Coarse Aggregate. This is gravel, crushed stone or other suitable materials larger than 0.25" diameter. Suitable coarse aggregates are hard and durable. Flaky, soft, or easily worn materials should not be used. It is most efficient to use the largest size aggregate possible, however, the largest size aggregate should never be larger than 75% of the width of the narrowest space through which the fresh concrete must pass. For slabs, the largest aggregate should not exceed 1/3 the slab thickness. For walls, the largest aggregate should not exceed 1/5 the wall thickness. In some cases where reinforcement is spaced very closely, pea gravel is specified as the coarse aggregate. Common maximum coarse aggregate sizes are: 3/8", 3/4", 1", and 1-1/2".

6.6.2. Fine Aggregates. This constituent of concrete is normally sand, with a maximum particle size of 1/4". The best fine aggregates are well-graded, being of various sizes from very fine sand up to larger sand particles. It is important that fine aggregates are clean and completely granular, and do not contain extremely small particles such as clays or silts.

6.6.3. Light-Weight Aggregate. This type of aggregate is used to make light-weight concrete (it is primarily the aggregate and not special cement or admixtures that makes light-weight concrete weight less than standard concrete). Light-weight aggregates are those which weigh less than standard sand and gravel aggregates. Crushed volcanic rock is sometimes used for this purpose; contact your concrete supplier for more information as to the type of light-weight aggregates used in your area.

A word of caution about light-weight concrete; its allowable compressive strength is often less than for standard concrete, so be sure to factor this into your design.

6.6.4. Water. All water used in the making of concrete should be clean and free of alkalis, acids, organic compounds, silts, or clays. In general, any water fit to drink is suitable for concrete.

Question: *"I like to use lots of water in my concrete, that way it's so much easier and faster to work with. Anything wrong with this?"*
Answer: *Yes, plenty. The strength of concrete is directly related to the amount of water used. <u>The more water, the lower the strength</u>. Also, the more water, the more shrinkage (and cracking) will occur. There is always a tendency to add water at the construction site to get a more flowable mix, particularly if the concrete has been in the truck too long. Bad, bad idea. In doing so, you weaken the concrete considerably. Just a small increase in the water-cement ratio (see next section), say from 0.35 to 0.53 will cut the strength approximately in half.*

6.7. Water-Cement Ratio. This is a measure of how much water is added to the concrete mix per the amount of cement used. One 94 pound sack of cement requires only 3-1/2 gallons of water for complete hydration. This translates to a water-cement ratio of 0.31. Water-cement ratio is probably the most important variable in the determination of the concrete's strength. The more water used, the weaker the concrete. A good rule of thumb is to use the least amount of water necessary to provide a workable mix. This usually results in a water-cement ratio of between 0.45 to 0.55. Admixtures can be used to enhance the workability of concrete without increasing the water-cement ratio, see the following sections.

6.8. Slump. Slump is an indirect measure of the amount of water in the mix. It is determined by filling a standardized metal 'slump cone' with fresh concrete, rodding (compacting) it a specified amount, removing the cone, and measuring how much the concrete 'slumps' downward. In general, the higher the slump, the higher the water-cement ratio. A slump of 3" - 4" is normally specified for standard concrete mixes with Type I cement. If you check the slump from a new batch with no water being added at the jobsite, and the slump is too high, you should consider rejecting the load because the strength will be reduced. Remember, you can always add water to your mix, but you can never take it away.

6.9. Admixtures. These are any compounds added to the concrete before or during mixing other than water, cement, and aggregate. Their purpose is to alter some property of the concrete such as: improve workability, reduce aggregate separation; add entrained air; accelerate the hydration process; retard the hydration process; or make the concrete more waterproof. Admixtures should be used with caution for the following reasons:

- In many instances, simply adjusting the ratios of cement, aggregate, or water can more economically and predictably perform the desired function of an admixture.
- Admixtures do not always react predictably with all cements, even those of the same type.
- Many admixtures affect more than just one property of the concrete; sometimes to a deleterious result.
- Admixtures can behave unpredictably due to such variables as wetness and richness of mix, aggregate gradation, type of mixing, and length of mixing.

Those cautions being said, there are several basic types of admixtures commonly used. They are discussed below.

6.9.1. Air-entrainment. Of all admixtures, this is probably the most prevalent. It has many well documented benefits, and is required by many engineers in all concrete. It consists of a foaming agent that is added either directly to the cement during the manufacturing process (resulting in Types IA, IIA, and IIIA cements), or as an admixture during batching. Regardless of which method of addition is used, the result is the same: innumerable microscopic voids are formed in the cement paste. Several benefits of air-entrainment follow:

- Improved workability without added water
- Improved durability
- Protection against freezing and frost action

- Improves resistance to salts, such as those used for snow and ice removal
- Improved water-tightness
- Reduces 'bleeding' (excess water which leaches out during curing)

The amount of air-entrainment is expressed as a percentage of volume of concrete. In general, the smaller the aggregate, the more air-entrainment may be used. For 3/8" maximum aggregate, the recommended amount of air-entrainment is between 6 – 10%. For 1-1/2" maximum aggregate, the recommended amount of air-entrainment is 3 – 6%.

6.9.2. Accelerators. These either accelerate the set time of freshly poured concrete, or impart high early strength to the concrete, or both. The purposes generally include:

- Reduces pressure on forms, particularly in cold weather. This is useful in cases where forms are tall (such as walls), which if filled would result in the fresh concrete exerting huge outward pressures on the forms, potentially causing bursting
- Permits early removal of forms
- Reduces the amount of time for curing and protection of the green concrete, permitting early finishing
- Compensates for the effect of low temperature on strength development
- Permits placing the structure in service sooner than would otherwise be allowed
- Greatly improves the concrete's resistance to abrasion and erosion at early stages of curing

The principle accelerator ingredient most commonly used is calcium chloride. The maximum amount of calcium chloride that should be used is 2% by weight of the cement.

Concrete with calcium chloride added is approximately twice as strong as without at 3 days, and approximately 70% stronger at 7 days. The difference is still evident at 28 days, but tapers off after that. Setting time is typically reduced by about a third to a half. Early hydration heat is increased, but calcium chloride must never be considered an antifreeze. There are several warnings that go with this admixture; they follow:

- Increased drying shrinkage
- Increased corrosion potential of regular reinforcement steel, and must not be used at all in prestressed concrete
- Increased corrosion potential if there is embedded aluminum (such as conduits), particularly if the aluminum is in the presence of steel
- It may defeat air-entrainment unless added to the batch separately
- Reduces concrete's resistance to sulfate attack, so don't use it in concrete exposed to alkaline soils
- Should not be used in hot environments

6.9.3. Water Reducers. This type of admixture is used to reduce the amount of water in the mix, while maintaining good workability. Recall that the less water used results in greater strength concrete (assuming the amount of cement does not change). Water reducers should **not** be used to lower the amount of cement used (while keeping a constant water-cement ratio). Water reducers are commonly used for very high strength concrete, because they allow the water-cement ratio to stay low, thereby helping to ensure high strength.

Chemicals commonly used for water reduction are the lignosulfonates (calcium, sodium or ammonium) and salts of hydroxy-carboxylic acids. It should be noted that these chemicals may also act as retarders (see below). A water reducer should be used only after field tests have demonstrated it's effect on the concrete, particularly if used in conjunction with air-entrainment or other admixtures.

Mid-range water reducers are similar to normal water reducers, except that they can reduce water requirements slightly more (approximately 8 – 12% reduction in added water) than conventional water reducers. More importantly, mid-range water reducers also reduce the 'stickiness' of mixes with high cement content, which makes finishing easier.

Superplasticizers, plasticizers, or high-range water reducers are a relatively new admixture (mid – 1970's) in the water reducer line. They can reduce added water by up to 30%. These are commonly used as follows:

- As a water reducer to give concrete a very low water-cement ratio, but normal workability and high strength
- To provide normal workability and strength at a reduced cement content but normal water-cement ratio.
- As a plasticizer to produce very workable concrete; that is, a flowing, self-leveling concrete with a high slump and high compressive and flexural strength. Consolidation (vibration) effort becomes less critical. More fines (such as fly-ash) are typically added for this type of application.
- In cases where closely spaced rebar is used, and / or in cases where pumping of concrete is required. The increased flowability helps ensure that concrete fills all the voids and tight places in and around the rebar.

Superplasticizers have minimal effects on the other properties of cement, but as always, should be field tested prior to service use.

6.9.4. Retarders. This type of admixture acts to slow the hydration process, thus increasing the time that the concrete remains plastic and workable. They are the opposite of accelerating admixtures. Once the concrete starts to set, however, strength gain is at approximately the normal rate.

Retarders are used primarily when placing concrete in hot weather, especially if the pours are in small increments. Recall that hydration occurs faster with an increase in ambient temperature. Retarders are not generally needed for small to normal sized pours as long as concrete placement occurs at temperatures less than 75 degrees F. Retarders are commonly used for mass concrete pours to help keep the heat of hydration low.

Retarding agents include inorganic compounds such as borax, boric acid, calcium borate, sodium bicarbonate and certain phosphates. Extreme care must be used with this class of retarders because they can have erratic and unreliable results. Some of the same admixtures used for water reduction (metallic salts of lignosulfonic acid or salts of organic hydroxy-carboxylic acid) are also used as retarders. As with other admixtures, the effects of retarders depends on other mix properties of the concrete, as well as temperature. For these reasons, field tests should be made to determine the actual results of the admixture.

6.9.5. Dampproofers, Waterproofers, Permeability Reducing Agents. This class of admixture is intended to reduce water permeability through hardened concrete. There are a myriad of proprietary products available for this purpose. Some are excellent and some are nearly useless - some will actually make the concrete less water resistant if not used exactly per the manufacturer's recommendations; so beware.

Plain concrete has been used successfully for many years in the construction of water storage tanks and vessels. In these cases, it is important to use a rich mixture, and ensure that there are enough fines in the fine aggregate. Leaks are almost always due to poor workmanship, or bad mix design.

It is interesting that there are many products available that are excellent in sealing hardened concrete against water intrusion. Some of these are silica based compounds that are applied in an aqueous solution which penetrate the pores then harden thus forming an impenetrable barrier. Some products can actually permanently plug flowing leaks through concrete from the inside.

6.9.6. Pozzolans. This is an admixture that mimics (to some degree) the cement portion of concrete, and thus is commonly used to reduce the amount of cement added. A common definition of a pozzolan is as follows: "A siliceous or siliceous and aluminous material, which in itself possesses little or no cementitous value, but will in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties." The word 'pozzolan' comes from the town of Pozzuoli, Italy, situated near the source of volcanic ash (a pozzolan) used by the Romans in the construction of many of their structures.

Pozzolans are either naturally occurring: volcanic tuff, volcanic ash, pumicite and obsidian; calcined or burnt clays or shales, or man-made: crushed and ground blast furnace slag, fly ash or silica fume.

Pozzolans react with the hydrated lime or calcium hydroxide (byproducts of hydration) in hydrated concrete. This reaction increases the tensile and compressive strength, lowers the permeability, reduces leaching and improves the sulfate resistance of concrete (all good things). Summarized below are the general effects of pozzolans.

- Can be used to reduce the amount of cement required for equivalent strength
- Reduced heat of hydration due to less cement used
- Increased workability
- Reduced segregation of aggregates and bleeding
- Durability of concrete varies depending on the pozzolan used. Check with your supplier
- Strength is usually improved with lean mixes
- Drying shrinkage – little effect

- Improved sulfate resistance, particularly those with high silica contents. Not a substitute for Type V cement, however
- Slightly reduced resistance to scaling from de-icers

The amount of pozzolan used varies commonly from 10 to 30 percent replacement of the cement. No pozzolan should be used without complete understanding of it's character or without trial testing.

6.10. Shrinkage. As concrete cures, it shrinks. The amount of shrinkage depends mostly on the amount of water used in the mix; the more water used, the greater the shrinkage. In dry climates, most of the shrinkage will occur within the first 2 – 3 months. In humid, moist climates, most of the shrinkage will occur within the first year. Shrinkage does not generally depend on the stress level of the concrete, but rather on moisture conditions and time.

Shrinkage causes internal stresses within concrete; however, code compliant reinforced concrete will have enough rebar in it to adequately resist these stresses.

Slabs on grade will crack due to shrinkage (note use of the word 'will', not 'may'). Such cracking can be controlled, however, through the correct placement of contraction and isolation joints (see the following section on slabs on grade for more on these joints).

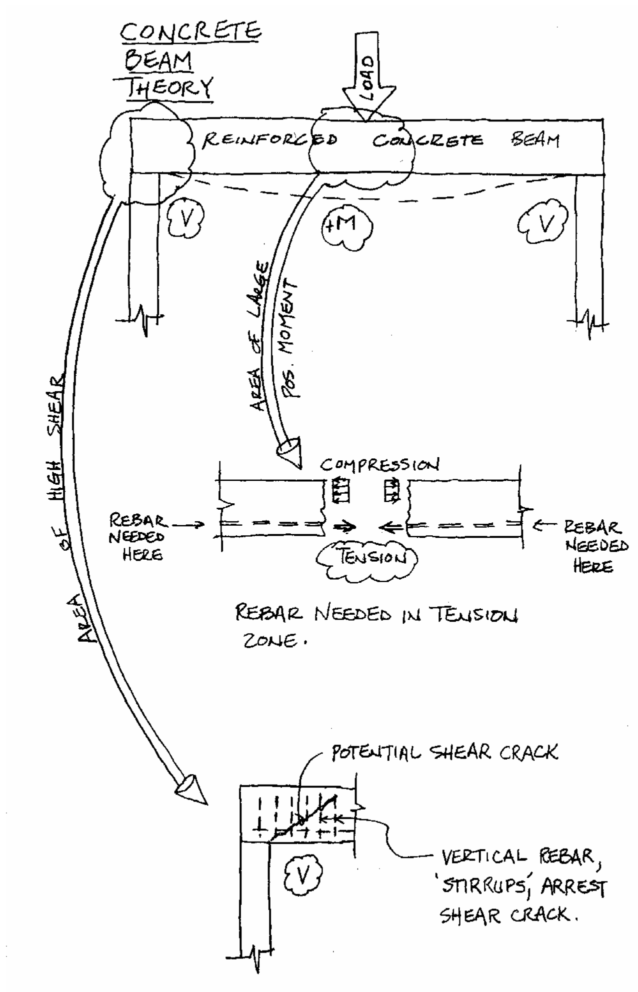
6.11. Creep. Creep is very minute compression movement (strain) that occurs in permanently loaded concrete. Unlike shrinkage, creep <u>does</u> depend on the amount of permanent stress present in concrete. Interestingly, properly designed standard reinforced concrete structures are not normally affected by creep. However, any pre-stressed or post-tensioned concrete structures likely will be. Engineers take this into consideration during the design process of those types of structures.

6.12. Temperature Expansion and Contraction. Concrete will expand and contract with the raising and falling of ambient temperature. In small or short concrete members (say 20 feet long or less) there are generally no special provisions for this because the amount of movement and internal stresses are relatively low. This also tends to be true in the case of retaining walls or other structures that do not experience wide fluctuations in temperature.

In the case of long elements, slabs, and walls (masonry walls as well) that are exposed to the weather, provisions must be made to accommodate temperature induced strains; or undue cracking or buckling may occur. Engineers typically call for expansion or isolation joints at strategic locations to accommodate the movement and relieve the stresses associated with temperature fluctuations. For example, I require expansion joints in any free standing masonry or concrete wall at no more than 25 foot intervals.

6.13. Stress in Concrete. Concrete was invented to resist compressive (squeezing) forces only. To this day, it is still used primarily for compression. It will not take much in the way of tension or tensile forces. Consider a 6"x12" square concrete column. If the ends were well fixed, and it was short, you could place approximately 90 tons on it without failure. If you took that same column and pulled on the ends, it would pull apart, failing in tension at a measly 3 tons or so. Concrete is so weak in tension, in fact, that it's tensile strength is not normally even considered in design.

Now, how about a 6x12 concrete beam. Recall that when simple beams are loaded, there is tension at the bottom of the beam, and compression at the top. If concrete can take no tension, how in the world could a concrete beam ever work? The answer is that, by itself, a concrete beam could not. If however, you place reinforcing steel (rebar) in the bottom of the beam, then pour the concrete around it so that the rebar is embedded in concrete – now you have something in the bottom of the beam that can take tension (the rebar). This is the basic concept of how concrete, more correctly - reinforced concrete, works. You strategically place steel reinforcement in the areas that are subjected to tension, and let the concrete take compressive forces only.



GARRISON'S FIRST LAW OF CONCRETE: <u>Concrete is a compression material only. Never place concrete in tension unless steel reinforcement is present in the right places and amounts to take the tensile (tension) stresses</u>.

6.14. Concrete Bending Members. All concrete bending members must have adequate steel reinforcement embedded in them in the tension zones. If they do not, they will break and fail. Examples include:

6.14.1. Simple Beams. These must have steel reinforcement in the bottom portion of the beam.

6.14.2. Continuous Beams. These must have steel reinforcement in the bottom portion of the beam between the supports; and steel reinforcement in the top of the beam over the supports. Remember your negative moments!

There are numerous rules in the UBC about how far tension bars must extend beyond the tension zones (see following section on development length), so don't try to outguess your engineer in this area. Conversely, if you think that there is insufficient steel in a tension area, absolutely bring it to the engineer's attention.

6.14.3. Cantilever Retaining Walls. These are retaining walls that are not connected to anything at the top of the wall; i.e. they 'cantilever' upward from the base of the wall. In these walls, the tension in the concrete occurs on the soil side of the wall, and thus that is the portion of the wall where there must be vertical steel reinforcement (see the next chapter on foundations and retaining walls for a thorough discussion of this concept).

6.14.4. Braced Retaining Walls. These are retaining walls that are positively connected to a floor diaphragm or other lateral load resisting element at the top of the wall. Tension in the wall is on the non-soil side of the wall, and thus that is where the vertical reinforcement must go (see the next chapter on foundations and retaining walls for a thorough discussion of this concept).

6.15. Steel Reinforcement, 'Rebar'. There are two basic types of rebar: plain and deformed. Most rebar you see on a construction site is of the deformed variety; and is the default type if nothing is specified. This type is easily recognizable by the heavy ribs or deformations in the surface; whose purpose it is to bond positively with the cement paste in the fresh concrete. Plain bars have no deformations and are used in special circumstances like dowels in expansion joints, where they transfer shear stress, but not tension.

6.15.1. Strength Grade. Rebar is typically specified by its minimum allowable yield stress. The two most common grades used are grade 40 and grade 60, which corresponds to minimum yield stresses of 40,000 and 60,000 psi. respectively. Rebar is also available in grades 50 and 75.

6.15.2. Types of Steel. Rebar is manufactured from 4 different types of steel: billet steel – ASTM A615; rail steel – ASTM A616; axle steel – ASTM A617; and low alloy 'weldable' steel – ASTM A706. Some of the more common sizes and grades available are shown as follows:

- Billet Steel - A615
  - Sizes 3-6, Grade 40
  - Sizes 3-18, Grade 60
  - Sizes 6-18, Grade 75
- Rail Steel - A616. Sizes 3-11, Grades 40 and 60
- Axle Steel - A617.
  - Sizes 3-6, Grade 40
  - Sizes 3-11, Grade 60
- Weldable Steel, Low Alloy - A706. Sizes 3-18, Grade 60.

The UBC 1921.2.5.1, requires that A706 rebar be used in seismic resisting frames and wall boundary elements (billet steel may also be used, under certain circumstances).

6.15.3. Size. Size of rebar is expressed in eighths of an inch. For example #4 rebar is 4 - eighths of an inch in diameter which equals 1/2" diameter. A #8 rebar is 8 - eighths of an inch in diameter equaling 1".

6.16. Welded Wire Fabric. This is a heavy wire mesh typically used in slabs as temperature reinforcement. The wire is cold drawn and may be smooth or deformed. The wires are 'resistance welded' at their joints. The size of the mesh openings and the wire size may vary in either direction. Fabric is available in rolls for the lighter gages only, and must be ordered in flat sheets for the thicker gages (check with your steel supplier for specifics).

Welded wire fabric is available in smooth wire and deformed wire varieties. It is called out by 'WWF' followed by the spacing in inches of the longitudinal wire, the spacing of the transverse wire in inches, the cross sectional area of the longitudinal wire, and the cross sectional area of the transverse wire. Cross sectional area is expressed in hundredths of a square inch. If the wire is deformed, there is a 'D' in front of the wire size; if the wire is plain, there is a W. Some common sizes follow:

- Smooth wire <u>rolls</u> with 6x6 or 4x4 grid: W1.4xW1.4 (10 Ga.), W2.0xW2.0 (8 Ga.), W2.9xW2.9 (6 Ga.), W4.0xW4.0 (4 Ga.). If these would have been with deformed wire, the 'W' would have been replaced by a 'D'.
- Smooth wire <u>sheets</u> with 6x6 grid: W2.9xW2.9 (6 Ga.), W4.0xW4.0 (4 Ga.), or W5.5xW5.5 (2 Ga.). Sheets with 4x4 grid: W4.0xW4.0 (4 Ga.). If these would have been with deformed wire, the 'W' would have been replaced by a 'D'

For example, a common rolled fabric is 6x6-W1.4xW1.4. This is a mesh that is 6" x 6" (square meshes) and smooth wires (both longitudinal and transverse) that are .014 square inches (10 gage) in area. A common heavy deformed wire sheet fabric is 4x4-D4.0xD4.0. This indicates a 4" x 4" square mesh, deformed wires (both longitudinal and transverse) that are .04 square inches (4 gage) in cross sectional area.

WWF Splices. There are numerous rules for splice length of welded wire fabric. If the fabric is used in a structural slab (spanning some distance, i.e. not a slab on grade) to resist tension (as opposed to temperature reinforcement in a slab on grade) overlap should be calculated, based on certain formulas specified in the code. Typically this type of lap is at least two meshes. For non-structural mesh applications, it is advised to provide at least one mesh plus 2 inches overlap at all splices. Less splice overlap may be used in certain cases, particularly with deformed fabric, but such should be called out by the engineer.

6.17. Minimum Steel Reinforcement / 'Temperature Steel'. Now you know the main purpose of steel reinforcement / rebar in concrete – to resist tension forces. Rebar has other purposes as well, however. As concrete changes temperature and moisture content (during hydration, and after hardening due to normal daily temperature and humidity fluctuations) it shrinks or expands. As it does so, tensile stresses are induced. These stresses are everywhere in the concrete, not just in the externally loaded 'tension zones'. There must be reinforcement present to resist these 'incidental' tension stresses. This is the main reason that all structural concrete must, by code, have a certain minimum amount of reinforcement present.

The minimum amount of steel reinforcement required is expressed as a ratio of reinforcement area to concrete area.

For example, for walls, the minimum amount of vertical steel is .0012 for deformed rebar not larger than #5. For an 8 " thick wall, this translates to one #4 vertical every 20" (Note that there are other rules for minimum reinforcement for walls which would require the minimum spacing of any rebar be not greater than 18". See the chapter on retaining walls for more.). Remember, this ratio is a minimum; calculations based on loads and stress could very well require more rebar than this.

There are many rules for minimum reinforcement, which differ depending on the type of element and stress being resisted. At the least everyone should be aware that there are different rules for minimum reinforcement for nearly all types of concrete elements. This is true regardless of how much or little is required by stress calculations. Following are some elements requiring minimum reinforcement:

- Structural slabs (but not necessarily slabs on grade) require minimum reinforcement in both directions.
- Beams and bending members require a minimum amount of tension reinforcement. Shear reinforcement is also required in certain situations. If the beam is to resist earthquake forces, reinforcement is required both in the bottom and top of the beam.
- Columns require a certain minimum amount of vertical reinforcement. Hoops, stirrups, or spiral ties are also required. If the column is designed to resist earthquake forces, the reinforcement requirements are much more stringent than if the column only resists gravity loads.
- Walls. There are minimum requirements for both vertical and horizontal reinforcement. And, if the wall is greater than 10" thick, a minimum of two mats of reinforcement are required.

In actuality, there are minimum reinforcement requirements for all types of concrete in the western United States except for certain residential foundations and certain slabs on grade.

Question: *"I always put lots of rebar in my slabs, but they crack anyway. What's going on here?"* **Answer**: *First of all, good for you for putting rebar in your slabs – that is an excellent practice that will be discussed later in the section on slabs. In answer to your question, all concrete, (slabs, beams, walls, columns, etc.) is going to crack a small amount (perhaps microscopically) no matter what you do. This is because as concrete hydrates, it shrinks, and shrinking leads to cracking. The trick is to control the cracking, and /or make it crack where you want it to. Temperature steel as described in this section will not stop cracking. It will, however distribute the tensile stresses brought on by shrinking throughout the concrete to ensure that the cracking is kept at a minimum.*

6.18. Concrete Structures Which Resist Earthquake Forces. Concrete by itself is not very ductile, that is, it tends to behave in a brittle manner. In an earthquake, shaking motion induces cyclic tension and compression stresses in structures that require ductile behavior for the structure to survive. The only way concrete gets ductility is through the use of steel reinforcement. Furthermore, ductility in concrete depends on the reinforcement being placed in the correct locations, spliced correctly, and being tied together with hooks at the ends of hoops, ties and stirrups. Ample experience has shown that special attention to reinforcement detailing and provision is necessary to avoid catastrophic failure of concrete structures during strong ground motion.

The UBC has a separate subsection within the concrete section dedicated to structures which utilize concrete elements (beams, columns, shear walls, slabs, and / or trusses) to resist earthquake forces. This section primarily places additional requirements on the provision and detailing of the steel reinforcement used. For example, transverse reinforcement (hoops, stirrups, and ties) requirements are much more stringent than for simple gravity load resisting concrete elements. There are also more stringent requirements for rebar splicing. Longitudinal reinforcement is required in both the tension and compression zones of all elements. All of these requirements are there to ensure that during strong shaking, the concrete behaves in a ductile manner, and does not fail catastrophically.

6.19. Development of Reinforcement. In order for steel reinforcement to do its job, it must not be pulled or pushed through the concrete when subjected to service or earthquake loads. This is called development of the rebar, and there are several ways that it is accomplished:

- Embedment length
- Hooks, or
- Mechanical devices

Development of rebar is not the same as splicing of rebar. Both will be covered in the following subsections.

6.19.1. Development of Deformed Bars in Tension. Rebar that is subjected to tension must have enough bar length extended past the tension zone to prevent it from being pulled out of the concrete. This is called development length. Development length is usually defined in terms of bar diameter. For example, a development length of 40 bar diameters is commonly referred to; which for #4 bar equals 20 inches. Bear in mind, development length is not necessarily the same as lap splice length. If certain rules are not met, lap splice length will be 1.3 times development length (see following section on lap splices).

The actual method used to calculate development length is complicated, and depends on: the location of the rebar, the coating used on the rebar - if any, the rebar size, the rebar yield strength, and the concrete compressive strength.

If rebar is being used in an earthquake resisting element, longer development lengths are normally required than for non-earthquake resisting elements. For example, the development length of a horizontally oriented #4 or #5 grade 40 bar, with 2,500 psi concrete, non-earthquake application, non-epoxy coated, with less than 12” of concrete placed below it, with adequate cover is 32 bar diameters (UBC 1912.2.2). If this member were to resist seismic loads, and all of the rules were followed correctly, the development length would be about the same. If, however, any of the rules (cover, spacing, etc.) could not be followed, the development length for the seismic case could go up to about 45 bar diameters.

6.19.2. Development of Deformed Bars in Compression. There are separate rules for development length of compression bars, which usually results in a shorter development length than for tension. Beware, however of compression bars that may go to tension during an earthquake.

6.20. Standard Hook. A standard hook (UBC 1907.1) is either of the following:

- 180 degree bend plus 4 x bar diameter extension, but not less than 2.5” at free bar end
- 90 degree bend plus 12 x bar diameter extension
- For stirrups and tie hooks, #5 bar and smaller = 90 degree bend + 6 bar diameter extension .
- For stirrups and tie hooks, #6 - #8 bar, see first bullet.

- For stirrups and tie hooks, #8 bar and smaller = 135 degree bend plus 6 bar diameters extension.
- For stirrups and tie hooks in seismic zones 3 and 4 (Western Washington) you must conform to the seismic hoop provisions of UBC 1921.1 (see next section).

6.21. Seismic Hooks, Stirrups, and Ties. (UBC section 1921.1) For stirrups and tie hooks in seismic zones 3 and 4 (Western Washington), the following apply:

- **Seismic Hook** is a hook on a stirrup, hoop or crosstie having a bend not less than 135 degrees with a six bar diameter extension (but not less than 3 inches) that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop.
- **A Hoop** is a closed tie or continuously wound tie. A closed tie can be made up of several reinforcing elements, each having seismic hooks at both ends. A continuously wound tie must have a seismic hook at both ends.
- **A Crosstie** is a continuous reinforcing bar having a seismic hook at one end and a hook of not less than 90 degrees with at least six diameter extension at the other end. The hooks must engage peripheral longitudinal bars. The 90 degree hooks on two successive crossties engaging the same longitudinal bar must be alternated end for end.

6.22. Tension Bars Terminating in Standard Hook. UBC 1912.5. Tension bars may terminate in a standard hook in cases where insufficient development length exists otherwise. This development length is the length of straight bar <u>before</u> the standard hook plus the radius of the hook plus one bar diameter. Technically speaking this is the straight portion of bar between the hook and the 'critical section'. But what is 'critical section'? The UBC definition in 1921.1 defines it as… "the location where the strength of the bar is to be developed". My interpretation of this is the location where normal, unhooked development length would start. The point is, where tension bars terminate in a hook, adequate rebar length must be provided on each side of the bend.

Calculation of this development length depends on the grade of rebar, concrete strength, concrete cover, enclosure within ties or stirrups, coating, and whether or not lightweight aggregate is used. For example, for a grade 40 bar, size #11 or smaller, non-seismic application, with adequate cover, non-epoxy rebar, non-lightweight aggregate the minimum development length of a tension bar terminating in a hook is 12 x bar diameter. For a seismic resisting element, conforming to all the 'rules' the minimum development length would be 13 x bar diameter. In no case should the development length be less than 8 bar diameters or 6".

6.23. Splices. There are three acceptable ways to splice rebar.

6.23.1. Lap Splice. This method consists of placing the ends of the rebar along side of each other, and overlapping them a certain amount. The overlapped portion should be tied together with tie wire to ensure that the pieces do not slip away or apart during pouring.

The UBC <u>does</u> allow non-contact lap splices. Bars so spliced must not be farther apart than

1/5 the length of the splice, and cannot exceed 6" apart.

One of the most important consideration in a lap splice is that the concrete, particularly the cement paste, is well adhered to the bars being spliced. Consolidation (vibration) and correctly sized aggregate will help ensure that this occurs correctly.

There are two classes of tension lap splices:

- **Class A Tension Lap Splice**. This is a lap splice with a splice length equal to the development length of the rebar being spliced. Class A splices are allowed only when: (1) The area of reinforcement provided is at least twice that required by analysis over the entire length of the splice; and (2) One half or less of the total reinforcement is spliced within the required lap length. What this is saying is that Class A splices are okay where the amount of rebar being provided is per the engineer's calculations, and that the lap splices must be staggered.
- **Class B Tension Lap Splice**. This is a lap splice with splice length equal to 1.3 times the development length of the rebar being spliced. Class B splices are to be used in all cases where Class A splices do not qualify. To be safe, you can always use a Class B splice where lap splices are allowed.

There are many restrictions on lap splices. Some of the more important follow:

- Lap splices of bars in a bundle may not overlap, and the splice length is based on the individual bar size, not the bundle size.
- Lap splice in 'tension tie members' may not be lap spliced, but must be mechanically spliced or welded; and such splices must be staggered at least 30 inches.

In earthquake resisting beams:

- Lap splices are permitted only if bound by hoop or spiral reinforcement over the lap length.
- Lap splices are not permitted in or near joints of beam to column.
- Lap splices are not permitted in areas of flexural yielding caused by inelastic lateral displacements.

In earthquake resisting columns:

- Lap splices are only allowed in the center half of the column, and must be proportioned as tension splices (not compression splices).

6.23.2. Welded Splices. These are allowed anywhere in non-earthquake resisting elements. They must meet the requirements of UBC Standard 19-1, and must develop at least 125% of the specified yield strength, fy, of the rebar. In earthquake resisting elements, welded splices are not allowed in the vicinity of an anticipated plastic hinge region (plastic hinges are predicted by engineering analysis).

6.23.3. Mechanical Splices. There are a number of proprietary mechanical rebar splices available on the market. There are two types:

- **Type 1 Mechanical Splice**. This mechanical splice will develop 125% of the specified yield strength, fy, of the rebar in tension or compression. Recall that fy is the minimum allowable yield strength, and not the ultimate strength which is about 30% higher. Type 1 mechanical splices are allowed to be used anywhere, except within or near plastic hinge regions in earthquake resisting elements.

- **Type 2 Mechanical Splice**. This type of splice must develop at least 160% of the specified minimum yield strength of the rebar, fy, or it must develop at least 95% of the ultimate tensile strength of the rebar. This type of splice is allowed anywhere in any member.

6.24. Shear Reinforcement. Concrete has relatively good shear strength without any reinforcement at all. Shear reinforcement refers to rebar that is placed perpendicular to the main tension or compression rebar. Hoops, stirrups, and ties are common examples of shear reinforcement. The main purpose of shear reinforcement is to keep diagonal cracks from occurring in areas of high shear, such as near the ends of beams.

In non-earthquake resisting structures, if the concrete cross section is large enough, it is possible that no shear reinforcement would be required; though it is always a good idea to include at least a minimal amount.

Earthquake resisting elements must have shear rebar, and in large quantities.

6.25. Minimum Bend Diameters. Rebar must be bent cold, and may not have any tighter (smaller) bend diameter than the following to avoid kinking. Bend diameters are as measured across the inside of the bend:

- For #3 through #8 = 6 x bar diameter
- For # 9 through #11 = 8 x bar diameter
- For #5 and smaller stirrups = 4 x bar diameter.

6.26. Surface Condition of Reinforcement. Rebar must be free of grease, oil, mud and other non-metallic coatings (except for epoxy coating). Rust and hard mill scale is okay for rebar (not prestressing tendons) so long as it hasn't decreased the bar diameter.

6.27. Spacing of Rebar. Minimum clear spacing between parallel bars in a layer equals the bar diameter, but not less than 1". This applies to splices as well. For spirally reinforced or tied reinforced compression members, this spacing is 1.5 x the bar diameter, but not less than 1.5". Where parallel reinforcement is in more than one layer, the upper bars must be placed directly over the lower bars.

In walls and structural slabs (other than concrete joist construction), primary flexural reinforcement must be spaced no farther apart than 3 x wall or slab thickness or 18".

6.28. Bundled Bars. The maximum number of bars bundled together, in contact with each other is four bars in one bundle. Bundled bars must be enclosed in stirrups or ties, and all splices must be staggered at least 40 x individual bar diameter.

6.29. Cover Over (Protection of) Rebar. For field poured, non-prestressed, rebar size #11 and smaller there must be at least the following amount of cover provided for all rebar:

- Concrete cast against and permanently exposed to earth = 3". For example: the bottoms of footings.
- Concrete exposed to earth or weather; #6 and larger = 2". For #5 and smaller = 1-1/2". For example: formed retaining walls;

formed sides of footings; or exterior beams and columns.

- Concrete not exposed to weather or in contact with ground: slabs, walls, joists = 3/4”; beams and columns = 1-1/2”. This covers all formed interior applications.
- Concrete tilt-up panels cast against a rigid horizontal surface, such as a slab, exposed to weather: #8 and smaller = 1”; for #9 and larger = 2”.

6.30. Placing Concrete. The correct terminology for depositing wet concrete into it’s final location is ‘placing’ not ‘pouring’. However, the use of the term ‘pouring’ is a faux pas that has become so widely used that it is somewhat acceptable to even the most particular of concrete purists. There are many tricks of the trade in placing concrete that are beyond the scope of this book, however, there are a few items that are important to the structural integrity of the finished concrete that will be discussed herein.

6.31. Segregation. Because concrete is made up of different materials with different weights and sizes, it tends to segregate or separate under certain conditions. There are several rules of thumb that must be followed to minimize this.

- Place all concrete by dropping vertically, regardless of the type of equipment it is coming out of or going into. This is accomplished by using vertical chutes or ‘elephant trunks’ at the end of the delivery chute.
- Do not allow falling concrete to hit rebar or other obstructions in the forms. This causes segregation. It can be minimized by: using an elephant trunk and limiting the vertical drop distance.
- Minimize the vertical drop distance to a few feet, say 4 or 5 feet at the most; and less when the forms have rebar obstructions present.
- Minimize the thickness of pouring to 18” lifts at the maximum. When pouring a wall, for example it is prudent to fill forms in 12” – 18” lifts and go around or back and forth over the length of the forms rather than stay in one place until the form is full.

6.32. Consolidation. Concrete must be consolidated (compacted) in order to avoid rock pockets, sand streaks, or voids. The best way to accomplish this is with mechanical, power driven ‘spud vibrators’. The vibrating head of these should be placed into the wet concrete as near to vertical as possible, and slightly penetrate any lower previously placed lifts.

Vibration can sometimes be applied externally to the forms or extended ends of rebar with successful results.

Care must be taken to vibrate the concrete enough to eliminate all voids and rock pockets, <u>but not so much as to cause segregation</u>. Overvibration reduces the strength of the concrete, and unfortunately commonly occurs particularly with inexperienced workers and overly wet mixes.

6.33. Slabs on Grade. One of the most common uses for concrete is the humble slab constructed on the ground or ‘slab on grade’. Following are some of the more important issues relative to these.

6.33.1. Subgrade. The slab will only be as good as the subgrade (soil below the slab). If the subgrade settles, shrinks, or swells, the concrete will follow suit and crack, settle or raise. It is imperative that the subgrade be prepared properly, which includes the following:

- Removal of loamy organic topsoil
- Removal of weeds, branches, roots or other organic material that will in time rot and cause voids below the slab
- Removal of soft spongy soil and replacement with compacted granular soil (sand or sand / gravel mix)
- Compaction of all fill soil in 10" maximum (uncompacted depth) lifts. If native cut soil (undisturbed soil that has been exposed by cutting away the topsoil) is the subgrade, compaction may not be necessary; consult your engineer or geotechnical consultant.

6.33.2. Drainage. It is important that groundwater be kept from underneath the slab because soil bearing capacity is reduced if the soil is saturated. Perimeter footing drains are an excellent method for accomplishing this.

Regarding surface water draining from the top of slab, recall that water is essential for hydration of concrete. So, if the concrete gets wet and stays wet, it will not cause a problem with the concrete. It may, however cause other problems such as ponding – freezing, therefore the top of slabs exposed to water should be sloped. The minimum slope that should be used for slabs is 1-1/2%, though 2% (1/4" per foot) is normally recommended. Less slope, to as little as 1/2% (about 1/16" per foot) is okay, so long as extreme care is taken in preparation of forms, and placing concrete such that no sags or 'bellies' are created in the surface of the finished concrete.

Interior slabs are frequently placed level (no slope). If surface drainage is a concern, 1% (approximately 1/8" per foot) is recommended.

6.33.3. Vapor Barrier. Any interior slab should be constructed with a vapor barrier between the bottom of slab and ground to ensure that groundwater will not wick upward through the pores in the slab and cause problems with flooring material. There are plenty of examples where this was not done, the result being that rigid floor tiles blistered and came unglued. A good heavy visqueen, 6-mil or thicker is recommended, with at least 6" overlap at all seams. If subgrade is rocky, a 2" layer of sand should be placed on top of ground, then the visqueen placed over that. A 2" layer of sand should also be placed over the visqueen to ensure that it is not punctured during rebar placing and concrete pouring. Even a pinhole can cause moisture to migrate through the slab.

In the cases where high groundwater is not a problem, and a certain amount of water vapor can be tolerated through the slab, a 4 – 5 inch thick layer of pea gravel is recommended under the slab. This will act as a capillary break to retard wicking of unanticipated ground moisture upward to the bottom of slab.

6.33.4. Reinforcement. Although it is not required by code, I personally recommend that all slabs on grade be reinforced. This will help to control cracking due to temperature and moisture fluctuations, and will impart additional strength to the concrete as well.

For non-vehicular slabs, a 4" thick concrete section with light gage welded wire fabric is okay.

For any slab that will experience vehicular traffic, particularly if the subgrade soils are marginal, I recommend a 5-1/2" minimum thickness slab with #4 rebar at 16” on center each way. I also recommend a perimeter footing of at least 12” deep and 8” wide with continuous #4 or #5 rebar in the top and bottom. If subgrade soils are bad, or heavy wheel loads are anticipated, see your engineer about heavier reinforcement and thicker concrete.

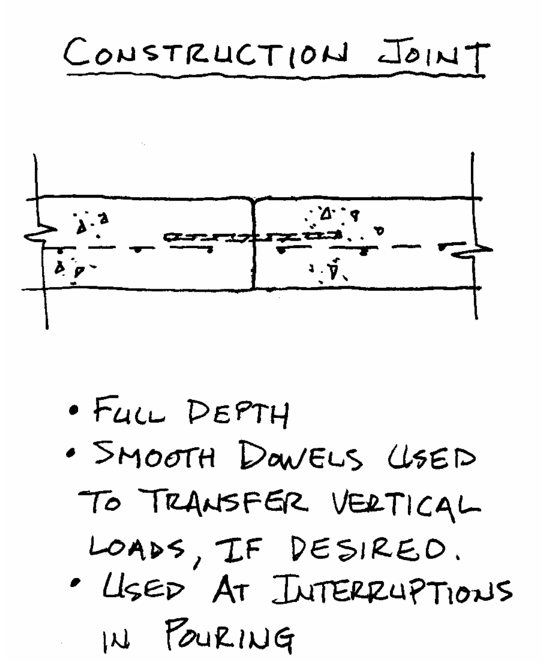
All slab reinforcement should be supported at approximately mid-height of slab on mortar blocks or chairs (sometimes called ‘dobies’). If welded wire fabric is used, it is recommended that it also be pulled upward during pouring to ensure that it winds up toward the middle of the slab. If the slab is heavily reinforced, the rebar purpose and location may vary, consult with your engineer.

6.33.5. Concrete Mix. It is recommended that slab concrete have a 28 day compressive strength, f’c of at least 2,500 psi. Cement content of at least 5 sacks per cubic yard for non-vehicular applications and at least 5-1/2 sacks for vehicular areas is recommended. Air entrainment is a must for exterior slabs and is recommended also for interior slabs.

6.33.6. Joints in Slabs on Grade. There are three basic types of joints used in slab on grade construction.

- **Construction Joint**. This is a full depth joint that is used whenever concrete placing is interrupted or discontinued.

  If bond across the joint is not desired, but transfer of vertical load is, smooth dowels across the joint should be used at some regular spacing, preferably the same spacing as the slab reinforcement. If bond across the joint is desired, deformed dowels are recommended. Shear keys are recommended in lieu of smooth dowels only if the slab will not be subjected to wheel or other heavy point loads (the key makes a weakened plane from the top of the key to the top of slab that can crack and break under heavy load).

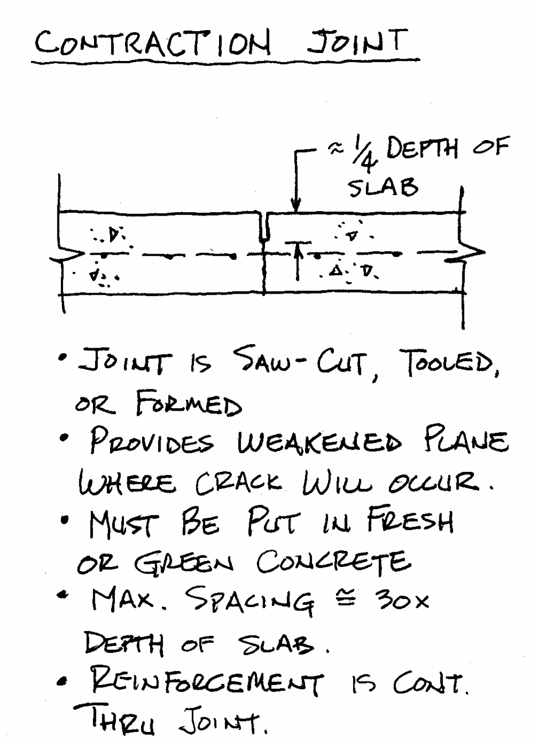


- **Contraction Joint**. As all concrete cures, it shrinks. In slabs, this will cause random cracking unless contraction joints are constructed into the slab.

  Contraction joints do not stop the slab from cracking, they just predetermine where the cracks will be. A contraction joint is a grooved, sawed, or formed line that is deliberately placed in the slab approximately one fourth the thickness of the slab. The slab is weakened (thinner concrete section) at the joint, and thus shrinkage cracks will occur there. The maximum spacing of contraction joints is approximately 30 times the thickness of the slab. They should be placed at regular intervals, in both orthogonal (perpendicular) directions, such that a 'square panel' look is obtained.

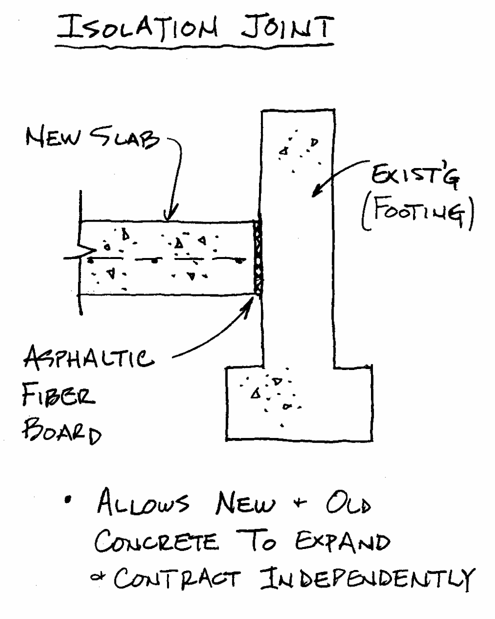
If contraction joints are saw cut, they must be cut just as soon as the concrete is stiff enough to support the saw.

There are proprietary formed contraction joints that are placed into the concrete, then removed, leaving a joint. Other types are designed to be left in place. They all perform the same function: providing a weakened plane in the slab so that the resulting shrinkage cracking will occur there and not randomly.



- **Isolation Joint**. When a new slab is placed against an existing slab, footing or other rigid surface, an isolation joint is used at the interface.

  Isolation joints consist of a bituminous soaked fiber 'board' that is the full depth of the slab, and is 'stuck' to the existing surface with mastic, and left in place. The new concrete is poured directly against it. The isolation joint allows both the new and old concrete to expand and contract independently of each other, and thus not create accidental stresses in either.



6.34. Hot and Cold Weather Concreting. The old notion that concrete can not be placed in hot or cold weather is obsolete. It is not necessary to delay concrete placing due to hot or cold weather so long as the correct protective measures are taken. It should be noted however, that enactment of these protective measures nearly always results in more expensive concrete.

6.34.1. Hot Weather Limitations. So when is it too hot to place concrete without extra protective measures? When the ambient temperature reaches 80 degrees F, and the relative humidity is 25% or lower, caution is advised. When the temperature reaches 90 degrees F, (and particularly if the humidity is low) additional protective measures must be taken.

6.34.2. Hot Weather Effects. In the absence of special precautions, the following undesirable effects of placing concrete in hot weather may occur:

- Water demand will increase. For each 10 degree rise in temperature, about 7 additional lbs of water per cubic yard is required to maintain the same slump.
- Weakened concrete due primarily to the extra water required
- Accelerated set and rapid loss of slump
- Cracking is exacerbated. This is caused by shrinkage due to the additional water required, and by rapid evaporation from the surface of the green concrete. This is particularly evident in thin slabs
- Increased permeability of the concrete. This can lead to water penetration problems, and corrosion of rebar
- Reduced bond of concrete to reinforcement

6.34.3. Hot Weather Precautions. Following is a summary of precautions that can be followed to minimize the effects of hot weather concreting:

- Plan carefully, and be prepared so that delivery and placement is performed as quickly as possible
- Keep aggregates cool, particularly coarse aggregates. This can be done by shading, sprinkling with cool water, and forced air fanning over the wet aggregate
- Use chipped ice in the mixing water. All ice should be melted before the concrete leaves the mixer
- Use cold mixing water
- Use light colored mixers
- Use water reducing admixtures to avoid having to add excess water
- Use a retarding admixture to slow or extend setting time
- Avoid over-mixing. If a mixer is delayed, stop the mixer and agitate intermittently
- Work at the coolest times of the day or night
- For slabs on grade or foundations, thoroughly wet the ground prior to placing concrete. This keeps the ground from absorbing the concrete's hydration water
- Use fog nozzles in the vicinity of the work area to raise humidity and lower air temperature
- Protect the concrete with sunshades and / or wind breaks
- Start curing as early as possible. Apply the selected curing material as soon as the concrete surface will not be marred by it
- Use fog nozzles (not spray) directly over the fresh and green concrete

6.34.4. Cold Weather Concreting. There is no technical reason that concrete can not be placed in cold or freezing weather. Precautions must be taken, however, which will most likely increase the cost.

6.34.5. Cold Weather Effects. The rate of hydration slows with a lowering of temperature. If concrete is placed in cold, but not freezing temperatures, it will cure very slowly. Of course this means that it will gain strength very slowly too. Concrete placed in cold weather (but not allowed to freeze) will only gain approximately 20% of it's design strength after one week. At two weeks it will be at about 60%, and at two months it will finally reach it's design strength.

If fresh or green concrete is allowed to freeze (for even a short amount of time), it will be permanently and irreversibly damaged. It will lose approximately half (or more) of it's design strength.

If fresh concrete is protected from freezing for 2 or 3 days, then allowed to freeze, it will gain only as much strength as it gained during those 2 or 3 days. New concrete should be kept at a minimum temperature of 40^0 F for one full week.

There is no material or admixture that can be added to fresh concrete to lower it's freezing temperature, or act as an anti-freeze.

6.34.6. Cold Temperature Limitations. If freezing will not occur, but temperatures can dip below 40 degrees F, using hot water in the mix is normally all that is required to ensure good concrete. Remember though, cure time will be longer if temperatures remain cold.

For structural concrete for buildings, it is recommended that small members be poured at 55 degrees F, minimum ambient temperature; and for larger members 50 degrees F is the recommended minimum. The concrete should be batched at about 10 degrees higher temperature to compensate for the temperature loss that will occur during transport.

6.34.7. Cold Weather Precautions.

- Plan ahead; provide heaters, enclosures, insulated forms, etc.
- Use extra Type I cement, or use type III (high early strength) cement to speed hydration; or use an accelerating admixture
- Use air entrainment
- Heat the mix water. Heat to no more than 175 degrees F
- Do not use frozen aggregate
- Do not place concrete in frozen forms or on frozen ground
- Leave forms in place as long as possible to trap the heat of hydration
- Maintain the temperature at 55^0 F, 50^0 F, or 45^0 F for thin, medium, or heavy sections for three days; two days for Type III cement or accelerated Type I
- Maintain concrete temperature at 40^0 F for four additional days

6.35. Marine Applications. Concrete is an excellent material for saltwater marine construction, provided that the correct materials are selected, and high quality construction methods are used.

6.35.1. Marine Concreting Design. Special attention to wave action and scour must be given to the design (discuss this with your engineer). Joints which allow saltwater access to reinforcement must be carefully designed or avoided all together. Use of chamfers and fillets at corners to shed standing water is good practice.

6.35.2. Marine Concreting Materials. Use only Type V cement, well-graded non-reactive (inert) aggregates, and a low water-cement ratio. Air-entrainment is recommended. Use of a well tested pozzolan is sometimes recommended. Water reducing admixtures are sometimes recommended to improve workability without the addition of extra water.

Normal deformed rebar is okay to use, so long as at least 3” of cover is provided to all surfaces, and the concrete is properly placed and mechanically consolidated (vibrated). Epoxy coated rebar is sometimes recommended, however it is no substitute for proper workmanship.

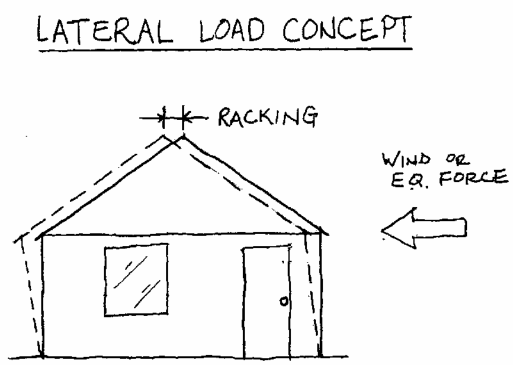
Question often arises as to whether or not seawater may be used to make concrete. Because seawater contains about 3-1/2 percent chlorides (salt), it will cause corrosion of reinforcing steel. Therefore, it is not recommended for use in reinforced concrete. If, however, an unreinforced concrete structure is contemplated with seawater, this may be okay, but keep the water-cement ratio as low as possible, and be sure to use air-entrainment.

6.35.3. Marine Concreting Curing. Concrete should be cured for at least 7 days prior to saltwater exposure.

CHAPTER 7
WIND AND SEISMIC (LATERAL) LOAD DESIGN

7. Introduction. A very high percentage of structural failures occurs due to the natural forces of wind and earthquakes. This chapter will explain how these forces of nature act on structures, and how structures can be designed to resist them.

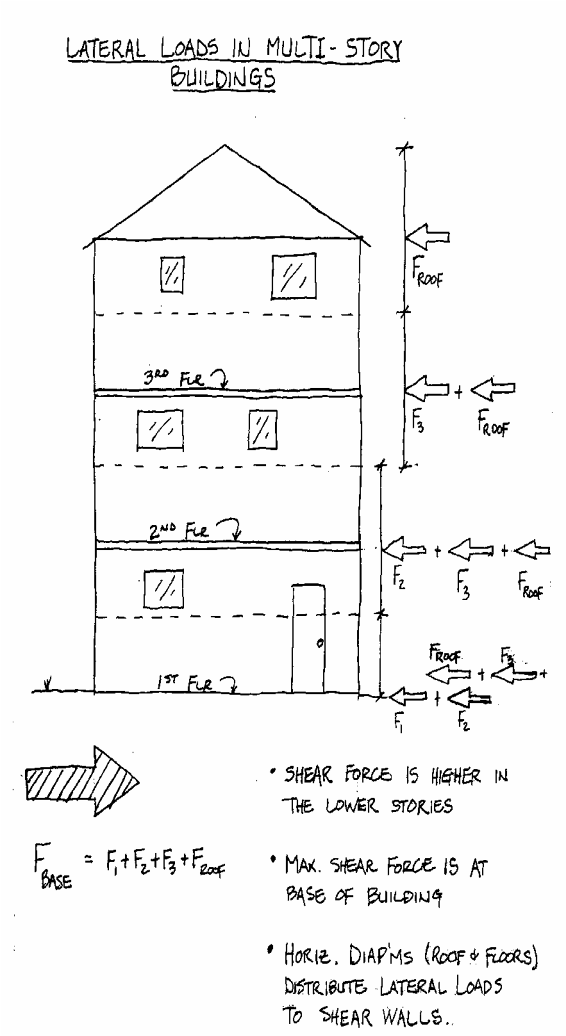
Both wind and earthquakes act similarly on structures. They apply sideways, or *lateral* loads.



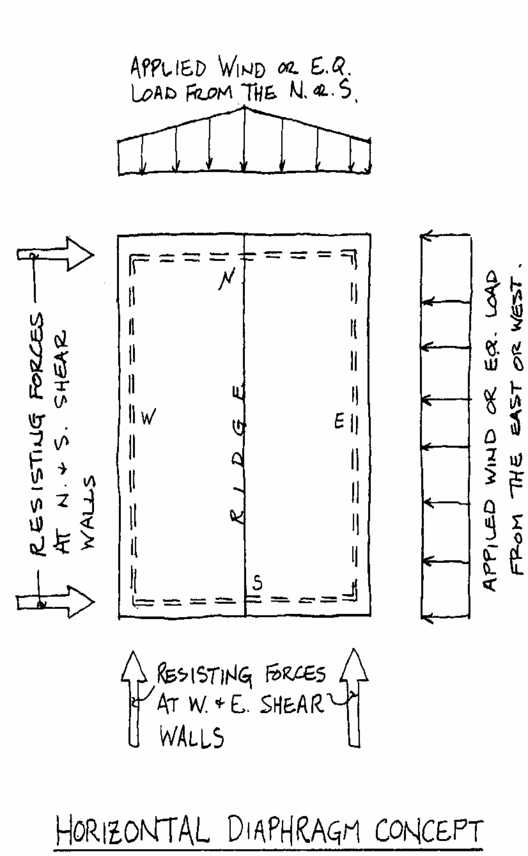
There are also upward or uplift components of force applied to structures from wind or seismic action. Both of these loading phenomenon (lateral and uplift) are quite different than downward loading due to the weight of things (gravity loading). One of the most common mistakes I see among non-engineers is that they get so wrapped up in worrying about gravity loading that they completely forget or neglect lateral loading. The simple truth is that structural failures due to wind and earthquakes kill many more people, and destroy much more property every year than failures due to gravity loads. The western United States is both wind and earthquake prone, thus common sense - and the Uniform Building Code require that every structure be built to resist lateral loading.

7.1. Distribution of Lateral Loads. When wind or earthquake strikes, those lateral loads are absorbed by a structure and are distributed throughout based on weight (in the case of earthquake), tributary wind sail area (in the wind loading case), building stiffness, and other factors that will be discussed in following sections. It is important, however, that you first understand in general how those loads are accumulated and dispersed.

7.1.1. Vertical Distribution of Lateral Loads. In the case of a single story structure, it is relatively simple to determine the amount of lateral load that will go to the walls or frames resisting the racking forces; it is the amount tributary to the roof and upper half of the walls. But what about multi-story structures? The UBC dictates a complicated method to determine how much lateral load goes to each story in a multi-story structure, which must be left to the engineer. Conceptually, however, the non-engineer should know that the *lower stories must carry the lateral load tributary to them, plus the lateral load of all of the stories above*. So, the first story will always be the most heavily loaded from wind or earthquake because it has to take its own tributary load plus the entire building above. *In short, the lower you go in the structure, the higher the lateral loads.* This very issue causes innumerable problems and failures in multi-story structures because it is so common for building designers to want the first floor to be very 'open', i.e. have lots of storefront windows and / or doors (laterally weak); and yet the first floor gets hammered the hardest during a lateral load event. The UBC has recognized this, and has developed strict rules for soft or weak stories, as discussed later in this chapter.



7.1.2. Horizontal Distribution of Lateral Loads. Now we know how lateral loads are distributed vertically over a structure, but how about horizontally? In other words, once we determine how much load goes to each story, how do we go about getting that load to each shear wall, frame, or other load resisting element? The answer in nearly all types of building construction is through horizontal diaphragms. These are nothing more than floors or roofs. There is another section in this chapter that discusses this concept in detail, but for now, just understand that the horizontal diaphragms absorb lateral load and distribute it to the vertical load resisting elements such as shear walls or frames.



7.2. Wind.

7.2.1. Wind Speed. The UBC specifies the design wind speed and forces produced therefrom for all locations in the U.S. Design wind is based on the 'Fastest Mile Wind Speed'. This is defined in the UBC as, *"… the highest sustained average wind speed based on the time required for a mile-long sample of air to pass a fixed point."* Note that this is an *average* wind speed, and does not directly reflect gusts, which can have much higher short term 'bursts' of speed. The wind speed maps included in the UBC show wind 'contours' of basic wind speeds, which are the speeds used in design. Basic

Wind Speed is defined as: *"… the fastest-mile wind speed associated with an annual probability of 2% measured at a point 33 feet above the ground for an area having exposure 'C'."* To simplify, this means that <u>structures must be designed for an average sustained wind that will theoretically occur once every 50 years</u>.

7.2.2. Wind Exposure. The wind force on your structure depends on how well shielded it is from the wind by obstructions. Obstructions are permanent wind blocks such as forests, buildings, or ground surface irregularities. The UBC has three categories of exposure:

7.2.2.1. Exposure B. This is the most shielded exposure. It has obstructions covering at least 20% of the ground level area extending 1 mile or more from the site on all sides. If you are building in a wooded area, or in the middle of a city, and your building is no taller than the trees or buildings around you, this is your exposure.

7.2.2.2. Exposure C. This is the basic open, exposed condition. It has terrain that is flat and generally open extending 1/2 mile or more from the site in any full quadrant. If you are building in the country, and there is open pasture on any side of your house extending 1/2 mile or more, this is your exposure.

7.2.2.3. Exposure D. This is the most exposed, highest wind force situation. It only applies to areas with basic wind speeds of 80 mph or more, and to shoreline conditions. It is defined as flat and unobstructed terrain facing large bodies of water over 1 mile or more in width; and extending inland 1/4 mile or 10 times the building height. So, if you are building on the beach or shore of a very large lake or the ocean, this is your exposure.

Exposure makes a huge difference in the amount of wind force used in design. For example, the difference in exposure B to C for a residence is about 80%. From C to D is about 50%; and from B to D is a whopping 104%. All this boils down to significant added cost in structural bolstering as your wind exposure goes up.

7.2.3. Building Height. As the height of the building goes up, the wind force increases on the higher portions of the building. In general, the wind speed at the surface of the ground is lower than at higher elevations. So if you are constructing a 4 story building, the top story will experience greater wind force than the bottom floor; approximately 50% higher at 40 feet above the ground for an exposure C condition.

7.2.4. Closed, Partially Closed, or Unenclosed Buildings. The wind force used in design depends on the number of openings in the exterior walls. *Openings* are defined in the UBC as all doors and windows that are not specifically designed and installed to resist wind loads. For nearly all residential and light commercial construction, doors and windows are classified as openings because they are not specifically designed and installed to resist extreme wind loads; and if they are (garage and shop doors for example), their cost is much higher.

Design wind forces are about the same for enclosed and unenclosed buildings. Partially enclosed buildings, however are subject to about 30% higher design forces for elements and components.

Partially enclosed is defined in the UBC as *"… a structure or story that has more than 15% of any windward projected area open; and the area of* *openings on all other projected areas is less than half of that on the windward projection".*

Unenclosed is defined as *"…. A structure or story that has 85% or more openings on all sides".*

7.2.5. Discontinuities. When strong winds strike, the vast majority of structural damage that occurs does so at discontinuities in buildings. Discontinuities are irregularities in the lines or surfaces of walls or roofs. These include: corners, eaves, ridges, and gable ends. The wind that strikes discontinuities tends to pull and tear at things rather than push. The UBC has special rules for design of elements and components at discontinuities. Generally speaking, engineers are required to apply <u>up to double</u> the wind loads to these areas (roof edges, overhangs, and ridges more so than wall corners) compared to loads applied to the rest of the structure. This is why, for example, you may see extra strong truss to wall connectors near gable ends.

7.2.6. Essential and Hazardous Facilities. The UBC requires that the design wind forces used for essential facilities and hazardous facilities be greater than for standard occupancy structures. This means that engineers design essential and hazardous facilities 15% stronger than standard occupancy structures. See UBC table 16-K for listings of the various types of facilities.

7.3. Earthquake Design.

7.3.1. General. When an earthquake strikes, the ground is caused to move; both horizontally and vertically. Any structure in the path of the earthquake ground wave is going to be shaken proportionally to the magnitude and frequency of the earthquake itself. Magnitude refers to the force of the wave (strong or weak) and frequency refers to the speed of the shaking. Basic earthquake design mimics this shaking by the application of lateral loads to the building and providing structural elements in the right places and quantities to resist them. Actually, wind and earthquake loading act similarly on a structure. In simple terms, they both tend to push sideways, thus causing racking. Lateral design of light structures, like wood framed residences are usually controlled by wind forces, and heavier structures are typically controlled by earthquake forces; but you can't be sure unless you run the calculations. In design, the engineer calculates the force from both, selects the higher force, and applies it; from that point on, the non-controlling force is no longer considered.

It is interesting to note that frequently wind will control the design for forces acting in one direction, say perpendicular to the ridge line, and seismic will control in the other direction. In these cases, the controlling forces, both wind and earthquake, are used to design lateral load resisting elements tributary to them. The loads are applied separately, however, because it is assumed that a major wind will not occur at the same time as a major earthquake.

7.3.2. Weight. There are many factors that affect a building's response to earthquake. The most important is the <u>weight</u> of the building. The higher the weight, the more seismic load is applied during an earthquake. So, if you build a house out of 2x4's, composition roofing, vinyl flooring, and wood siding (relatively light construction), it will not be nearly as affected by earthquake as one built from solid core masonry walls, hydronic heated tiled floors, and tile roofing. This is not to say the masonry house will collapse while the stick framed house will stand; not at all. How well the structure behaves depends on how strong and well built it is. The point of this section is that heavier structures must resist higher earthquake loads because it is the weight of the structure that causes the lateral forces when the earth shakes.

7.3.3. Flexibility or Ductility. In general, the more flexible and ductile a structure, the better it will fare in an earthquake. To understand this, assume that there were two identical fifty year old structures side-by-side; except that one was built of bricks and mortar, and one was built of wood. Wood is much more ductile than masonry; hence it can bend, sway, and deflect a certain amount without cracking and breaking whereas masonry is brittle, and tends to crack and break under dynamic loading (loading associated with motion). I would expect the wood structure to fare much better in a strong earthquake, because it would tend to bend and sway; moving with the motion of the ground. The brick structure would behave in a brittle fashion; not able to bend much without cracking. If there was not sufficient steel reinforcement in the right places, the brick building would be more likely to sustain catastrophic damage than the wood building.

This is not to say, however, that all wood framed buildings are safe from earthquakes. Actually, quite the contrary is true. Many older wooden buildings have certain design flaws that make them very vulnerable to earthquakes. Two common problems are the *soft story* and the *window wall*. Both of these items will be covered in later sections.

Concrete buildings behave similarly to masonry; they tend to be brittle and prone to failure in an earthquake, particularly if they do not have adequate steel reinforcement. If, however, your concrete or masonry structure is built in accordance with current code, it will be heavily reinforced, and <u>will</u> be extremely strong and resistant to earthquakes. The steel reinforcement creates <u>ductility</u> in the masonry or concrete.

7.3.4. Seismic Zones. It is understood that the closer you are to the earthquake, generally, the more seismic force your building will experience. So it would be uneconomical to require all buildings be designed to resist a high magnitude earthquake if they were built in non-earthquake country. The U.S. is broken down into six zones which reflect this. The zones are classified from least hazardous to most hazardous as follows: 0, 1, 2A, 2B, 3, and 4. The zone 4 regions are western and southern California, central Nevada, southern Alaska, and southern Hawaii. Western Washington is a zone 3 region, and Eastern Western Washington is a zone 2B region.

7.3.5. Soil Classification. Another important factor in determining how much effect an earthquake will have on a structure is the type of soil on which the structure is founded. The UBC lists six type of soils that cover all situations; they follow:

- **Soil Type S_A**. This is hard rock.
- **Soil Type S_B**. This is rock, but not as solid and competent as hard rock. It may be fractured and or weathered to a certain extent.

- **Soil Type S_C.** This is very dense soil and soft rock.
- **Soil Type S_D.** This is a stiff soil profile.
- **Soil Type S_E.** This is a soft soil profile. It includes any soil profile with more than 10 feet thickness of soft clay.
- **Soil Type S_F.** This is the worst type of soil for earthquakes. It includes liquifiable soils (see the next section on liquefaction), weak clays, peats, and thick layers of stiff clays. This soil type requires site specific evaluation.

The UBC requires that the soil type be based on *"…. properly substantiated geotechnical data using the site categorization procedure listed in section 1636 and Table 16J."* What this requires is specific geotechnical information consisting of: average shear wave velocity, and / or standard penetration resistance, and / or average undrained soil shear strength. This information can only be supplied by a competent seismologist, geotechnical engineer, or engineering geologist. There is an exemption in the UBC that allows the use of soil type S_D when the soil properties are not known in sufficient detail to determine the soil profile type. The exemption goes on to say that soil types S_E and S_F need not be used unless the building official or geotechnical data indicate that they may be present.

7.3.6. Liquefaction. This is a soil condition whereby normally safe, sound soils turn to mush during the shaking associated with an earthquake. The soils involved in liquefaction are typically loose granular sands, silty sands, or sands and gravels. If you place a heavy structure on these soils, everything may be perfectly fine until the earthquake hits. At that point, the bearing capacity of the soil goes to about zero, and down goes the structure. If you suspect that liquefiable soils may be present at your building site, a geotechnical investigation is required. There are several remedies for liquefiable soils, which include pilings or vibro-compaction. Both are expensive.

7.3.7. Essential and Hazardous Facilities. The UBC requires that the design seismic forces used for essential facilities and hazardous facilities be greater than for standard occupancy structures. This means that engineers design essential and hazardous facilities 25% stronger than standard occupancy structures (elements and components are designed 50% stronger). You will note that the importance factor for seismic design is greater than for wind design (25% versus 15%). Examples of essential or hazardous facilities are: hospitals, fire and police stations, emergency preparedness shelters, aviation towers, or any structure housing toxic or explosive chemicals. See UBC table 16-K for a complete listing of these types of facilities.

7.3.8. Irregularities. In general, plain square box type structures behave better in earthquakes than architecturally pleasing angled, discontinuous, 'cut-up' structures. The UBC recognizes this and places stringent requirements on such architectural wonderments. In general, the UBC allows such structural irregularities, but additional analysis (which is expensive) is required; and the analysis will likely result in more restrictive and expensive construction. Following are the ten irregularities recognized and their consequences.

7.3.8.1. Stiffness Irregularity – Soft Story. A soft story is one in which the lateral *stiffness* (not necessarily strength) is less than 70% of that in the story above or less than 80 percent of the average stiffness of the three stories above. An example might be a lower story with lateral load resisting steel frames, and an upper floor of numerous shear walls. The walls are much stiffer than the frames, thus the lower floor will deflect much more than the upper; thereby creating a soft story. The 'Penalty Requirement' is that analysis must be a complicated and expensive *dynamic analysis* rather than the much simpler and cheaper *static analysis*.

7.3.8.2. Weight (mass) Irregularity. This exists where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered. An example might be if an upper floor supports the load of vats of liquid; or a parking structure on the roof. The 'Penalty Requirement' is that analysis must be a complicated and expensive *dynamic analysis* rather than the much simpler and cheaper *static analysis*.

7.3.8.3. Vertical Geometric Irregularity. This exists where (when viewing from the side, or profile view) the horizontal dimension of the lateral force resisting system in any story is more than 130% of that in an adjacent story. An example of this is where you have a short shear wall directly under a very long upper floor shear wall. The result is similar to a soft story. The 'Penalty Requirement' is that analysis must be a complicated and expensive *dynamic analysis* rather than the much simpler and cheaper *static analysis*.

7.3.8.4. In-Plane Discontinuity in Vertical Lateral Force Resisting Element. This exists where an in-plane offset of the lateral load resisting elements (on the same floor) is greater than the length of those elements. There is an exemption for certain types of light wood and steel framed constructions. This applies more to concrete structures. The 'Penalty Requirement' for this is that additional seismic load cases must be analyzed during design.

7.3.8.5. Discontinuity in Capacity – Weak Story. A weak story is one in which the story strength is less than 80 percent of that in the story above. The story strength is the total strength of all seismic resisting elements sharing the story shear for the direction under consideration. An example of this could be the classic window wall commonly seen in storefronts, when the upper floors do not have window walls. The 'Penalty Requirement' is that the height of the structure is limited to two stories or 30 feet where the weak story has a calculated strength of less than 65% of the story above.

7.3.8.6. Torsional Irregularity. This is to be considered when horizontal diaphragms are not flexible. In general, this will apply only to buildings with floors or roofs of: concrete, or very stiff steel or plywood diaphragms (the calculation to determine whether or not the diaphragm is flexible is quite complicated). A torsional irregularity exists when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure. This will occur when the shear walls or other lateral load resisting elements are much stiffer at one side of the building than the other (on the same floor). In an earthquake, the building will tend to twist, with the flexible side allowing more movement relative to the stiff side. The 'Penalty Requirements' for this are: lateral analysis must be made for other than the principal axes as well as for the principal axes; and no short term allowable stress increases are allowed for connections of vertical elements to horizontal diaphragms.

7.3.8.7. Re-entrant Corner. A re-entrant corner is an 'inside' corner. For example, where the exterior wall line of a building jogs inward and creates an alcove, the corners at the inside of the alcove are re-entrant. To be affected by this paragraph, the re-entrant corner must have both wall projections beyond the corner be of a length greater than 15% of the plan dimension of the structure in the given direction. So whenever you have an inside corner that is deep within the outline of the building, this condition applies. The 'Penalty Requirements' for this are: no short term allowable stress increases are allowed for connections of vertical elements to horizontal diaphragms; and the horizontal diaphragm chords and drag members must be designed considering independent movement of the wings in the same and opposite directions.

7.3.8.8. Diaphragm (horizontal diaphragm) Discontinuity. This exists where horizontal diaphragms have abrupt discontinuities or variation in stiffness, including those having cut-out or open areas greater than 50% of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50% from one story to the next. An example of this could be a floor with a large opening for an atrium below. The 'Penalty Requirement' for this is no short term allowable stress increases are allowed for connections of vertical elements to horizontal diaphragms.

7.3.8.9. Out-of-Plane Offsets. These exist where there are discontinuities in the lateral force path, such as out-of-plane offsets of the vertical elements. The 'Penalty Requirements' for this are: additional seismic load cases must be analyzed during design lateral analysis; no short term allowable stress increases are allowed for connections of vertical elements to horizontal diaphragms; and special rules are enacted if your lateral load system includes steel 'special concentrically braced frames'.

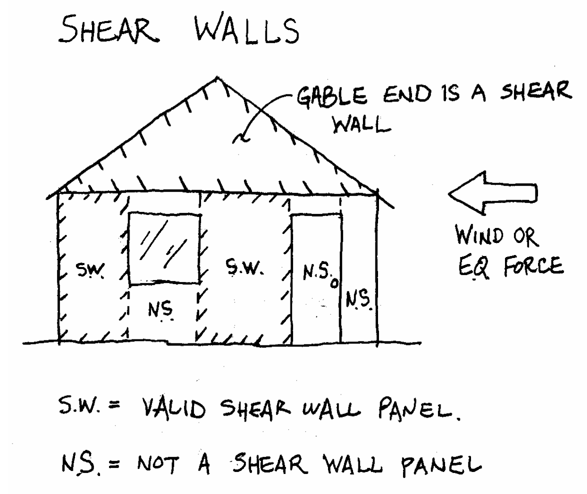
7.3.8.10. Nonparallel Systems. This exists where the vertical lateral load resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force resisting system. An example of this could be where interior shear walls are at odd angles to the exterior walls. The 'Penalty Requirement' for this is the lateral analysis must be made for other than the principal axes as well as for the principal axes.

7.4. Structural Systems. There are various structural systems that are incorporated into buildings to provide resistance to wind and earthquake loads. Over the years, the UBC has become progressively stricter in the requirements of construction of these various systems when they are intended to resist earthquake forces. The UBC recognizes that different types of systems behave differently in earthquakes, and the potential for failure is different as well. As a result, engineers must use different earthquake design forces depending on the type of system used. For example, a heavy timber braced frame which holds up gravity load and is also intended to resist earthquake loads must be designed with approximately double the seismic force as a common plywood shear wall. There are about 25 possible basic types of structural systems (not including the 15 or so possible combinations of systems) that the UBC recognizes. The bottom line is that certain structural systems are penalized by the earthquake code and certain systems are encouraged. Table 16-N in the UBC lists the various systems and their corresponding 'R' values (the lower the 'R' value, the higher the applied load, i.e. the more this type of construction is penalized) . Following is a listing of the more common types of structural systems.

7.4.1. Shear Walls. A shear wall is a wall that is constructed to resist the racking from the sideways (lateral) loading associated with wind or earthquakes. In actuality, all walls provide a certain amount of racking resistance, but in order to be counted in design they must be constructed in a certain manner. Shear walls may be constructed of wood, gypsum board, metal, or concrete.

7.4.1.1. Plywood or Oriented Strand Board (OSB) Shear Walls Over Wood Framing. These are light framed wood stud walls covered with plywood or OSB. The shear strength (resistance to lateral loads) depends on:

- thickness of plywood or OSB (the thicker – the stronger)
- the length of nails used (the longer – the stronger)
- the spacing of the nails (the more closely spaced – the stronger, however spacing may not be too close as to cause stud splitting)



- the spacing of the studs (the closer – the stronger)

There is a whole laundry list of rules for plywood and OSB shear walls that exists as *footnotes* to Table 23-II-1 in the UBC. These are quite important to ensure proper construction, but are very frequently ignored. They are summarized below:

- All edges must be blocked. This means that all plywood or OSB edges must be nailed to a stud, plate, or 2x blocking. [There is an exemption to this, however, for prescriptive shear walls that qualify as conventional light framed construction. See following section on conventional light framed construction.]
- Sheets may be installed either vertically or horizontally.
- Field nailing (i.e. non-edge nailing) must be no greater than 6" O.C. for 3/8" or 7/16" thick sheathing with studs at 24" O.C.; and 12" O.C. for other conditions.
- If other than Douglas Fir-Larch or Southern Pine studs are used, shear wall strength values must be reduced.
- In two sided shear walls where edge spacing is less than 6" O.C., edges must be

offset (staggered) from side to side, or 3" wide studs must be used.

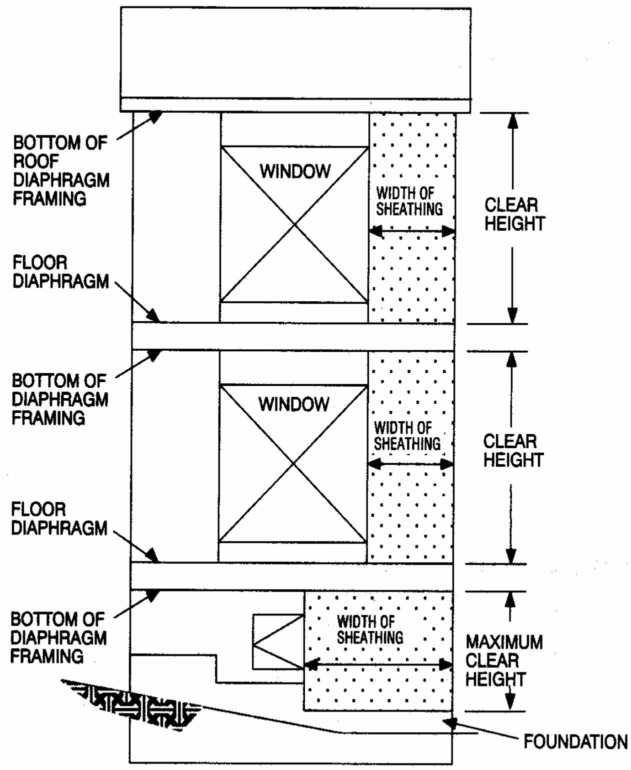
- In seismic zones 3 and 4 (this includes western Washington) where shear loads exceed 350 pounds per lineal foot (this includes 1/2" plywood nailed with 8d's at 3" O.C.; 1/2" plywood nailed with 10d's at 4" O.C.; and all stronger configurations) all edge nailed members and mud sills must be at least 3x material. The only exception to this is the allowance of 2x sill plate material if double the number of anchor bolts are used, and 2"x2" washers are installed on the anchor bolts.

7.4.1.2. Aspect Ratio of Wood Diaphragms. The UBC does not allow tall skinny shear walls or long narrow roof diaphragms. There is a Table, 23-II-G which shows the maximum diaphragm dimension ratios allowed. Following are the most common aspect ratio restrictions that would apply to seismic zone 3 (Western Washington):

- For plywood or OSB shear walls, all edges nailed, the maximum height to width ratio is 3-1/2:1
- For plywood and OSB floor and roof diaphragms, the maximum span to width ratio is 4:1

As an example in determining if a shear wall panel may be counted or not, if you have a garage with a 7' high door, in order for shear wall panels at the sides of the door to be counted, they must be at least 2' wide. If they are narrower than 2', they may not be counted.

There are other ways of providing lateral resistance in the absence of compliant shear walls, but they are all expensive, and should be engineered. In general, the code wants you to use wide shear walls; the more - the better.

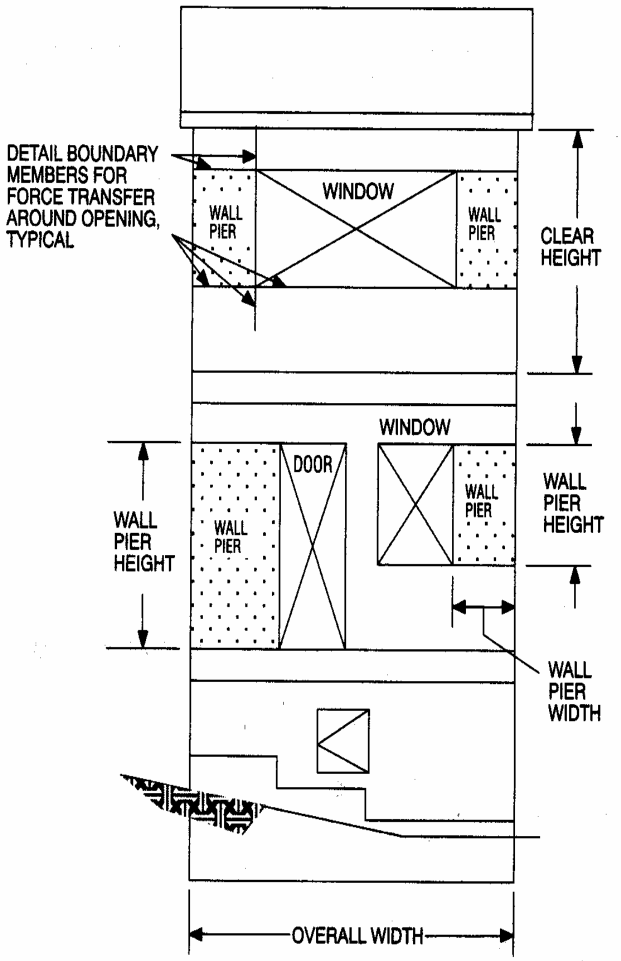


(a) HEIGHT TO WIDTH RATIO

Question arises as how to measure the height and width of a legal shear wall panel. The two sketches from the UBC, Figure 23-II-1, shows how to do this.

The conservative approach is shown in sketch (a) above where the height is from plate to plate.

But you may also use the approach (b) shown below – the difference being that the height is from top of opening to bottom of opening. Note, however, in this case you must provide design and detailing for force transfer around the openings.



(b) HEIGHT TO WIDTH RATIO WITH DESIGN FOR FORCE TRANSFER AROUND OPENINGS

7.4.1.3. Prescriptive (Braced) Shear Walls. The UBC allows certain types of shear walls to be constructed without engineered design. The UBC calls these 'braced walls', and they may be constructed of wood boards placed diagonally, plywood and OSB sheathing, fiberboard sheathing, gyp board, particleboard, or plaster. Of course there are rules for construction of these in the code (see below section for rules pertaining to plywood and OSB braced walls). Prescriptive braced walls are allowed for 'Conventional Light-Framed Construction' only as defined in Division IV of Chapter 23. Question naturally arises as to exactly what qualifies as Conventional Light-Framed Construction. The answer is not so straightforward, but the following bullets will provide some guidance.

CONVENTIONAL LIGHT-FRAMED CONSTRUCTION:

- One, two, or three story buildings of group R Occupancies (residential).
- Single story buildings of other selected occupancies (Category 4 per UBC Table 16-K) with slab on grade floors.
- Portions of buildings that are of non-conventional construction do not qualify.
- High wind areas do not qualify unless specifically allowed by the local jurisdiction.
- In seismic zone 3, with maximum wind speed of 80 mph, buildings must be provided with interior **and** exterior braced wall lines. Interior braced wall lines must be provided at 34' O.C. max in each direction in all stories. Where wind speed is higher, the maximum spacing is reduced to 25' O.C.
- Where buildings have unusual shapes (offsets, framing irregularities, split levels, walls at odd angles, openings in horizontal diaphragms, or other discontinuities as described in UBC section 2320.5.4) they do not qualify as conventional, and must be engineered per UBC Chapter 16.
- All braced walls must be clearly indicated on the Plans.

Following is an explanation of what the aforementioned prescriptive section of code says for plywood and OSB braced walls.

PLYWOOD AND OSB PRESCRIPTIVE BRACED WALLS:

- Must meet all of the requirements of the bullets in the above section to qualify as conventional light-frame construction.
- In seismic zones 3 and 4 (Western Washington and California) minimum plywood thickness must be at least 3/8”, and stud spacing must be 24” or less.
- All braced wall panels must be at least 4’ wide (They may be narrower if constructed as *alternate braced wall panels*, see next section).
- Load path must be completed at the top and bottom of braced wall panels with adequate nailing and anchor bolts per other UBC sections. **This is a huge item, particularly for interior walls that must have positive load path to the roof diaphragm**!
- So long as proper thickness and span ratings of sheathing are used, and stud spacings are per code (i.e. per Table 23-IV-D-1), horizontal blocking of joints is not required.
- Braced panels must start within 8’ of each end of the wall.
- Braced walls must be in-line (or offset) no more than 4’.
- For the top story of a two or three story building, or for a one story building: you must have at least each end of the wall as a braced wall, and there must be braced wall panels at least 25’ OC.
- For the first story of a two story, or the second story of a 3 story: the above requirements apply, plus at least 25% of the building length must be braced wall panel(s).
- For the first story of a three story building: the above bullet applies, except that at least 40% of the building length (not 25%) must be braced wall panels.

7.4.1.4. Alternate Braced Wall Panels. The UBC allows a narrower prescriptive braced wall panel than the typical 4’ wide braced panel if such panel(s) conforms with the following:

7.4.1.4.1. Alternate Braced Wall Panel For a Single Story Building:

- Must meet all of the requirements of the bullets in the previous section to qualify as conventional light-frame construction.
- Panels must be at least 2’ – 8” wide.
- Panels may not be more than 10’ high. (This bullet and the one above violate the UBC’s own aspect ratio limit of 2 : 1 or 3-1/2 : 1 (depending on your seismic zone), as set forth in Table 23-II-G!)
- Sheathing must be 3/8” min thickness, applied to at least one side, and be blocked at all edges.
- There must be 2 anchor bolts (1/2” diameter in seismic zone 3 (Western Washington) or 5/8” diameter in seismic zone 4) per panel at quarter points.
- There must be a holdown capable of 1,800 pounds uplift installed at each panel end.
- The footing under the panel must have at least one #4 rebar continuous in the top and bottom of the footing.

7.4.1.4.2. Alternate Braced Wall Panel For the First Story of a Two Story Building:

- All of the above applies except that sheathing must be applied to both sides of the panel; three anchor bolts, at fifth points are required; and holdown capacity is 3,000 pounds each.

7.4.1.5. Gypsum Board (Drywall) Shear Walls. The UBC allows interior gyp walls to be used to resist wind and seismic loads. The strength values are generally a little better than half those for plywood walls - and this assumes gyp on both sides of the wall and that shear wall design is controlled by wind not seismic (see last bulleted item below for seismic strength reduction). If only one side is sheathed with gyp, strength values are about 1/3 of plywood (tabulated gyp strength values may be added if both sides are sheathed). A summary of other rules pertaining to gyp shear walls follows:

- The strength of the gyp may not be added to the strength of plywood in the case of an exterior wall sheathed on opposite sides with each.
- Gyp walls may be blocked or unblocked, but if unblocked, the strength is reduced about 25%.
- Nailing or screwing of gyp shear walls is uniform throughout; i.e. all edge and field nailing / screwing distances are the same.
- Gyp shear walls may not be used to resist lateral loading by concrete or masonry construction.
- Gyp shear values are reduced 50% for seismic loading (but not wind) in seismic zones 3 and 4 (includes Western Washington).

Gyp shear walls are not frequently used because of their relatively low strength; and it can be difficult to connect them positively to horizontal diaphragms (thereby ensuring proper load path – see following section on load path).

7.4.1.6. Metal Stud Shear Walls. There is a new section in the 1997 UBC (2219) that allows and describes shear walls constructed with metal studs under plywood, OSB, and gypsum board. In general, the shear strength values for this type of construction are approximately equal to wood framed construction. (Beware, the design tables, 22-VII-A, -C show <u>ultimate strength</u> values. These must be reduced per section 2219.3!) Also, there are many restrictions and rules in using metal stud shear walls – be sure you know them first. At this time, only 15/32" structural type 1 plywood, 7/16" OSB, and double sided 1/2" gyp board are allowed per the code.

7.4.1.7. Concrete and Masonry Shear Walls. Shear walls constructed of concrete or masonry offer excellent resistance to lateral loading - but only if they are constructed properly. Alternatively, *unreinforced or minimally reinforced* concrete or masonry walls can be the worst kind (most dangerous) in resisting lateral loads. The reasons for this have been discussed previously, but to reiterate, concrete and masonry by themselves are very brittle. They depend on steel reinforcement (rebar) for ductility and to take the tension associated with racking and other dynamic induced motions. Without rebar (or with insufficient rebar) they are extremely prone to catastrophic failure from dynamic lateral loads. Following are some of the more important points to remember when depending on concrete or masonry walls to resist wind or earthquake loads.

- All concrete and masonry shear walls **must** have sufficient *quantities* of rebar in the right *locations*. The UBC specifies the minimum amount of both vertical and horizontal rebar which must be present. Engineering design may dictate more rebar than the UBC specified minimum.

- At the intersection of floor and roof diaphragms with concrete or masonry shear walls, positive connection(s) must exist which adequately transfer the lateral loads from the horizontal to vertical elements.
- If shear walls are heavily stressed with in-plane bending and axial loads as well as with shear loads, additional reinforcing and increased wall thickness may be required in the 'boundary zones' (the zones at ends of the walls or wall sections).

7.4.2. Moment Frames (non-braced frames). These are frames that develop resistance to lateral loading by virtue of rigid connections of the beam to column portions of the frame. These connections are said to be 'fixed', 'restrained', or 'moment resisting'. There are no intermediate bracing (diagonal) members in moment frames. There are two types of moment frames recognized by the UBC: Ordinary Moment Resisting Frames (OMRF's) and Special Moment Resisting Frames (SMRF's). The main difference is that special moment resisting frames are designed and constructed with much stricter requirements as to their expected performance in earthquakes. With SMRF's, greater attention is required for all of the following: beam-to-column welding and / or bolting; web strength of beams and columns in the vicinity of joints; continuity plates (web stiffeners) in columns at beam flange intersection locations; width-thickness ratios of compression elements; and lateral buckling of beam and column flanges. The tradeoff for all of the added trouble and expense in using SMRF's is that the code required earthquake force used in design is about half that for OMF's (which would likely result in smaller less expensive members).

7.4.3. Braced Frames. These are frames that resist lateral loading by virtue of diagonal braces that attach to the beam and column elements of the frame. They are generally stiffer than moment frames, which means that they control deflection or drift better. There are two basic types of braced frames: Concentrically Braced Frames (CBS's) and Eccentrically Braced Frames (EBF's). Concentrically braced frames are braced systems whose worklines essentially intersect at points, i.e. two diagonal braces connect to either a beam or column at the same location (point). There are 'V', inverted 'V', and 'K' type CBF's.

An eccentrically braced frame differs in that the diagonal braces do not intersect with beams at a point. Rather there is a length of beam between the ends of diagonal braces. This length of beam is called a 'link'. In a severe earthquake, EBF's are designed to plastically yield (but not break) within the link portion of the frame, while the rest of the frame remains elastic.

The UBC allows all of the above types of frames to be used to resist lateral loads. With each type of frame, the UBC dictates a different seismic load to be used in design. This, in effect, rewards certain types of frames and penalizes other types of frames thereby making them less or more costly to build. The 'reward or penalty' is based on strength and ductility, and how well the various systems have historically behaved in documented earthquake events. You can differentiate the better types of systems by examining their 'R' values as listed in the UBC Table 16-N. The higher the R value, the better the system at resisting earthquake loads. You will also note in this table the height limit of the various systems as shown in the rightmost column. For example, the maximum height of a light wood framed shear wall system is 65 feet, but there is no limit to the height of a SMRF system of either steel or concrete.

7.4.4. Horizontal Diaphragms. Now we understand that vertical elements such as shear walls and frames resist the racking associated with sideways loading from winds and earthquakes. But, how do those lateral forces get to the shear walls or frames? Do they just automatically wind up there? No, they do not. The answer (in nearly all building construction) is through *horizontal diaphragms*. Horizontal diaphragms are typically roofs and floors that 'absorb' lateral loads and distribute them to the various shear walls or frame systems. To understand this, imagine a one story building with no roof. Imagine further that all the walls were hell-for-stout plywood shear walls, but they did not join together at the corners. Now, let's say an 80 mile per hour wind kicked up. It would not matter whether the walls were the strongest shear walls in the world, they would simply tip over because there is no roof or ceiling at their top to absorb the wind and transfer it throughout. Now, let's stand the walls back up, and throw a simple truss roof on the building. Let's cover the trusses with plywood, and nail it off well. Let's connect the trusses to the top plates with framing clips. Okay, now let's blow that same 80 mile per hour wind on the building. Now the building will easily withstand the wind force.

This same concept holds true with multi-story structures, except that each floor (as well as the roof) becomes a horizontal diaphragm. Each horizontal diaphragm absorbs and distributes the lateral loads to the shear walls or frames below it.

Question typically arises as to how steeply-pitched roofs can behave as horizontal diaphragms. After all, they are not really horizontal are they? The answer is not simple, perhaps that is why you will not find it in many (if any) textbooks. Textbooks normally use flat roofs in their examples (real world?).

Typically, pitched roofs exist on structures having pre-manufactured trusses on them. Most pre-manufactured trusses have horizontal bottom chords, or at the least shallowly sloping bottom chords in the case of a vaulted ceiling. The 'horizontal diaphragm' is most likely a combination of the roof <u>and</u> the ceiling, but only a very complex analysis could reveal the true answer. Regardless, where manufactured trusses are used, there have been very few cases of structural problems due to failure of a horizontal diaphragm.

If, however, you have a steep roof with no ceiling (i.e. a rafter system), there could be cause for concern because the horizontal diaphragm not only is far from horizontal, but there is no horizontal ceiling to assist in distribution of lateral loads. In the worst case, the roof system could crease along the ridge and fold inward.

7.4.4.1. Wood Horizontal Diaphragms. The UBC allows various types of wood diaphragms, including:

- diagonal planks (at 45 degrees)
- premanufactured structural panels
- plywood
- OSB

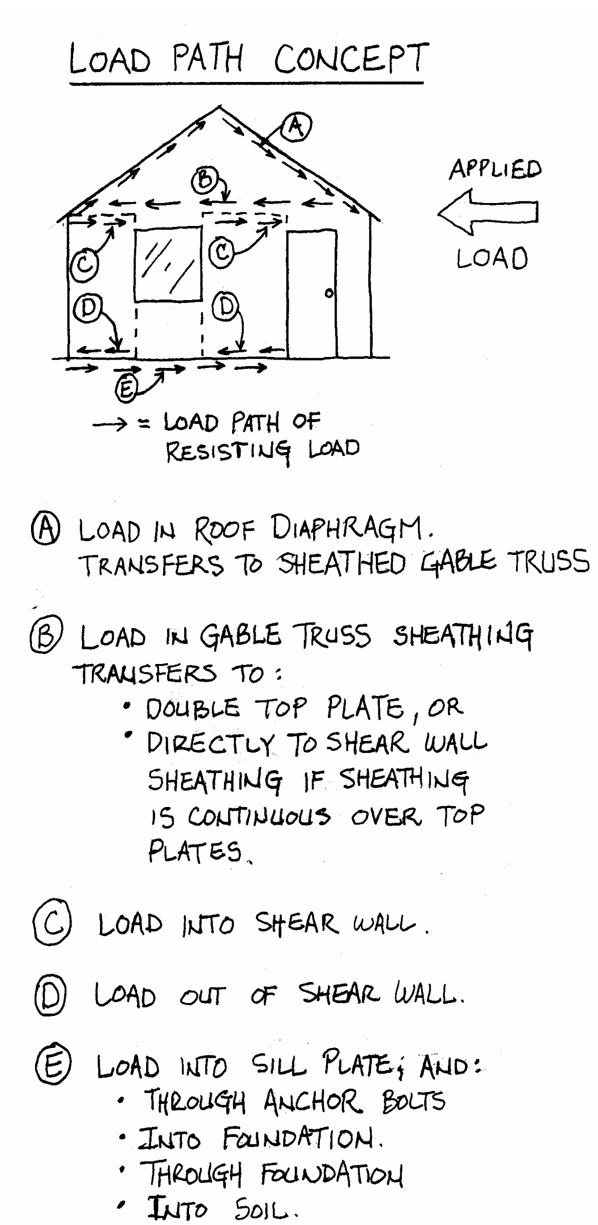
- particleboard

The most common are plywood and OSB. There is a Table 23-II-H that shows the allowable in-plane shear loads for plywood and OSB diaphragms, and the various ways in which the panels may be laid out. The most common way of laying out 4'x8' panels is called 'Case 1', where the panel joints are staggered, and panels are placed 'longways' across the roof framing. This layout results in about double the amount of allowable shear load as opposed to any other layout; so use Case 1 whenever possible.

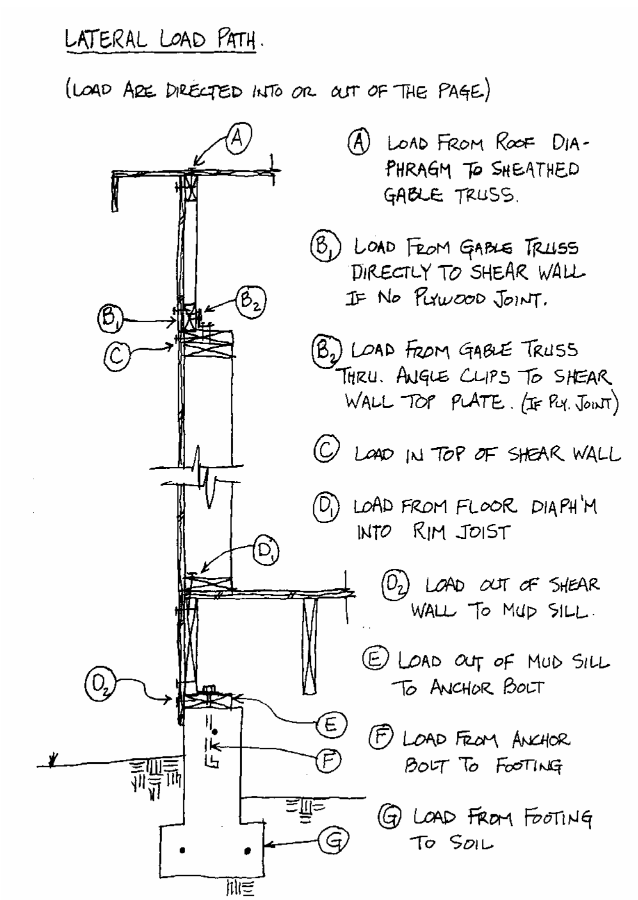
In contrast to engineered wall diaphragms made of plywood or OSB, *roof* diaphragms do not have to be blocked. Unblocked roof diaphragms are perfectly legal, however, their allowable shear load is about half as much as if blocked. In order to know whether or not your roof diaphragm is strong enough, or whether blocking is required, or if your non-Case 1 layout is adequate, an engineered analysis is required. On low rise residential construction, Case 1 - unblocked is normally okay.

Problems with plywood roof diaphragms usually begin to occur when the structure is long and narrow (say 2:1 or more). The maximum dimension ratio allowed by the UBC in Table 23-II-G is 4:1. This means, for example, if you are building a long rectangular shop, who's footprint is 80'x20' (4:1), you would just be allowed to use a plywood roof diaphragm. If you wanted to make your shop 85'x20', you would exceed the 4:1 maximum roof diaphragm ratio, and you would either have to make the shop wider (at least 1.25' wider) or you would have to add an interior shear wall parallel to the 20' end walls.

7.4.5. Load Path. Once we determine what system to use to resist lateral loads, it becomes imperative that we make sure that the lateral loads get to the systems. In other words, we could have the strongest steel frame in the world built into our structure, but if it is not connected properly to the ceiling and floor (horizontal diaphragms), the building could literally fall down around it. *In short, load path is all about connections*.



GARRISON'S FIRST LAW OF WIND AND EARTHQUAKE DESIGN. **You can design yourself a Fort Knox, but if you don't connect it together properly, it is nothing more than a house of cards.**



When engineers design structures for lateral loads, they really earn their pay in the design of the connections. The old adage "A chain is only as good as it's weakest link" applies directly here. It is a well documented fact that the majority of structural failures occurs at the *connections*, not in the rupture or breakage of beams or columns. Every shear wall or frame system must be positively connect at it's top and bottom to another structural link in the lateral load resisting system, or the entire system is defeated. With wood shear wall systems, framing clips, anchor bolts, and holdowns typically complete the load path from horizontal diaphragms (floors and roofs) to walls. With steel frame systems, load path is normally completed through bolting or welding of the horizontal diaphragm to the flanges of the beams. With concrete frame systems, reinforcement connecting floors and beams typically completes the path. The point is, regardless of the lateral resisting system used, there must be a defined method of positively transferring loads from horizontal diaphragms (floors and roofs) to vertical elements (frames or shear walls). And furthermore, this connection system must be strong enough to fully transfer the loads such that any failures occur in the frames or shear walls, and not in the connections themselves.

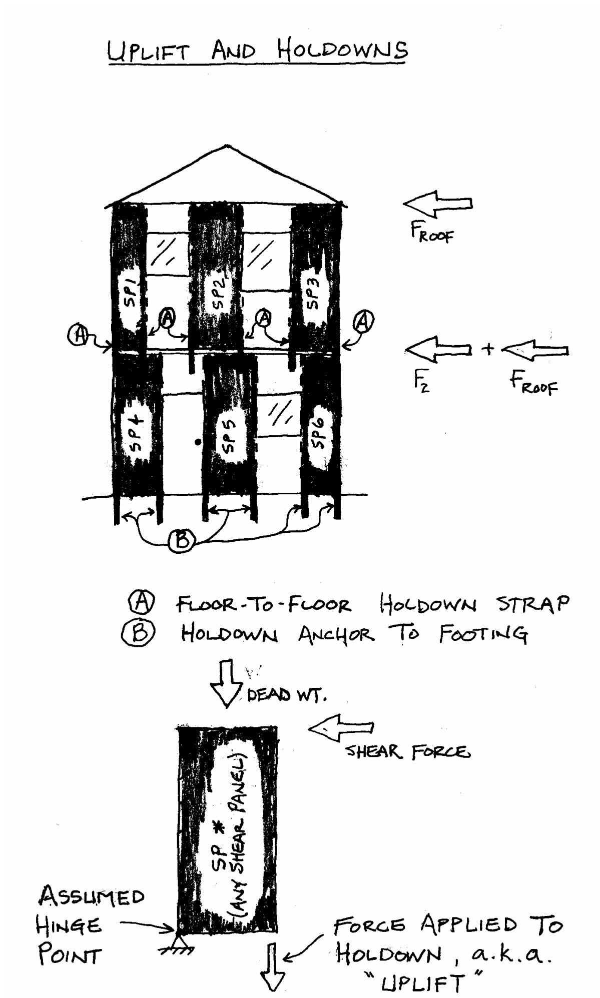
7.4.6. Collectors and Drag Struts. In the real world of construction, it is (unfortunately) common for the load path between horizontal and vertical lateral load resisting elements to be discontinuous. For example, a roof diaphragm may end on top of a beam instead of on top of a shear wall. In this case, the roof is bringing the lateral load it has collected, and is depositing it to the beam; but the beam is not a shear wall and cannot resist the lateral load. If the beam is not connected positively to a shear wall or frame, it will do nothing in a lateral load event other than just go along for the ride. There must be some type of connection to effectively transfer the lateral load from the roof to the beam, to the shear wall. The beam in this case is called a collector, and the connection of the beam to the shear wall may be called a drag connection.

Basically, any time there is a discontinuity between the horizontal diaphragm (roof or floor) and the vertical load resisting element (shear wall or frame), you have a drag or collector condition. These terms are sometimes used interchangeably, but they both refer to essentially the same thing - a discontinuity between the horizontal and vertical load resisting elements.

If nothing is done to complete the *load path* via collectors or drag struts, a weakness is built into the lateral load resisting system; and this will be the point of failure. Being able to spot these discontinuities takes considerable knowledge and experience. This is probably the single most important area of neglect by non-engineers when it comes to lateral design.

7.4.6.1. Uplift and Holdowns. In order for shear walls to be stable in a lateral loading event, they must be adequately connected at their bases. Recall that lateral loading is applied to the top of a shear wall. This tends to make the wall want to slide off of its base, and also rotate in-plane in the direction of the applied load. The sliding is typically resisted by anchor bolts in the case of first floor shear walls, and nailing through the bottom plate into the plywood floor diaphragm in the case of upper story shear walls.

Uplift is another story all together. To understand this, consider an isolated shear wall panel and imagine pushing it, in-plane, at its top. It will want to lift up at one of the base corners, and push down at the other. The pushing down is typically not a problem, because the wall panel is presumably sitting on a solid foundation or other bearing wall. (When a shear wall is bearing on a beam or doubled floor joist, the pushing <u>can</u> be a problem, consult your engineer.) The uplift must be resisted in order to ensure stability of the wall panel. This is typically done via pre-manufactured holdown devises.



The holdowns must be positively connected to the edge framing (studs) of the panel, and also connected to the foundation or wall below. It should be evident that the holdown is only as good as it's connection to the edge framing of the shear panel, and it's connection to the foundation or wall framing below. So, follow the manufacturer's instructions and recommendations to the tee during all holdown installations.

Also, it is very important that the holdown be installed as close to the edge of the panel as possible. This is because the holdown was sized and designed for the uplift forces at the edges. As you move away from the panel edges toward the center of the panel, the uplift forces go way up – beyond the capacity of the holdown.

There are many sizes and styles of holdowns available; your engineer must be the one to determine the forces involved, and specify the correct holdown for each panel. But, as the builder, you should tell your engineer whether you prefer strap type or bolted type; commonly, either will work. It is interesting to note that there are many, many factors involved in selection of the proper holdown, including: panel height, panel width, dead load on the panel, openings adjacent to the panel, intersecting walls, whether or not there is a solid wall below, etc. Even shear panels in the same wall line can require different holdowns.

7.4.7. Distribution of Lateral Loads Throughout the Structure. Question often arises as to how much lateral load will go to the various shear walls and / or frames in a structure. I like to joke that when I design, I look at the plans, and have a little discussion with them. I like to tell each wall how much load will be apportioned to it during an earthquake or severe wind. Since the plans can't talk back, the discussion is usually short and sweet, and I get my way. The unfortunate reality is that no one knows for sure which shear walls or frames get how much lateral load (though a good engineer can make a decent guess). There are several mechanisms at work here:

7.4.7.1. Tributary Area. During a wind storm, it is assumed that the amount of wind sail area, i.e. the side exposure of walls and roofs, contributes directly to the lateral load on the structure. The more exposure, the more force that is applied. This is common sense. For example with a rectangular single story house without interior walls, when the wind is blowing to the north, half of the total wind load on the house will go to, and be resisted by the west exterior wall, and the other half will go to and be resisted by the east exterior wall (the walls parallel to the direction of the wind). And the roof diaphragm is assumed to disburse it there evenly. Of course, when the wind is blowing to the west, the roof diaphragm distributes the load evenly to the north and south exterior walls. This seems quite simple, and it is. But, what happens when we add a second or third story, and lots of angles to the walls, and a bunch of interior walls? Where exactly does the wind load go then? Well, we don't know for sure, but a reasonable approach is to apportion it relative to the tributary area of the exterior of the building and roof. This means that each shear wall used in design takes the amount of wind tributary to it's wind sail area on the exterior of the building. The engineer decides whether or not to use the interior walls parallel to the wind direction under consideration to resist the wind load.

What? Yes, it is true, the engineer decides which interior walls to use, if any, as shear walls. But, does the wind or earthquake care which walls the engineer selected? Nope, the lateral forces will be distributed based on the laws of physics, regardless of what the engineer has done. We can only hope that the engineer has predicted well.

It is absolutely true that any interior wall that is connected to the ceiling and floor will take some lateral load whether the engineer says so or not. The goal of most engineers is not to use interior walls as shear walls, and depend only on the exterior walls to take the lateral load. The interior walls in this case are not counted, but do of course, soak up some lateral load, thereby acting as a *factor of safety* to the whole lateral load resisting system. The interior walls are said to be *redundant*, because they are not counted in design, but actually will contribute to the structural system.

One of the main reasons that interior walls are not normally used in design has to do with load path. While it is generally not a problem to construct a gyp shear wall, it is sometimes difficult and expensive to positively connect one to the floor and / or roof diaphragms, ensuring proper load path. Many times there will be uplift forces at the ends of these walls, which must be resisted by some structural element below. So, rather than go through the trouble of resolving these load path issues via extra beams, footings, clips, holdowns, plywood, braces, etc., use of interior shear walls is avoided.

7.4.7.2. Tributary Weight. The amount of force delivered to a structure during an earthquake is directly dependant on the weight of the structure (and to some extent the weight of the contents of the structure). So, to determine how much earthquake force goes to which shear walls or frames, you must determine the amount of <u>weight</u> that is tributary to each. For example, with a single story rectangular house, during an earthquake that shakes in the north-south direction, the force arising from the weight of the roof and the weight of half the height of the walls will be distributed through the roof diaphragm to the east and west exterior shear walls. If there is snow on the roof, the force will be greater proportional to the amount of snow (the UBC does not require the full snow load to be included in earthquake design). Adding more stories and interior walls adds to the weight and thus seismic force, but in the end, each shear wall or frame must resist the lateral load based on the amount of weight tributary to it.

There are instances where a lot of weight may be supported by some structural system, say a beam, column and pad footing, but the beam, column, and footing are not designed to resist lateral load. What then? This is another instance where the lateral load tributary to that weight must be collected in a collector element (typically a beam or strut) and transferred positively (through connectors) to a shear wall or frame. Again, this is easily overlooked by the non-engineer, and if not addressed, will become a severe weakness in the structure when an earthquake occurs. The amount of load transferred through the collector or strut is typically considerable (thousands of pounds is common) and results in heavy duty collector / connection elements.

7.4.7.3. Stiffness. Generally speaking, in the case where structural elements of different material or stiffness are subjected to some type of loading (either lateral or gravity loads) the **stiffest elements always absorb the load first**. This means, for example, if you have a building with an interior masonry wall that extends from floor to roof, and the exterior walls are plywood shear walls, during a wind or earthquake, the lateral loads will go to the masonry wall before they go to the plywood walls - even if the masonry wall is not designed to withstand them. Masonry walls are much stiffer than plywood, thus will 'attract' lateral load first. Assuming that the building is somewhat symmetrical, it is likely the plywood shear walls may never even see any lateral load unless the masonry wall(s) crack first.

Another example of this could be where a particular exterior wall has a large section of shear wall, then a bunch of windows and doors, then a small shear wall. Assuming that the horizontal diaphragm (roof or floor) above is well connected to a continuous top plate, the small shear wall section will not likely ever see any lateral load. This is because it is not nearly as stiff as the large shear wall section at the other end of the wall, and the stiffer element will 'soak up' the load first. In order for the smaller wall segment to experience significant lateral loading, the stiffer wall must actually fail to some minor extent first and deflect, thus allowing a portion of load to transfer to the more flexible smaller element.

The point of all of this is that lateral load goes to the stiffest element first, regardless of whether the designer includes them in the load resisting system or not. This is particularly important where different types of construction materials (i.e. masonry and wood) are used, because the lateral loads may not wind up where they are assumed.

7.4.7.4. Horizontal Torsion. When viewing a building from the top (plan view), if you notice that any exterior side of the structure is much stiffer, or much more flexible than another, you have a potential problem with horizontal torsion. A stiff side could be one with a lot of shear wall area or stiff frames, and a weak or flexible side is one with many doors and / or windows without much in the way of lateral resistance. The problem arises when a lateral loading event occurs, the stiff side does not deflect much, while the flexible side deflects a lot. This tends to cause the building to twist (as viewed from the top). There are two problems here. First, this twisting adds shear force (due to moments resulting from the eccentricity between the center of mass and the center of rigidity) to the entire lateral load resisting system; and second, the large deflections on the weak / flexible side can cause damage or failure there that can 'unhinge' the whole structure.

The amount of additional shear load due to horizontal torsion depends on whether the horizontal diaphragm at the top of the story is flexible or not. Wood diaphragms are usually considered flexible, and concrete are not. The problem is worse with non-flexible (concrete) horizontal diaphragms. The UBC specifies an amount of additional lateral load to be added to the lateral resisting system to account for this, however the calculation of same is quite complex. The maximum amount of horizontal deflection (drift) is limited by the UBC to ensure that things don't come apart. See the next section.

In summary, where there is a 'soft' or weak side to a structure relative to a strong or stiff side, there will be horizontal torsion during a lateral load event. This will increase the shear loads and deflections, and must be considered in design.

7.4.8. Story Drift. During a wind or seismic event, there will be a certain amount of deflection of the structure sideways due to the lateral loads imparted. This is known as story drift. The UBC defines story drift as: *"…. Lateral displacement of one level relative to the level above or below."* In <u>earthquake</u> design, the maximum allowable story drift is either .025 or .020 times the story height, depending on the type of construction (or 'fundamental period' of the structure). So, for a three story structure (30' high, say) this means that the maximum drift at the top of the third story would be .025 x 30' x 12"/ft. = 9 inches. For a 30 story building, the maximum drift at the top would be about .020 x 300' x 12"/ft. = 72 inches or 6 feet! This is a lot of sideways movement, but considering that you are dealing with thousands of tons of steel and concrete and hundreds of thousands of pounds of sideways force, and a 300 foot tall building, 6 feet, relatively speaking, isn't that much.

For wind design, there is no UBC limitation on story drift per se. There are, however, numerous limitations on the amount of deflection for various structural elements. For example in the wood shear wall section there is a paragraph that reads *"… Permissible deflection shall be that deflection up to which the diaphragm and any attached distributing or resisting element will maintain its structural integrity under assumed load conditions, i.e. continue to support assumed loads without danger to occupants of the structure."* This simple paragraph says a mouthful about the true goal of lateral load design **– to make sure that things don't come apart and hurt someone during sideways movement.**

Many times during winds or earthquakes, there are failures, but as was mentioned previously, the main members (beams, columns, shear walls, etc.) didn't fail, the connections failed. And one of the main cause of connection failure is excessive deflections during sideways movement. When things move sideways dramatically, it becomes easy for beams to slip off of their posts and joist or rafters to come out of their hangers. It is these 'sneaky' types of failures that are the most prevalent, and wreak the most havoc. What is tragic is that nearly all of them are avoidable, if only sound engineering practices are adhered to during the design phase, and things are built per the plans and per the UBC.

7.4.9. Redundancy. One of the main reasons that many older, non-code compliant structures have been able to weather wind storms and mild seismic events is the redundancy that is 'accidentally' built into the lateral load resisting systems. *Redundancy refers primarily to interior walls or other secondary lateral load resisting elements that share in the resistance of lateral loads, whether they were intended to or not*. In general, the more redundancy you have in your structure, the safer it is in a wind or seismic event.

When I analyze a structure, I get particularly nervous when I see a structure that barely meets code and is wide open on the inside, i.e. has no interior walls – no redundancy. These types of structures depend wholly on the exterior walls or primary frames to resist all lateral loads. Calculations can tell us that these exterior walls or primary frames will do the job, but what if someone reads the plans wrong, or the steel frame welder is hung over? There is no redundancy or 'safety net' to make up for error during construction. Matters are compounded when inspection by the Architect, Engineer, or Building Official is minimized or omitted; as is so often the case.

To summarize, the safest structures are those which are highly redundant. Those which are not should be designed and constructed with extra care to ensure that the lateral load resisting systems function exactly as intended.

CHAPTER 8
FOUNDATIONS AND RETAINING WALLS

8.1. Soils. All foundations and retaining walls interact with soils, so a basic understanding of them is an important prerequisite to further study.

8.2. Soil Types. In general there are two basic types of soils: *cohesive* and *cohesionless*. In nature, however, it is common that these basic soil types are mixed together such that it becomes difficult to define exactly what type of soil or soils you have at the building site. This is one reason that a geotechnical investigation is important for every building project.

8.2.1. Cohesive Soils. These are generally non-granular, and will keep their shape if molded or remolded in the presence of a slight amount of water. These types of soils are comprised of clays or silts, though they may have sands and / or gravels mixed in with them. Cohesive soils are sensitive to changes in moisture content, i.e. they tend to shrink or swell depending on how much groundwater is present. This can be detrimental to foundations because shrinkage leads to settlement and swelling can lead to heaving.

The allowable bearing capacity (i.e. the capacity of the soil to support vertical loads from footings) for cohesive soils is generally not as good as for cohesionless soils. For example, in the absence of a geotechnical study, the UBC allows only 1,000 pounds per square foot (psf) allowable bearing capacity for clays, sandy clays, silty clays, and clayey silts (all cohesive), whereas sandy (cohesionless) soils are allowed 1,500 psf. If cohesive soils are kept dry, however, they can serve as excellent foundations for structures, with bearing capacities well over 2,000 psf. This is one reason that footing drains and tightlined downspout drains are so important in areas with cohesive soils (like some of the hillside areas of the Puget Sound region).

Clays and silts which have some moisture content and are heavily loaded will settle, but the amount of settlement, and the time it takes to settle are very difficult to predict. Generally clayey soils take much longer to settle (years is common) than granular soils. The amount of load applied, the amount of groundwater present, the relative amount of clay versus granular material, and the prevalence of other types of soil strata all dictate settlement rate and amount. Only a competent geotechnical consultant can make meaningful predictions of settlement in cohesive soils.

8.2.2. Cohesionless Soils. These are soils that will not maintain their shape when molded or remolded, regardless of their moisture content. They consist of sands and gravels. Engineers generally prefer cohesionless soils for bearing of footings because they have good bearing capacities and behave more predictably than clays or silts.

The bearing capacity of cohesionless soils depends on how tightly compacted (loose or dense) they are. Loose granular soils have a greatly reduced bearing capacity (as compared with tightly compacted), and they will settle as soon as substantial load is applied. So the goal for a good foundation in granular soils is to ensure that they are well compacted to begin with. Compaction can be done artificially via vibratory compactors, or the weight of the soil itself can be used. Geotechnical engineers sometimes require that entire building sites are *preloaded* – that is covered with several feet (usually 7 –10 feet high) of soil to compact the existing soil and cause most of the settlement to occur prior to the building being constructed.

Presence of groundwater will also reduce the bearing capacity of sands and granular soils. Generally, if the water table is at least the same distance below the footing as the width of the footing, there is no reduction in bearing capacity, but as the water table rises toward the bottom of the footing, the bearing capacity goes down. When the water table reaches the bottom of the footing, the allowable bearing capacity is about half of the dry case; and as the water table rises over the top of the footing, the bearing capacity continues to drop. This concept is very important if you are building in a sandy area that is prone to seasonal flooding or seasonal high groundwater table. You will want expert advise on sizing your footings.

8.3. Liquefaction. There is one property of cohesionless soils (particularly loose, uniformly graded, i.e. particle sizes are all about the same) that can pose a severe threat to a structure. It is called *liquefaction*, and occurs during an earthquake. Liquefaction is the settlement that occurs as a loose granular soil is shaken and remolds itself. Any structures that bear on such soils will settle along with the remolded soils – not good. There are several remedies for these types of soils, including vibrocompaction and piling.

8.4. Geotechnical Investigation. I always say that geotechnical design is as much an art as it is a science. I say this for two reasons: first, soils were made by the violent and turbulent forces of nature, and are rarely uniform or predictable; and second, you can't see worth a damn under the ground to know what you're dealing with. It continually amazes me the resistance I see to spending a little money up front for a good geotechnical investigation prior to construction of buildings. All you need to do is remember that your building is only as good as the foundation to help you realize that a geotechnical investigation is money well spent. Furthermore, if a foundation problem ever should occur, repair and fixing it will cost orders of magnitude more than simply checking it out initially and designing correctly the first time.

Of the remedial (structural damage and upgrade) work I do, a very high percentage has to do with foundation problems. The irony is that nearly every foundation failure is 100% preventable, if only a geotechnical investigation had been done first. So the bottom line is: <u>'Don't build anything that interacts structurally with soil unless you've had a geotechnical expert advise you first.'</u>

8.5. Organic Materials in Soil. Organic materials are anything that was or is living. Lots of times there are organic materials (leaves, grass, branches, roots, animals, etc.) mixed in with or buried in soils. The problem with organics is that they decompose; and when they do, they greatly reduce in size. So if you've built over anything organic, be prepared for future foundation settlement and problems. The best remedy is to remove all organic material in the first place.

I know of many settlement problems in the City of Burlington where old sloughs were filled with logs and other organic debris. Now, some years later, the organics are decomposing and the structures built thereon are going down. The fix involves piling, and is very expensive.

8.6. Footings. There are two basic types of concrete footings commonly used in residential and commercial construction: *continuous footings* and *pad footings*. Continuous footings are long and narrow and typically support walls. Pad footings are typically square or rectangular, and support point loads like posts or columns. The design of each is similar; basically, the bottom of the footing is made large enough to support the live and dead loads to be placed thereon. Some of the more important points to remember in <u>all</u> footing design follow:

- The base of the footing must be large enough to support all intended live and dead loads. This directly depends on the bearing capacity of the soil. The higher the bearing capacity – the smaller the footing may be. Minimum continuous footing sizes are shown in Table 18-1-C of the UBC for up to three story wood framed buildings. A geotechnical report should be completed for the sizing of all footings, regardless of minimum UBC requirements.
- If uplift is anticipated (such as footings for steel frames in pre-manufactured metal buildings), there must be enough concrete weight in the footing to keep the footing down during a severe wind event. Soil weight and the weight of other attached footings may also be used to resist uplift, but only if specifically included in the design.
- Reinforcement must be present in the footing to ensure that all bending stresses (either intended or accidental) are resisted.
- Continuous footings must have at least 1 continuous rebar in the bottom (minimum of 2 is recommended) and one in the top; and there should be vertical rebar at 24" maximum spacing extending from the bottom of the footing into and through the height of the stem wall. Heavily stressed footings and footings with heavy holdowns (uplift forces) will have more rebar, as specified by the engineer.
- Pad footings must have a grid of rebar toward the bottom of the pad, and if there is any uplift expected, there must also be a rebar mat in the top portion (sometimes a single grid in the middle will do if the footing is thick enough). Reinforcement size and spacing depends on loading magnitude, concrete strength, soil bearing capacity, whether moment loads are present or not, and required development length.
- Pad footings must be thick enough to resist punching shear from the point loads of column / post bases.
- There must be adequate drainage installed at the base of footings to keep groundwater away.
- Continuous footings on slopes must be stepped so that the bottom of the footing is horizontal. The only exception to this is if footings are installed on sloped bedrock. In this case, rebar dowels should be installed to anchor the footing to the rock and to ensure that no sliding of the footing occurs.
- Anchor bolts into treated sill plates should be minimum 5/8" diameter (*required* in seismic zone 4), and must be embedded at least 7" into the concrete stem wall or

footing. Maximum spacing of anchor bolts is recommended at 4' (6' is required), and at least two anchor bolts are required per sill plate. In seismic zones 3 and 4 (western Washington included) all anchor bolts through sill plates must be equipped with washers of minimum size 2" x 2" x 3/16".

8.7. Retaining Walls. Any vertical or near vertical wall that holds back earth is a retaining wall. There are several different types, and they behave structurally differently. Retaining walls are probably the most universally misunderstood structural element that I see in my practice.

8.7.1. Forces on Retaining Walls. The main force that retaining walls must resist is the lateral or sideways force that the retained soil applies. It is assumed in design that this force starts out at zero at the top of the wall, and increases uniformly with depth to a maximum at the bottom of the wall. This is called a *triangular force distribution*. For example, if you have a 10 foot high wall, and examine a 1 foot strip of wall, the triangular lateral force distribution starts at zero at the top of the wall, and is about 400 pounds at the bottom. Remember, this is a sideways force that is pushing on the wall trying to slide it and tip it over. The actual force depends on the soil type and other factors that will be discussed in the following sections.

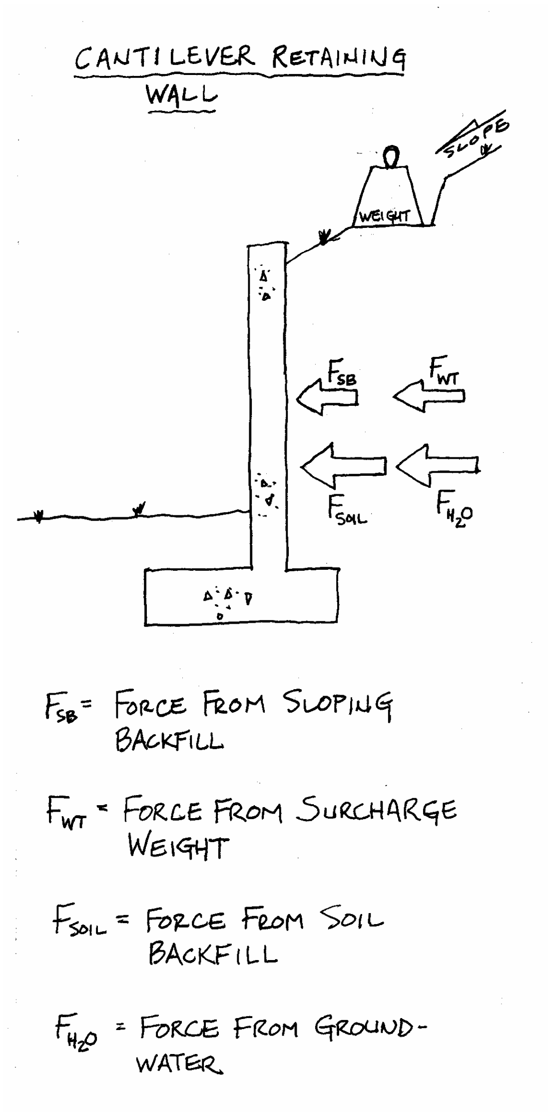
8.7.2. Groundwater. If groundwater is present, the lateral force exerted on the wall *is greatly increased*. This is why nearly all retaining walls have 'behind wall drains' installed that intercept the groundwater and route it away from the backside of the wall. Groundwater also can reduce the bearing capacity of the soil under the footing portion of the retaining wall. For this reason, the 'behind wall drain' should be installed just below the level of the bottom of the footing.

Groundwater not only increases the lateral force on the back of the wall, it has a way of finding chinks or cracks in the wall, and seeping through. This is a definite problem if your retaining wall is a basement wall (occupied space on the non-retained side). This is another reason for a good behind-wall drain system. I have been involved as an expert witness in a law suit where this very problem was the issue. Unfortunately for the contractor, he did just about everything wrong in the construction of the wall. It leaked, and was structurally unsound. The fix (which he was responsible for) cost on the order of $50,000.

8.7.3. Surcharge. If a retaining wall is built in such a location that a vehicle can drive on the retained earth within about 10' or less of the wall, the weight of vehicle will add to the lateral load that the wall must resist. This is called *surcharge loading*. Of course, any heavy gravity (vertical) load that can be applied to the retained soil within about 10' (horizontally) of the wall becomes a surcharge load. Sometimes houses or garages are built next to one another such that one has a retaining wall which will be influenced by surcharge loading from the other's footing. This situation is particularly of concern, because the surcharge load is permanent as opposed to intermittent as from a vehicle.

In general, the closer the load is to the wall, the greater resultant lateral load is applied to the wall; and the heavier the gravity load – the more lateral load on the wall. So, if you ever have a retaining wall designed, make sure to tell the engineer whether or not there will be any surcharge load.

8.7.4. Sloped Backfill. If the retained soil is sloping upward from the wall, greater lateral force is exerted on the wall: the steeper the slope, the higher the force.



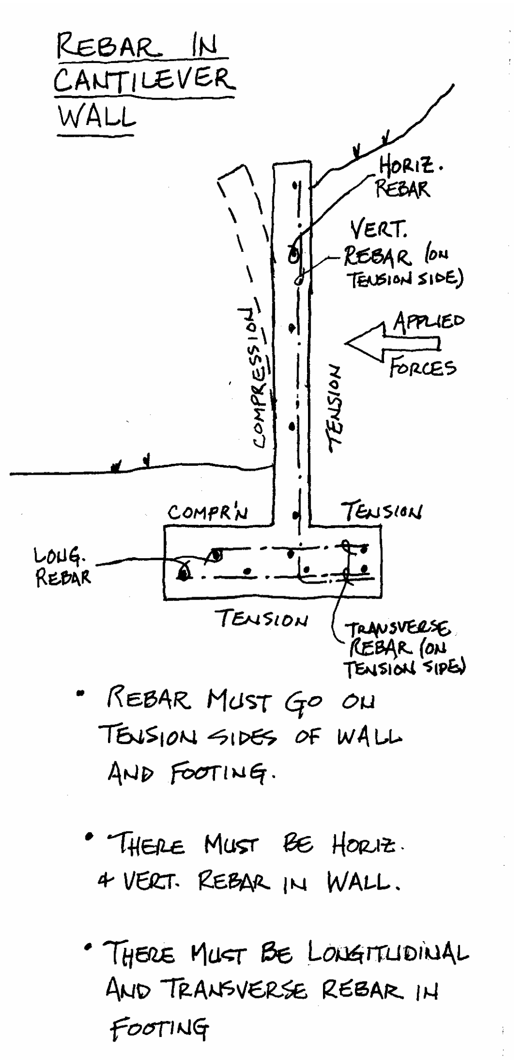
8.7.5. Earthquake. It is believed that lateral loads on retaining walls are increased during earthquake ground motion. The UBC, however, does not specifically require additional load on retaining walls due to earthquake; your building department may though. To be conservative, it is a good idea to include them regardless. My experience is that lateral soil loading will increase approximately 25% - 50% if moderate earthquake loading is included.

8.7.6. Weight on Top of the Wall. If the retaining wall acts to support a building, slab, or other structure, the additional gravity load must be included in the design. It is interesting that this load may actually help hold the wall against sliding and overturning; but it will increase the load on the soil beneath the footing. If the load is not applied directly to the middle of the top of wall (i.e., an eccentricity is introduced between the load and the center of the wall) it will induce additional moment in the wall that the wall itself must resist.

8.7.7. Types of Retaining Walls. There are several types of retaining walls commonly used. Each is classified by it's structural mechanism in resisting lateral earth pressure. Following is a discussion of the most common.

8.7.7.1. Cantilever Retaining Wall. This is a concrete or masonry wall that has a large, wide concrete footing connected with heavy rebar to the wall section. <u>There is no other structural element at the top of the wall to resist the sideways pushing of the earth</u>. The lateral force of the earth is resisted entirely by the moment connection of the wall to the footing. With this type of wall, the footing must be heavily reinforced in the correct places, as must the wall portion. It is critical that the engineer and contractor understand how these walls function so that the <u>rebar is placed on the *tension* sides of the wall and footing</u>. If this is not done correctly, the wall is pretty well guaranteed to crack and likely ultimately fail (a **very** expensive proposition to correct).

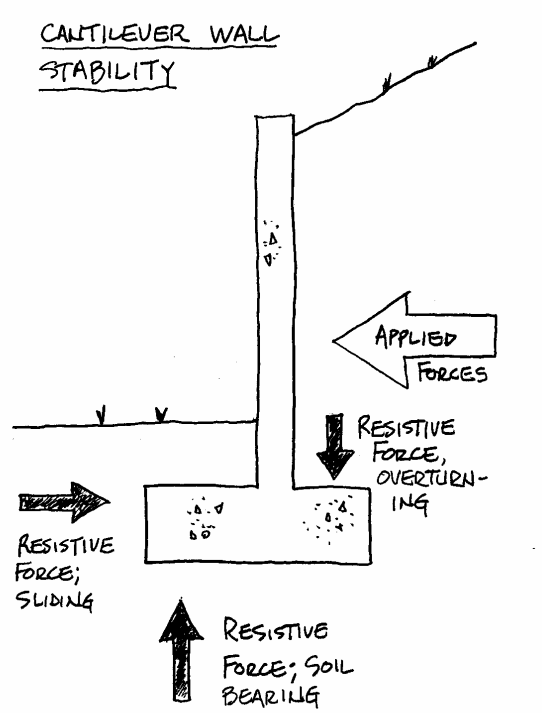
Cantilever walls are so named because the wall portion is fixed at it's base and 'cantilevers' upward. That is, there is no other lateral support at the top of the wall.



8.7.7.1.1. Overturning. Cantilever walls resist overturning primarily by having a heel portion of the footing extend back into the retained earth. The weight of the earth over the heel keeps the wall from tipping over. Also, the front of the footing (toe) extends outward from the wall providing additional stability against overturning. The UBC requires a factor of safety of 1.5 against overturning, meaning that there must be 50% again as much resistance to overturning as calculations strictly require.

8.7.7.1.2. Sliding. Cantilever walls must have either a generous depth of soil in front of them (in front of the footing on the non-retained side), or they must have a 'key' built into the bottom of the footing that extends down into the soil below the footing. The soil in front of the footing or in front of the key provides *passive soil resistance* against the concrete to keep it from sliding. The UBC requires a factor of safety of 1.5 against sliding, meaning that there must be 50% again as much resistance to sliding as calculations strictly require.

8.7.7.1.3. Soil Bearing Pressure. As lateral load is applied to a cantilever wall, it tends to tip over and rotate away from the retained soil. Because the connection between the footing and wall is fixed, this rotation is translated to a downward force applied to the soil at the toe of the footing and upward force at the heel of the



footing. We've discussed that the upward force is resisted by the weight of the soil piled over the heel. At the toe, however, the soil bearing capacity under the footing must be adequate to withstand the downward force. If the soil under the footing (toe) is bad for bearing, the footing will need to be much wider horizontally to spread the downward force over a larger area of soil.

Here is an excellent reason that a geotechnical report is in order for all retaining walls. Without one, you are gambling that the soil is strong enough to take this downward load. Further, if you *do* have a geotechnical report, you may find that the soil is quite good, which will directly *reduce* the required width of your footing, thus saving money.

8.7.7.1.4. Efficiency. There is a practical limit to the height of a cantilever retaining wall before a different type of wall begins to make more sense economically. It is my opinion that about 15' high of retained earth is the maximum height for efficiency. The problem with taller cantilever walls is that the footing size gets so large, and the wall needs to be so thick, that costs get out of hand. A good rule of thumb for estimating purposes is to assume that the footing width will be about 2/3 the wall height. So if you have a 12' high wall, figure that the footing will be about 12' x 0.67 = 8' wide. You can also assume that any wall over 10' high will be at least 10" wide with two layers of rebar, and could be wider depending on loading conditions and height. Shorter walls may also be surprisingly stout if there is groundwater, surcharge loads, or upsloping backfill.

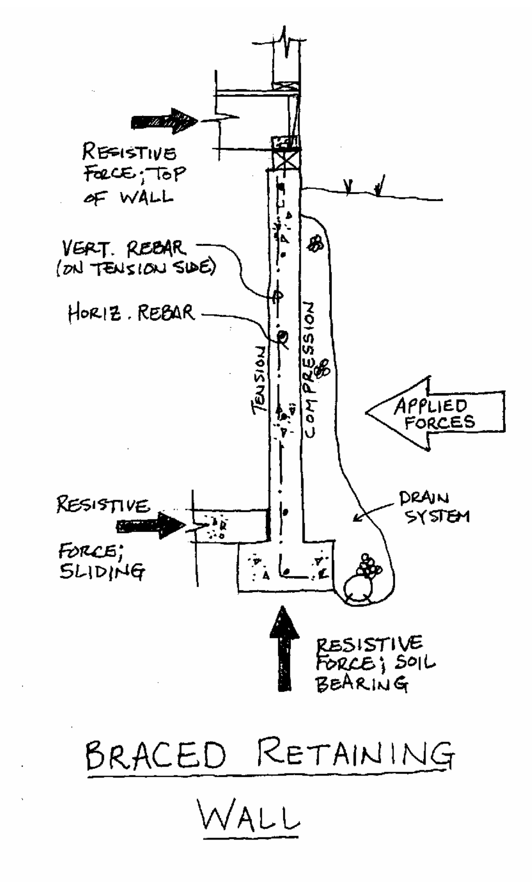
It is also noteworthy that a masonry cantilever wall gets very expensive with increase in height. The reason for this is that masonry is considerably weaker in out-of-plane bending than concrete, particularly if special inspection is not provided (without special inspection, engineers must use only 1/2 of the allowable strength of masonry in design).

8.7.7.2. Braced (or Propped) Retaining Walls. A braced (or propped) retaining wall looks similar to a cantilever retaining wall, except that the rebar is located differently within the wall and footing, and there is lateral support provided at the top of the wall. This is important enough to restate as follows. *There are two major structural differences between a braced wall and a cantilever wall*:

- <u>The tension side of a braced wall is on the non-retained side, while the tension side of a cantilever wall is on the retained earth side</u>. This is a huge difference structurally. Why? Because if you do not put the rebar toward the tension face of the wall, the wall will most likely crack and fail.
- <u>Secondly, a braced wall requires a positive connection at the top of the wall which must be strong enough (and have proper load path) to resist the entire inward force of the wall top.</u> This usually means that the top of the wall is positively connected to a floor diaphragm, which holds it from moving inward. A cantilever wall does not require this.

8.7.7.2.1. Overturning. Overturning is not a problem with a braced wall because the brace nullifies the overturning force. Beware however, that the inward force at the top of wall, which is transferred to some other structural element (floor diaphragm) must have a positive load path through proper connections all the way back down to the ground.

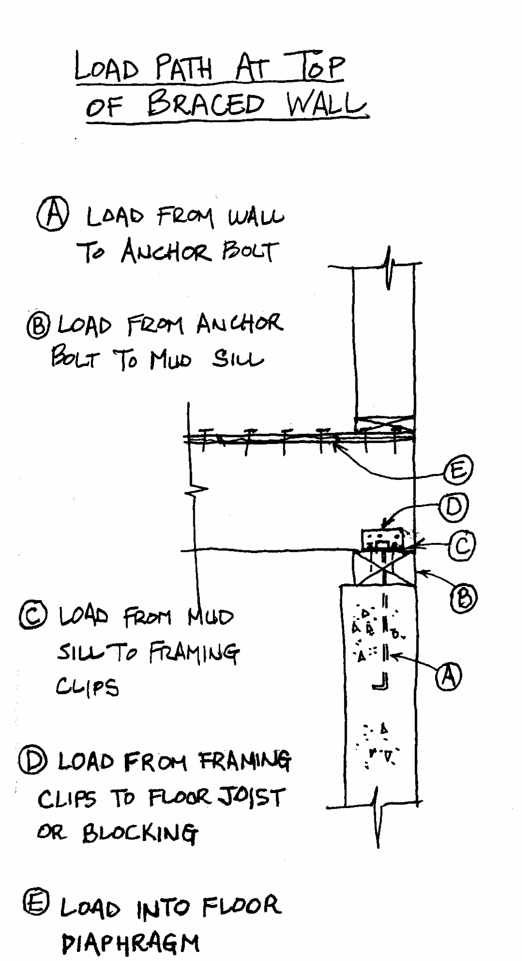
8.7.7.2.2. Sliding. Sliding is a major concern with a braced wall. It is generally resisted in the same way as a cantilever wall; with soil (or a slab) in front of the footing.



Braced walls very frequently have basement or floor slabs poured against them on the non-retained side, which normally provides ample resistance to sliding. So a ‘key’ or deep footing is not typically required.

8.7.7.2.3. Soil Bearing Pressure. Because there is no overturning to be resisted with a braced wall, the only thing the soil under the footing needs to resist is the weight of the wall and the structure it supports. This is usually not a problem, with the result being a modestly sized footing. A geotechnical report is highly recommended to ensure good performance and to minimize footing size if soils are good.

8.7.7.2.4. Wall Connection at the Top. A crucial element to any braced wall is it’s top connection to the lateral bracing system. Typically this lateral bracing system at the wall top is a floor diaphragm. If the floor diaphragm is wood joists and plywood subfloor, the critical connections are between the concrete wall top and the mud sill (assuming mud sill type of wall to floor connection, not ledger), and between the mud sill and the floor joist.



As an example, assume we have a 10’ high basement wall. The inward force at the top will be about 575 pounds per lineal foot of wall (assuming no surcharge, hydrostatic, or sloping backfill). If the floor joist are spaced at 16” OC, the inward force that must be transferred to each joist is about 770 pounds! Further, there must be enough anchor bolts to transfer this load from the concrete wall into the mud sill. If 5/8” anchor bolts are used, a 4x mud sill with anchor bolts at about 12” OC is required (don’t even think about trying to use a 2x mud sill – won't calc). If the floor joist are perpendicular to the wall, we need to provide framing clips to each joist that have a 770 pound capacity. This would need to be two Simpson L50's per joist (heavy duty). Pre-drilling of the nail holes is likely necessary to avoid splitting of the joist.

What if the floor joist are parallel to the retaining wall? The inward force of 550 pounds per foot of wall is still there and must be resisted. The same 4x mud sill and 5/8" anchor bolts at 12" OC is necessary. The best way to get the force from the mud sill to the floor diaphragm (plywood) is to install full depth blocking between the joist extending from the mud sill into the room at least between three joists. Blocking should be at least 16" centers along the mud sill. The blocks on top of the mud sill must be connected with the same L50's as used earlier. All of the blocks must be nailed to the plywood floor diaphragm to adequately transfer the load, say with 10d's at 4" spacing.

As can be seen from this example, the connection at the top of wall with a braced type of retaining wall is heavy-duty. Yet it is probably the most neglected detail in all retaining wall construction. Question arises as to why more basement walls don't therefore fail. Part of the answer normally has to do with redundancy. Contractors accidentally build in lateral load resisting elements that are frequently enough to keep the walls together. The above example assumes that no other mechanisms contribute to taking lateral load except the top and bottom of the wall. The bottom of the wall is assumed 'pinned' to the footing, i.e. no moment is assumed transferred between the wall and footing. It is through this pinned connection that lateral load is shifted to the top of the wall. In reality though, the connection of the footing to the wall actually is fixed to some degree; that is, it takes some moment. This will act to reduce the inward load at the top of the wall a certain amount, and crank in an overturning moment to the footing. Frequently, there is enough cumulative resistance to the lateral earth load that things hold together.

Two other important redundancies that keep braced walls from failing are corners in the wall, and intersecting perpendicular walls. These act as *counterforts*.

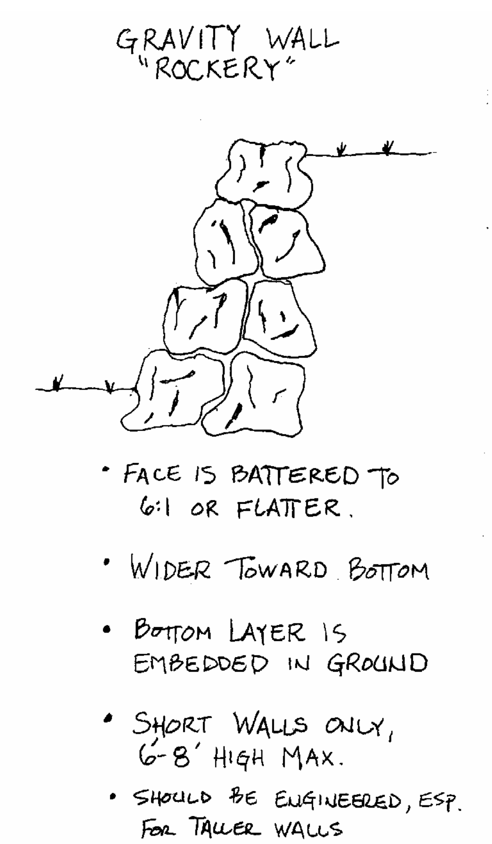
Regardless of these redundancies, it is much better to design and construct your braced walls such that you know where the load is going, and that there are structural elements present to resist the load. That way, you don't have to gamble on 'accidental' structural elements to provide redundant strength.

8.7.7.2.5. Efficiency. A braced wall is always more cost effective than a cantilever wall because the footing is smaller, and there may be less rebar. This 'reward' comes from the fact that another structural element (the brace at the wall top) is taking a big chunk of the lateral load, rather than the wall having to take it all itself.

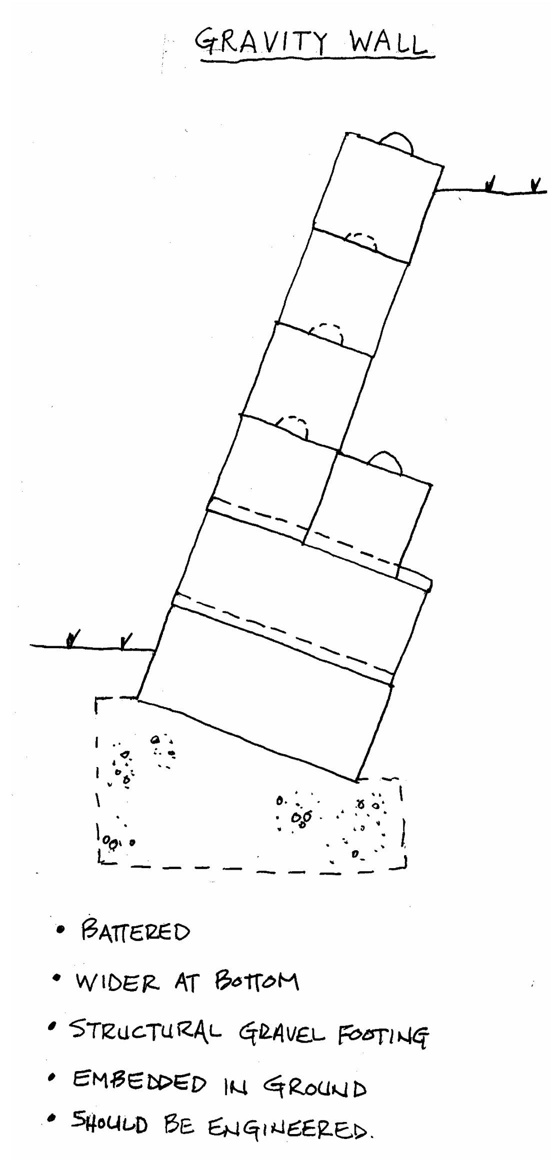
There is a practical limit to the height of a braced wall, probably somewhere between 15 to 20 feet. As braced walls get into this 'tall' range, there becomes a tremendous inward force at the top of the wall. It becomes difficult to transfer this high sideways force into wood framing members efficiently. Another problem with tall braced walls is the out-of-plane bending stresses in the middle section of the wall. The wall will need to be quite thick and heavily reinforced to resist these bending stresses as the height goes much beyond 15' or so.

8.7.7.3. Gravity Walls. A gravity wall is different altogether than a cantilever or braced wall. It depends on it's own weight, and nothing else to resist lateral earth forces (hence the name 'gravity' wall). For this reason, gravity walls are much more massive than cantilever or braced walls. Gravity walls are commonly made from the following:

- large concrete blocks (typically weighing 4,000 – 5,000 pounds each)
- gabions (wire baskets filled with small rock)
- large natural rocks (sometimes called a 'rockery').



In my experience, gravity walls are not normally engineered. Some think that if you pile a bunch of concrete or rock vertically, it will magically hold back any amount of earth. Of course, there is no magic involved. A gravity wall, just like a cantilever or braced wall, must be designed properly such that the earth forces are resisted by the weight of the wall, including a factor of safety of 1.5 (i.e. be 50% more stable than calculations show).



8.7.7.3.1. Overturning. The lateral earth forces on the back of the wall (which, as we've previously learned, will be exacerbated by surcharge loads and / or upsloping retained earth) tend to try to push the wall over. Simply put, the wall must be massive enough to resist this sideways earth loading. In my experience, non-engineered gravity walls, particularly rockeries and ecology block walls, rarely meet this criteria. One of the 'tricks' that engineers use in gravity wall design is to have the wall *battered* somewhat. Battered means that the wall itself slopes backward – into the retained earth a small amount. Typical batter ranges from 1:10 for short walls to 1:4 for taller walls. That is 1 unit horizontal to 10 or 4 units vertical respectively. If you ever see a rockery or ecology block retaining wall that is over 2' high and is not battered, you are looking at a problem waiting to happen, particularly if surcharge loading or sloped backfills are involved.

Question arises as to why so many of these 'potential problems' that you see out and about have not started to tip or lean. The most probable answer is that most natural soils have some cohesion to them, i.e. they contain a certain amount of clay, and are not purely granular in nature. This cohesion acts as glue within the soil, tending to keep it molded at a steep slope which is beyond it's real long term stability. But as the years go by, gravity has a way of overcoming this cohesion, relentlessly pulling at the soil. So, improperly designed rockeries or ecology block walls may hold for a few years, but give them time. Mother nature will catch up and have her way with them in the end.

It is also interesting to note how a poorly designed retaining wall fails. The outward lean usually occurs very gradually, i.e. it may take several years for a wall to develop significant lean. It may continue to lean and lean, never seeming to actually lay all the way over. This is due to the soil continually molding and remolding itself behind the wall on it's slow steady mission to gain internal stability. Many types of common soils are stable at angles ranging from 30 to 40 degrees to the horizontal. Unstable (overly steep) soil will continue to push sideways on anything in its way until it has reached it's own angle of internal stability (known as *angle of repose*). If there is a weak or light wall in the way, that structure will go along for the (slow) ride.

8.7.7.3.2. Sliding. Gravity walls must have resistance to sliding at their base just like all types of retaining walls. Because they are so wide horizontally at the base, and they are so heavy, sliding is generally not a problem; though it still must be examined in design. A good way to help ensure no sliding occurs is to embed the lowest layer of the wall a foot or more into the ground – well below the topsoil layer. This also helps to ensure that the soil the wall is bearing on is competent (topsoil frequently has a lot of organic material in it – not good).

Another sliding scenario that must be examined is the sliding that could occur between the individual blocks or rocks of the wall itself. It is certainly possible that a wall could come apart somewhere in it's middle if the base were adequately anchored, but there was insufficient resistance to sliding among the individual wall units.

8.7.7.3.3. Soil Bearing Under the Wall. Gravity walls weigh a lot. So, intuitively it is easy to understand that the soil under them has a lot of downward stress to resist. Couple this with the overturning forces at the toe of the wall (acts similar to a cantilever wall), and you may well have a problem in soil bearing. This must be carefully examined by the engineer during design. The basic remedy for overstressed or weak soils is to make the foundation row wide horizontally. This is another reason that a single wythe of ecology blocks or quarry rocks stacked vertically more than about 4' high will likely have problems. They will likely overstress the soil under the 'toe' in bearing as the retained soil applies lateral (tipping) pressure. Once a gravity wall starts to tip over, more and more pressure is delivered to the soil under the toe region, starting a failure cycle that could ultimately result in a downed wall.

8.7.7.3.4. Efficiency. It is my experience that gravity walls are efficient and cost effective for heights up to about 12 feet (maybe 15 feet with no surcharge load or upsloping backfill). Beyond this, the batter required for stability becomes excessive, and / or the base courses become inefficiently wide.

8.7.7.4. Reinforced Earth Retaining Walls. Where very tall retaining walls are called for, a reinforced earth wall is sometimes specified. These are constructed with a relatively thin outer face layer (ungrouted masonry units, or treated wood are common) which is connected at various levels up the wall to a strong geotextile fabric or structural mesh extending back into the retained earth several feet. Construction logistics requires that there is enough room behind the wall face for heavy equipment to place, spread, and compact the select fill material between the structural mesh layers as the wall goes up.

These types of walls are similar to gravity walls in that the reinforced earth behind the wall tends to act as a giant uniform mass which, as a whole, resists overturning and sliding.

There are various proprietary types of reinforced earth walls in use today. Several such systems use a masonry unit face pegged together with fiberglass dowels (no grout or rebar), and a high tensile strength polyethylene, polypropylene, or polyester grid blanket that hooks to the dowels and extends back into the earth behind the wall.

8.7.7.5. Cantilevered Pole Retaining Walls. Certain retaining walls are built by embedding poles into the ground deeply and cantilevering their ends into the air. A siding material is then attached to the wall to hold back retained earth. These types of walls are typically efficient only to relatively low heights, say 6 feet or so. Any taller, and the required post embedment becomes excessive.

The main problem with these types of walls is that the soil around the embedded posts does not offer much resistance to outward thrust unless the burial depth is considerable. A minimum of 4' embedment is recommended for short walls (4' and less in height), and as the height of the retaining wall goes up, the embedment depth gets greater; to a point where the embedment depth is more than the retained height. The actual depth of embedment depends on the soil type, post spacing, retained height, post hole diameter, and other factors. Corrosion or rotting of the posts is also a common problem.

8.7.7.6. Tie-Back or Bulkhead Retaining Walls. Perhaps the most efficient type of retaining wall for tall retained heights (over 15' or so) is the tie-back or bulkhead wall. These consist of a relatively thin facing material (sheet piling, or wood or steel vertical 'soldier' beams with wood or steel horizontal 'wale' beams, or a combination thereof) connected to tie-back rods that extend into the retained earth. The tie-back rods are installed typically by drilling horizontally or slightly below horizontal and then being grouted permanently within the drill hole. The number and spacing of tie-back rods is as determined by the engineer. If soil conditions allow, tie-backs can be of the helical screw anchor variety, which are literally screwed 10 or more feet into the soil with heavy machinery.

The design of a good tie-back system is expensive and complicated and depends on a thorough investigation of the retained soil.

Installation of tie-back rods is quite expensive because it involves horizontal drilling and pressure grouting (or specialized machinery in the case of helical anchors). The cost of this type of wall typically starts to make economic sense when retained heights exceed 15’ or so. For very tall walls (much greater than 15’), this is normally the most economical method. A drawback to this type of wall, however can be the final appearance of the wall itself. For this reason, some tall retaining walls are the masonry faced reinforced earth type, even though the cost may be greater.