INTRODUCTION TO STEEL DESIGN,
TENSILE STEEL MEMBERS
MODES OF FAILURE & EFFECTIVE AREAS

MORGAN STATE UNIVERSITY

SCHOOL OF ARCHITECTURE AND PLANNING

LECTURE I

Dr. Jason E. Charalambides

AVANT-GARDE ENGINEERING LLC
CONSULTING ARCHITECTS/ENGINEERS

WHY USE STEEL?

- This is an element that is offering its great natural characteristics to designers to show their level of understanding of materials for construction:

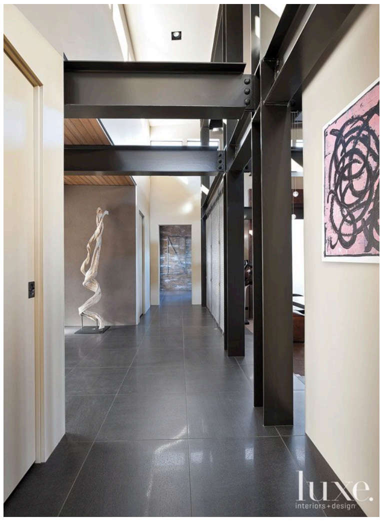
- The Farnsworth House. Architect: Mies van der Rohe, Chicago, IL 1951
- The Car Pavilion for Ben Rose. Architect: David Haid, Highland Park, IL 1974

Photo Credits: Left: http://solarcities.blogspot.com/2014/04/when-i-came-to-attention-ofnational.html
Right: courtesy Nikki A. Johnson Available in:
http://www.flickr.com/photos/8977254@N08/3618374222

WHY USE STEEL?

- ☐ Steel can be graceful if well designed

The roof spaceframe and central column will need constant attention. Note the mist, which made cleaning difficult.



WHY USE STEEL?

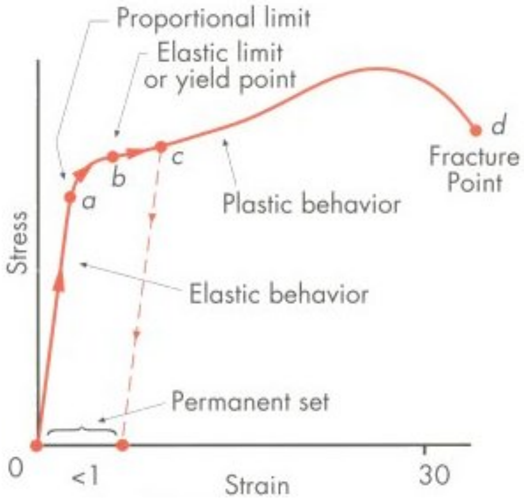
- ☐ Steel can let you get by designing something like this too!!! Structure of Panama's new Museum of Biodiversity!

Photo Credits: http://richardinpanama.com/2014/04/27/did-a-bomb-go-off/

Why Use Steel?

So let's get more technical here

- Main advantages:
  - Strength
  - Homogeny
  - Elasticity
  - Ductility
  - Speed of erection
  - Defined set of forms (dimensions)
  - Adaptability
  - Longevity
  - Simplicity
  - Quality control
  - Scrap value



Why Not?

- Main disadvantages:
  - Corrosion
  - Fireproofing
  - Susceptibility to buckling
  - Fatigue
  - Brittle fracture*

*A rapid propagation of cracks through a material that usually occurs so rapidly that allows no chance for plastic deformation to take place before fracture. Brittle fracture usually causes a failure in structural integrity. Due the lack of any warning signs between the start of failure and full rupture, given the rapid process, it shall likely lead to catastrophic failure.

Temperature, Section thickness, Stress concentration, Rate of loading, and material strength are factors that affect the ductile/brittle behavior of structural elements.

Factors Affecting Steel Strength

- Temperature
- Corrosion
- Element thickness (increasing brittleness – less ductile)
- Multiaxial stress induced by welding
- Dynamic loading altering the stress/strain properties (earthquakes etc.)

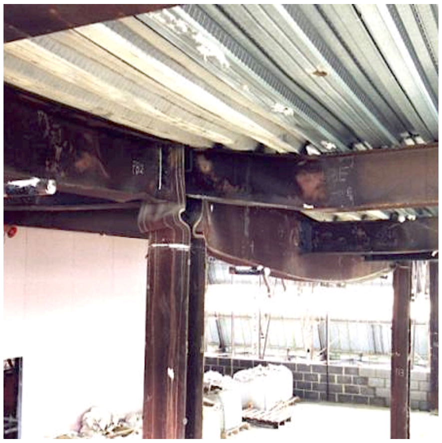


Photo: Column squashing in British Steel Test 2: Plane frame test
Credit: http://911research.wtc7.net/mirrors/guardian2/fire/cardington.htm

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Factors Affecting Steel Strength

- Lamellar tearing (tearing along the rolling axis)*
- Fatigue – a progressive failure caused by cyclic loading – not necessarily exceeding the yield stress – governed by:
  - Number of cycles of loading
  - Range of service stress (max stress - min stress)
  - The initial size of a flaw (discontinuity, like an extremely small crack or a bolt hole)

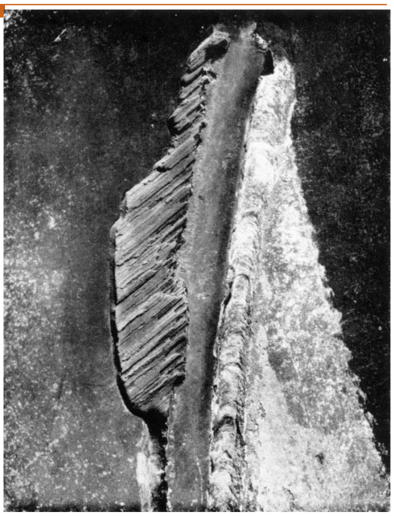

Photo: Failure of double fillet welded joint between vane and back plate disclosing lamellar tearing
Credit: http://products.asminternational.org/fach/data/fullDisplay.do?database=faco&record=1665&trim=false

WHAT CONSTITUTES STRUCTURAL FAILURE?

- A Structure fails when it does not do what it is intended to do.
  - (Definition of unsuccessful design!)
    - Fracture
    - Yielding
    - Buckling
    - Connection failure
    - Excessive displacement
    - Vibration

- Causes of failure:
    - Wrong estimation of loads
    - Mistakes in Analysis of elements
    - Connection failures
    - Imprecise processes during construction phase

Photo: From the 7.0 Richter scale earthquake in Haiti.
Credit: http://eqclearinghouse.org/co/20100112-haiti/general-information/fierro-photos-part/attachment/dsc_0112_2_resize

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Photo: From the 7.0 Richter scale earthquake in Haiti.
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THE SYSTEM FOR DESIGN AND ANALYSIS - LRFD

- The abbreviation stands for Load and Resistance Factor Design
  - System developed much earlier and implemented in late 80s replacing the Allowable Stress Design (ASD) that came back again in 2005 integrated in the same AISC Steel Construction Manual.

Nominal Loads

D = deal load
L = live load
L_R = roof live load
S = snow load
R = rain load
W = wind load
E = earthquake load

LRFD Load Combinations

1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_R \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_R \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.5W)$
4. $1.2D + 1.0W + 0.5L + 0.5(L_R \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + 0.5L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

- For simplification, in this class, combination #2 is a default pg. 2-10.

Exception: The load factor on *L* in load combinations 3, 4, and 5 shall equal 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf

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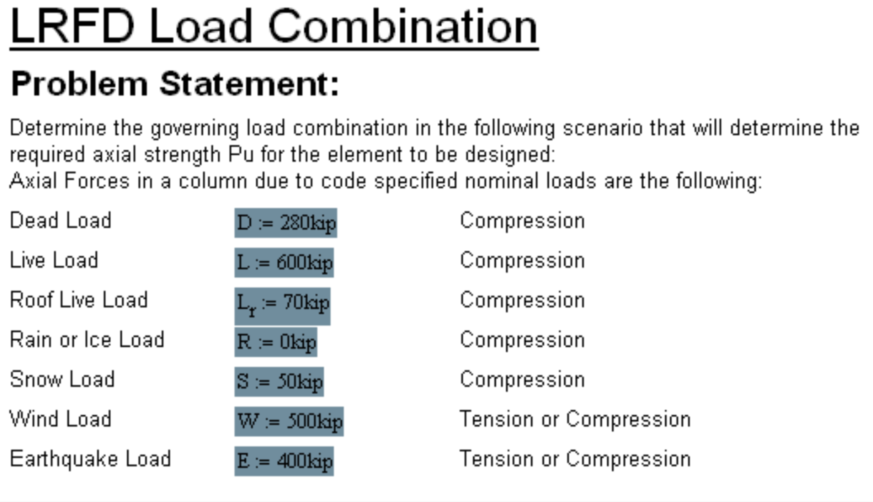
CALCULATING THE LOADS

- The main reference is the ASCE / SEI 7:
  - It provides the necessary information for determining basic loads other than the Dead Load.
  - Dead Load is easy to determine based on quantities and specific gravity of each material.
    - e.g. $\gamma_{concrete} \approx 150$ pcf

THE SYSTEM FOR DESIGN AND ANALYSIS – LRFD

IN CLASS EXAMPLE



LRFD Load Combination

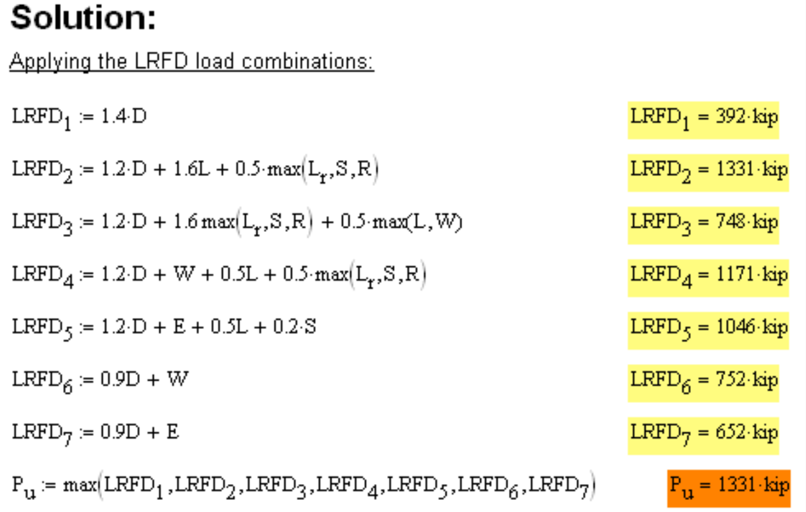
Problem Statement:

Determine the governing load combination in the following scenario that will determine the required axial strength Pu for the element to be designed:
Axial Forces in a column due to code specified nominal loads are the following:

Dead Load	$D := 280\text{kip}$	Compression
Live Load	$L := 600\text{kip}$	Compression
Roof Live Load	$L_r := 70\text{kip}$	Compression
Rain or Ice Load	$R := 0\text{kip}$	Compression
Snow Load	$S := 50\text{kip}$	Compression
Wind Load	$W := 500\text{kip}$	Tension or Compression
Earthquake Load	$E := 400\text{kip}$	Tension or Compression

THE SYSTEM FOR DESIGN AND ANALYSIS – LRFD

IN CLASS EXAMPLE



Solution:

Applying the LRFD load combinations:

$LRFD_1 := 1.4 \cdot D$ $\quad LRFD_1 = 392 \cdot kip$

$LRFD_2 := 1.2 \cdot D + 1.6L + 0.5 \cdot max(L_r, S, R)$ $\quad LRFD_2 = 1331 \cdot kip$

$LRFD_3 := 1.2 \cdot D + 1.6\,max(L_r, S, R) + 0.5 \cdot max(L, W)$ $\quad LRFD_3 = 748 \cdot kip$

$LRFD_4 := 1.2 \cdot D + W + 0.5L + 0.5 \cdot max(L_r, S, R)$ $\quad LRFD_4 = 1171 \cdot kip$

$LRFD_5 := 1.2 \cdot D + E + 0.5L + 0.2 \cdot S$ $\quad LRFD_5 = 1046 \cdot kip$

$LRFD_6 := 0.9D + W$ $\quad LRFD_6 = 752 \cdot kip$

$LRFD_7 := 0.9D + E$ $\quad LRFD_7 = 652 \cdot kip$

$P_u := max(LRFD_1, LRFD_2, LRFD_3, LRFD_4, LRFD_5, LRFD_6, LRFD_7)$ $\quad P_u = 1331 \cdot kip$

LRFD VS ASD

- The abbreviation stands for Load and Resistance Factor Design
  - System developed much earlier and implemented in late 80s replacing the Allowable Stress* Design (ASD) that came back again in 2005 integrated in the same AISC Steel Construction Manual.
  - In LRFD, the margin of safety is better tailored to different types of loads and load combinations.
  - The LRFD Margin of safety more realistically reflects levels of uncertainty associated with different load conditions, e.g. a larger load factor is used for live loads than for dead loads, based on the fact that there is greater uncertainty in live loads.
  - Overall one can assume that a more integrated building structure can be generated through the use of LRFD over ASD.

    - LRFD is the chosen system to be used during this course.

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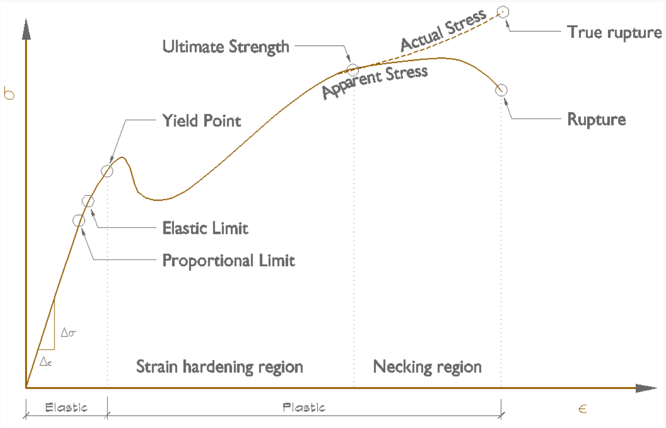
* Strength is used instead of Stress since 2005.

REVIEW OF MATERIAL PROPERTIES

- A typical Stress/Strain curve for structural Steel

$$\sigma = \frac{P}{A}$$

$$\varepsilon = \frac{L}{L_0}$$



Note: In the region between elastic limit and yield point,
the material deforms plastically, but on a smaller scale.

REVIEW OF MATERIAL PROPERTIES CONT.

- Strength
  - Fy = yield stress - Maximum stress in Elastic region
  - Fu = ultimate tensile strength - Maximum stress prior to fracture
- Stiffness
  - A constant factor on calculating the stiffness of Steel is its Young's modulus of Elasticity (stress "σ" divided by strain "ε"): E=29,000 ksi

$$E=\frac{\sigma}{\varepsilon} \qquad \sigma=\frac{P}{A} \qquad \varepsilon=\frac{L}{L_0}$$

REVIEW OF MATERIAL PROPERTIES CONT.

- Strain at different ranges of behavior
  - $\varepsilon_y=\frac{F_y}{E}$ - ranging between 1% and 2% *(strain at yield pt.)*
  - ε_{sh} - ranging between 1% and 3% *(strain at hardening region)*
  - ε_u - ranging between 10% and 20% *(strain at ultimate stress pt.)*
  - $\varepsilon_{fracture}$ - ranging between 20% and 30% *(strain at fracture)*

REVIEW OF MATERIAL PROPERTIES CONT.

- Typical Grades of Steel:
  - Structural steels are identified by ASTM designations
  - For Applicable ASTM Specifications for Rolled Shapes and Plates and Bars please refer to Table 2-4 and Table 2-5 in the AISC Users Manual pp. 2-48 – 49.
  - Common structural steels:

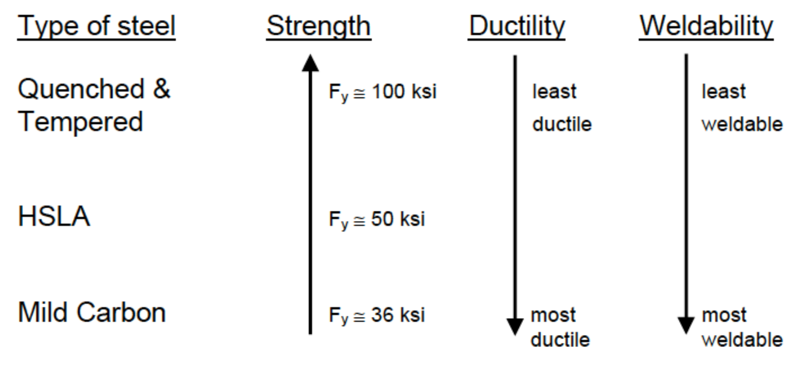
A36	Fy = 36 ksi *	Fu = 58 ksi (plates & shapes)	8 in thickness maximum
A572	Fy = 50 ksi	Fu = 65 ksi (shapes)	
	Fy = 42 ksi	Fu = 60 ksi (plates)	
A992	Fy = 50 ksi	Fu = 65 ksi	

  - A992 is an enhanced version of A572 Gr 50, currently available only for rolled wide-flange shapes
  - AISC Approved Steels: see AISC Spec. Section A3.1(p16.1-6)

* 32ksi if >8 inches

REVIEW OF MATERIAL PROPERTIES CONT.

- General Trends in Steel Properties:



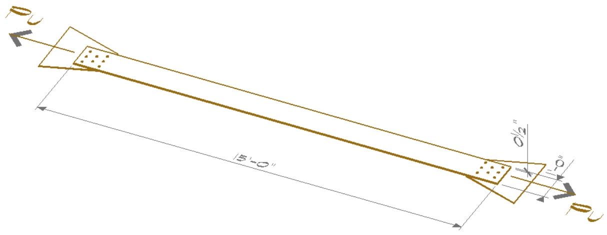
Type of steel	Strength	Ductility	Weldability
Quenched & Tempered	$F_y \cong 100$ ksi	least ductile	least weldable
HSLA	$F_y \cong 50$ ksi		
Mild Carbon	$F_y \cong 36$ ksi	most ductile	most weldable

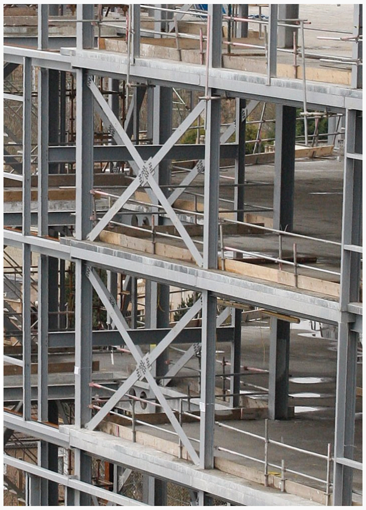
TENSION MEMBERS

- **Limit States:** (Modes of Failure)

  There are three ways that axially loaded steel members in tension can fail:

  - Yield of Gross Area
  - Fracture of Net Area
  - Block Shear Fracture

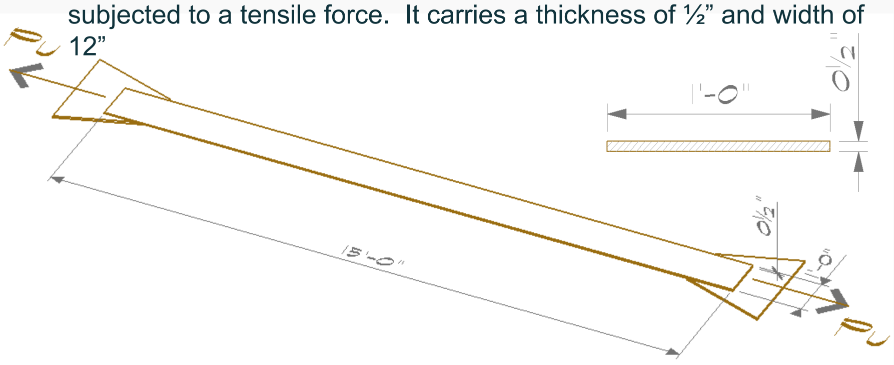




TENSION MEMBERS
SHORT EXAMPLE

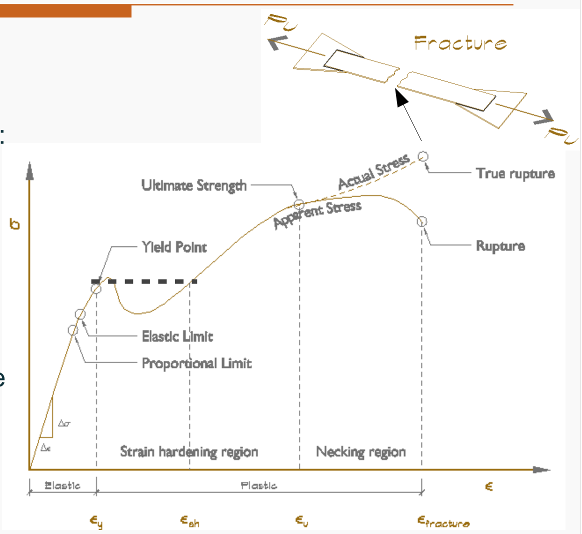
1) Yield of Gross Area: $\varphi R_n = \varphi * A_g * F_y$ (J4-1)

- This occurs when the cross sectional area of the steel member yields and deforms within its plastic region
- Let's consider a 15' long plate of A-572 steel, welded to gussets and subjected to a tensile force. It carries a thickness of ½" and width of 12"



TENSION MEMBERS
SHORT EXAMPLE

- Stress/Strain curve used for a member in Tension.
- Let's assume the following based on average strain values:
  - $\varepsilon_y = 0.001$
  - $\varepsilon_{sh} = 0.02$
  - $\varepsilon_u = 0.15$
  - $\varepsilon_{fr} = 0.25$
- Let's measure the elongation that this element will experience at the different stages of material behavior:



TENSION MEMBERS
SHORT EXAMPLE

- Result of applied stress:
- Due to strain we shall observe the following elongations at the given points:
- $L_{fin} = L_0 + \Delta L = L_0 + (\varepsilon * L_0)\ OR\ (1+\varepsilon) * L_0$
  - @ Yield point: $L_y = 1.001 * 180^{inch} = 180.18^{inch}$ $\Delta L_y = 0.18^{inch}$
  - @ Strain hardening: $L_{sh} = 1.02 * 180^{inch} = 183.6^{inch}$ $\Delta L_{sh} = 3.6^{inch}$
  - @ Ultimate strain: $L_u = 180^{inch} * 1.15 = 207^{inch}$ $\Delta L_u = 27^{inch}$
  - @ Fracture point: $L_{fr} = 180^{inch} * 1.25 = 225^{inch}$ $\Delta L_{fr} = 45^{inch}$
- The peak capacity of this element could be given by: $\cancel{P_u = A_g * F_u}$
  - (Where the Gross Area is subjected to the ultimate stress)
- To achieve this peak capacity in tension this element reaches an elongation of 27” which shall be excessive in almost any scenario!

TENSION MEMBERS
SHORT EXAMPLE

- Result of applied stress:
- The lower limit of this condition is at the Yield point, hence the term “Yield of Gross Area.”
- Given that the peak load in this condition would be at Fu not Fy, it has already been established that the Fu will be practically unachievable due to the excessive deformation, the above formula is the one used.
- Thus Failure is at the point of loss of stiffness due to yield of gross section and at the nominal axial tensile strength Pn as expressed:

$$\boxed{\varphi R_n = 0.9 * A_g * F_y} \quad \text{(J4-1)}$$

- Thus, the capacity of this element in yield of gross area is:

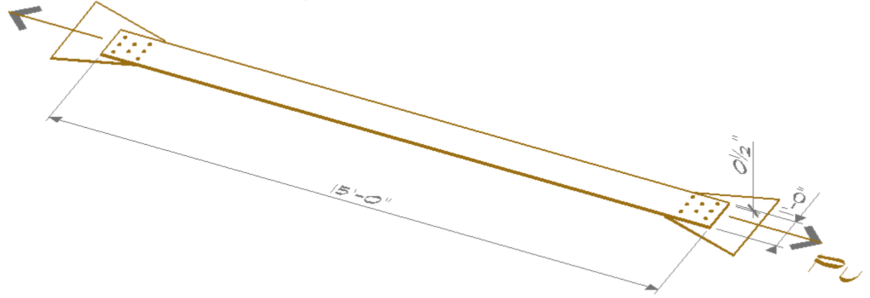
$$\underline{\varphi R_n = 0.9 * (0.5^{inch} * 12^{inch}) * 42^{ksi} \approx 227^{kip}}$$

Note: For tensile yielding the safety factor ϕ=0.90

TENSION MEMBERS
SHORT EXAMPLE

2) Fracture of Effective Net Area: $\varphi R_n = \varphi * A_e * F_u$ (J4-2)

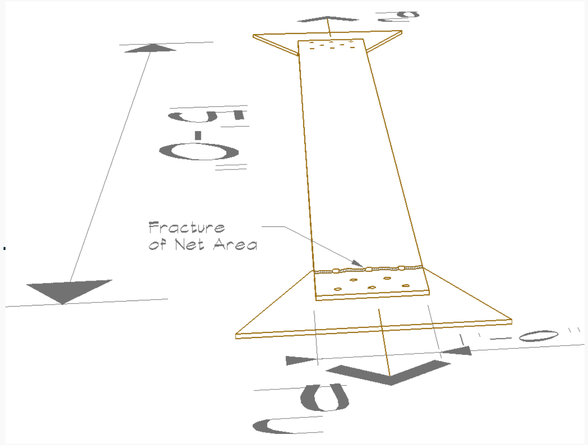
- This occurs when the cross sectional area of the steel member fractures between the points where bolts are located
- The element shall not fracture where there are no bolt holes. Nature always finds the easiest and most comfortable locations for solid materials to fail. They shall not fail at the second weakest point.



Note: For tensile rupture the safety factor ϕ=0.75

TENSION MEMBERS
SHORT EXAMPLE

- Yet, the aforementioned scenario is not to focus on, especially in case there are many bolts set in a row.
- Instead of the bolts shearing and tearing through the steel one by one, it is possible that the plate shall tear exactly there where the least area is holding against the stress, i.e. the line with the most bolts and thus the least net area withstanding the tension
- That is the effect of "Fracture of Effective Net Area."



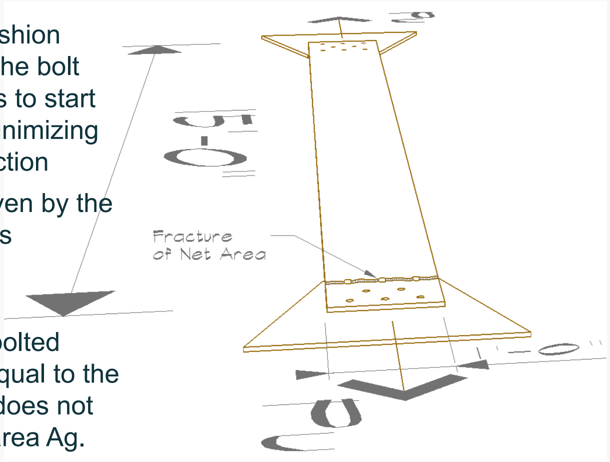
$$\boxed{\varphi R_n = 0.75 * A_e * F_u}$$ (J4-2)

TENSION MEMBERS
SHORT EXAMPLE

- Fracture of Net Area:
  - This failure occurs in the fashion indicated in the diagram. The bolt holes offer ideal weak spots to start the fracture, significantly minimizing the strength of the steel section
  - The effective net area is given by the following formula where U is essentially 100%

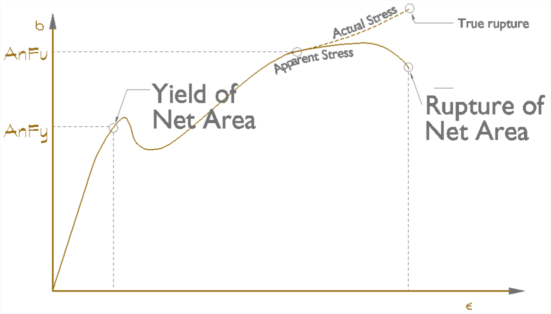
    $$A_e = A_n * U$$
  - The effective area"Ae" for bolted connections is practically equal to the net area "An" as long as it does not surpass 85% of the gross area Ag.

    $$A_e = A_n \leq 0.85 * A_g$$



TENSION MEMBERS
SHORT EXAMPLE

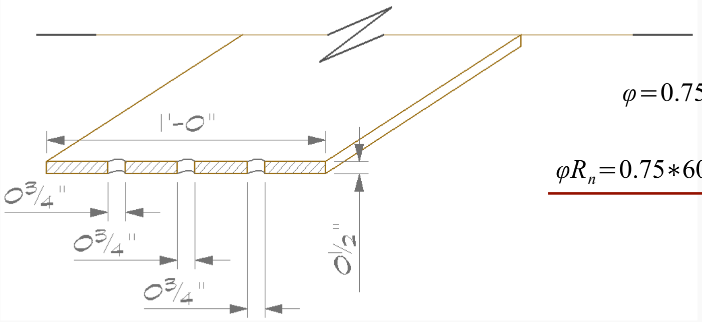
- Fracture of Net Area:
  - The elongation that may occur at the yield point will take place in a very short section, e.g. 1” or so. Therefore the effect will be minimal if “ε” is about 0.15, i.e. Δ=1”*0.15=0.15”, ..close to negligible
  - Even at the point of rupture, the deformation shall not be very significant. The actual failure will be the fracture



TENSION MEMBERS
SHORT EXAMPLE

- The Net Area is the area seen in the diagram. The effect of the “Shear Lag coefficient” U is inconsequential in most cases given a value of “1” (see Table D3.1) of the AISC User's manual.
- For the given scenario the following can be applied:

$$A_n = t*w - 3*d*t = 0.5^{inch}*12^{inch} - 3*0.75^{inch}*0.5^{inch} = 4.875^{inch^2}$$



$$\varphi = 0.75$$

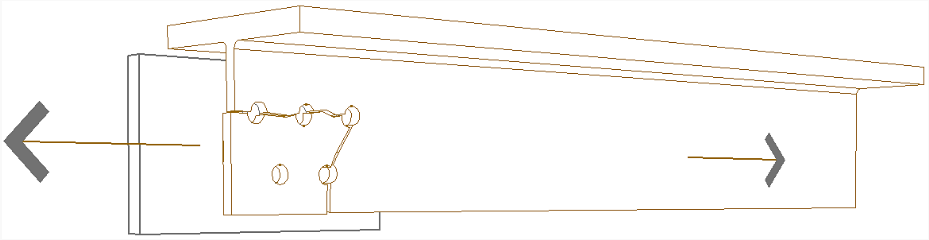
$$\varphi R_n = 0.75*60^{ksi}*4.875^{inch^2} \approx 219^{kip}$$

TENSION MEMBERS
SHORT EXAMPLE

3) Block Shear fracture:

$$\varphi R_n = \varphi * 0.6 * F_u * A_{nv} + U_{bs} * F_u * A_{nt} \leq \varphi * 0.6 * F_y * A_{gv} + U_{bs} * F_u * A_{nt} \qquad \text{(J4-5)}$$

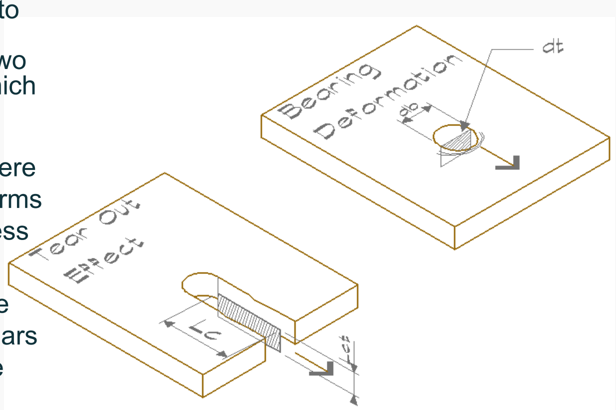
- This is a more complicated scenario and it shall be addressed during the next lecture.



Note: For Block Shear rupture the safety factor ϕ=0.75

BEARING STRENGTH AT BOLT HOLES

- By addressing the effect of stress transferred from bolts to plates that are not strong enough to resist the stress, two main effects occur, one of which may lead to another.
  - Bearing deformation, where the plate yields and deforms to accommodate the stress transferred
  - Tear out effect, where the plate can not hold and tears out whilst the bolt ant the element it is attached on move through the plate.



BEARING STRENGTH AT BOLT HOLES

- The available bearing strength, φRn at bolt holes shall be determined for the limit state of bearing as follows:
- (a) For a bolt in a connection with standard, oversized, and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force:
  - (i) When deformation at the bolt hole at service load is a design consideration

$$\varphi R_n = 0.9\, L_c * t * F_u \le 2.4\, d * t * F_u \qquad \text{(J3-6a)}$$

  - (ii) When deformation at the bolt hole at service load is not a design consideration

$$\varphi R_n = 1.125\, L_c * t * F_u \le 3.0\, d * t * F_u \qquad \text{(J3-6b)}$$

where

- d = nominal bolt diameter.
- Fu = specified minimum tensile strength of the connected material.
- Lc = distance between the edge of the hole and edge of the material.
- t = thickness of connected material.

Note: factor ϕ=0.75 has already been incorporated in the formulae above.

BEARING STRENGTH AT BOLT HOLES

- (b) For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force:

$$\varphi R_n = 0.75 * L_c * t * F_u \le 2.0\, d * t * F_u \qquad \text{(J3-6c)}$$

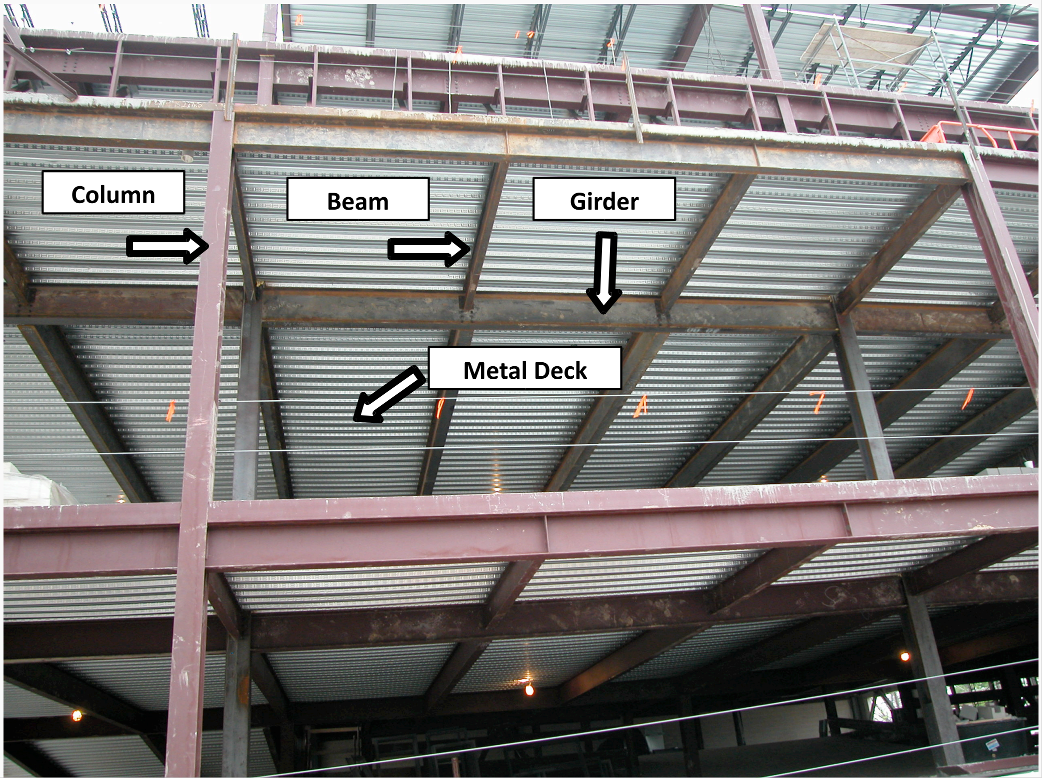
- (c) For connections made using bolts that pass completely through an unstiffened box member or HSS, see Section J7.

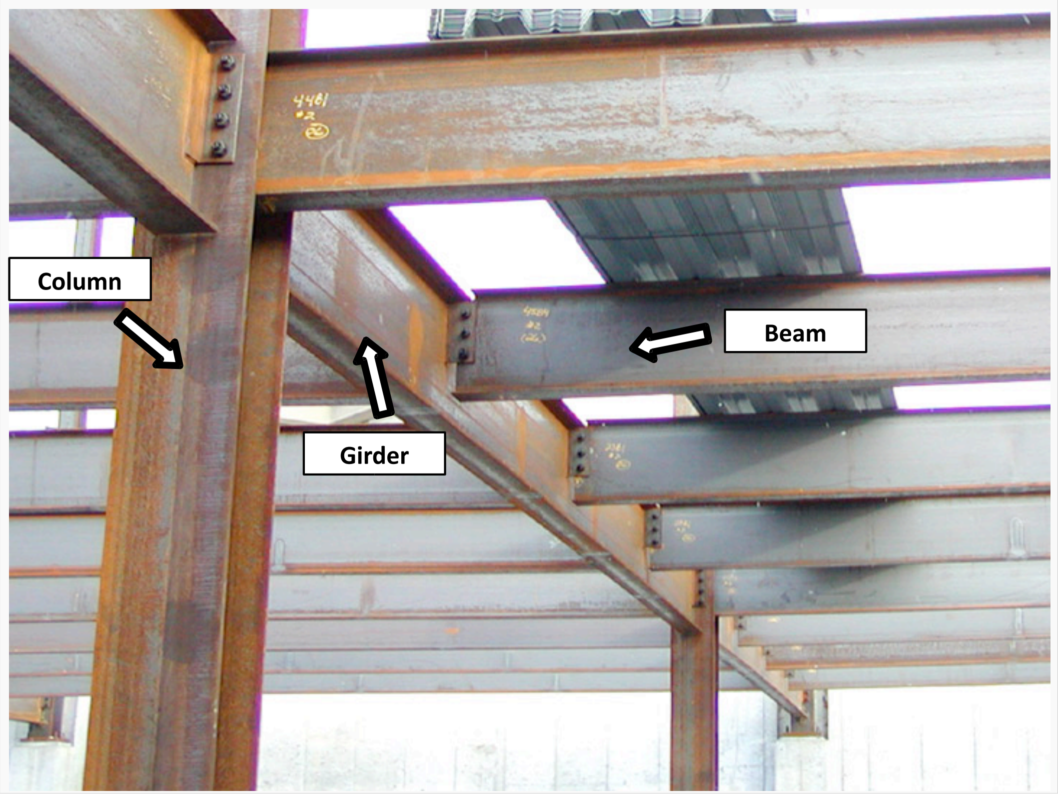
$$\varphi R_n = 1.35 * F_y * A_{pb} \qquad \text{(J7-1)}$$

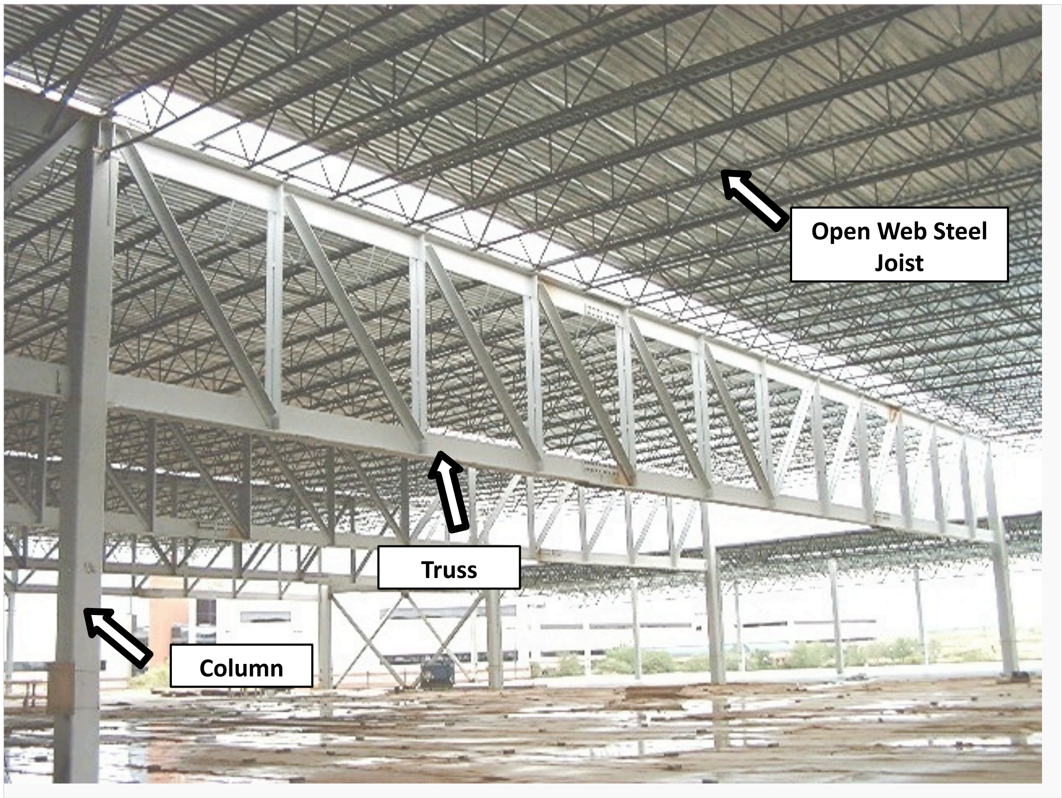
- For connections, the bearing resistance shall be taken as the sum of the bearing resistances of the individual bolts.
  - Where
  - Apb = Projected area in bearing.
  - Fy = specified minimum yield stress.

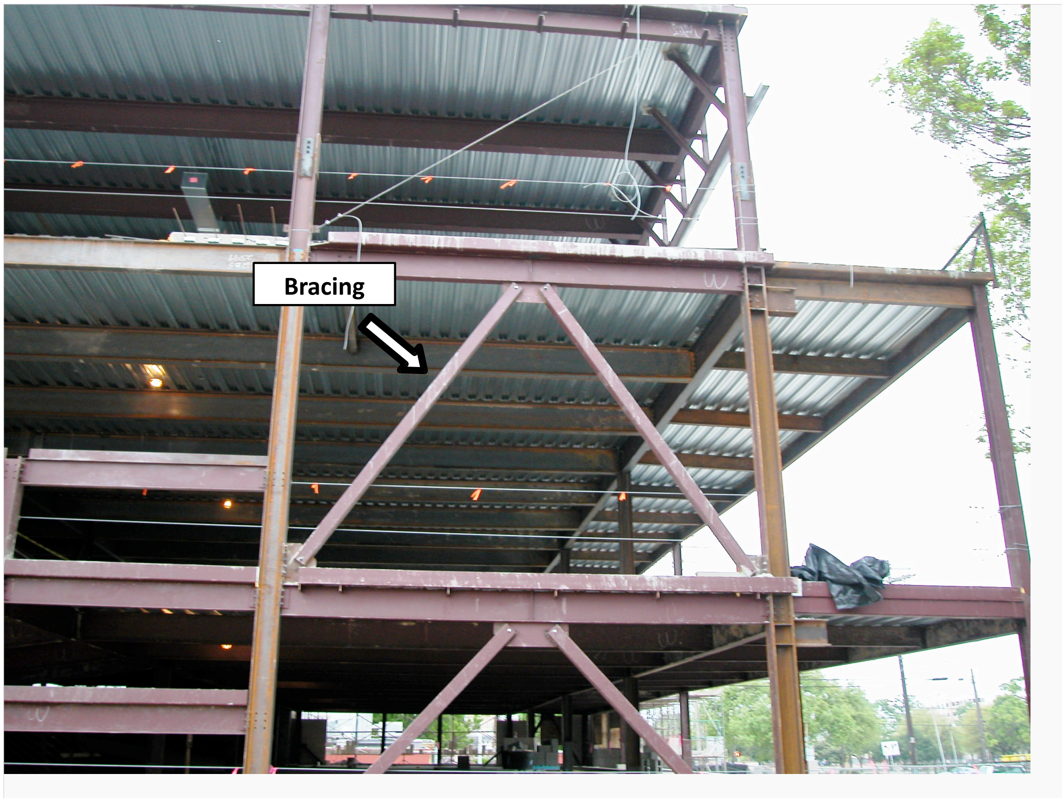
TERMINOLOGY AND CONSTRUCTION

- The following slides are provided as introduction to the terminology applied to different parts and elements used for the design and construction of Steel Structures,
  - Courtesy of Dr. M. Engelhardt
    - Dept. of Architectural, Civil, and Environmental Engineering at the University of Texas at Austin

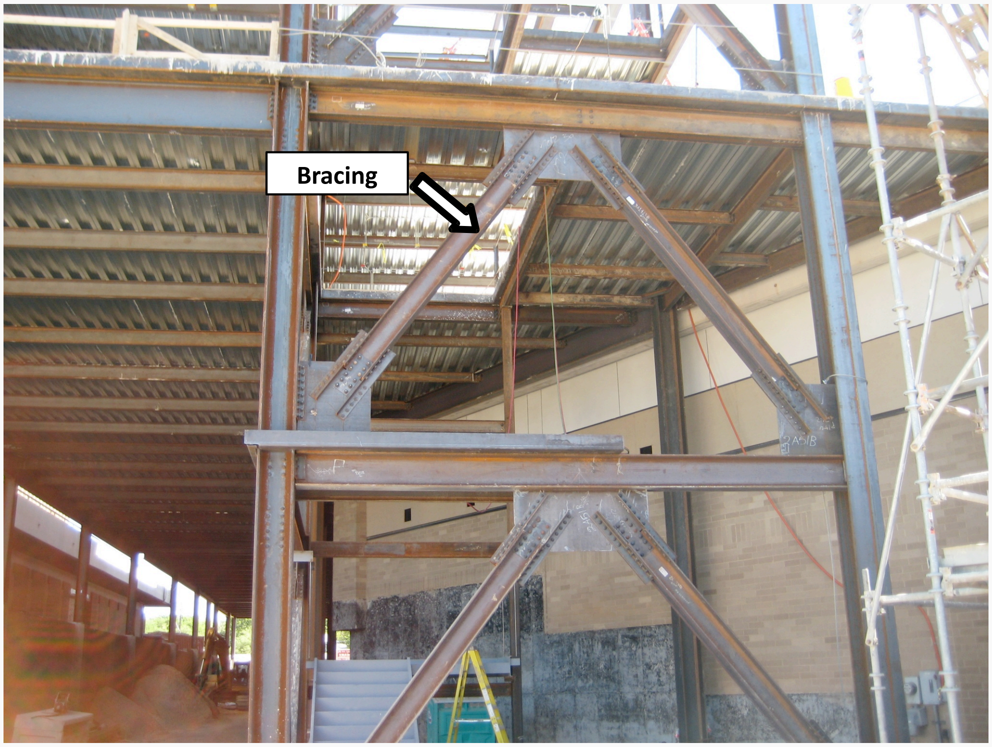


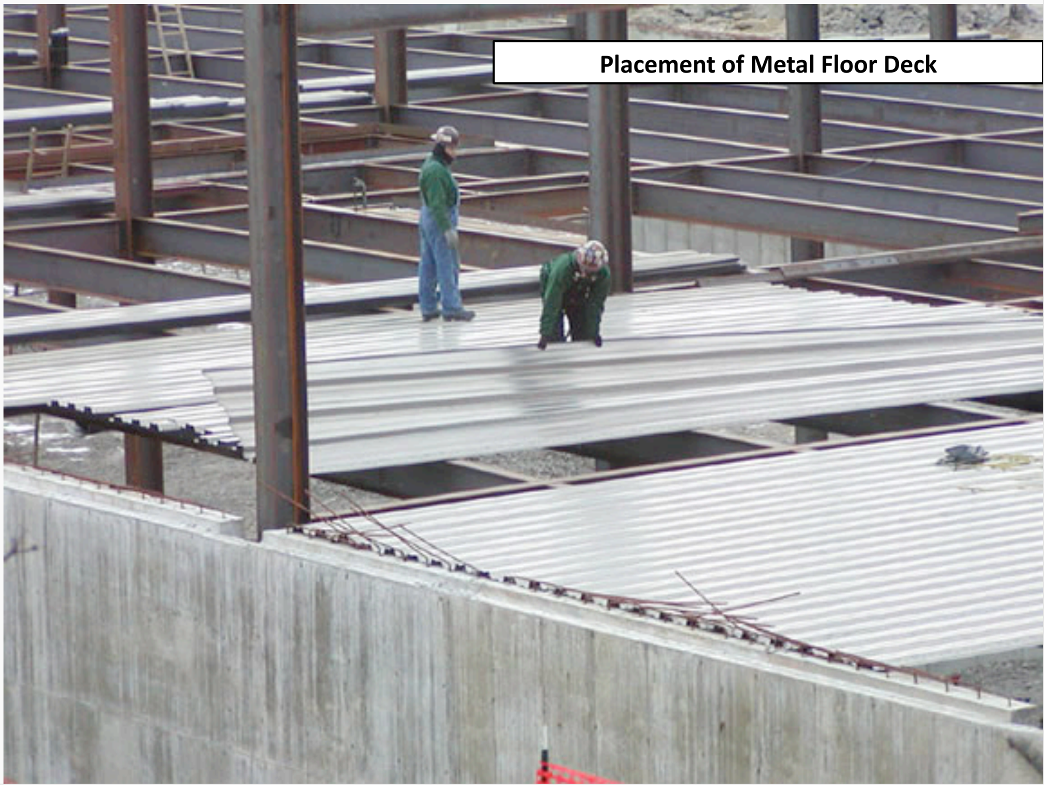

Column
Girder
Beam


Open Web Steel Joist
Truss
Column


Bracing

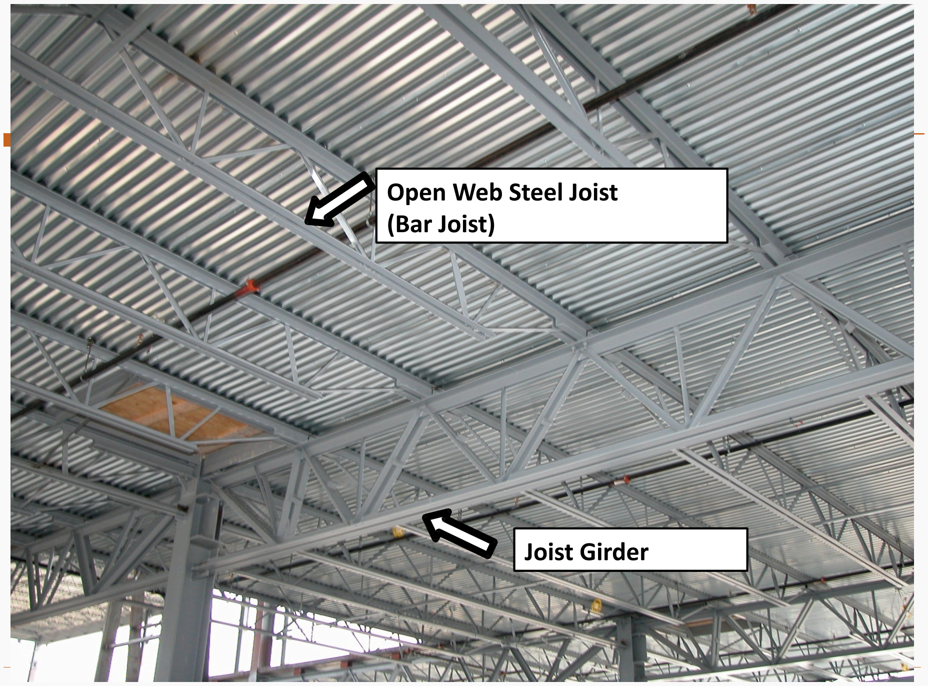

Bracing


Bracing


Placement of Metal Floor Deck

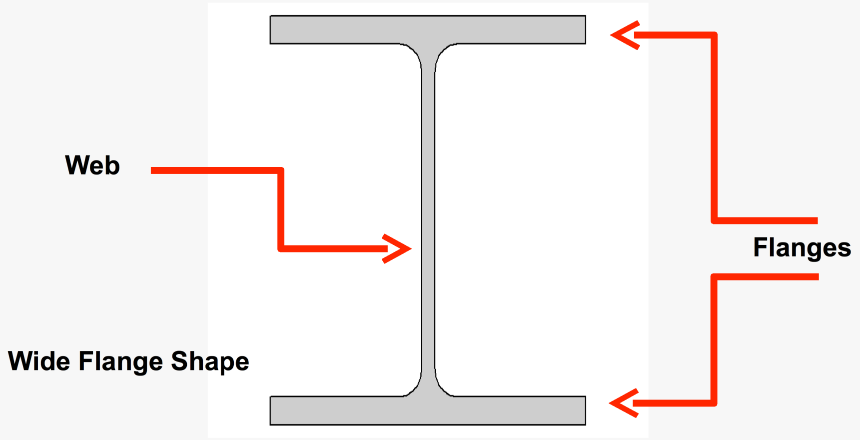

Shear Stud


Welded Wire Mesh Reinforcement


Open Web Steel Joist
(Bar Joist)
Joist Girder

Terminology for Standard Steel Sections

Rolled Sections



Web

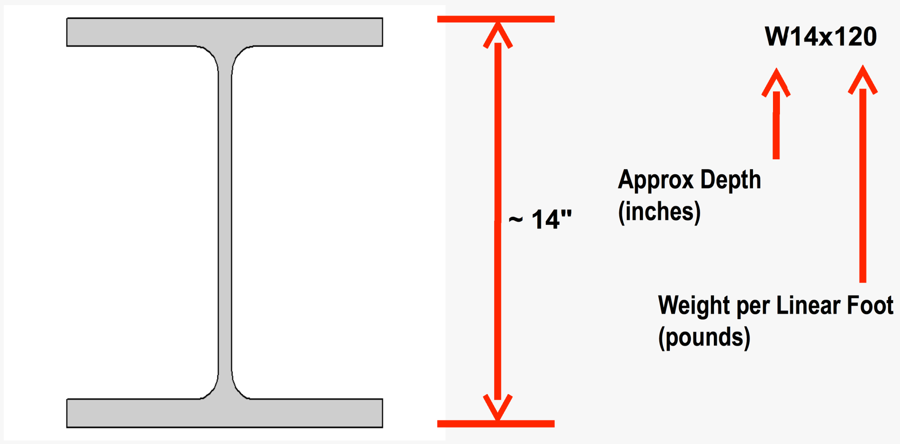
Flanges

Wide Flange Shape

Terminology for Standard Steel Sections

Rolled Sections

Example Designation:



W14x120

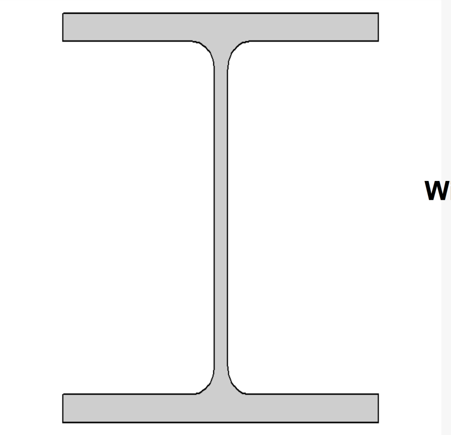
~ 14"

Approx Depth (inches)

Weight per Linear Foot (pounds)

Wide Flange Shape

Terminology for Standard Steel Sections
Rolled Sections

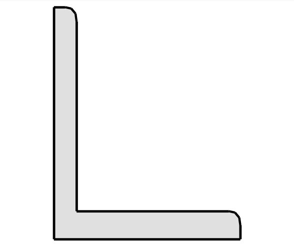


Wide Flange Shape

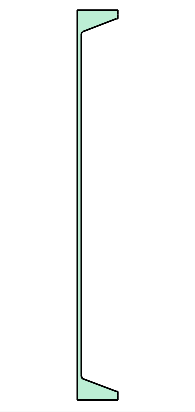
Wide flange shapes available in depths from 4" to 44"

And weights from 8 lbs/ft to 800 lbs/ft

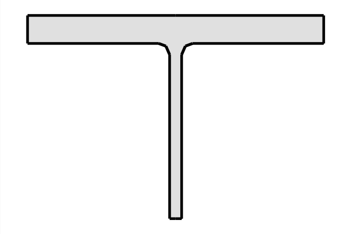
Terminology for Standard Steel Sections
Rolled Sections



Angle

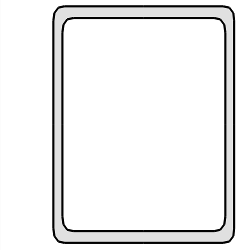


Channel

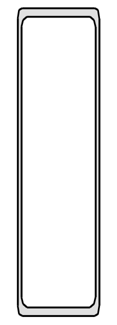


Tee

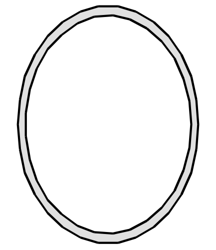
Terminology for Standard Steel Sections
Rolled Sections



Square Tube



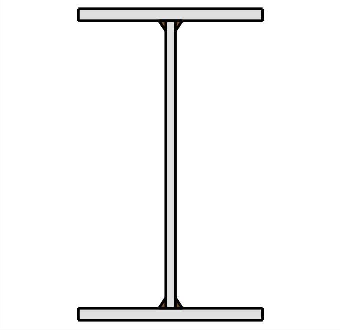
Rectangular Tube



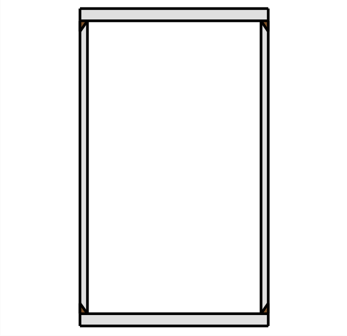
Round Tube (Pipe)

Also called "HSS" section (Hollow Structural Sections)

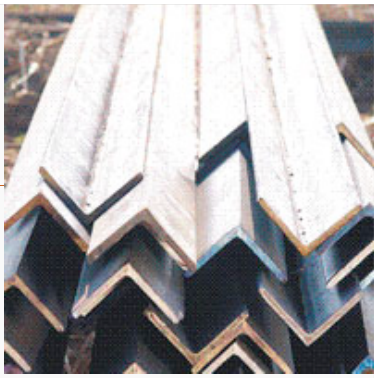
Terminology for Standard Steel Sections
Welded Shapes

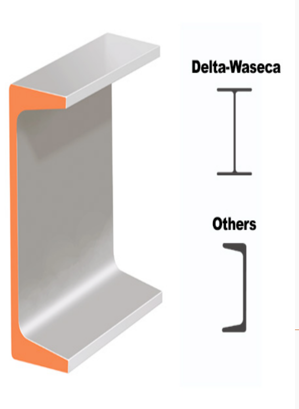


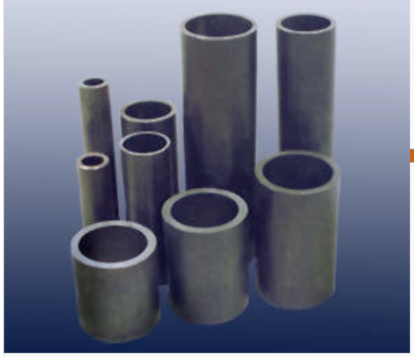
Welded I-Shape

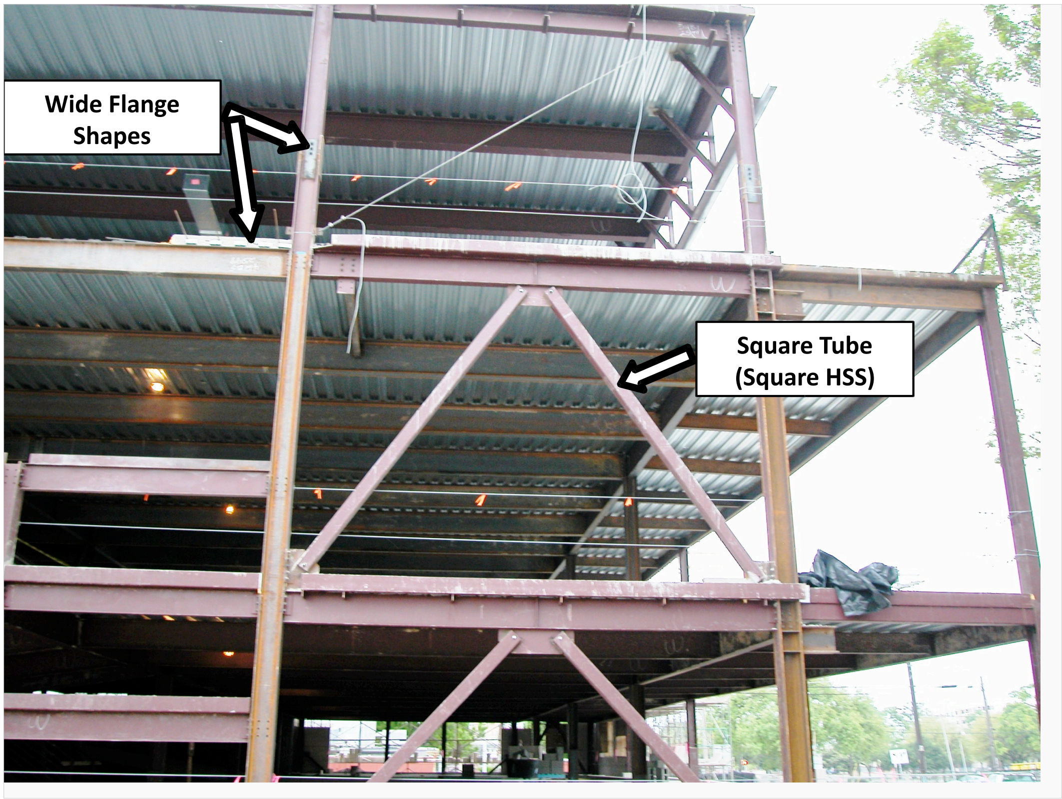


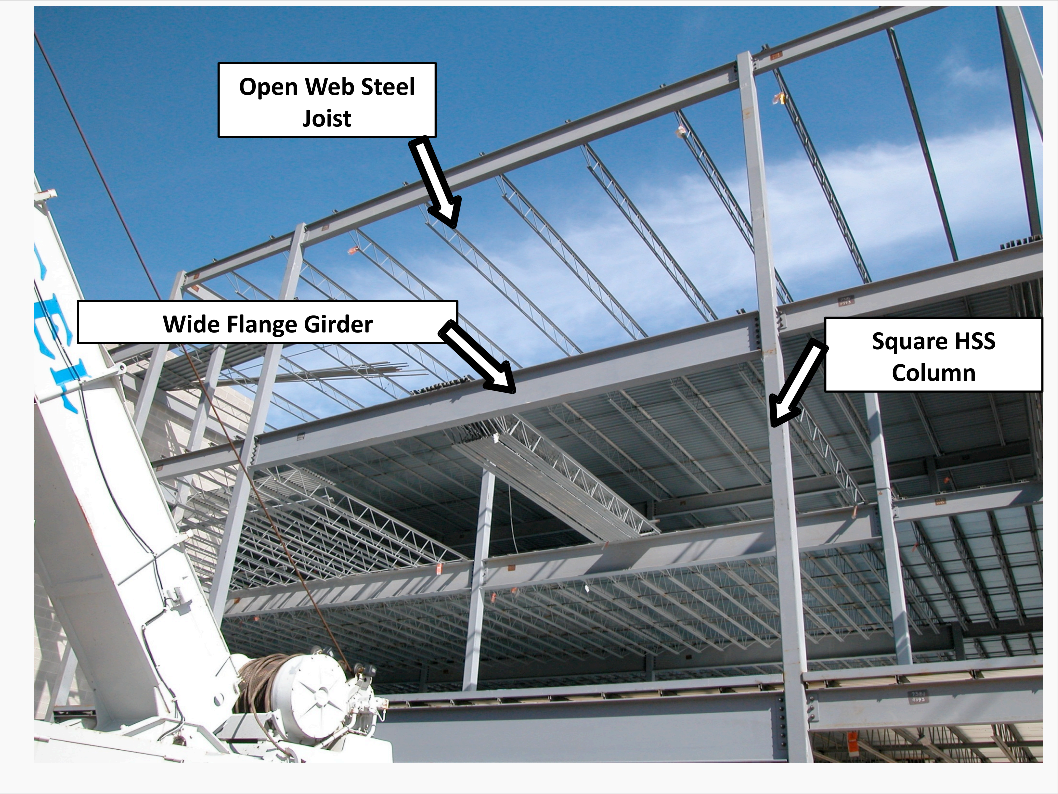
Welded Box

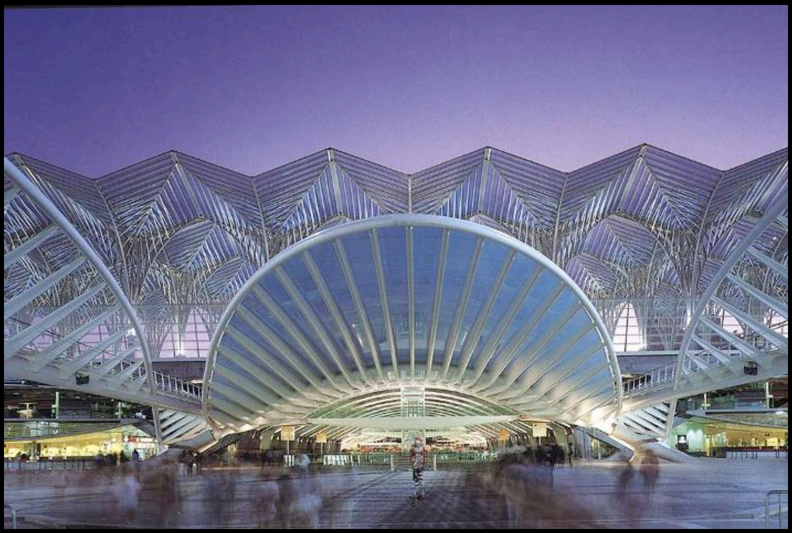


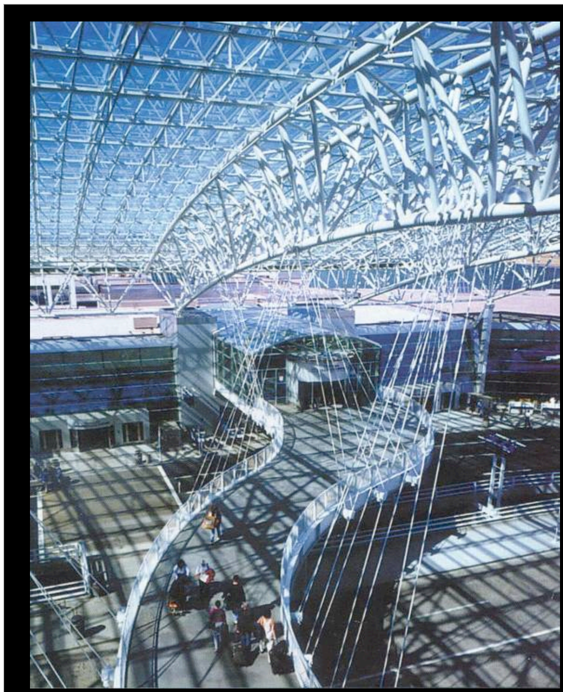

Delta-Waseca
Others




Wide Flange Shapes
Square Tube (Square HSS)


Open Web Steel Joist
Wide Flange Girder
Square HSS Column





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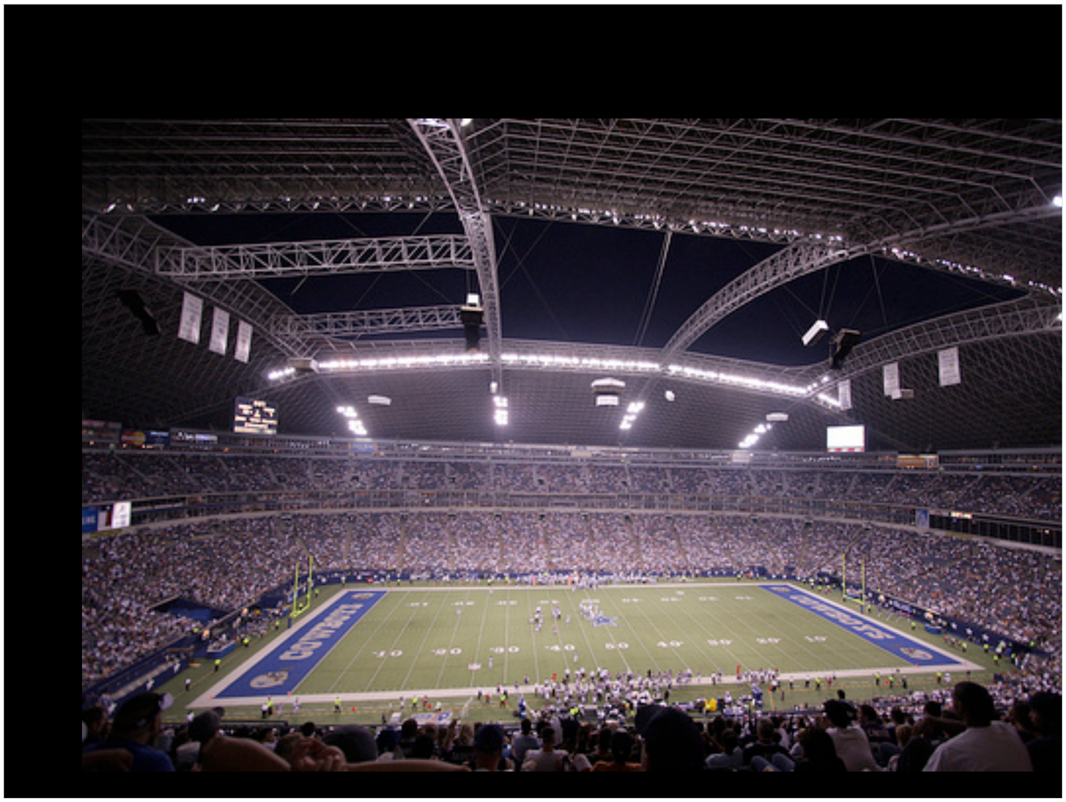

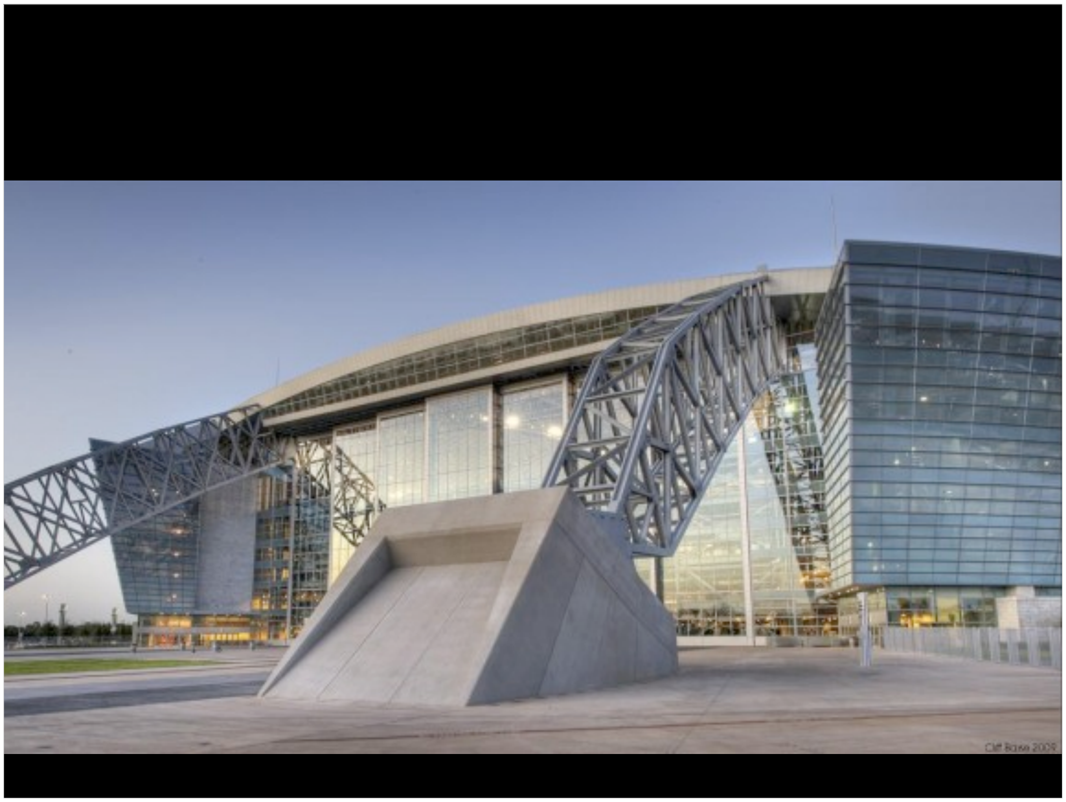

Cliff Baise 2009