# University of California, San Diego
# Faculty of Engineering

# INTRODUCTION

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# TEXTBOOKS

1. Çelik Yapıların Tasarım, Hesap ve Yapım Esaslarına dair yönetmelik-2016

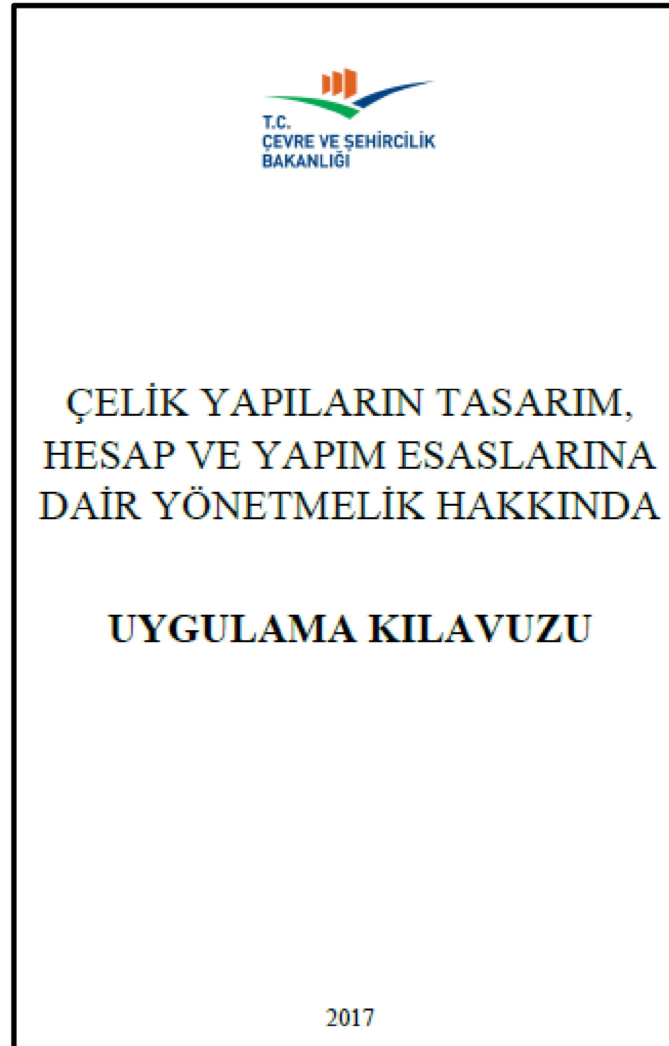
4 Şubat 2016 – Sayı : 29614 RESMÎ GAZETE Sayfa : 3

# ÇELİK YAPILARIN TASARIM, HESAP VE YAPIM ESASLARI

## İÇİNDEKİLER

# TEXTBOOKS

**2. Çelik Yapıların Tasarım Hesap ve Yapım Esaslarına Dair Yönetmelik Hakkında Uygulama Kılavuzu**, T.C. Çevre ve Şehircilik Bakanlığı, 2017.



T.C.
ÇEVRE VE ŞEHİRCİLİK
BAKANLIĞI

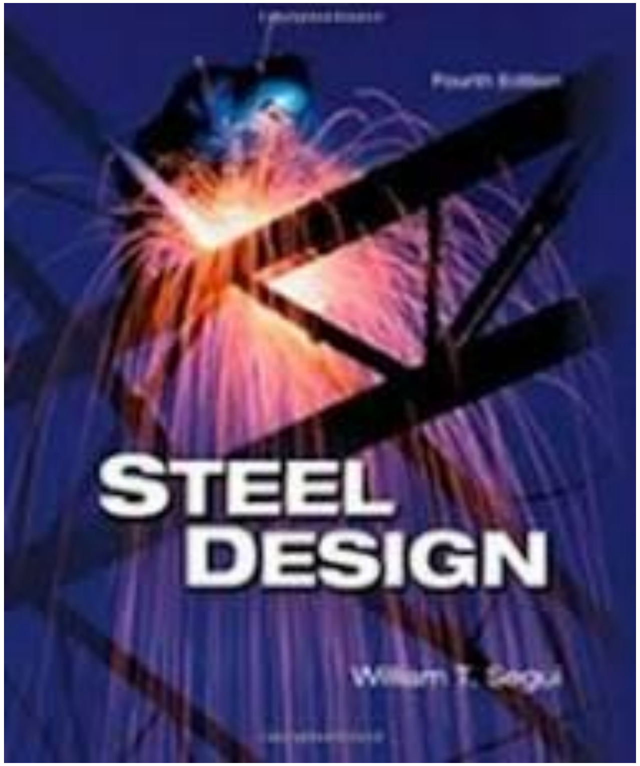
ÇELİK YAPILARIN TASARIM, HESAP VE YAPIM ESASLARINA DAİR YÖNETMELİK HAKKINDA

**UYGULAMA KILAVUZU**

2017

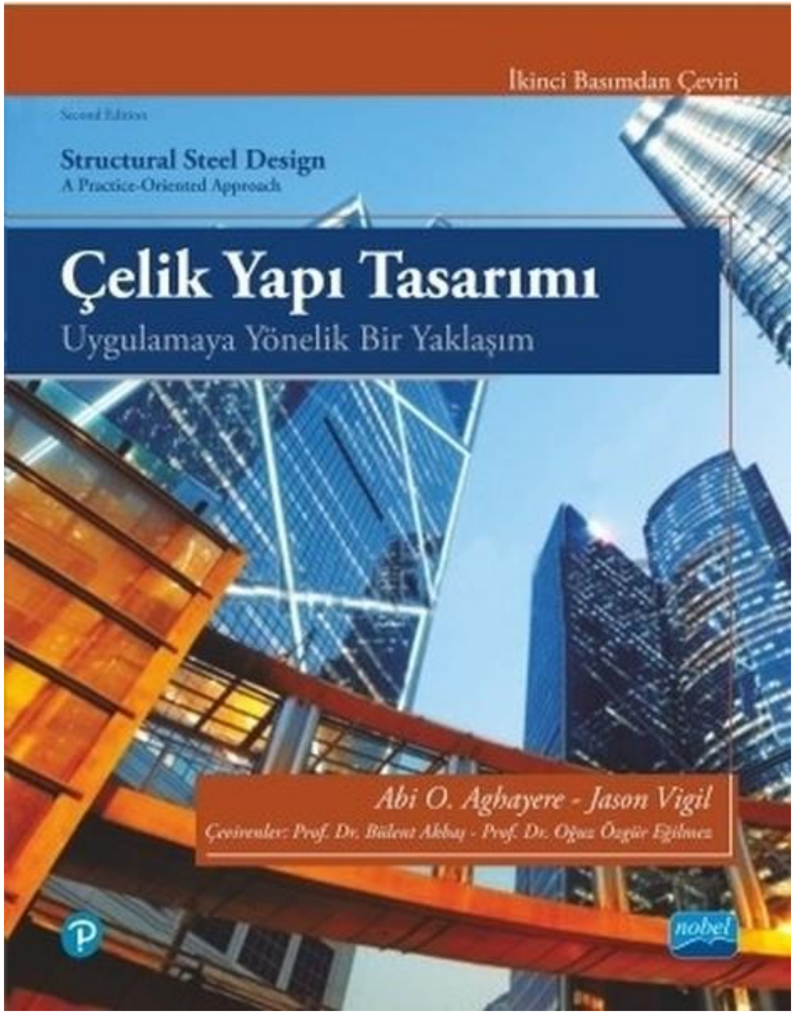
# TEXTBOOK

3. Segui, W.T., *Steel Design*, 5th Edition, Cengage Learning, 2013



# TEXTBOOKS

4. Abi O. Aghayere, Jason Vigil
Çeviren: Prof. Dr. Bülent Akbaş, Prof. Dr. Özgür Eğilmez
**Çelik Yapı Tasarımı Uygulamaya Yönelik Bir Yaklaşım**, Nobel Yayınevi.



# STRUCTURAL STEEL AS A BUILDING MATERIAL

The forerunners to structural steel were cast iron and wrought iron and these were used widely in building and bridge structures until the mid-nineteenth century.

In 1856, steel was first manufactured in the United States and since then it has been used in the construction of many buildings and bridge structures.

Structural steel is an alloy of iron and carbon and is manufactured in various standard shapes and sizes by steel rolling mills, and has:

$\rho$ = unit weight = 7850 kg/m$^3$ (490 lb/ft)

E = modulus of elasticity = 200,000 MPa (29000 ksi)

$\nu$ = Poisson's ratio $\cong$ 0.30.

# STRUCTURAL STEEL AS A BUILDING MATERIAL

Carbon Content of Structural Steel:

0.15% to about 0.30% by weight

Iron Content of Structural Steel:

As high as 95%

The higher the carbon content, the higher the yield stress, and the lower the ductility and weldability.

Higher carbon steels are also more brittle.

# STRUCTURAL STEEL AS A BUILDING MATERIAL

Some of the **advantages** of structural steel as a building material include the following:

1. Steel has a high strength-to-weight ratio.

2. The properties of structural steel are uniform and homogeneous, and highly predictable.

3. It has high ductility, thus providing adequate warning of any impending collapse.

4. It can easily be recycled. In fact, some buildings have a majority of their components made of recycled steel.

5. Steel structures are easier and quicker to fabricate and erect, compared with concrete structures.

# STRUCTURAL STEEL AS A BUILDING MATERIAL

Some of the **advantages** of structural steel as a building material include the following:

6. The erection of steel structures is not as affected by weather as is the use of other building materials, enabling steel erection to take place even in the coldest of climates.

7. It is relatively easier to make additions to existing steel structures because of the relative ease of connecting to the existing steel members.

# STRUCTURAL STEEL AS A BUILDING MATERIAL

Some of the **disadvantages** of structural steel as a building material include the following:

1. Steel is susceptible to corrosion and has to be protected by galvanizing or by coating with zinc-rich paint, especially structures exposed to weather or moisture. Although corrosion-resistant steels are also available. Consequently, maintenance costs could

be high compared to other structural materials.

2. Steel is adversely affected by high temperatures and therefore often needs to be protected from fire.

3. Depending on the types of structural details used, structural steel may be susceptible to brittle fracture due to the presence of stress concentrations, and to fatigue due to cyclic or repeated loadings causing reversals of stresses in the members and connections.

# PROPERTIES OF STRUCTURAL STEEL: MECHANICAL

The three most important properties of structural steel used in structural design are:

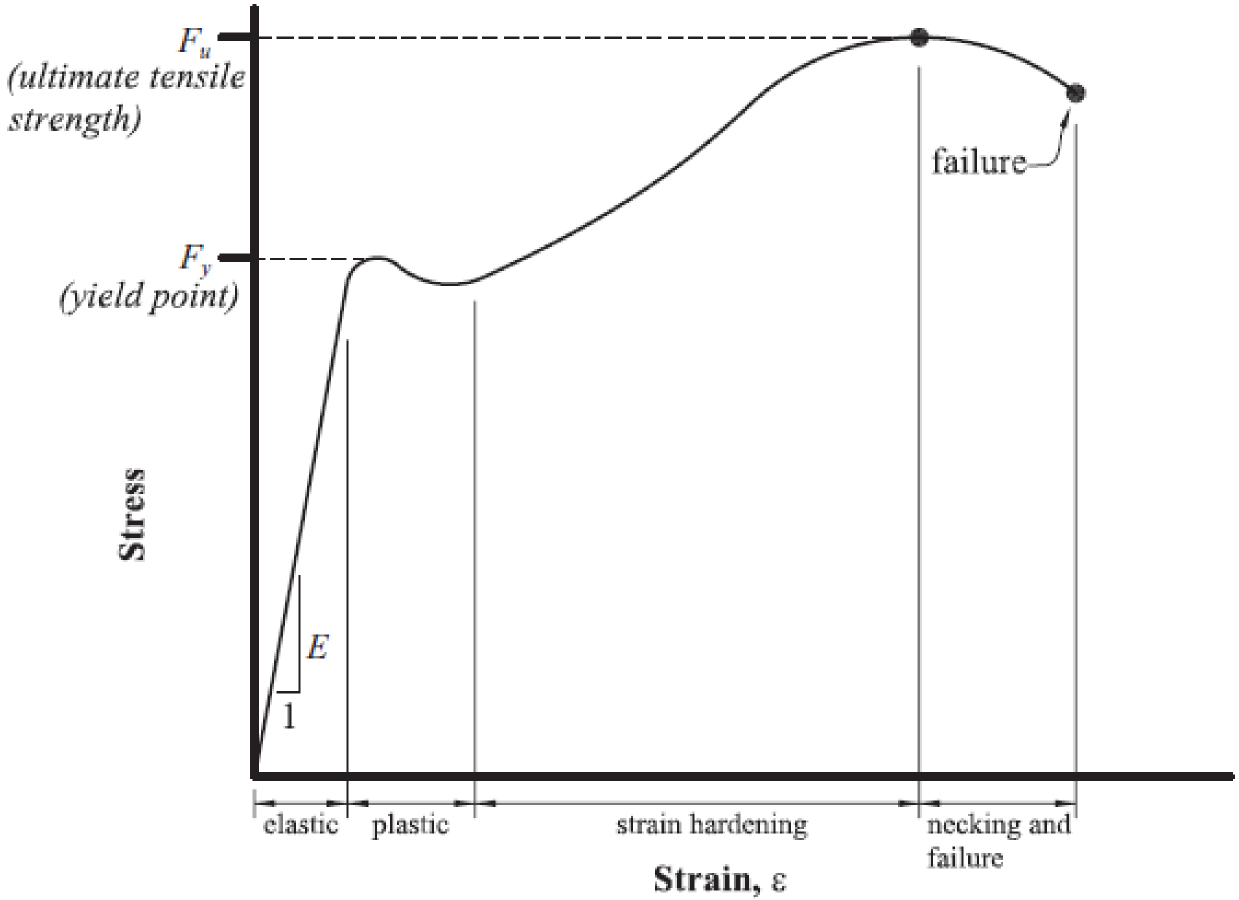
1. Tensile strength
2. Ultimate strength
3. Percent elongation

These are determined by a tensile test that involves subjecting a steel specimen to tensile loading and measuring the load and axial elongation of the specimen until failure.

$$\sigma = \frac{P}{A} \qquad \varepsilon = \frac{\Delta L}{L_0} \qquad \%Elongation = \frac{L' - L_0}{L_0} x100$$

# PROPERTIES OF STRUCTURAL STEEL: MECHANICAL

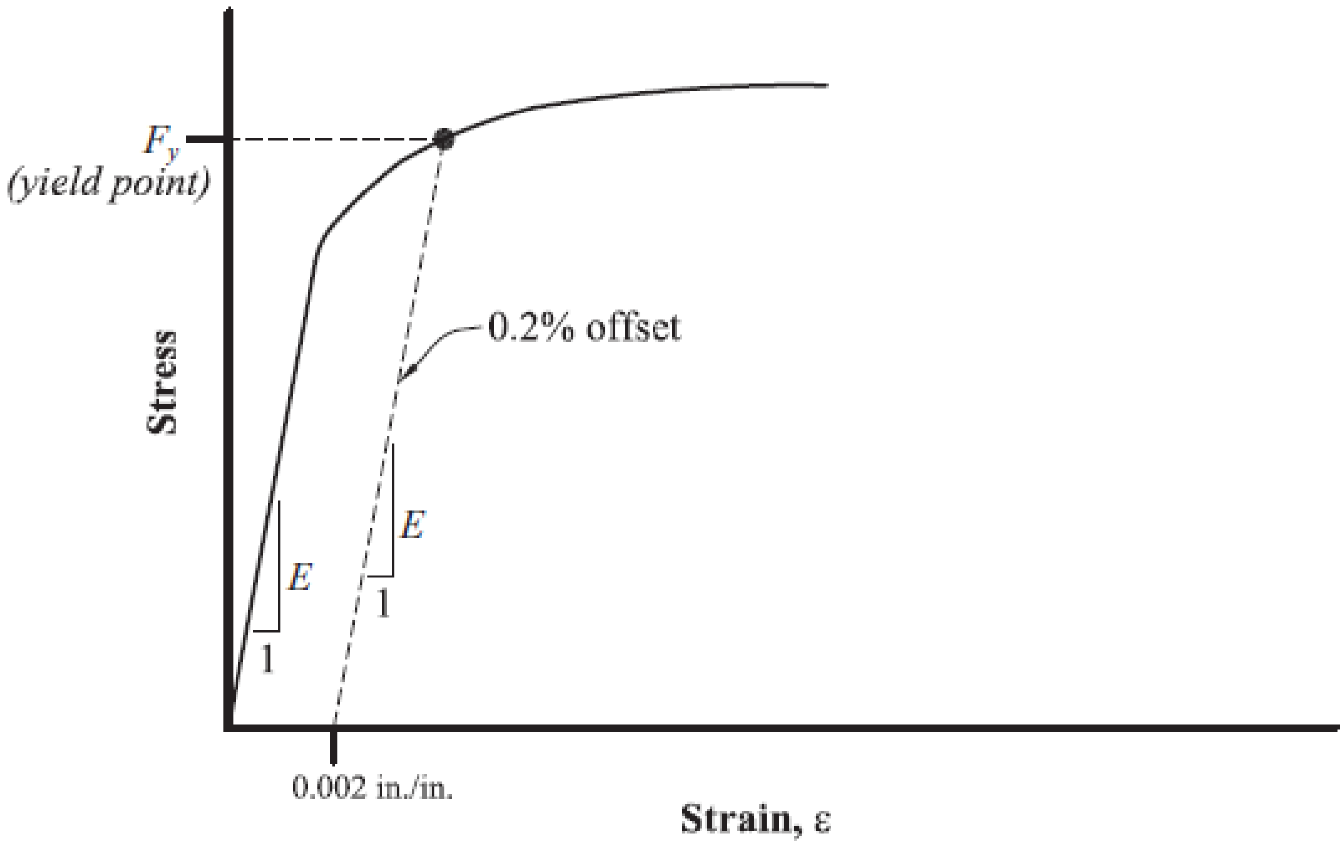
Typical stress-strain curves for structural steel




*a. Steel with a defined yield point*

# PROPERTIES OF STRUCTURAL STEEL: MECHANICAL

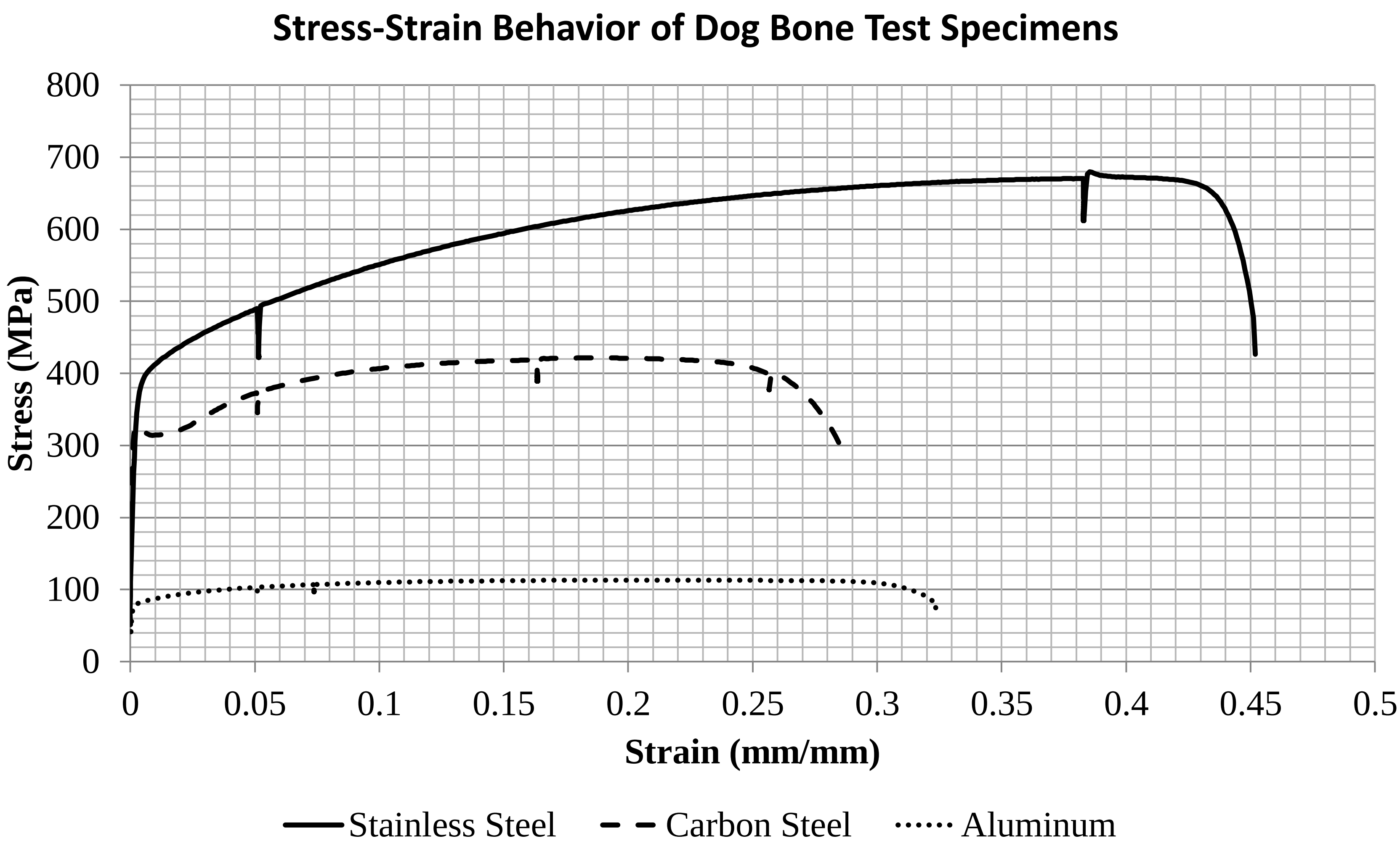
Typical stress-strain curves for structural steel




*b. Steel without a defined yield point*

# PROPERTIES OF STRUCTURAL STEEL: MECHANICAL

Typical stress-strain curves for structural steel



# PROPERTIES OF STRUCTURAL STEEL: MECHANICAL

The 2016 Structural Steel Design Specification enforces the use of structural steel that meets the following specifications:

| | |
|---|---|
| **Rolled Sections and Plates** | **TS EN 10025** |
| **Hallow Structural Steel (HSS) and Box Sections** | **EN 10210**<br>**EN 10219** |

Concrete and reinforcement steel have to be manufactured in accordance with TS500 and TS708 standards, respectively.

European Steel Grades:

**TABLO 2.1A – SICAK HADDELENMİŞ YAPISAL ÇELİKLERDE KARAKTERİSTİK AKMA GERİLMESİ, $F_y$ VE ÇEKME DAYANIMI, $F_u$**

| Standart ve Çelik Sınıfı | Karakteristik Kalınlık, $t$ (mm) | | | |
|---|---|---|---|---|
| | $t \leq 40$mm | | 40mm $< t \leq$ 80mm | |
| | $F_y$ (N/mm$^2$) | $F_u$ (N/mm$^2$) | $F_y$ (N/mm$^2$) | $F_u$ (N/mm$^2$) |
| **EN 10025-2** | | | | |
| S235 | 235 | 360 | 215 | 360 |
| S275 | 275 | 430 | 255 | 410 |
| S355 | 355 | 510 | 335 | 470 |
| S450 | 440 | 550 | 410 | 550 |
| **EN 10025-3** | | | | |
| S275 N/NL | 275 | 390 | 255 | 370 |
| S355 N/NL | 355 | 490 | 335 | 470 |
| S420 N/NL | 420 | 520 | 390 | 520 |
| S460 N/NL | 460 | 540 | 430 | 540 |

# PROPERTIES OF STRUCTURAL STEEL: EUROPEAN

**Weldable fine grain structural steels according to EU Stand.**

**EN 10025-4: S 355 ML**

| | |
|---|---|
| S | Structural Steel |
| XXX | Min. Yield Strength (Mechanical Characteristic |

| Physical Characteristics | |
|---|---|
| L | for low temperatures |
| M | thermomechanical rolling |
| N | normalized rolling |
| W | wheathering |

# BİNA GEOMETRİSİNİN BELİRLENMESİ

## Weldable fine grain structural steels according to EU Stand. EN 10025-4:

| Grade | Min. yield strength (MPa) | | | | | | Tensile Strength (MPa) | | | | | Min. elongation |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Nominal thickness (mm) | | | | | | Nominal thickness (mm) | | | | | |
| | ≤16 | >16 ≤40 | >40 ≤63 | >63 ≤80 | >80 ≤100 | >100 ≤125 | ≤40 | >40 ≤63 | >63 ≤80 | >80 ≤100 | >100 ≤125 | |
| S 275 M | 275 | 265 | 255 | 245 | 245 | 240 | 370-530 | 360-520 | 350-510 | 350-510 | 350-510 | 24 |
| S 355 M<br>S 355 ML | 355 | 345 | 335 | 325 | 325 | 320 | 470-630 | 450-610 | 440-600 | 440-600 | 430-590 | 22 |
| S 460 ML<br>S 460 ML | 460 | 440 | 430 | 410 | 400 | 385 | 540-720 | 530-710 | 510-690 | 500-680 | 490-660 | 17 |

# PROPERTIES OF STRUCTURAL STEEL: TOUGHNESS

**Charpy-V-Notch Toughness: Section 2.1.2 (ÇYTE 2016)**

To prevent brittle failure, the min. average Charpy-V-Notch (CVN) toughness value should be at least 27 J at the min. temperature that the steel will be exposed to during its lifetime.

# PROPERTIES OF STRUCTURAL STEEL: TOUGHNESS

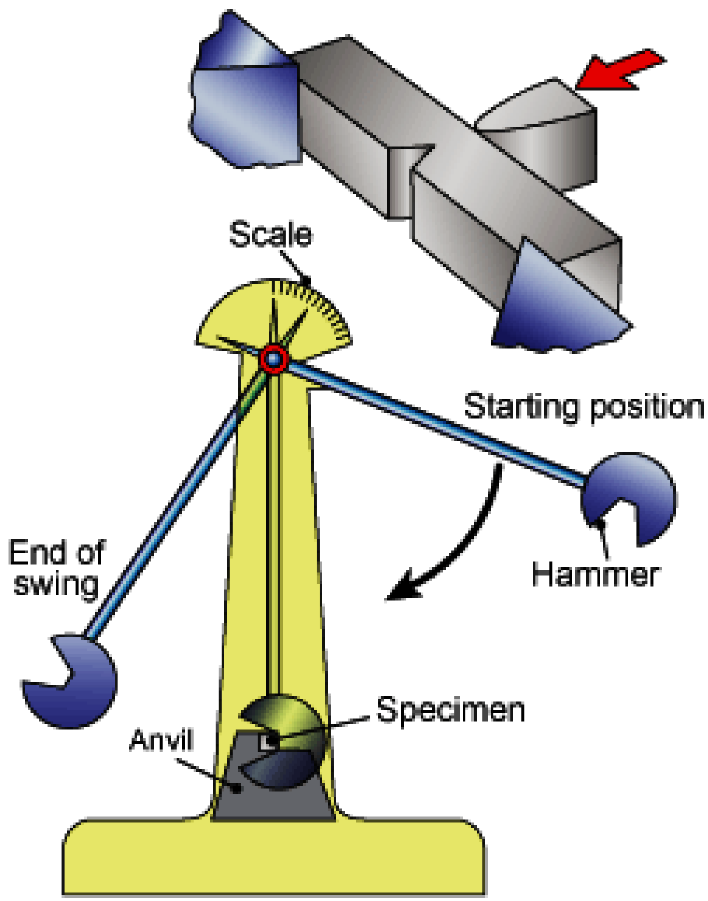
**Charpy-V-Notch Toughness: Section 2.1.2 (ÇYTE 2016)**

In addition, built-up sections (or rolled) consisting of plates (or parts) with a thickness exceeding 50 mm are considered built-up heavy sections. Built-up heavy shapes used as members subject to primary tensile forces due to tension or flexure and spliced or connected to other members using complete-joint-penetration groove welds (<u>tam penetrasyonlu küt kaynak</u>) that fuse through the thickness of the plate shall meet a min. average CVN value of 27 J at 21°C.

# PROPERTIES OF STRUCTURAL STEEL: TOUGHNESS

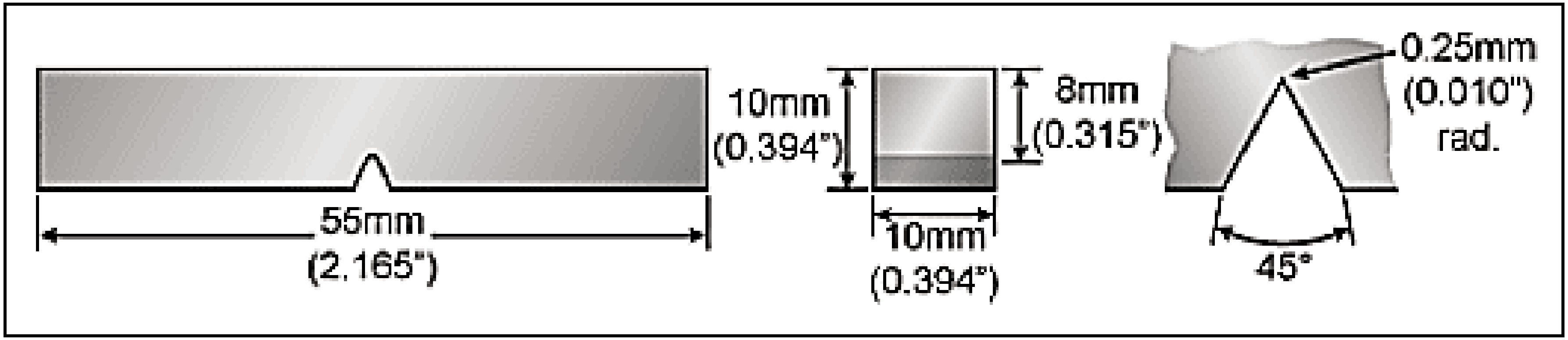
**Charpy-V-Notch Toughness: Section 2.1.2 (ÇYTE 2016)**

Charpy V-Notch impact test must be conducted in accordance with ASTM A6/A6M, Supplementary Requirement S5, Charpy V-Notch Impact Test.



**Charpy-V-Notch Toughness: Section 2.1.2 (ÇYTE 2016)**

The amount of energy required to fracture the specimen is recorded (energy absorbed by the specimen until fracture).



# PROPERTIES OF STRUCTURAL STEEL: ALLOYS

| Chemical Element | Major Advantages | Disadvantages |
|---|---|---|
| Carbon (C) | Increases the strength of steel. | Too much carbon reduces the ductility and weldability of steel. |
| Copper (Cu) | When added in small quantities, it increases the corrosion resistance of carbon steel, as well as the strength of steel. | Too much copper reduces the weldability of steel. |
| Vanadium (V) | Increases the strength and fracture toughness of steel and does not negatively impact the notch toughness and weldability of steel. | |
| Nickel (Ni) | Increases the strength and the corrosion resistance of steel. Increases fracture toughness. | Reduces weldability |
| Molybdenum (Mo) | Increases the strength of steel. Increases corrosion resistance. | Decreases the notch toughness of steel. |
| Chromium (Cr) | Increases the corrosion resistance of steel when combined with copper, and also increases the strength of steel. It is a major alloy chemical used in stainless steel. | |
| Columbium (Cb) | Increases the strength of steel when used in small quantities. | Greatly reduces the notch toughness of steel. |
| Manganese (Mn) | Increases the strength and notch toughness of steel. | Reduces weldability. |
| Silicon (Si) | Used for deoxidizing of hot steel during the steelmaking process and helps to improve the toughness of the steel. | Reduces weldability. |
| Other alloy elements found in very small quantities include nitrogen; those elements permitted only in very small quantities include phosphorus and sulfur. | | |

# PROPERTIES OF STRUCTURAL STEEL: ALLOYS

The carbon equivalent (CE) is useful in determining the weldability of older steels in the repair or rehabilitation of existing or historical structures where the structural drawings and specifications are no longer available, and determining what, if any, special precautions are necessary for welding to these steels in order to prevent brittle fractures.

To ensure good weldability already established above, the carbon equivalent, as calculated from the following equation, should be no greater than 0.5%

$$CE = C + \left(Cu + Ni\right)/15 + \left(Cr + Mo + V\right)/5 + \left(Mn + Si\right)/6 \le 0.5 \quad Eq.(1.1)$$

# PROPERTIES OF STRUCTURAL STEEL: ALLOYS

where:

C = Percentage carbon content by weight,

Cr = Percentage chromium content by weight,

Cu = Percentage copper content by weight,

Mn = Percentage manganese content by weight,

Mo = Percentage molybdenum by weight,

Ni = Percentage nickel content by weight, and

V = Percentage vanadium content by weight.

Si = Percentage silicon by weight.

**Example 1.1: Carbon Equivalent and Weldability of Steel**

A steel floor girder in an existing building needs to be strengthened by welding a structural member to its bottom flange. The steel grade is unknown and to determine its weldability, material testing has revealed the following percentages by weight of the alloy chemicals in the girder:

| | |
|---|---|
| C = 0.25% | Mo = 0.12% |
| Cr = 0.15% | Ni = 0.30% |
| Cu = 0.25% | V = 0.12% |
| Mn = 0.45% | Si = 0.20% |

Calculate the carbon equivalent (CE) and determine if this steel is weldable.

# PROPERTIES OF STRUCTURAL STEEL: EUROPEAN

Solution:

Using Eq. (1.1), the carbon equivalent is calculated as:

$$CE = 0.25\% + \left(0.25\% + 0.30\%\right)/15 + \left(0.15\% + 0.12\% + 0.12\%\right)/5$$
$$+ \left(0.45\% + 0.20\%\right)/6$$

$$CE = 0.47\% \leq 0.5\%$$

Therefore, the steel is weldable.

However, because of the high carbon equivalent, precautionary measures such as low-hydrogen welding electrodes and preheating of the member are recommended.

Since this is an existing structure, the effect of preheating the member should be thoroughly investigated so as not to create a fire hazard.

# PROPERTIES OF STRUCTURAL STEEL: EUROPEAN

**Non-alloy structural steels according to EU Stand.**

**EN 10025-2: S 355 J2 + Z35 + M**

| | |
|---|---|
| S | Structural Steel |
| XXX | Min. Yield Strength (Mechanical Characteristic |

| Notch Toughness (Mechanical Characteristic) | | |
|---|---|---|
| Min. 27 J | Min. 40 J | Temp. °C |
| JR | KR | +20 |
| J0 | K0 | 0 |
| J2 | K2 | -20 |

# PROPERTIES OF STRUCTURAL STEEL: EUROPEAN

**Non-alloy structural steels according to EU Stand.**

**EN 10025-2: S 355 J2 + Z35 + M**

| Special Requirements | |
|---|---|
| Z15 | min. 15% reduction of area |
| Z25 | min. 25% reduction of area |
| Z35 | min. 35% reduction of area |

| Treatment Conditions | |
|---|---|
| M | thermomechanical rolling |
| N | normalized rolling |
| AR | as rolled |

# PROPERTIES OF STRUCTURAL STEEL: EUROPEAN

## Non-alloy structural steels according to EU Stand. EN 10025-2:

<table>
<tr><th rowspan="3">Grade</th><th colspan="6">Min. yield strength (MPa)</th><th colspan="2">Tensile Strength (MPa)</th><th colspan="4">Min. Elongation %</th></tr>
<tr><th colspan="6">Nominal thickness (mm)</th><th colspan="2">Nominal thickness (mm)</th><th colspan="4">Nominal thickness (mm)</th></tr>
<tr><th>≤16</th><th>>16 ≤40</th><th>>40 ≤63</th><th>>63 ≤80</th><th>>80 ≤100</th><th>>100 ≤125</th><th>>3 ≤100</th><th>>100 ≤125</th><th>>3 ≤40</th><th>>40 ≤63</th><th>>63 ≤100</th><th>>80 ≤125</th></tr>
<tr><td>S 235 JR</td><td rowspan="3">235</td><td rowspan="3">225</td><td rowspan="3" colspan="3">215</td><td rowspan="3">195</td><td rowspan="3">360 - 510</td><td rowspan="3">350 - 500</td><td>26</td><td>25</td><td>24</td><td>22</td></tr>
<tr><td>S 235 J0</td><td>26</td><td>25</td><td>24</td><td>22</td></tr>
<tr><td>S 235 J2</td><td>24</td><td>23</td><td>22</td><td>22</td></tr>
<tr><td>S 275 JR</td><td rowspan="3">275</td><td rowspan="3">265</td><td rowspan="3">255</td><td rowspan="3">245</td><td rowspan="3">235</td><td rowspan="3">225</td><td rowspan="3">410 – 560</td><td rowspan="3">400 - 540</td><td>23</td><td>22</td><td>21</td><td>19</td></tr>
<tr><td>S 275 J0</td><td>23</td><td>22</td><td>21</td><td>19</td></tr>
<tr><td>S 275 J2</td><td>21</td><td>20</td><td>19</td><td>19</td></tr>
</table>

# PROPERTIES OF STRUCTURAL STEEL: AMERICAN

| Standard | Grade | Min. yield strength (MPa) | Tensile Strength (MPa) | Min. Elongation % Min. 50 mm |
|---|---|---|---|---|
| | | MPa (ksi) | MPa (ksi) | % |
| A36 | Grade 36 | ≥ 250 (36) | 400 – 550 (58-80) | 21 |
| A572 | Grade 42 | ≥ 290 (42) | ≥ 415 (60) | 24 |
| | Grade 50 | ≥ 345 (50) | ≥ 450 (65) | 21 |
| | Grade 55 | ≥ 380 (55) | ≥ 485 (70) | 20 |
| | Grade 60 | ≥ 415 (60) | ≥ 520 (75) | 18 |
| | Grade 65 | ≥ 450 (65) | ≥ 550 (80) | 17 |
| A588 | Grade B | ≥ 345 (50) | ≥ 485 (70) | 21 |
| | Grade C | ≥ 345 (50) | ≥ 485 (70) | 21 |

# PROPERTIES OF STRUCTURAL STEEL: AMERICAN

| Standard | Grade | Min. yield strength (MPa) | Tensile Strength (MPa) | Min. Elongation % Min. 50 mm |
|---|---|---|---|---|
| | | MPa (ksi) | MPa (ksi) | % |
| A709 | Grade 36 | ≥ 250 (36) | 400 – 550 (58-80) | 21 |
| | Grade 50 | ≥ 345 (50) | ≥ 450 (65) | 21 |
| | Grade 50S | 345 - 450 (50-65) | ≥ 450 (65) | 21 |
| A913 | Grade 50 | ≥ 345 (50) | ≥ 450 (65) | 21 |
| | Grade 65 | ≥ 450 (52) | ≥ 550 (80) | 17 |
| A992 | A992 | 345 - 450 (50-65) | ≥ 450 (65) | 21 |

# PROPERTIES OF STRUCTURAL STEEL: AMERICAN

| Structural Steel Shapes | ASTM Designation | Min $F_y$ (ksi) | Min $F_u$ (ksi) |
|---|---|---|---|
| W-shape | A913**<br>A992* | 50–70<br>50–65 | 60–90<br>65 |
| M- and S-shapes | A36 | 36 | 58–80 |
| Channels (C- and MC-shapes) | A36*<br>A572 Grade 50 | 36<br>50 | 58–80<br>65 |
| Angles and plates | A36 | 36 | 58–80 |
| Steel Pipe | A53 Grade B | 35 | 60 |
| Round HSS | A500 Grade B*<br>A500 Grade C | 42<br>46 | 58<br>62 |
| Square and Rectangular HSS | A500 Grade B*<br>A500 Grade C | 46<br>50 | 58<br>62 |

* Preferred material specification for the different shapes.

** A913 is a low-alloy, high-strength steel.

# PROPERTIES OF STRUCTURAL STEEL: THERMAL

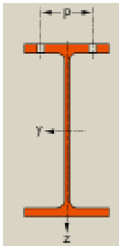
The behavior of structural steel discussed above **occurs at normal temperatures, usually taken as between -35°C and +50 °C**.

Steel loses strength when subjected to elevated temperatures.

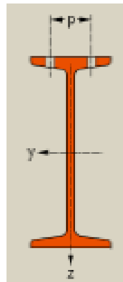
At a temperature of **approximately +700 °C, the strength and stiffness of steel is about 20% of its strength and stiffness at normal temperatures.**

As a result of the adverse effect of high temperatures on steel strength and behavior, structural steel used in building construction is often fireproofed by spray-applying cementitious materials or fibers directly onto the steel member or by enclosing the steel members within plaster, concrete, gypsum board, or masonry enclosures.

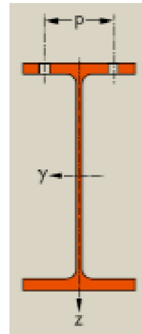
# STRUCTURAL STEEL SHAPES: EUROPEAN



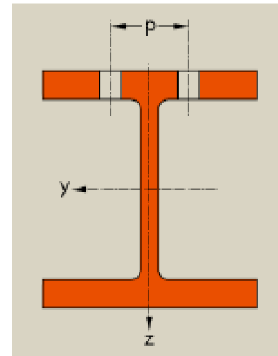
IPE beam (parallel flanges)(h=80-750mm)



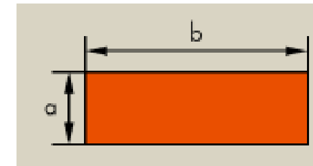
IPN section (european standart beam)(h=80-600mm)



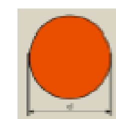
HE section (european wide flange beam)(h=100-1000mm)(HEL: extra wide flange)

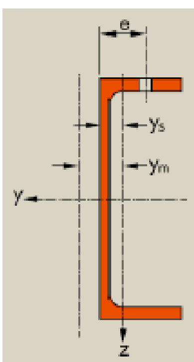


HD section (european wide flange column)(h=250-400mm)

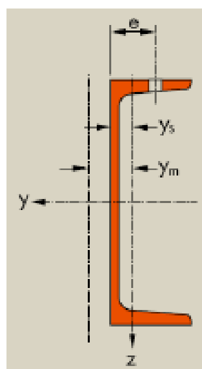


Flat bar (width 20-400mm)

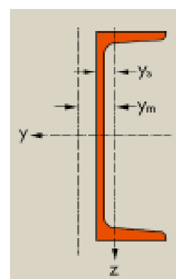

Round bar



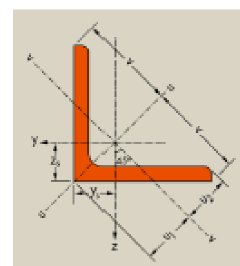
UPE section (channel with parallel flanges) (h=80-400mm)

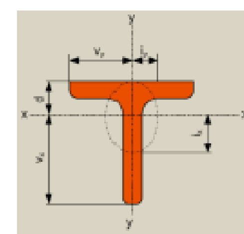


UPN section (european standart channel) (h=80-400mm)



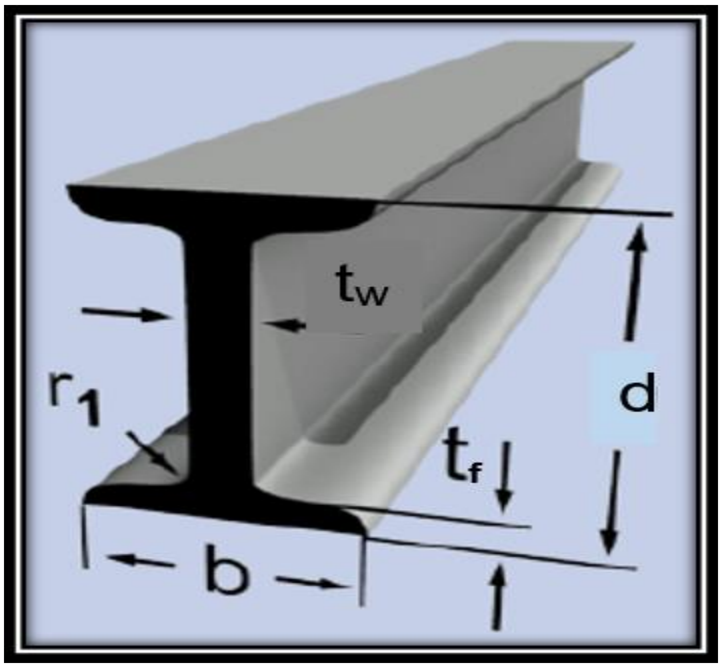
U section (channel with tapred flanges) (h=40-65mm)

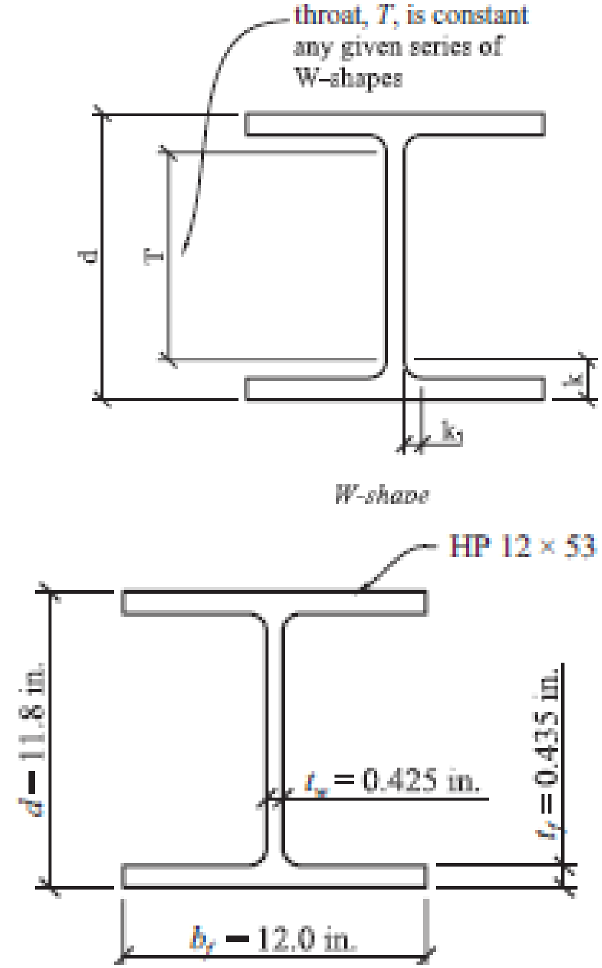

L section (equal leg angles) (h=20-200mm)


T section

# W-sections and HP sections

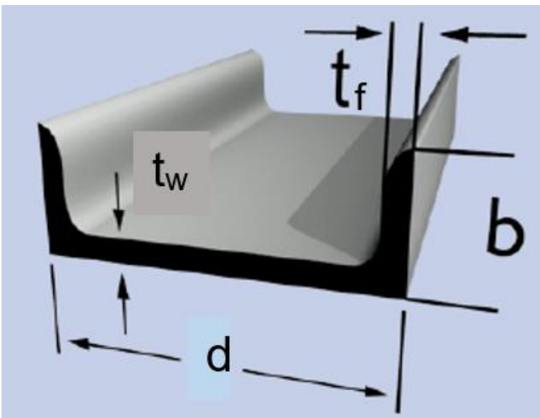
Abi Aghayere and Jason Vigil (2015)

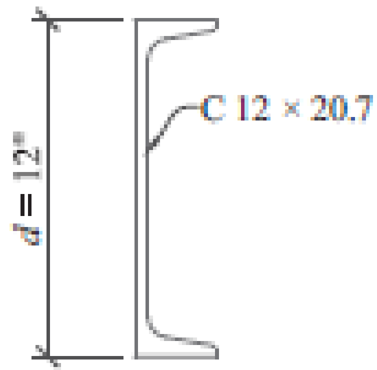


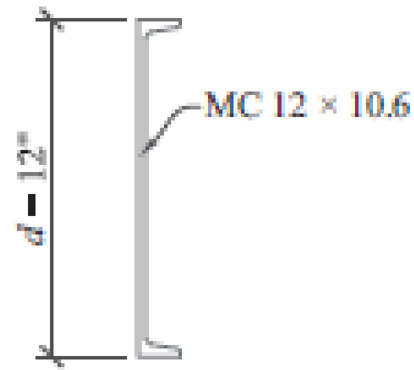
# Channels and Angles

Abi Aghayere and Jason Vigil (2015)

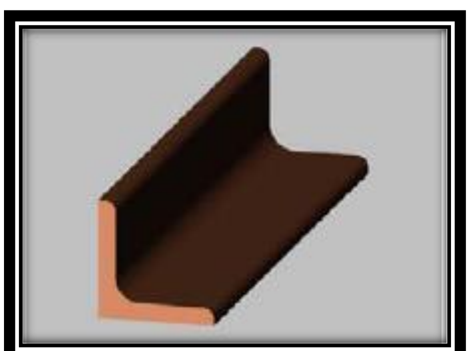


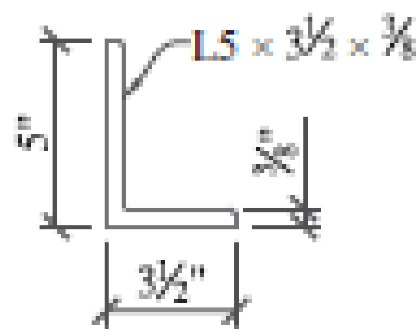



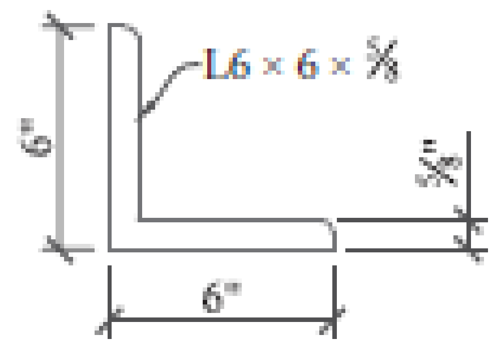

*a. C-shape*




*b. MC-shape*



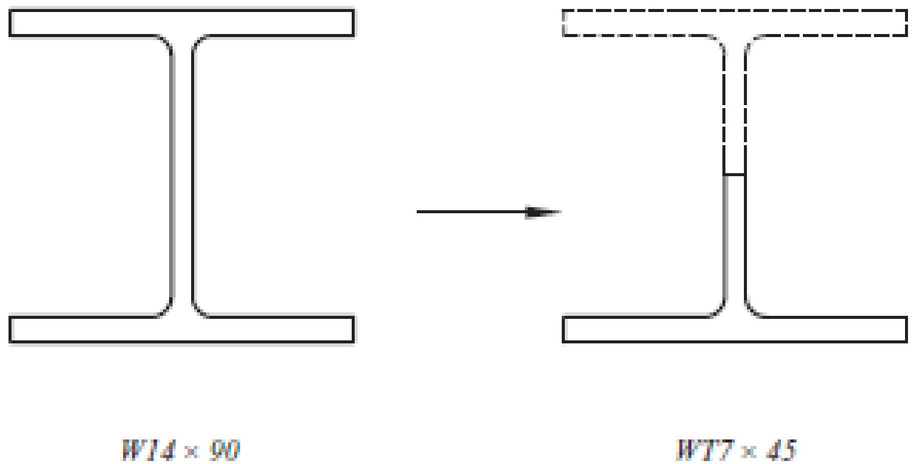


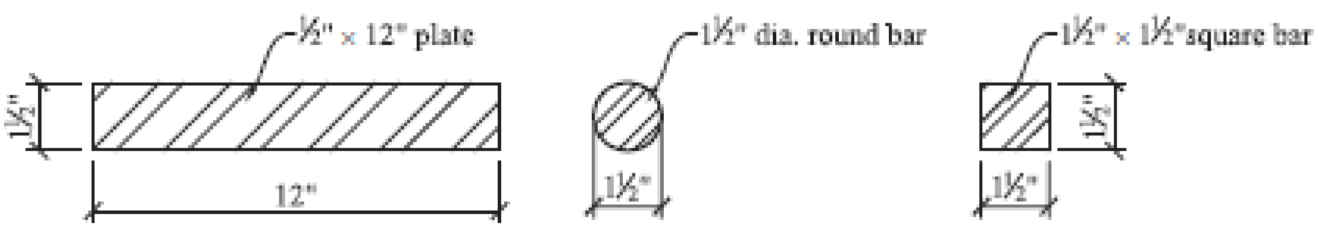



# STRUCTURAL STEEL SHAPES: AMERICAN

## WT, Plates and Bars

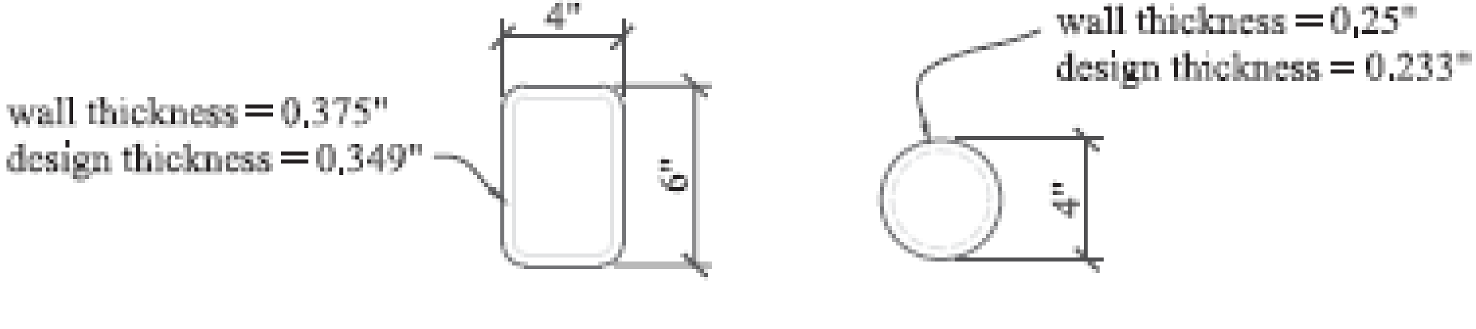
Abi Aghayere and Jason Vigil (2015)



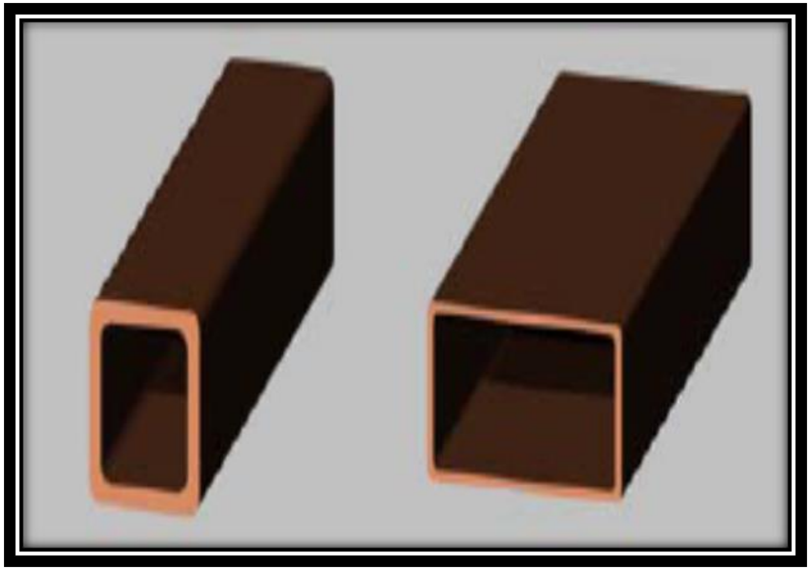



## Hollow Structural Shapes

Abi Aghayere and Jason Vigil (2015)

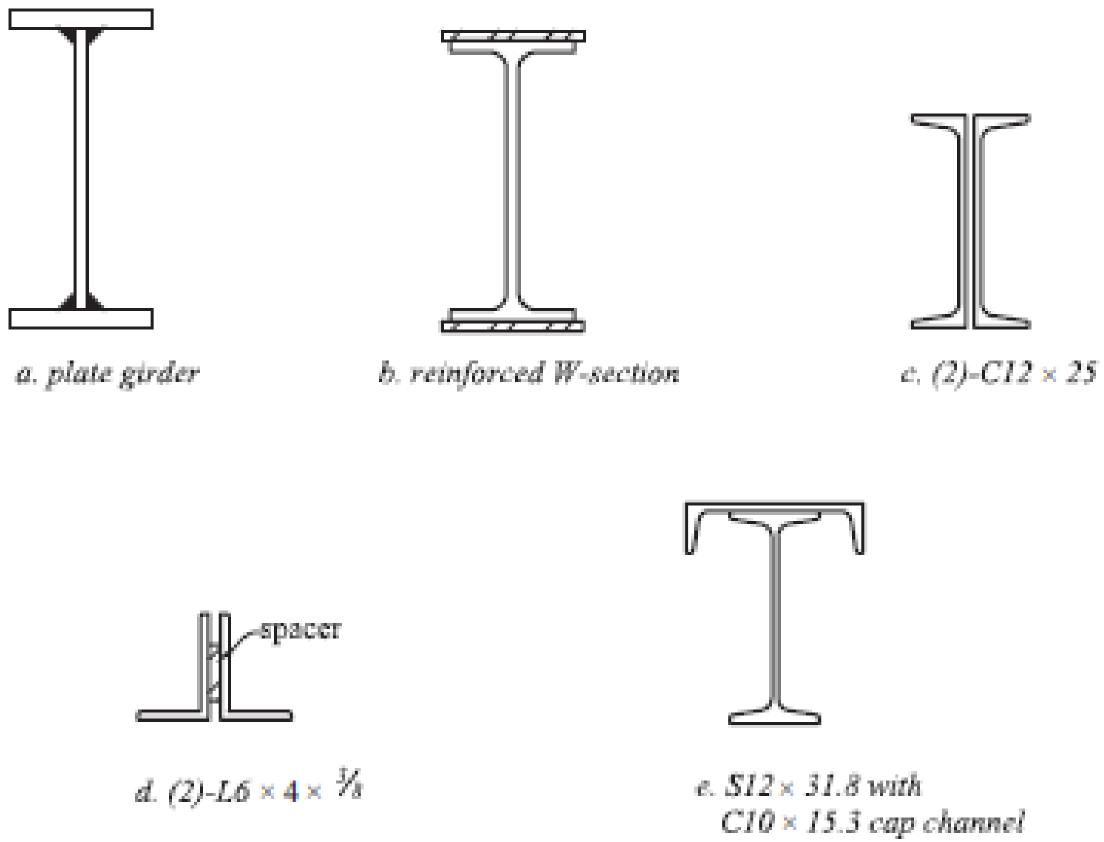



*a. HSS* 6 × 4 × ⅜ *b. HSS* 4.000 × 0.250



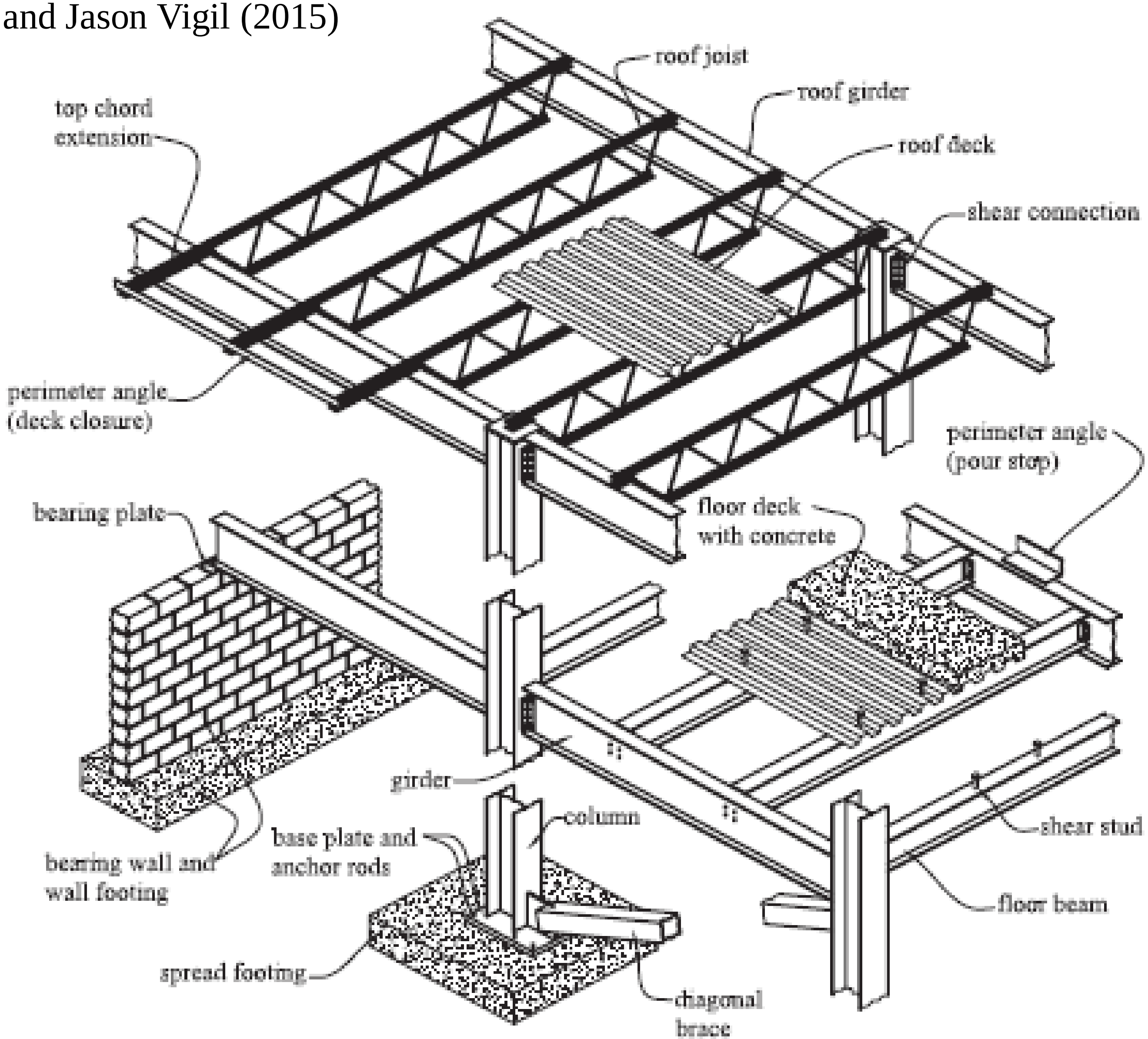
# Built-up Sections

Abi Aghayere and Jason Vigil (2015)



# BASIC STRUCTURAL STEEL ELEMENTS

Abi Aghayere and Jason Vigil (2015)



# BASIC STRUCTURAL STEEL ELEMENTS

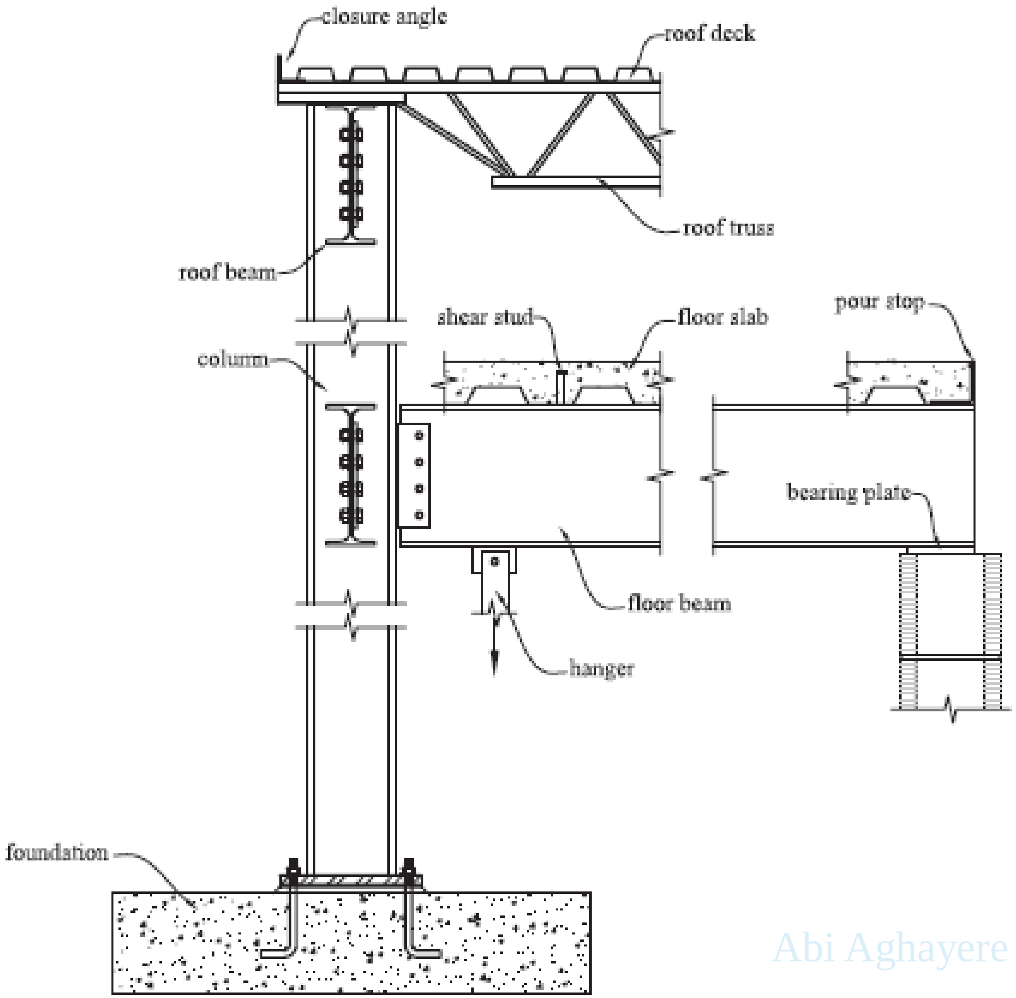



Figure 1-14 Typical steel building cross section—basic structural elements.

Abi Aghayere and Jason Vigil (2015)

# BASIC STRUCTURAL STEEL ELEMENTS

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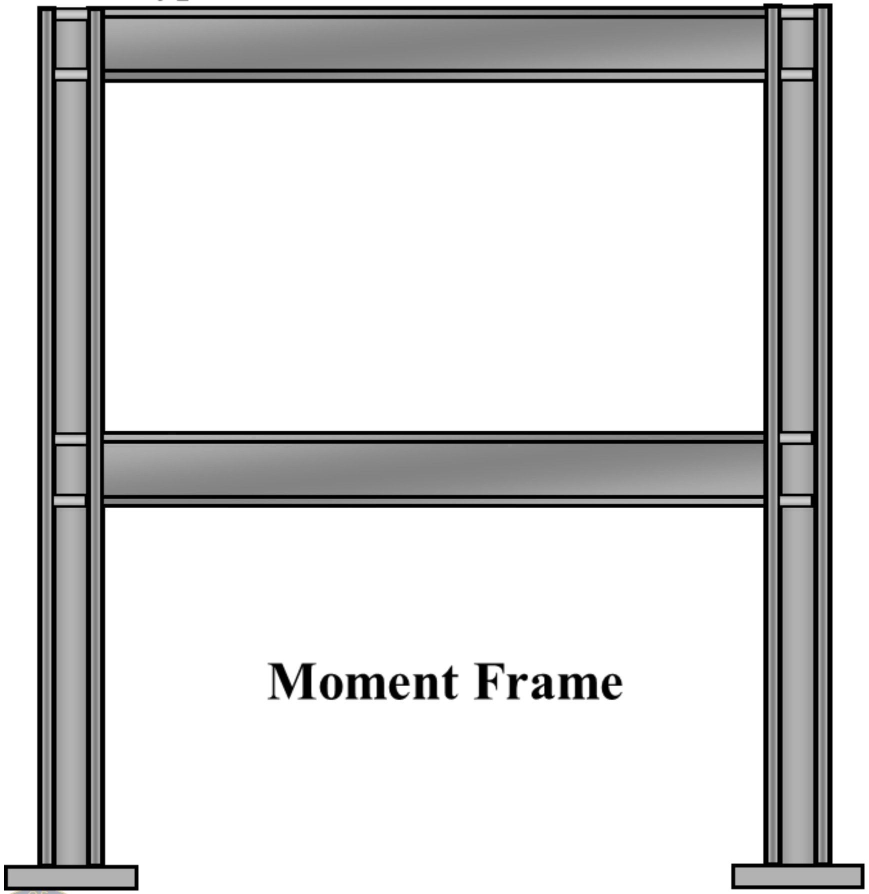
# Buildings: Trusses

Trusses may occur as roof framing members over large spans or as transfer trusses used to support gravity loads from discontinuous columns. Vertical and diagonal members are called web members. The top and bottom horizontal members are called chord members.

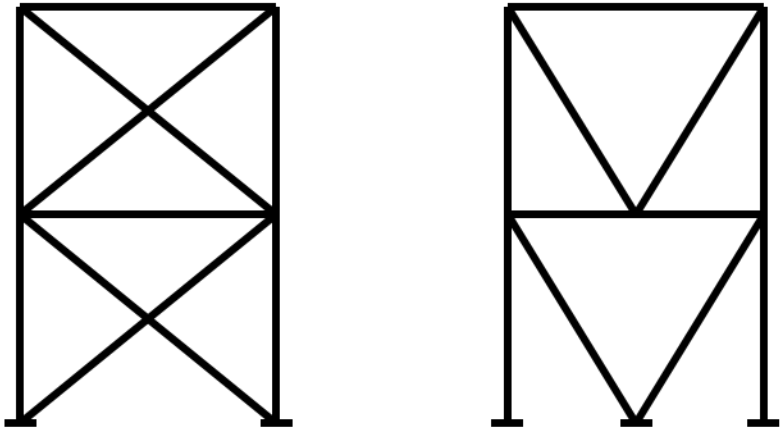
While the top and bottom chord members are generally continuous, the web members are connected to the top and bottom chords using bolted or welded connections.

## Buildings: Frames

Frames are structural systems used to resist lateral wind or seismic loads in buildings. Two main types are **moment frames** and **braced frames**.



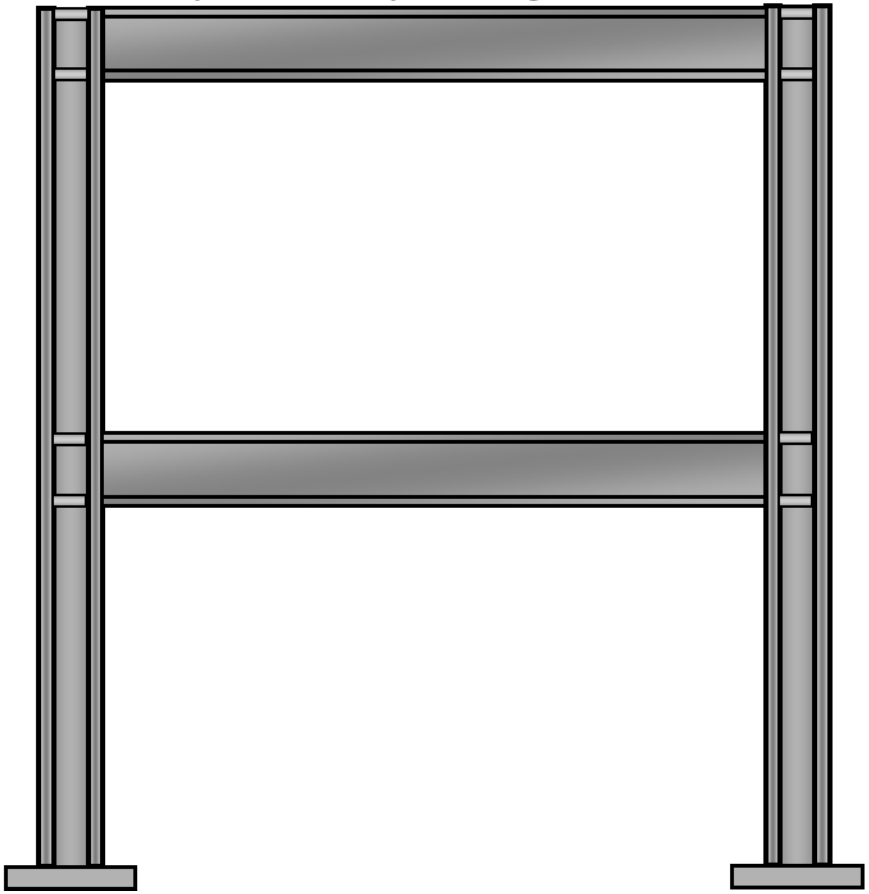
**Moment Frame**



**Braced Frames**

# Buildings: Moment Frames

Resist lateral loads through the bending rigidity of the beams/girders and columns. Ductility is normally achieved by flexural yielding at the ends of the beams.

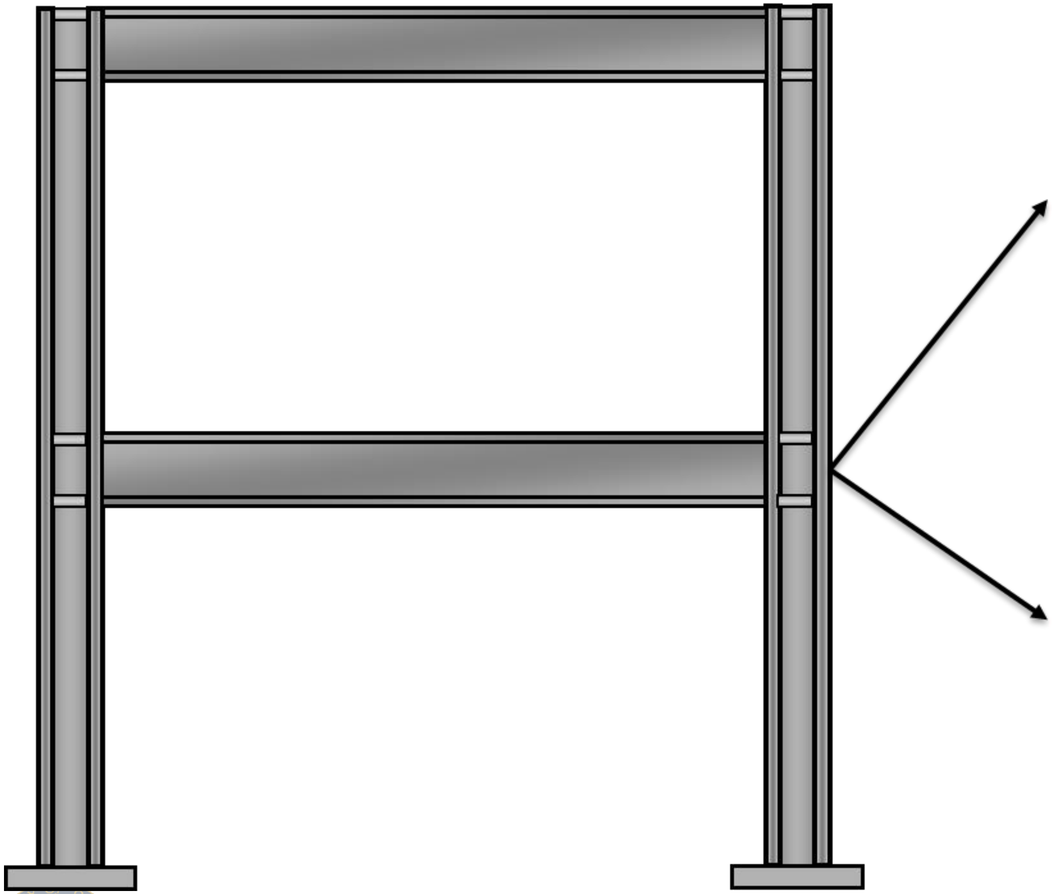


Advantages:
- Provide wide openings for architectural usage
- High ductility

Disadvantages:
- Low elastic stiffness

# Buildings: Moment Frames

Resist lateral loads through the bending rigidity of the beams/girders and columns. Ductility is normally achieved by flexural yielding at the ends of the beams.



Flange Welded Moment Connection

Bolted Moment Connection

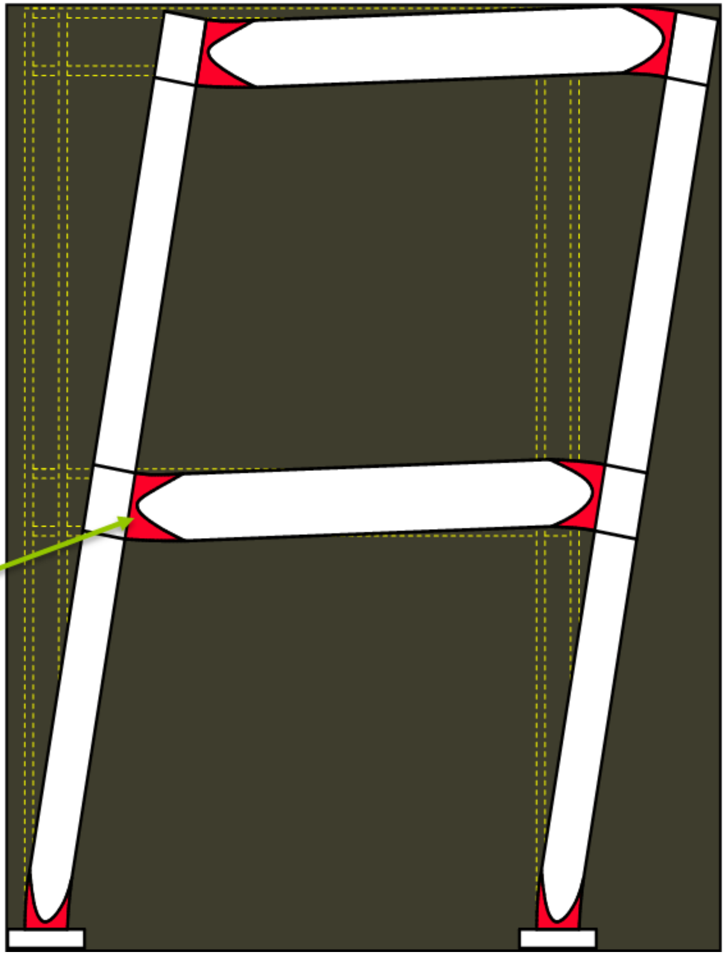
Photos: Courtesy of Prof. Engelhardt

# Buildings: Moment Frames

Resist lateral loads through the bending rigidity of the beams/girders and columns. Ductility is normally achieved by flexural yielding at the ends of the beams.

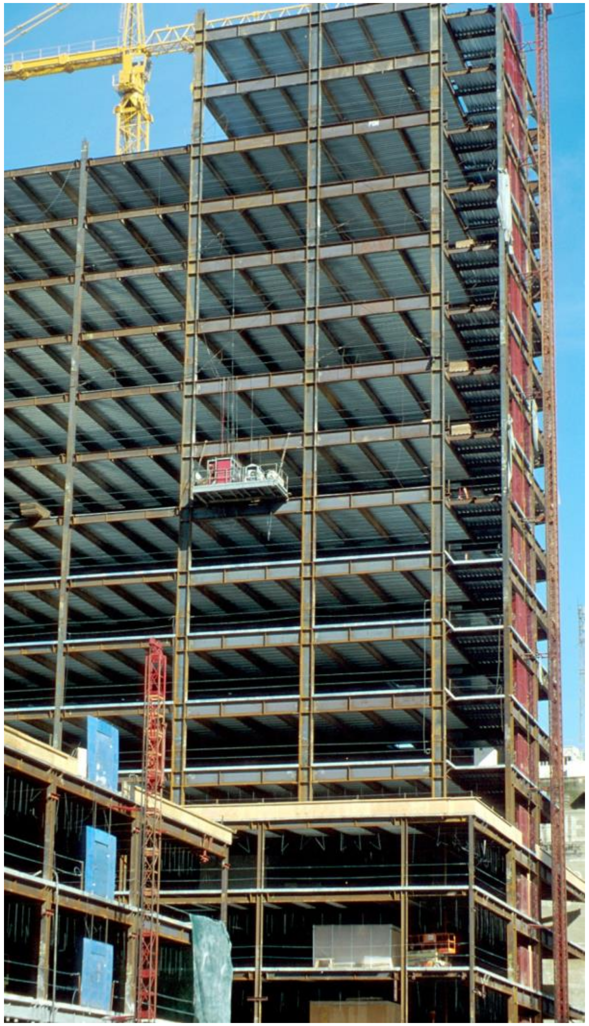
<u>Moment Frame</u>

Plastic hinge locations (energy dissipation)



# TYPES OF STRUCTURAL SYSTEMS IN STEEL

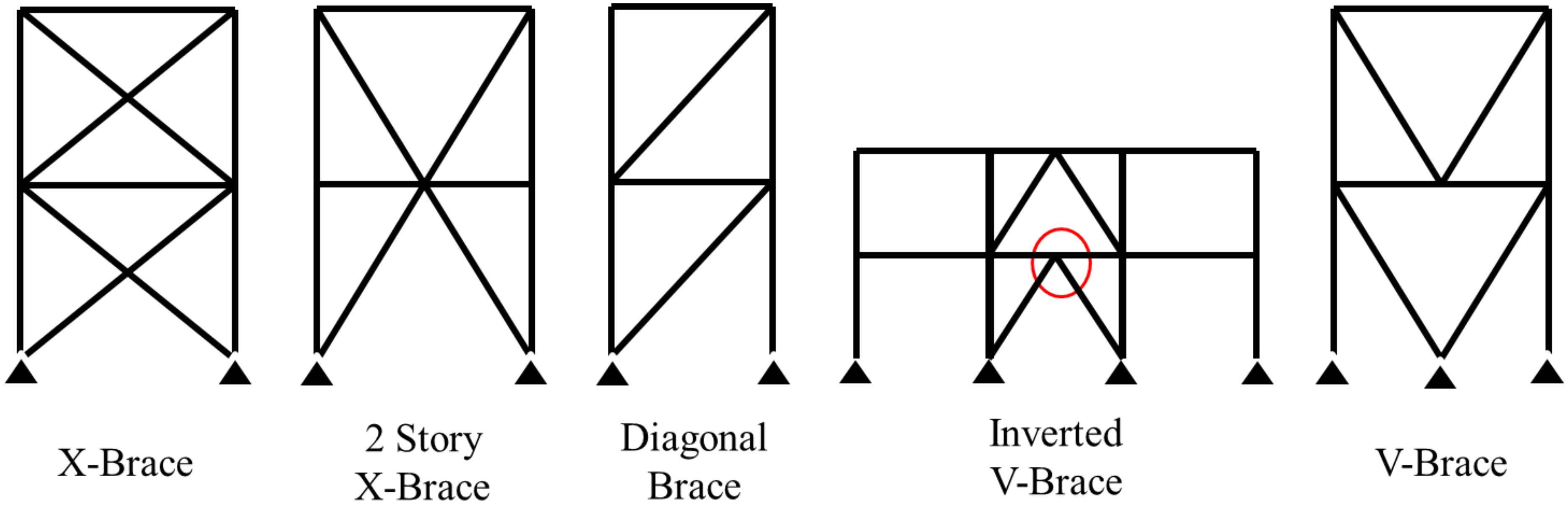
**Moment Frame**:
Only the two right frames are moment frames. Look at the sizes of the beams and columns as compared to the ones in other frames.



# Buildings: Braced Frames

Braced frames (centrically or eccentrically) are also used to resist lateral loads. While doing so, they also provide lateral support to the columns; enabling them to have a lower *KL* value.

**Centrically Braced Frames**



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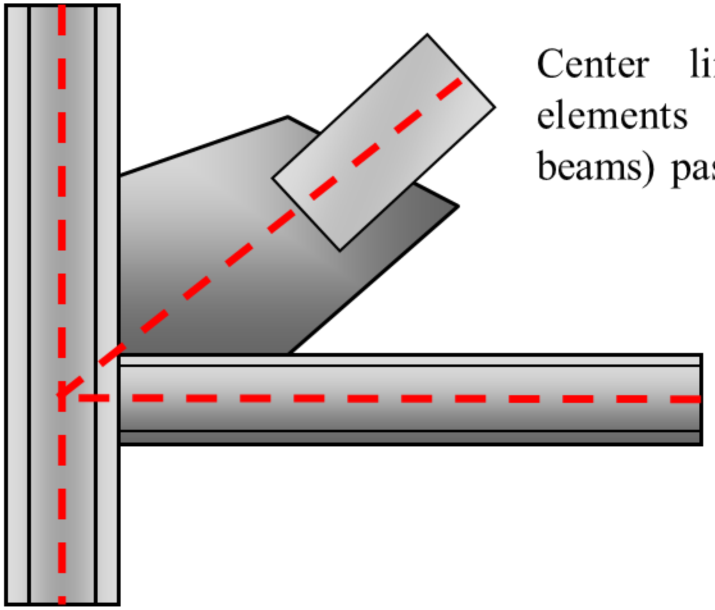
**Centrically Braced Frames**

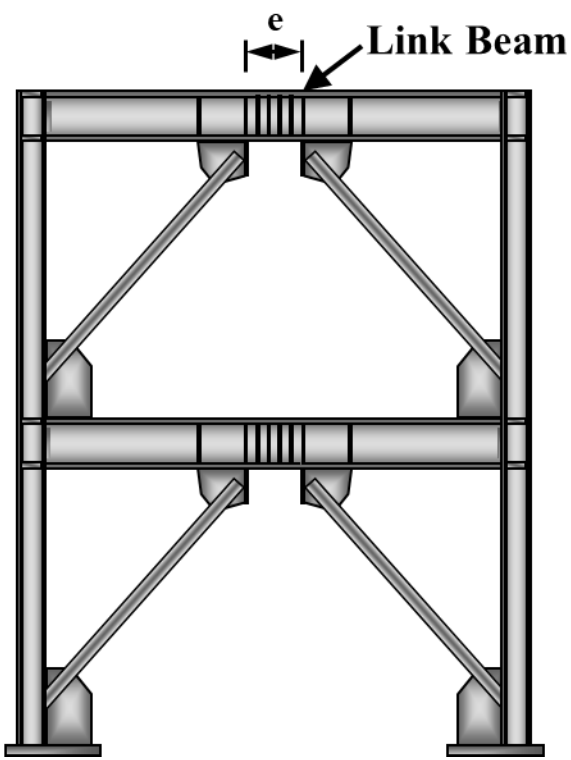


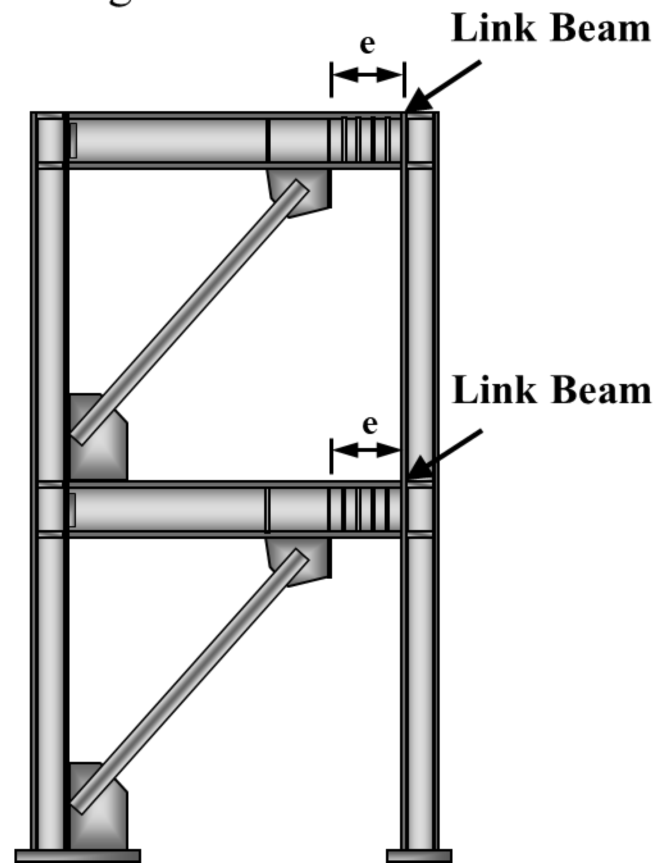
Center line of all structural elements (diagonals, columns, beams) pass from the same point

# Buildings: Braced Frames

Braced frames (centrically or eccentrically) are also used to resist lateral loads. While doing so, they also provide lateral support to the columns; enabling them to have a lower *KL* value.

**Eccentrically Braced Frames**



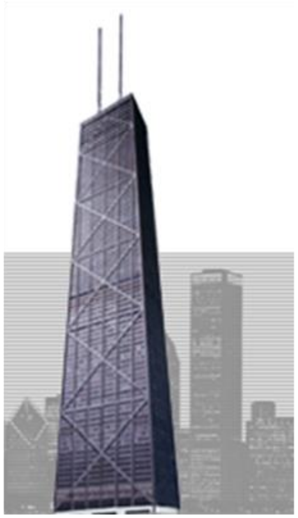



# Buildings: Braced Frames

Pros and Cons of Braced Frames

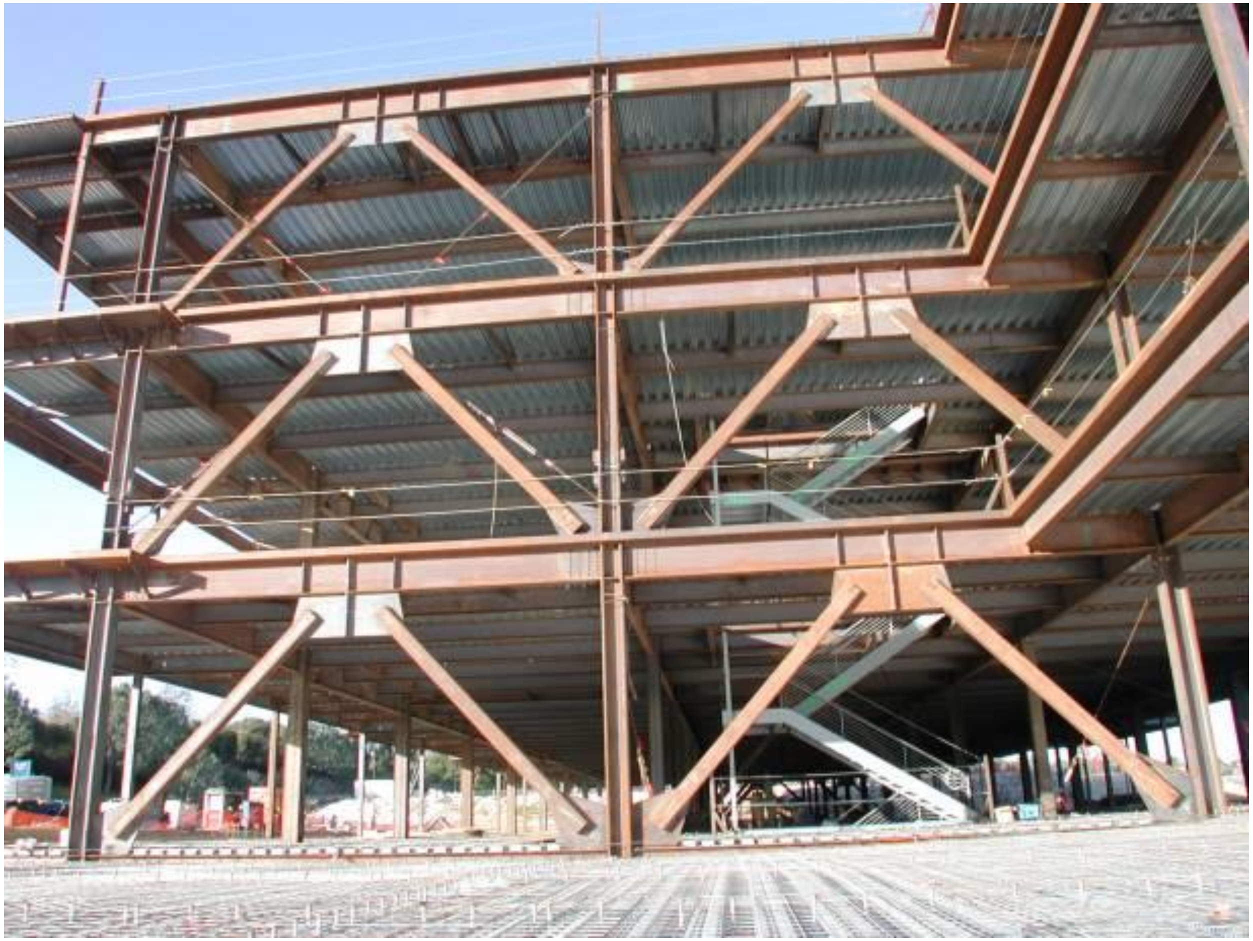
- They are stiffer than moment frames; which is preferred under service loads
- They are less ductile than moment frames due to brittle failure modes
- They might be cheaper than moment frames due to simple shear connections and potentially lighter columns (very tempting for owners)
- Less room for architectural openings
- In selecting the type of brace to be used, the following facts should be considered: (a) The cost of the braces and connections
  (b) Opening requirements of the architect
  (c) In high seismic regions a brace system with high ductility should be preferred

# Buildings: Centrically Braced Frames

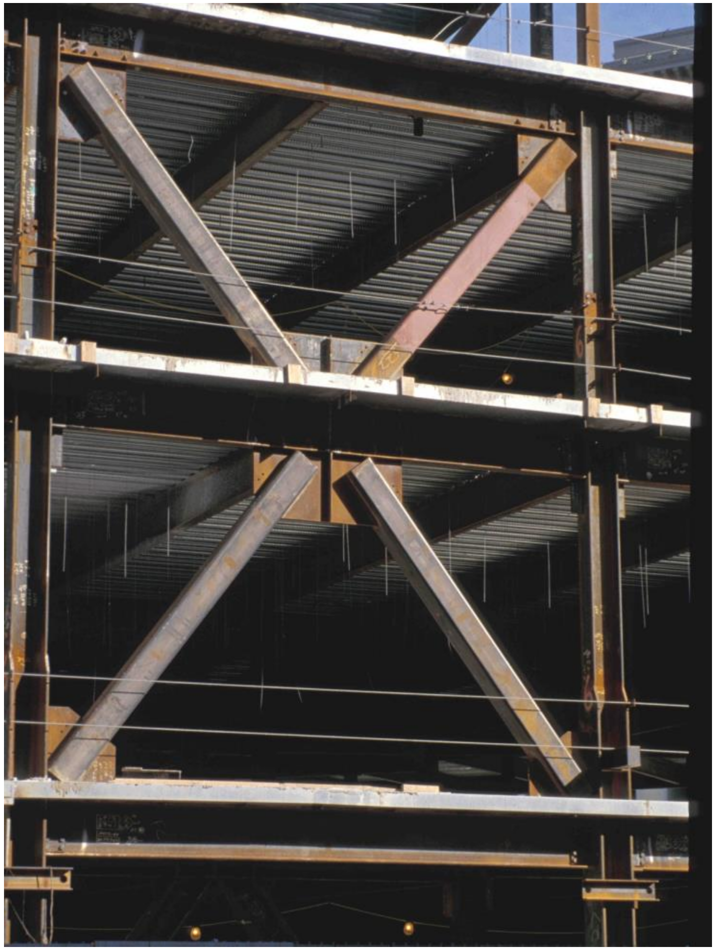



JOHN HANCOCK CENTER, Chicago, IL (100 story)

# Buildings: Centrically Braced Frames



## Buildings: Centrically Braced Frames



## Buildings: Centrically Braced Frames

2 Story X-Brace (X-brace elements are heavy I-sections): 70 story Landmark Tower, Japan

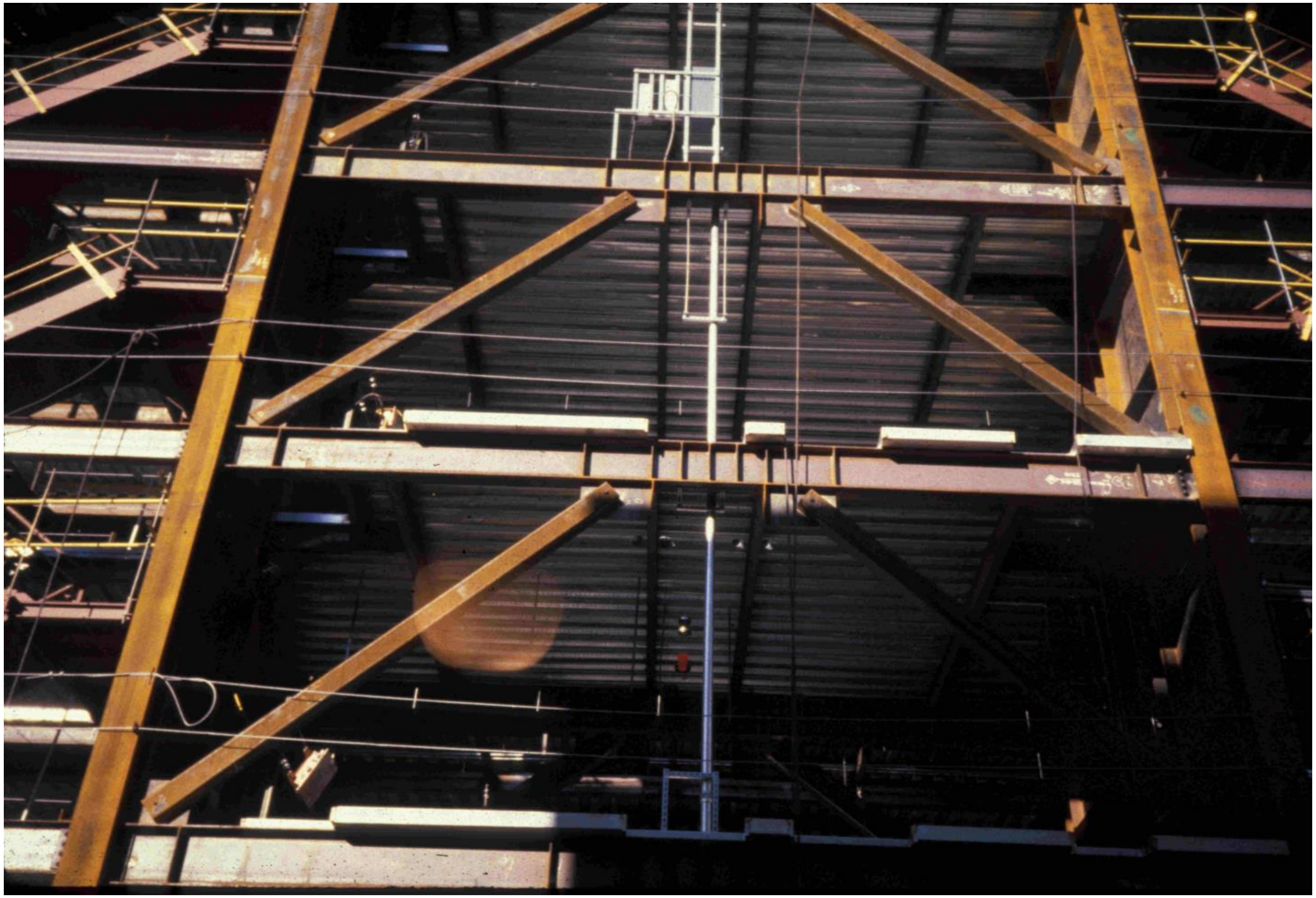
# Buildings: Centrically Braced Frames

2 Story X-Brace

## Buildings: Centrically Braced Frames

X-Brace in lower storys, Inverted V-Brace on upper story

## Buildings: Eccentrically Braced Frames



Link beams at beam mid-span, San Fransisco

# Buildings: Eccentrically Braced Frames

Single diagonal EBF in a high rise building,
Taipei, Taiwan
Columns are box-sections

# Buildings: Eccentrically Braced Frames

Link beam is connected to the box column with welds

## Buildings: Eccentrically Braced Frames


Link beams at beam mid-spans, San Francisco Airport

# BUILDING CODES AND DESIGN SPECIFICATIONS

**<u>Code</u>**

«Building construction in Turkey and in many parts of the world is regulated through the use of building codes that prescribe a consensus set of minimum requirements that will ensure public safety. A code consists of standards and specifications (or recommended practice), and covers all aspects of design, construction, and function of buildings, including occupancy and fire-related issues, but it only becomes a legal document within any jurisdiction after it is adopted by the legislative body in that jurisdiction. Once adopted by a jurisdiction, the code becomes the legal binding document for building construction in that country.»

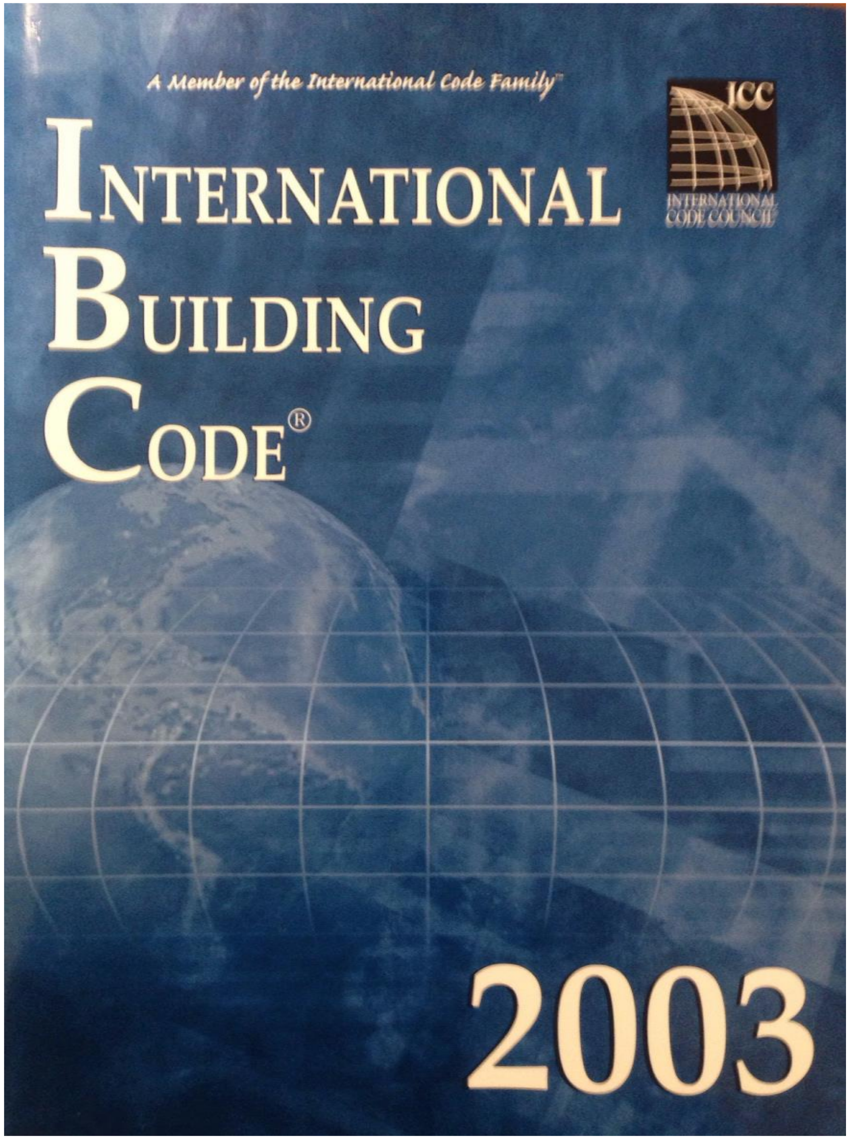
# BUILDING CODES AND DESIGN SPECIFICATIONS

**Code**

«The International Building Code (IBC 2006), published by the International Code Council (ICC), and is fast becoming the most widely used building code in the United States for the design of building structures. The current edition of the IBC 2006 now references the ASCE 7 load standard for calculation of all structural loads, including snow, wind, and seismic loads. The steel material section of the IBC references the AISC specifications as the applicable specification for the design of steel members in building structures.»

**Code**

The International Building Code (IBC 2006) addresses the design and installation of building systems through requirements emphasizing performance.



# BUILDING CODES AND DESIGN SPECIFICATIONS

**Specifications**

ANSI/AISC 360-15 Specification for Structural Steel Buildings

ANSI/AISC 360-10
An American National Standard

**Specification for Structural Steel Buildings**

June 22, 2010

Supersedes the
*Specification for Structural Steel Buildings*
dated March 9, 2005
and all previous versions of this specification

Approved by the AISC Committee on Specifications

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
One East Wacker Drive, Suite 700
Chicago, Illinois 60601-1802

# BUILDING CODES AND DESIGN SPECIFICATIONS

**Specifications**

AISC Steel Construction Manual


**Specifications**

Seismic Design Manual

# BUILDING CODES AND DESIGN SPECIFICATIONS

**Specifications**

ANSI/AISC 341-15 Seismic Provisions for Structural Steel Buildings

ANSI/AISC 341-10
An American National Standard

**Seismic Provisions for Structural Steel Buildings**

June 22, 2010

Supersedes the
*Seismic Provisions for Structural Steel Buildings*
dated March 9, 2005,
Supplement No. 1 dated November 16, 2005,
and all previous versions

Approved by the AISC Committee on Specifications

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
One East Wacker Drive, Suite 700
Chicago, Illinois 60601-1802

# BUILDING CODES AND DESIGN SPECIFICATIONS

**Specifications**

ANSI/AISC 358-15 Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications

ANSI/AISC 358-10
ANSI/AISC 358s1-11
An American National Standard

**Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications**

Including Supplement No. 1

2010
(includes 2011 supplement)

Supersedes ANSI/AISC 358-05 and ANSI/AISC 358s1-09

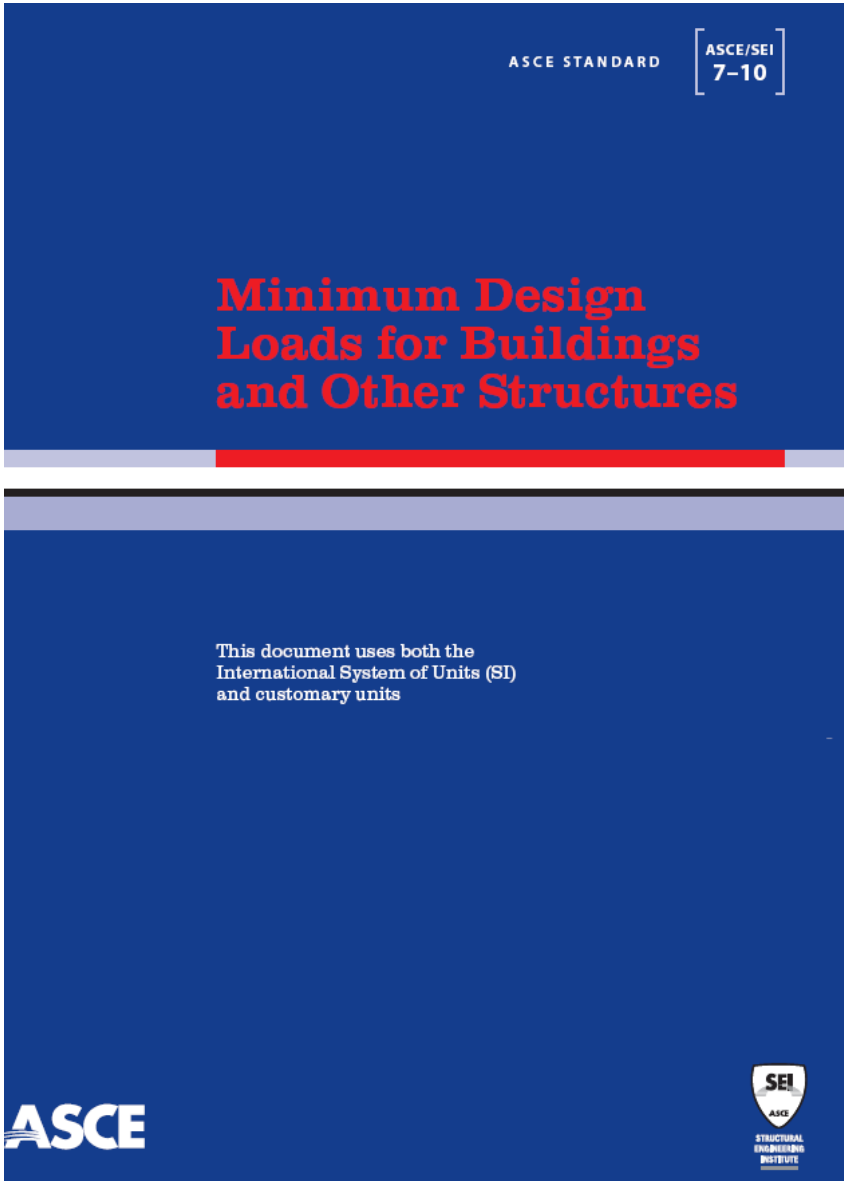
Approved by the AISC Connection Prequalification Review Panel
and issued by the AISC Board of Directors

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
One East Wacker Drive, Suite 700
Chicago, Illinois 60601-1802

# BUILDING CODES AND DESIGN SPECIFICATIONS

**<u>Specifications</u>**

ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures



# BUILDING CODES AND DESIGN SPECIFICATIONS

**AISC 2015**

A- General Provisions

B- Design Requirements

C- Stability Analysis and Design

D- Design of Members for Tension

E- Design of Members for Compression

**ÇYTE 2016**

1- Genel Esaslar

2- Malzeme

3- İmalat ve Montaj

4- Kalite Kontrolü

5- Çelik Yapıların Tasarımında Temel İlkeler

6- Stabilite Tasarımı

7- Eksenel Çekme Kuvveti Etkisi

8- Eksenel Basınç Kuvveti Etkisi

# BUILDING CODES AND DESIGN SPECIFICATIONS

| **AISC 2015** | **ÇYTE 2016** |
|---|---|
| F- Design of Members for Flexure | 9- Eğilme Momenti Etkisi |
| G- Design of Members for Shear | 10- Kesme Kuvveti Etkisi |
| H- Design of Members for Combined Forces and Torsion | 11- Bileşik Etkiler |
| I- Design of Composite Members | 12- Kompozit Elemanlar |
| J- Design of Connections | 13- Birleşimler ve Birleşim Araçları |
| K- Design of HSS and Box Member Connections | 14- Boru ve Kutu Enkesitli Elemanların Birleşimi |

# BUILDING CODES AND DESIGN SPECIFICATIONS

**AISC 2015**

L- Design for Serviceability

M- Fabrication, Erection, and Quality Control

Appendix 6. Stability Bracing for Columns and Beams

**ÇYTE 2016**

15- Kullanılabilirlik Sınır Durumları için Tasarım

16- Elemanlarda Stabilite Bağlantıları

# BUILDING CODES AND DESIGN SPECIFICATIONS

**AISC 2015 - Appendixes**

1. Inelastic Analysis and Design
2. Design for Ponding
3. Design for Fatigue
4. Structural Design for Fire Conditions
5. Evaluation of Existing Structures
6. Stability Bracing for Columns and Beams
7. Direct Analysis Method

**ÇYTE 2016 - Ekler**

1. Su Birikmesi (Göllenme) Etkisi
2. Yorulma Etkisi
3. Diyaframlar ve Yük Aktarma Elemanları
4. Önerilen Kaynaklar

**The Structural Design Process is Iterative**

1. Usually starts out with some schematic drawings developed by the architect for the owner of a building.

2. Using these schematic drawings, the structural engineer carries out a preliminary design to determine the preliminary sizes of the members for each structural material and structural system (gravity and lateral) considered. This information is used to determine the most economical structural material and structural system for the building.

**The Structural Design Process is Iterative**

3. After the structural material and systems are determined, then comes the final design phase where the roof and floor framing members and the lateral load systems are laid out and all the member sizes are proportioned to resist the applied loads with an adequate margin of safety.

4. This results in a set of construction documents that include structural plans, sections, details, and specifications for each of the materials used in the project.

**The Structural Design Process is Iterative**

5. After the final design phase comes the shop drawing and the construction phases during which the building is actually fabricated and erected.

   During the shop drawing phase, the steel fabricator's detailer uses the structural engineer's drawings to prepare a set of erection drawings and detail drawings that are sent to the structural engineer for review and approval.

## The Structural Design Process is Iterative

The shop drawing review process provides an opportunity for the design engineer to ensure that the fabrication drawings and details meet the design intent of the construction documents.

Once the shop drawings are approved, steel fabrication and erection can commence.

The importance of proper fabrication and erection procedures and constructible details to the successful construction of a steel project cannot be overemphasized.

**The Structural Design Process is Iterative**

In the United States and Canada, the design of simple connections (i.e., simple shear connections) is sometimes delegated by the structural engineer of record (EOR) to the steel fabricator, who then hires a structural engineer to design these connections using the loads and reactions provided on the structural drawings and/or specifications.

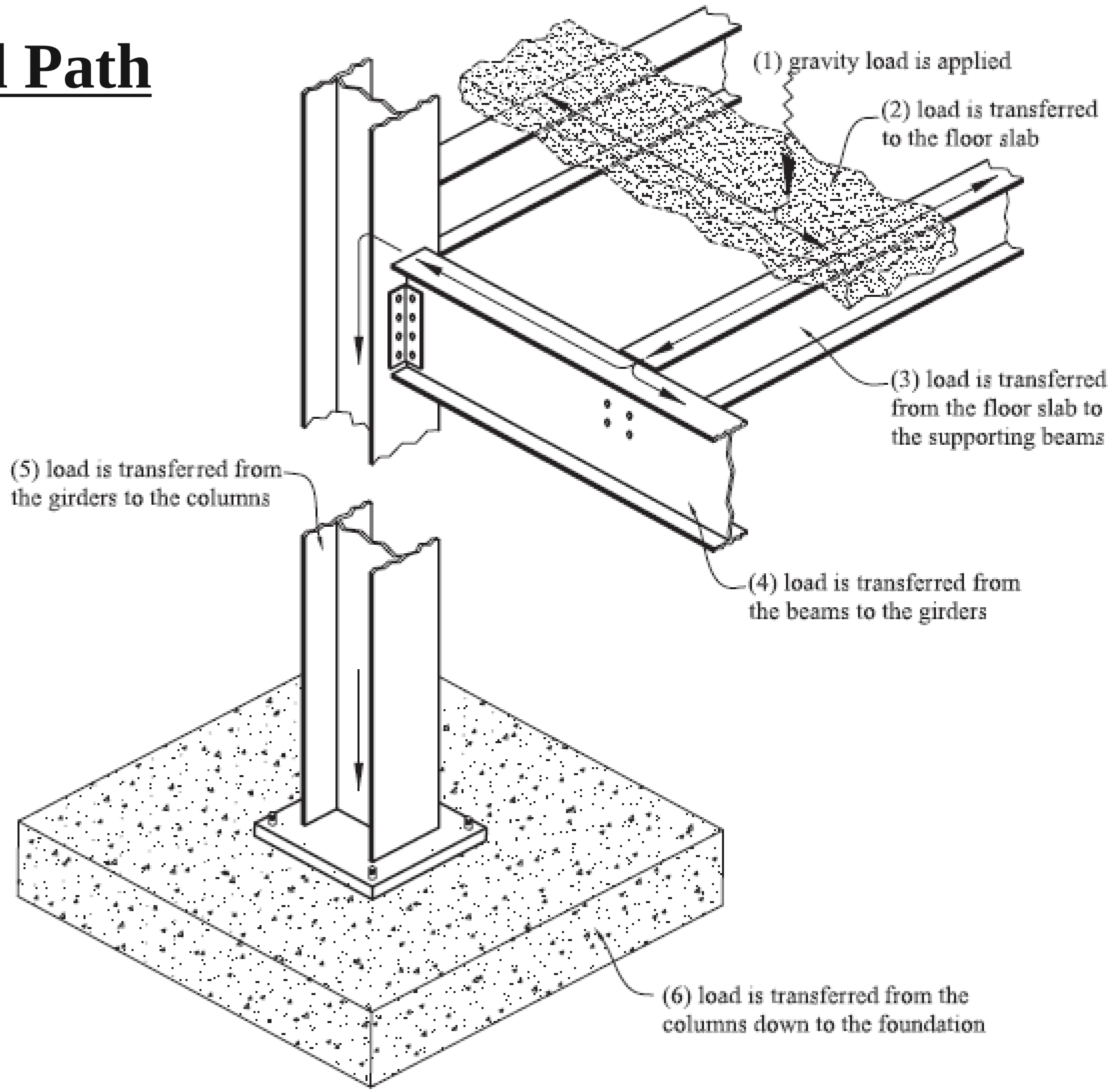
The connection designs and the detail drawings of the connections are also submitted to the structural engineer of record for review and approval. In other cases, especially for the more complicated connections such as moment connections, the EOR may provide schematic or full connection designs directly to the fabricator..

## Load Path

The load path is the trail that a load travels from its point of application on the structure until it gets to the foundation. Any structural deficiency in the integrity of the load path could lead to collapse; these commonly result from inadequate connections between adjoining structural elements rather than the failure of a structural member.

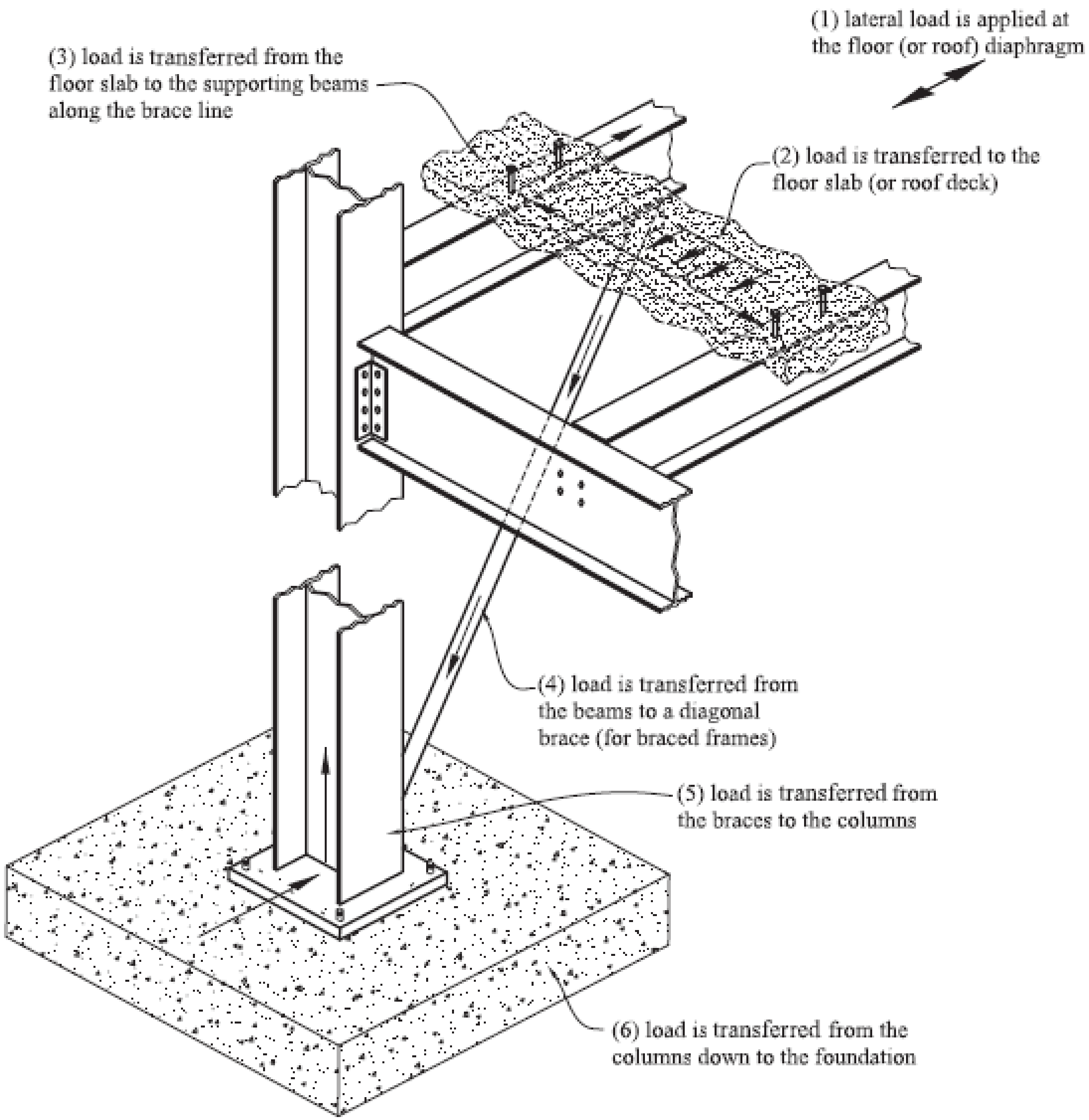
# GRAVITY AND LATERAL LOAD PATHS AND STRUCTURAL REDUNDANCY

## Vertical Load Path



# GRAVITY AND LATERAL LOAD PATHS AND STRUCTURAL REDUNDANCY

## Lateral Load Path



# GRAVITY AND LATERAL LOAD PATHS AND STRUCTURAL REDUNDANCY

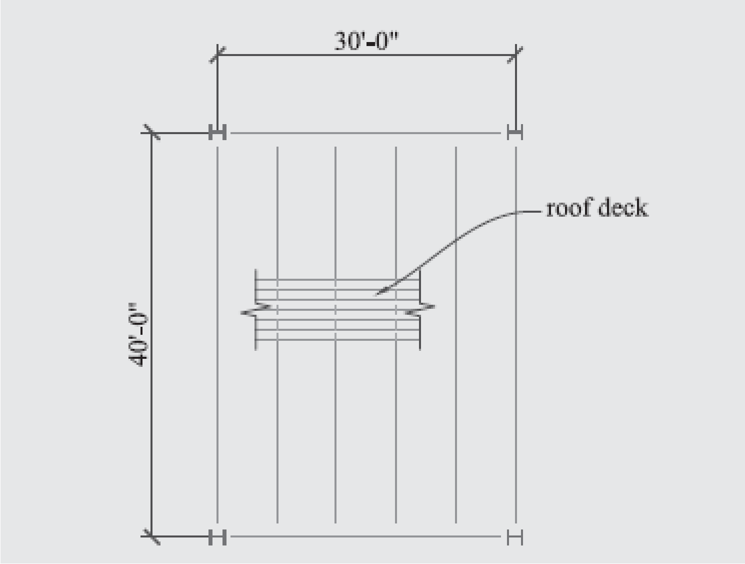
**Redundancy**

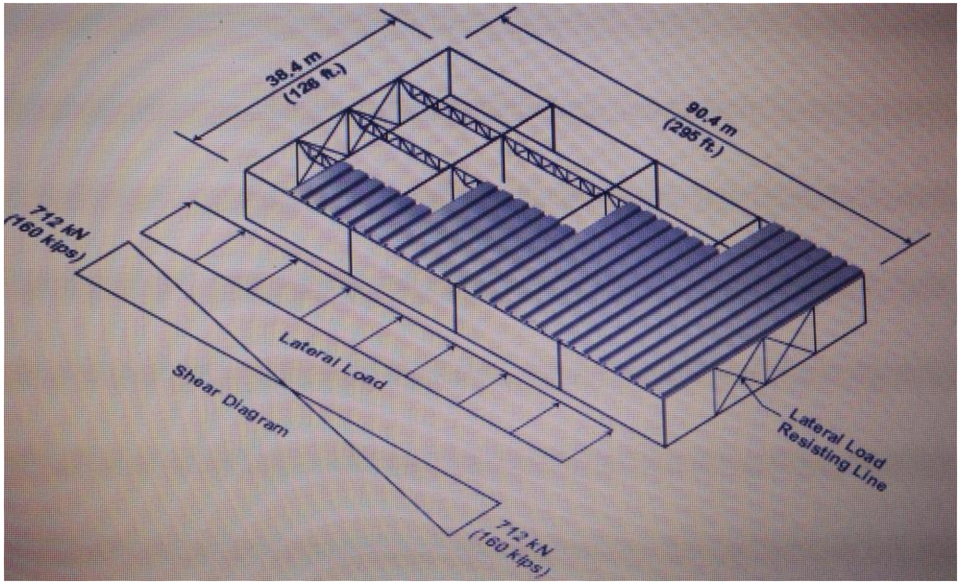
Structural redundancy—which is highly desirable in structural systems—is the ability of a structure to support loads through more than one load path, thus preventing progressive collapse. In a redundant structure, there are alternate load paths available so that failure of one member does not lead to failure of the entire structure; thus, the structure is able to safely transmit the load to the foundation through the alternate load paths.

# ROOF AND FRAMING LAYOUT

Once the gravity loads acting on a building are determined, the next step in the design process, before any of the structural members or elements can be designed, is the layout of the roof and floor framing and the lateral force-resisting systems.

<u>Floor Decking</u>



# INTRODUCTION

**The Intent of Structural Design**

To select member sizes and connections whose:

- Strength is higher than the effect of the applied loads
- Deflections and vibrations are within the prescribed limits.

**Design Methods Prescribed in the AISC and Turkish Specifications:**

- Allowable strength design (ASD) method (Güvenlik katsayıları ile tasarım, GKT, yöntemi)
- Load and resistance factor design (LRFD) method (Yük ve dayanım katsayıları ile tasarım, YDKT, yöntemi)

Appendix 1 of the AISC specification also allows inelastic methods of design such as the plastic design (PLD) method.

# INTRODUCTION

**History of the AISC Specification**

**Allowable Strength Design (ASD)**
**(Emniyet Gerilmeleri ile Tasarım (EGT))**

**Load and Resistance Factor Design (LRFD)**
**(Yük ve Dayanım Katsayıları ile Tasarım (YDKT))**

| ASD (EGT) | LRFD (YDKT) |
|---|---|
| 1920 1$^{st}$ Specification | 1986 1$^{st}$ Specification |
| 1978 8$^{th}$ Specification | 1993 2$^{nd}$ Specification |
| 1989 9$^{th}$ Specification (Supp.1-2001) | 1999 3$^{rd}$ Specification |
| | 2002 1$^{st}$ Seismic Specification |

# INTRODUCTION

**Allowable Strength Design (ASD)**
**(Güvenlik Katsayıları ile Tasarım (GKT))**

**Load and Resistance Factor Design (LRFD)**
**(Yük ve Dayanım Katsayıları ile Tasarım (YDKT))**

ASD (GKT) and LRFD (YDKT)

2010 14th Specification
2015 15th Specification

2010 3rd Seismic Specification
2015 4th Seismic Specification

# HISTORY OF AISC SPECIFICATION

**Prior to 2005**

Designers were free to use either the allowable stress design or load and resistance factor design method.

ASD was based on the concept that stresses under service loads should remain below certain allowable stress values.

In LRFD the applied loads are increased by load factors (ultimate loads) and limit state capacities are calculated.

ASD or LRFD, steel structures are safe, economical and easy to design.

**Post 2005**

Safety factors ($\Omega$) that are compatible with ASCE-7 (ASD)

Resistance factors ($\phi$) that are compatible with ASCE-7 load combinations (LRFD)

# HISTORY OF TURKISH SPECIFICATION

<u>Prior to 1980:</u> ???

<u>1980 TS-648:</u>
Çelik Yapıların Hesap ve Yapım Kuralları (Emniyet Gerilmeleri Yöntemi)

<u>2007 Deprem Bölgelerinde Yapılacak Binalar Hakkında Esaslar</u>
Bölüm 4 – Çelik Binalar için Depreme Dayanıklı Tasarım Kuralları

<u>2016 Çevre ve Şehircilik Bakanlığı</u>
Çelik Yapıların Tasarım, Hesap ve Yapım Esaslarına Dair Yönetmelik (GKT veya YDKT)

<u>2016 Deprem Bölgelerinde Yapılacak Binalar Hakkında Esaslar</u>

# HISTORY OF TURKISH SPECIFICATION

**Post 2016**

Bring all designers together under a single design standard

Allowable stress design is not allowed anymore

Consistent with the other standards such as ASCE 7, IBC or NFPA building codes.

Steel structures are safe, economical and easy to design.

# HISTORY OF TURKISH SPECIFICATION

**<u>Post 2016</u>**

Safety factors ($\Omega$) that are compatible with ASCE-7 (ASD)

Resistance factors ($\phi$) that are compatible with ASCE-7 load combinations (LRFD)

# LIMIT STATES

**The Intent of Structural Design**

To select member sizes and connections whose:

- Strength is higher than the effect of the applied loads
- Deflections and vibrations are within the prescribed limits.

**Design Basis: Section 5.2.1 of the Turkish Specification (AISC B3)**

Design shall be such that no applicable **strength** or **serviceability limit state** shall be exceeded when the structure is subjected to all applicable load combinations.

**A limit state is the point at which a structure or structural member reaches its limit of usefulness.**

# LIMIT STATES

- Strength Limit State
- Serviceability Limit State

Design for strength includes analysis to determine required strength and proportioning to have adequate available strength.

Design for serviceability includes analysis to determine displacements/vibrations and proportioning to have adequate available stiffness.

# LIMIT STATES: SERVICEABILITY (SECTION 15, ÇYTE 2016)

«Serviceability is a state in which the function of a building, its appearance, maintainability, durability, and the comfort of its occupants are preserved under typical usage.

Limiting values of structural behavior for serviceability (such as maximum deflections and accelerations) shall be chosen with due regard to the intended function of the structure.» (AISC 2015)

**15.1 Load Combinations**

Serviceability shall be evaluated using applicable load combinations.

(1) $G + Q$

(2) $G + 0.5\,S$

(3) $G + 0.5\,Q$

(4) $G + 0.5\,\mathrm{Q} + W$

## 15.2 Vertical Deflection Checks

Deflections in structural members and structural systems shall be limited so as not to impair the serviceability of the structure.

Deflections due to service loads shall not exceed:

For floors: $$\frac{\Delta}{L} \leq \frac{1}{360}$$

For roof floors: $$\frac{\Delta}{L} \leq \frac{1}{240}$$

**15.2 Vertical Deflection Checks**

Deflections in structural members and structural systems shall be limited so as not to impair the serviceability of the structure.

Deflections due to dead and snow loads shall not exceed:

Total deflection: $$\frac{\Delta}{L} \leq \frac{1}{300}$$

Roof floors: $$\frac{\Delta}{L} \leq \frac{1}{150}$$

**15.3 Drift Checks**

Drift shall be limited so as not to impair the serviceability of the structure.

Story drift:
(recommended value under service loads)

$$\frac{\Delta}{L} \leq \frac{1}{400}$$

## 15.4 Vertical Vibration Checks

The effect of vibration on the comfort of the occupants and the function of the structure shall be considered. The sources of vibration to be considered include occupant loading, vibrating machinery and others identified for the structure.

Natural vibration frequence of the slab system: (for limiting values look at Attachment 4)

$$f \cong \frac{18}{\sqrt{\delta}}$$

## 15.5 Wind Induced Motion

The effect of wind-induced motion of buildings on the comfort of occupants shall be considered (ex: wind induced vibrations)

## 15.6 Thermal Expansion and Contraction

The effects of thermal expansion and contraction of a building shall be considered (proper usage of expansion joints).

**Stability Limit States: Nominal Strength ($R_n$)**

1- Buckling of elements under axial load
- (a) Flexural buckling
- (b) Torsional buckling
- (c) Flexural torsional buckling
- (d) Local buckling

2- Buckling of elements under flexural loads
- (a) Lateral torsional buckling
- (b) Flange local buckling
- (c) Web local buckling
- (d) Shear buckling

3- Frame instability

4- Buckling due to concentrated loads

# LIMIT STATES: STRENGTH (SECTIONS 7-14, ÇYTE 2016)

**Fracture Limit States: Nominal Strength ($R_n$)**

1- Fracture along the effective net cross-sectional area

2- Block shear

3- Shear fracture of bolts

4- Weld fracture

**Yielding Limit States: Nominal Strength ($R_n$)**

Generally does not cause failure, but large deflections can arise.

# DESIGN METHODS FOR STRENGTH LIMIT STATES

Choose either of the following two design methods:

(a) Load and Resistance Factor Design (LRFD)

Yük ve Dayanım Katsayıları ile Tasarım (YDKT)

(b) Allowable Strength Method (ASD)

Güvenlik Katsayıları ile Tasarım (GKT)

Same strength limit states are checked for both of the methods (LRFD and ASD).

The nominal strengths (karakteristik dayanım) of the structural elements are the same in both methods.

There is a linear relationship between safety factors and resistance factors.

# DESIGN METHODS FOR STRENGTH LIMIT STATES

**<u>Design Philosophy for Strength Limit States</u>**

Required Strength ($R_a$ or $R_u$) $\leq$ Available Strength ($R_n/\Omega$ or $\phi R_n$)

| | |
|---|---|
| Required Strength: | $R_a$ (ASD) or $R_u$ (LRFD) |
| Nominal Strength (ASD or LRFD): | $R_n$ |
| Resistance Factor: | $\phi$ (LRFD) |
| Safety Factor: | $\Omega$ (ASD) |
| Allowable Strength: | $R_n/\Omega$ (ASD) |
| Design Strength: | $\phi R_n$ (LRFD) |

## 5.2.2 – Load and Resistance Factor Design (LRFD)
## (Yük ve Dayanım Katsayıları ile Tasarım (YDKT))

$$R_u \leq \phi R_n$$

$R_u = \Sigma Q_i \gamma_i$ = Required strength based on LRFD load combinations (YDKT yük birleşimi ile belirlenen gerekli dayanım)

$Q_i$ = Service load or load effect

$\gamma_i$ = Load factor (usually greater than 1.0)

$R_n$ = Nominal strength (Karakteristik dayanım)

$\phi$ = Resistance factor (Dayanım katsayısı)

$\phi R_n$ = Design strength (Tasarım dayanımı)

## 5.2.2 – Load and Resistance Factor Design (LRFD) (Yük ve Dayanım Katsayıları ile Tasarım (YDKT))

$$R_u \leq \phi R_n$$

$R_u = \Sigma \gamma_i Q_i$ = Required strength based on LRFD load combinations (YDKT yük birleşimi ile belirlenen gerekli dayanım)

$Q_i$ = Service load or load effect

$\gamma_i$ = Load factor (usually greater than 1.0)

Note that the service load (i.e., the unfactored or working load), $Q_i$ is the load applied to the structure or member during normal service conditions, while the factored or ultimate load, $R_u$, is the load applied on the structure at the point of failure or at the ultimate limit state.

# DESIGN METHODS FOR STRENGTH LIMIT STATES

## 5.2.2 – Load and Resistance Factor Design (LRFD) (Yük ve Dayanım Katsayıları ile Tasarım (YDKT))

$$R_u \leq \phi R_n$$

| Limit State | Resistance Factor ($\phi$) |
|---|---|
| Shear | 1.0 or 0.9 |
| Flexure | 0.9 |
| Compression | 0.9 |
| Tension (Yielding) | 0.9 |
| Tension (Rupture) | 0.75 |

## 5.2.3 – Allowable Strength Design (ASD) (Güvenlik Katsayıları ile Tasarım (GKT))

$$R_a \leq \frac{R_n}{\Omega}$$

$R_a = \Sigma Q_i$ = Required strength based on ASD load combinations (GKT yük birleşimi ile belirlenen gerekli dayanım)

$R_n$ = Nominal strength (Karakteristik dayanım)

$\Omega$ = Factor of safety (Güvenlik katsayısı)

$R_n / \Omega$ = Avalilable strength (Güvenli dayanımı)

## 5.2.3 – Allowable Strength Method (ASD) (Güvenlik Katsayıları ile Tasarım (GKT))

$$R_a \leq \frac{R_n}{\Omega}$$

| Limit State | Load Factor ($\Omega$) |
|---|---|
| Shear | 1.5 or 1.67 |
| Flexure | 1.67 |
| Compression | 1.67 |
| Tension (Yielding) | 1.67 |
| Tension (Rupture) | 2.0 |

**5.3 Loads and Load Combinations (Strength Limit State)**

Loads that are used in load combinations:

$G$ ($D$) : dead load

$Q$ ($L$) : live load

$Q_r$ ($L_r$) : roof live load

$S$ ($S$): snow load

$R$ ($R$) : rain load

$W$ ($W$) : wind load

$E$ ($E$) : seismic load

$F$ (F) : fluid loads

$T$ ($T$) : self straining load (ex: temperature change or support settlings

H ($H$) : lateral soil pressure, hydrostatic pressure and pressure of bulk materials

# DESIGN METHODS FOR STRENGTH LIMIT STATES

**Difference Between LRFD and ASD**

‘In the load and resistance factor design (LRFD) method, the safety margin is realized by using load factors and resistance factors that are determined from probabilistic analysis based on a survey of the reliability indices inherent in existing buildings and a preselected reliability index. The load factors vary depending on the type of load because of the different degrees of certainty in predicting each load type, and the resistance factors prescribed in the AISC specification also vary depending on the load effects.

For example, dead loads are more easily predicted than live or wind loads; therefore, the load factor for a dead load is generally less than that for a live load or a wind load. The load factors account for the possibility of overload in the structure.’

**Difference Between LRFD and ASD**

‘In the ASD method, the safety margin is realized by reducing the nominal resistance by a factor of safety, and a single load factor is generally used for all loads.

Since the LRFD method accounts for the variability of each load by using different load factors and the ASD assumes the same degree of variability for all loads, the LRFD method provides more uniform reliability and level of safety for all members in the structure, even for different loading conditions.

In the case of the ASD method, the level of safety is not uniform throughout the structure.’

# INTRODUCTION TO DESIGN LOADS

‘Structural loads are the forces that are applied on a structure (e.g., dead load, floor live load, roof live load, snow load, wind load, earthquake or seismic load, earth and hydrostatic pressure). The magnitude of these loads are specified in the **ASCE 7 load standard**, or TS498; in this standard, buildings are grouped into different occupancy types, which are used to determine the importance factor, *I*, for snow, wind, seismic, or ice load calculations.

The importance factor is a measure of the consequence of failure of a building to public safety; the higher the importance factor, the higher the snow, wind, or seismic loads on the structure.

ASCE 7, Table 1-1 should be used with ASCE 7, Tables 6-1, 7-4, 10-1, and 11.5-1 to determine the importance factors for wind loads, snow loads, ice loads, and seismic loads, respectively.’

## Gravity Load Resisting Systems

The two main types of floor systems used to resist gravity loads in building structures are:

one-way and two-way load distribution systems.

For steel structures, the one-way load distribution system occurs much more frequently than the two-way system; consequently, only one-way systems are discussed further.

## Gravity Load Resisting Systems

‘There are several one-way load distribution systems that are used in steel buildings. These systems support loads in one-way action by virtue of their construction and because the bending strength in one direction is several times greater than the strength in the orthogonal direction.

These types of one-way systems span in the stronger direction of the slab panel regardless of the aspect ratio of the panel. Examples of one-way systems used in steel buildings include

- Metal roof decks (used predominantly for roofs in steel buildings),
- Composite metal floor decks (used predominantly for floors in steel buildings), and
- Pre-cast concrete planks.’

**Gravity Load Resisting Systems**

The common types of gravity loads that act on building structures: - Roof dead load

- Floor dead load
- Roof live load
- Snow load
- Floor live load

**Tributary Widths and Tributary Areas**

‘These concepts are used to determine the distribution of floor and roof loads to the individual structural members.

The tributary width (TW) of a beam or girder is defined as the width of the floor or roof supported by the beam or girder, and is equal to the sum of one-half the distance to the adjacent beams to the right and left of the beam whose tributary width is being determined.’

**Tributary Widths and Tributary Areas**

'The tributary width is calculated as:

TW = 1⁄2(Distance to adjacent beam on the right) + 1⁄2(Distance to adjacent beam on the left).

The tributary area of a beam, girder, or column is the floor or roof area supported by the structural member.

The tributary area of a beam is obtained by multiplying the span of the beam by its tributary width.

The tributary area of a column is the plan area bound by lines located at one-half the distance to the adjacent columns surrounding the column whose tributary area is being calculated.'

**Tributary Widths and Tributary Areas**

- Beams are usually subjected to uniformly distributed loads (UDL) from the roof or floor slab or deck.
- Girders are usually subjected to concentrated, or point, loads due to the reactions from the beams. These concentrated loads or reactions from the beams have their own tributary areas.
- The tributary area of a girder is the sum of the tributary areas of all the concentrated loads acting on the girder.
- Perimeter beams and girders support an additional uniform load due to the loads acting on the floor or roof area extending from the centerline of the beam or girder to the edge of the roof or floor.

**Calculation of Tributary Width and Tributary Area: Example**

‘Using the floor framing plan showni determine the following:

a. Tributary width and tributary area of a typical interior beam,

b. Tributary width and tributary area of a typical spandrel or perimeter beam,

c. Tributary area of a typical interior girder,

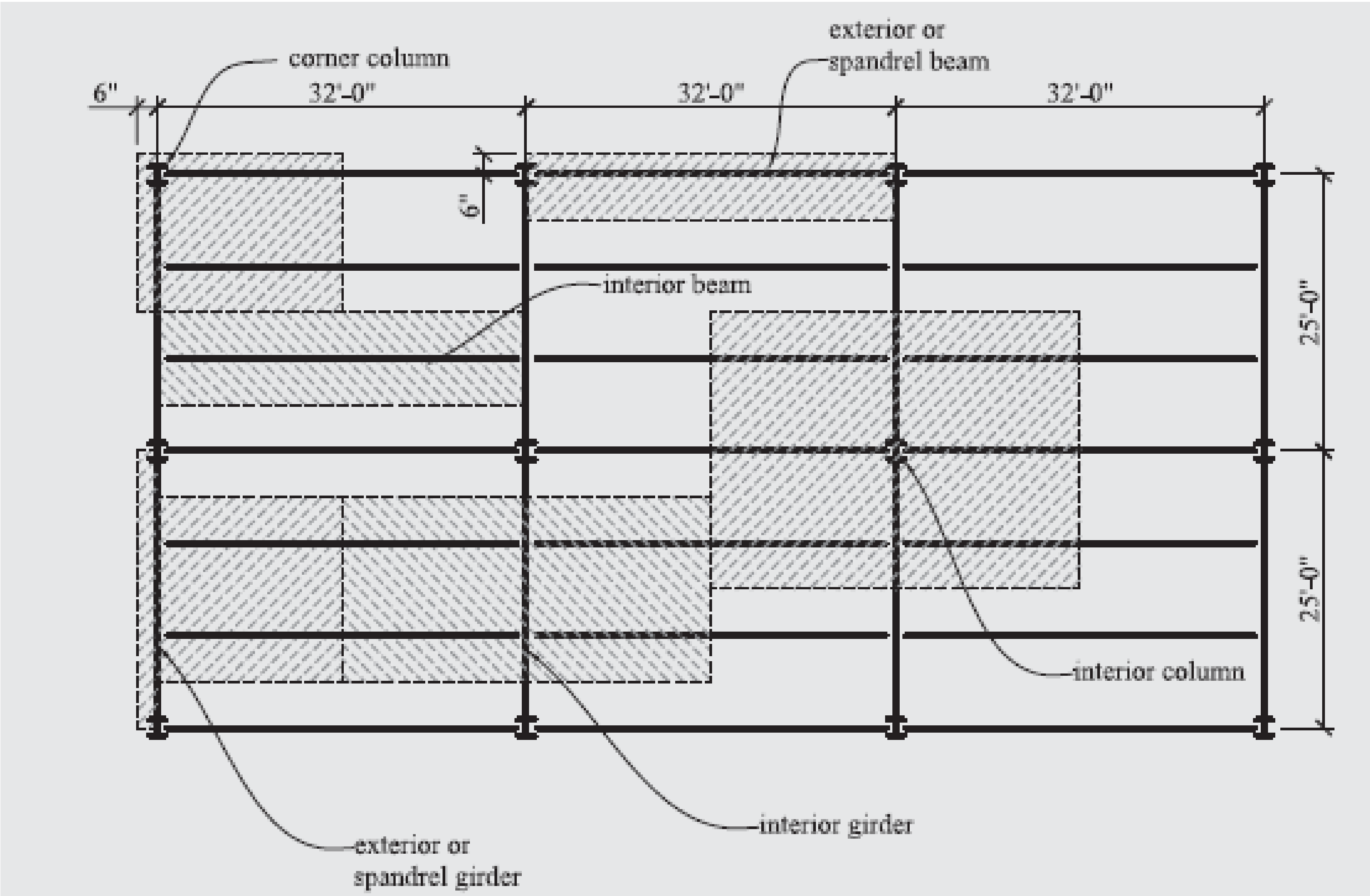
d. Tributary area of a typical spandrel girder,

e. Tributary area of a typical interior column,

f. Tributary area of a typical corner column, and

g. Tributary area of a typical exterior column.’

## Calculation of Tributary Width and Tributary Area: Example




6″ = 15 cm
32′ = 9.75 m
25′ = 7.60 m

**Live Loads**

'In general, any load that is not permanently attached to the structure can be considered a live load. The three main types of live loads that act on building structures are:

- Floor live load ($Q$ or $L$)
- Roof live load ($Q_r$ or $L_r$)
- Snow load.

Floor live loads, $L$, are occupancy loads that are specified in:

- ASCE 7, Table 4-1
- IBC, Table 1607.1
- TS 498, Çizelge 7'

**Live Loads**

'The magnitude of live loads, *L*, depends on the use of the structure and the tributary area (TA).

Floor live loads are usually expressed as uniform loads in units of pounds per square foot (psf) (in ASCE 7 or IBC) or kN/m$^2$ (in TS 498) of horizontal plan area. .

In certain cases (ASCE 7 and IBC), the code also specifies alternate concentrated floor live loads (in lb. or kip) that need to be considered in the design; but in most cases, the uniform loads govern the design of structural members.'

## Live Loads

‘The two types of live loads that act on roof members are roof live load, $L_r$, and snow load, $S$. From the load combinations presented earlier, it becomes apparent that **roof live loads and snow loads do not act together at the same time.**

Roof live loads rarely govern the design of structural members in higher snow regions, except where the roof is a special purpose roof used for promenades or as a roof garden.’

# FLOOR LIVE LOADS IN BUILDING STRUCTURES

‘Floor live loads are occupancy loads that depend on the use of the structure. These loads are assumed to be uniform loads expressed in units of $kN/m^2$ in TS 498 and pounds per square foot (psf) in IBC and the ASCE 7 load standard.

The live loads are determined from statistical analyses of a large number of load surveys. The IBC and ASCE 7 codes also specify alternate concentrated live loads for some occupancies because for certain design situations, live load concentrations—as opposed to uniform live loads—may be more critical for design. For example punching shear may be an issue, such as in thin slabs or in the design of stair treads where the concentrated loads instead of the uniform loads control the design of the member.’

# FLOOR LIVE LOADS IN BUILDING STRUCTURES

## 12 - DÜŞEY HAREKETLİ YÜKLER

### 12.1 - DÜZGÜN YAYILI HAREKETLİ YÜKLER (Çatı, Döşeme, Merdiven İçin)

ÇİZELGE 7 - Düzgün Yayılı Düşey Hareketli Yük Hesap Değerleri

| Kullanma Şekli | | | | Hesap Değeri |
|---|---|---|---|---|
| | ÇATILAR Yatay veya 1/20'ye kadar eğimli | Döşemeler | MERDİVENLER (Sahanlık ve merdiven girişi dahil) | $kN/m^2$ |
| 1 | | Çatı arası odalar | | 1,5 |
| 2 | Zaman zaman kullanılan çatılar | Konut, teras oda ve koridorlar, bürolar, konutlardaki 50 $m^2$'ye kadar olan dükkanlar, hastane odaları | | 2 |
| 3 | Konut toleranslarının kullanılması ve çiçeklik (bahçe yapılması) | Hastanelerin mutfakları, muayene odaları, poliklinik odaları, sınıflar, yatakhaneler, anfiler | Konut Merdivenleri | 3,5 |

# FLOOR LIVE LOADS IN BUILDING STRUCTURES

| | ÇATILAR Yatay veya 1/20'ye kadar eğimli | Döşemeler | MERDİVENLER (Sahanlık ve merdiven girişi dahil) | Hesap Değeri $kN/m^2$ |
|---|---|---|---|---|
| 4 | | – Camiler<br>– Tiyatro ve sinemalar,<br>– Spor dans ve sergi salonları,<br>– Tribünler (oturma yeri sabit olan)<br>– Toplantı ve bekleme salonları<br>– Mağazalar,<br>– Lokantalar<br>– Kütüphaneler<br>– Arşivler<br>– Hafif ağırlıklı atölyeler<br>– Büyük mutfaklar, kantinler<br>– Mezbahalar<br>– Fırınlar,<br>– Büyükbaş hayvan ahırları<br>– Balkonlar 10 $m^2$'ye kadar<br>– Büro, hastane okul, tiyatro sinema kütüphane depo vb. genel yapı koridorları | Umuma açık yapılarda büro hastane okul, tiyatro, kütüphane kitaplık vb. | 5 |
| 5 | | – Tribünler (oturma yeri sabit olmayan) | | 7,5 |
| 6 | | – Garajlar (Toplam ağırlığı 2,5 tona kadar olan araçlar için) | | 5 |

# FLOOR LIVE LOADS IN BUILDING STRUCTURES ASCE 7

**Table 4-1 Minimum Uniformly Distributed Live Loads, $L_o$, and Minimum Concentrated Live Loads**

| Occupancy or Use | Uniform psf (kN/m²) | Conc. lb (kN) |
|---|---|---|
| Apartments (see Residential) | | |
| Access floor systems | | |
| Office use | 50 (2.4) | 2,000 (8.9) |
| Computer use | 100 (4.79) | 2,000 (8.9) |
| Armories and drill rooms | 150 (7.18)[a] | |
| Assembly areas and theaters | | |
| Fixed seats (fastened to floor) | 60 (2.87)[a] | |
| Lobbies | 100 (4.79)[a] | |
| Movable seats | 100 (4.79)[a] | |
| Platforms (assembly) | 100 (4.79)[a] | |
| Stage floors | 150 (7.18)[a] | |
| Balconies and decks | 1.5 times the live load for the occupancy served. Not required to exceed 100 psf (4.79 kN/m²) | |
| Catwalks for maintenance access | 40 (1.92) | 300 (1.33) |

# FLOOR LIVE LOADS IN BUILDING STRUCTURES ASCE 7

| | | |
|---|---|---|
| Residential | | |
| One- and two-family dwellings | | |
| Uninhabitable attics without storage | 10 (0.48)[l] | |
| Uninhabitable attics with storage | 20 (0.96)[m] | |
| Habitable attics and sleeping areas | 30 (1.44) | |
| All other areas except stairs | 40 (1.92) | |
| All other residential occupancies | | |
| Private rooms and corridors serving them | 40 (1.92) | |
| Public rooms[a] and corridors serving them | 100 (4.79) | |
| Roofs | | |
| Ordinary flat, pitched, and curved roofs | 20 (0.96)[n] | |
| Roofs used for roof gardens | 100 (4.79) | |
| Roofs used for assembly purposes | Same as occupancy served | |
| Roofs used for other occupancies | [o] | [o] |
| Awnings and canopies | | |
| Fabric construction supported by a skeleton structure | 5 (0.24) nonreducible | 300 (1.33) applied to skeleton structure |

# FLOOR LIVE LOAD REDUCTION

‘To account for the low probability that floor structural elements with large tributary areas will have their entire tributary area loaded with the live load at one time, the IBC, ASCE 7 and TS 498 load specifications allow for floor live loads to be reduced provided that certain conditions are satisfied.’

In this course we will use the reduction outlined in ASCE 7.

# FLOOR LIVE LOAD REDUCTION: ASCE 7

The reduced design live load of a floor, $L$, in kN/m$^2$, is given as:

$$L = L_o\left(0.25 + \frac{4.57}{\sqrt{K_{LL}A_T}}\right)$$

where,

$L$ = reduced design live load per m$^2$ of area supported by the member

$L_o$ = unreduced design live load per m$^2$ of area supported by the member (see Table 4-1)

$K_{LL}$ = live load element factor (see Table 4-2)

$A_T$ = tributary area in m$^2$

$L$ shall not be less than $0.50L_o$ for members supporting one floor and $L$ shall not be less than $0.40L_o$ for members supporting two or more floors.

# FLOOR LIVE LOAD REDUCTION: ASCE 7

The reduced design live load of a floor, $L$, in kN/m$^2$, is given as:

$$L = L_o\left(0.25 + \frac{4.57}{\sqrt{K_{LL} A_T}}\right)$$

**Table 4-2 Live Load Element Factor, $K_{LL}$**

| Element | $K_{LL}$[a] |
|---|---|
| Interior columns | 4 |
| Exterior columns without cantilever slabs | 4 |
| Edge columns with cantilever slabs | 3 |
| Corner columns with cantilever slabs | 2 |
| Edge beams without cantilever slabs | 2 |
| Interior beams | 2 |
| All other members not identified, including: | 1 |
| Edge beams with cantilever slabs | |
| Cantilever beams | |
| One-way slabs | |
| Two-way slabs | |
| Members without provisions for continuous shear transfer normal to their span | |

[a]In lieu of the preceding values, $K_{LL}$ is permitted to be calculated.

# FLOOR LIVE LOAD REDUCTION: ASCE 7

**4.7.3 Heavy Live Loads**

Live loads that exceed 100 lb/ft$^2$ (4.79 kN/m$^2$) shall not be reduced.

EXCEPTION: Live loads for members supporting two or more floors shall be permitted to be reduced by 20 percent.

**4.7.4 Passenger Vehicle Garages**

The live loads shall not be reduced in passenger vehicle garages.

EXCEPTION: Live loads for members supporting two or more floors shall be permitted to be reduced by 20 percent.

**4.7.5 Assembly Uses**

Live loads shall not be reduced in assembly uses.

**4.7.6 Limitations on One-Way Slabs**

The tributary area, $A_T$, for one-way slabs shall not exceed an area defined by the slab span times a width normal to the span of 1.5 times the slab span.

# FLOOR LIVE LOAD REDUCTION: ASCE 7

'Roof live loads, $L_r$, are the weight of equipment and personnel on a roof during maintenance of the roof or the weight of moveable nonstructural elements such as planters or other decorative elements, or the use of the roof for assembly purposes. Like floor live loads, the unreduced roof live loads are also tabulated in the ASCE 7 load standard.'

**Roof Live Load Reduction**

'Only live loads on ordinary flat, pitched, or curved roofs can be reduced.'

$L_r = L_o R_1 R_2$ where $0.58 \leq L_r \leq 0.96$

where,

$L_r$ = reduced roof live load per $m^2$ of horizontal projection supported by the member

$L_o$ = unreduced design roof live load per $m^2$ of horizontal projection supported by the member (see Table 4-1)

## Roof Live Load Reduction

The reduction factors $R_1$ and $R_2$ shall be determined as follows:

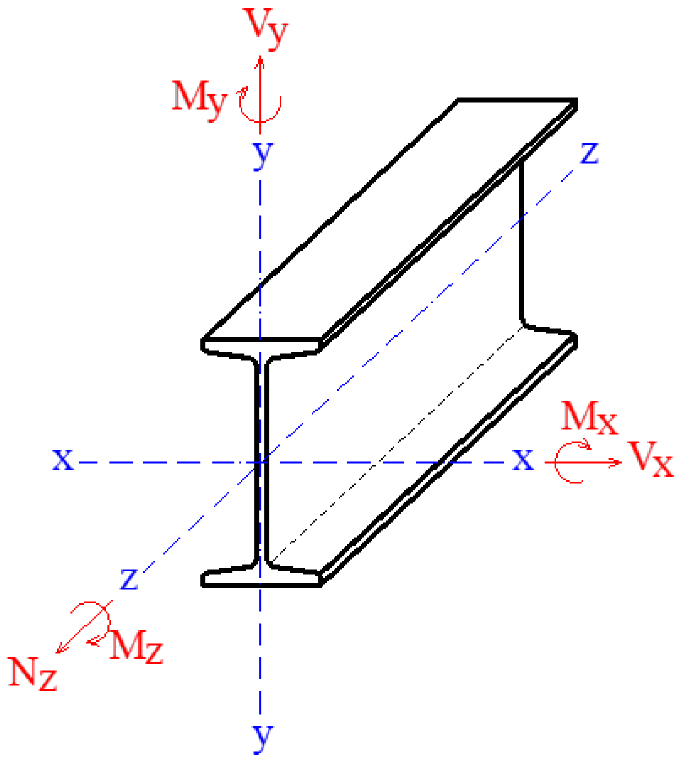
$$R_1 = \begin{cases} 1 & \text{for } A_T \leq 18.58 \text{ m}^2 \\ 1.2 - 0.011A_T & \text{for } 18.58 \text{ m}^2 < A_T < 55.74 \text{ m}^2 \\ 0.6 & \text{for } A_T \geq 55.74 \text{ m}^2 \end{cases}$$

where $A_T$ = tributary area in m$^2$ supported by the member and

$$R_2 = \begin{cases} 1 & \text{for } F \leq 4 \\ 1.2 - 0.05F & \text{for } 4 < F < 12 \\ 0.6 & \text{for } F \geq 12 \end{cases}$$

where, for a pitched roof, $F$ = number of inches of rise per foot (in SI: $F = 0.12 \times$ slope, with slope expressed in percentage points) and, for an arch or dome, $F$ = rise-to-span ratio multiplied by 32.

# SECTIONAL PROPERTIES




$N_z$ : Normal Force → Normal Stress ($\sigma$)

$V_y$ : Shear Force→ Shear Stress ($\tau$)

$V_x$ : Shear Force→ Shear Stress ($\tau$)

$M_x$ : Flexural Moment→ Normal Stress ($\sigma$)

$M_y$ : Flexural Moment → Normal Stress ($\sigma$)

$M_z$ : Torsional Moment→ Shear Stress ($\tau$)

# SECTION PROPERTIES

## ➢ Polar Moment of Inertia (J):

Polar Atalet Momenti

The **polar moment of inertia**, also known as **second polar moment of area**, is a quantity used to describe resistance to **torsional** deformation.

$$I_p = \int_A r^2 dA = \int_A (x^2 + y^2) dA \Rightarrow I_p = I_x + I_y$$

## ➢ Radius of Gyration, [$r_x$, $r_y$] ($i_x$, $i_y$):

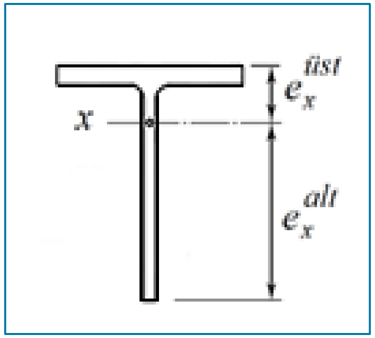
Atalet yaricapi

$$i_x = \sqrt{\frac{I_x}{A}} \quad , \quad i_y = \sqrt{\frac{I_y}{A}}$$

## ➢ Section Modulus, [$S_x$] ($W_{ex}$):

Elastik Mukavemet Momenti

$$W_{ex}^{alt} = \frac{I_x}{e_x^{alt}} \quad , \quad W_{ex}^{üst} = \frac{I_x}{e_x^{üst}}$$



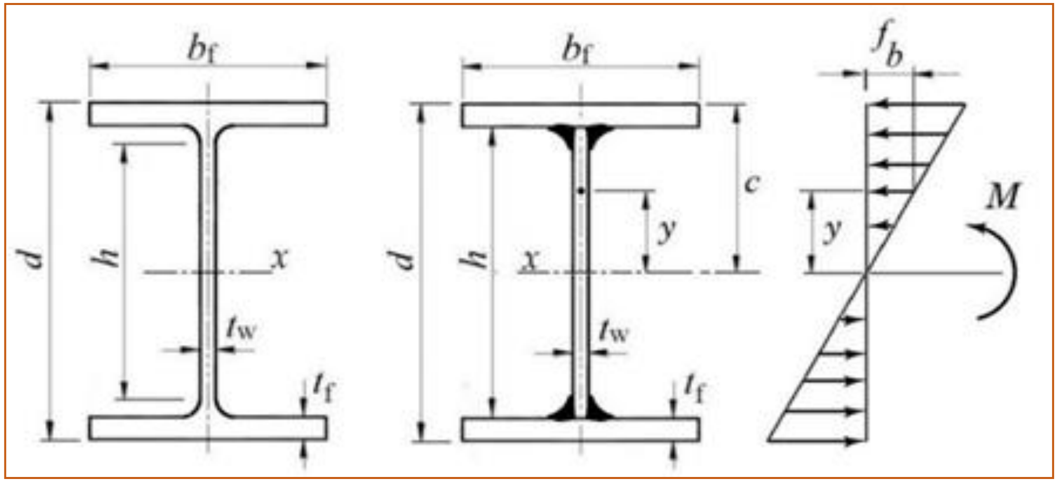

## ➢ Plastic Sectional Moment

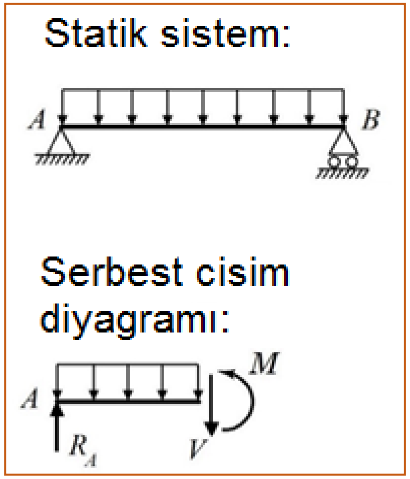
Plastik Moment

Elastic stress at any point of flexural member

$$f_b = \frac{M}{I_x} y$$

# SECTION PROPERTIES




$$f_{\max} = \frac{M}{I_x} c = \frac{M}{I_x / c} = \frac{M}{W_{ex}}$$

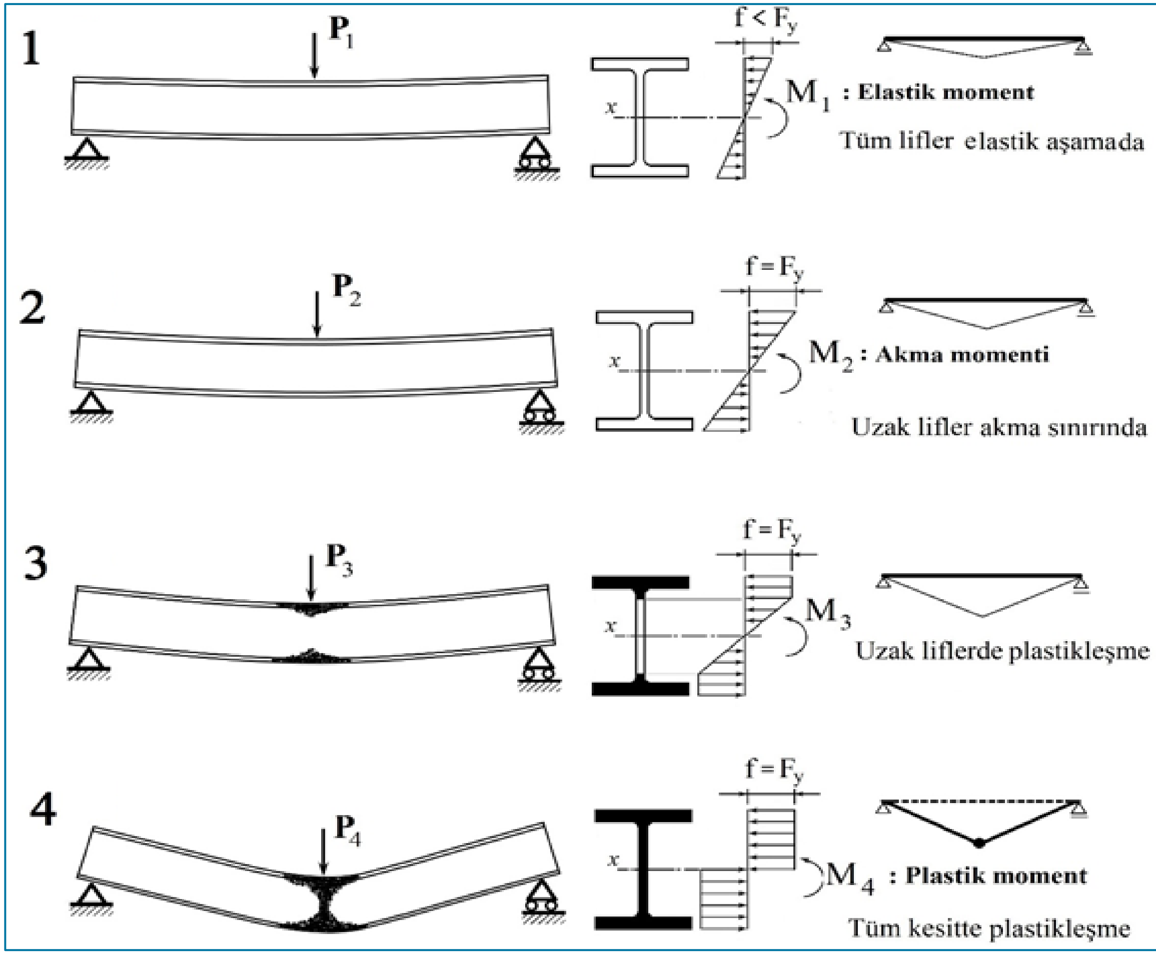
$$M_y = F_y \, W_{ex}$$

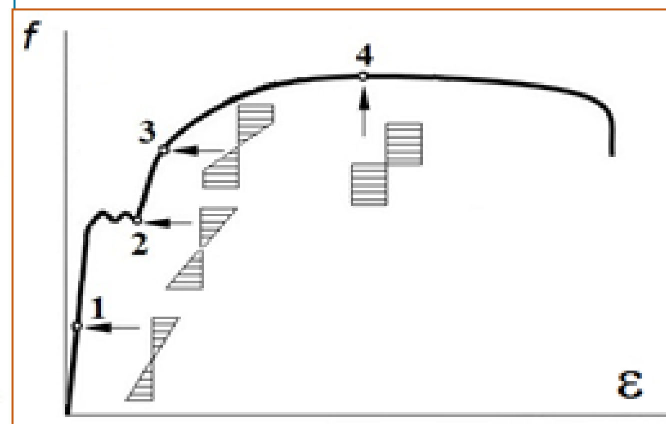
$f_{max} \leq F_y$

$M_y$ : Yield Moment,
$W_{ex}$ : Elastic Sectional Modulus

# SECTION PROPERTIES

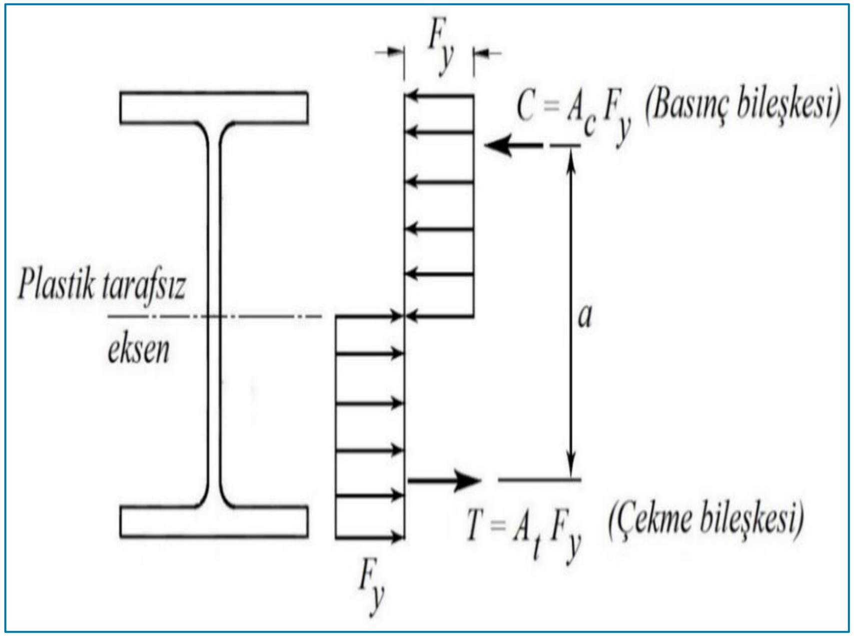
If you increase the loading at the middle of the beam



# SECTION PROPERTIES

***<u>The plastic neutral axis divides the cross section into two equal areas.</u>*** Elastic and plastic neutral axes are the same in sections symmetrical with respect to the bending axis.




$$C = T \qquad A_c F_y = A_t F_y \rightarrow A_c = A_t$$

$$M_p = F_y A_c a = F_y A_t a = F_y \frac{A}{2} a = F_y W_{px}$$

$A$ :Total sectional area

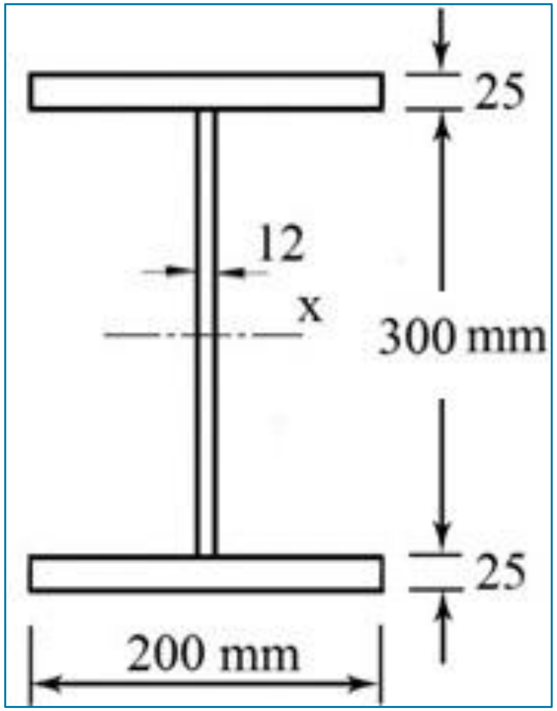
$Ac, At$ : Compression and Tension section area

$Wpx$ : Plastic section modulus: $Wpx = (A/2)a$

# EXAMPLE 1

What is the cross section
a) Yield moment,
b) Plastic Section (Malzeme: S335).

**Solution: a)**




$$F_y = 355\ \text{N/mm}^2 \ \ (35.5\ \text{kN/cm}^2)$$

$$I_X = \frac{1.2 \times 30^3}{12} + 2\left[\frac{20 \times 2.5^3}{12} + 20 \times 2.5 \times \left(\frac{30}{2} + \frac{2.5}{2}\right)^2\right]$$

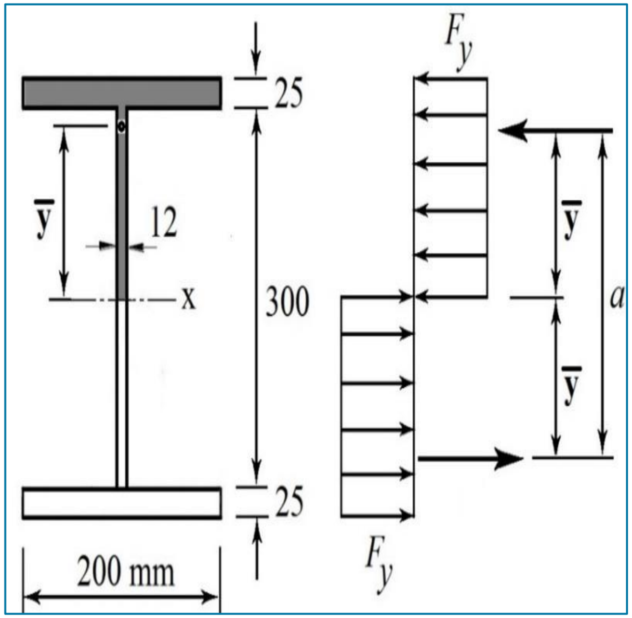
$$= 29158.3\ \text{cm}^4$$

$$W_{ex} = \frac{I_X}{c} = \frac{29158.3}{(30/2) + 2.5} = 1666.2\,\text{cm}^3$$

$$M_y = F_y\, W_{ex} = 35.5 \times 1666.2 = 59150\ \text{kNcm}$$

**b)** Since the section is symmetrical with respect to the x-axis, this axis divides the section into two equal parts. Hence the x-axis is also the plastic neutral axis.

$$\bar{y}=\frac{\sum A\times y}{\sum A}=\frac{20\times 2.5\times\left(\frac{30}{2}+1.25\right)+\frac{30}{2}\times 1.2\times\frac{30/2}{2}}{20\times 2.5+1.2\times(30/2)}=13.93\ cm$$




$$a=2\bar{y}=2\times 13.93=27.86\ cm$$

$$W_{px}=\frac{A}{2}a=\frac{136}{2}\times 27.86=1894\ \text{cm}^3$$

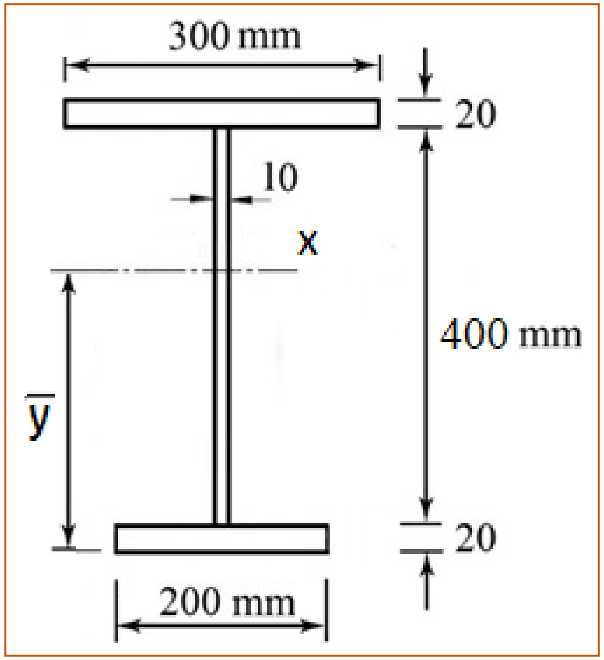
$$M_p=F_y\,W_{px}=35.5\times 1894=67237\ \text{kNcm}$$

# EXAMPLE 2

Calculate the elastic ($W_{ex}$) and plastic ($W_{px}$) strength moments of the build-up section in the figure.

**Solution**

Elastic Neutral Axis: (center of gravity)




$$\bar{y} = \frac{\sum A \times y}{\sum A}$$

$$= \frac{20.0 \times 2.0 \times 1.0 + 40.0 \times 1.0 \times 22.0 + 30.0 \times 2.0 \times 43.0}{20.0 \times 2.0 + 40.0 \times 1.0 + 30.0 \times 2.0}$$

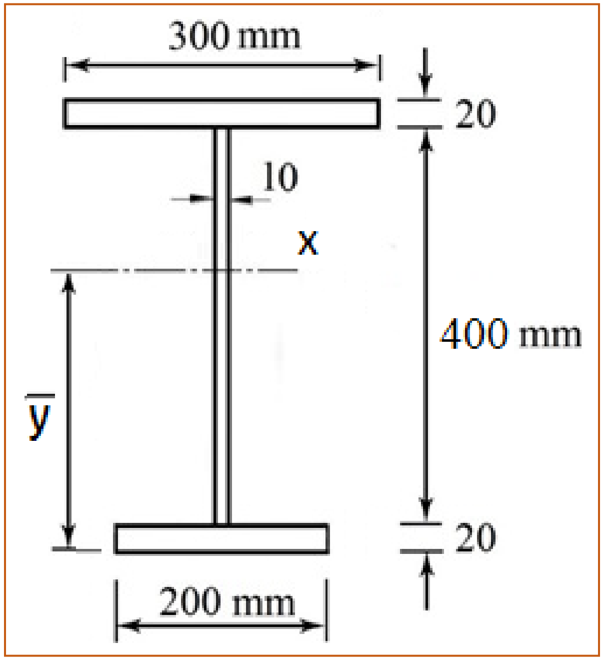
$$= 25.0 \text{ cm}$$

# EXAMPLE 2

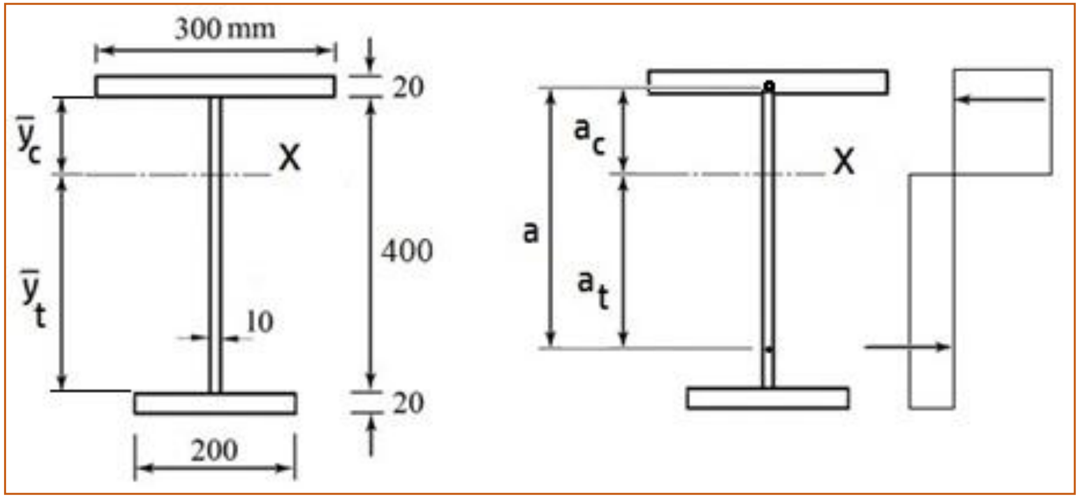
$$I_x = \frac{20.0\times 2.0^3}{12} + 20.0\times 2.0\times 24.0^2 + \frac{1.0\times 40.0^3}{12} + 1.0\times 40.0\times 3.0^2 + \frac{30.0\times 2.0^3}{12} + 30.0\times 2.0\times 18.0^2$$

$$= 48206.67\ cm^4$$

$$W_{ex}^{alt} = \frac{I_x}{25} = \frac{48206.67}{25} = 1928.27\ cm^3 \quad , \quad W_{ex}^{üst} = \frac{I_x}{19} = \frac{48206.67}{19} = 2537.2\ cm^3$$



# EXAMPLE 2




$$A = 30\times2 + 40\times1 + 20\times2 = 140\ cm^2, \qquad A_c = A_t = 140/2 = 70\,cm^2$$

$$1\times\bar{y}_c = 70 - 30\times2 \ \rightarrow\ \bar{y}_c = 10\ cm, \qquad 1\times\bar{y}_t = 70 - 20\times2 \ \rightarrow\ \bar{y}_t = 30\ cm$$

$$a_c = \frac{\sum A\times y}{\sum A} = \frac{10\times1\times5 + 30\times2\times11}{70} = 10.14\ cm, \qquad a_t = \frac{30\times1\times15 + 20\times2\times31}{70} = 24.14\ cm$$

$$a = a_c + a_t = 10.14 + 24.14 = 34.28\,cm \ \rightarrow\ W_{px} = \frac{A}{2}a = 70\times34.28 = 2400\ cm^3$$