CHAPTER II

LITERATURE REVIEW

2.1 Steel

Steel is an alloy of iron and carbon. The earliest use of iron, the main component of steel, was for small tools in approximately 4000 B.C. or the Bronze Age. Steel was in the form of wrought iron and produced from the process of heating metal ores in a charcoal fire or we know now as smelting. (Segui, 2013). Steel, as a structural material, was firstly built in England in 1777-1779. During the period from 1780 until 1820, arch-shaped structures are being built by cast-iron pieces forming bars or trusses in which these pieces were compiled as main girders of the structure. (Salmon, et al., 2009).

According to McCormac & Csernak (2012), the advantages of steel as a structural material follows:

1. Steel has high strength in order to support structures such as long-span bridges, tall buildings, structures situated in poor foundation, and workshops.
2. Steel has a consistent property which do not change in a considerable amount of time compared to reinforced-concrete structure.
3. The moments of inertia of a steel structure can be calculated accurately as it follows Hooke's law up to high stresses, hence engineers are likely to use steel as a design material.
4. If steel structures are properly maintained, the structure will last indefinitely. Maintenance such as annual inspection of corrosion and paint will increase the longevity of the structure.
5. Steel can withstand substantial deformation without failure under high tensile stress.
6. Even though steel member is loaded until it has large deformation, it is still able to withstand large forces loaded upon them.

7. Steel structures are easy to have additions made to the structure itself.
8. Steel structures have simple connection devices such as welds and bolts that can be fastened together easily.
9. Construction time is shortened due to its speed upon erection.
10. Steel can be reuse for another structure if disassembled, or can be sold as scrap if it is not reusable.

As for the disadvantages of steel, McCormac & Csernak (2012) concluded that:

1. Steel are prone to corrosion when it is exposed to air and water. Corrosion may lead to reduction of strength of the structure. Hence, maintenance such as painting is needed to prevent corrosion
2. Steel’s strength can be reduced dramatically at the burning of other materials around it. With the characteristic of steel being a good conductor, heat can be easily transferred to other members of the steel structure. In order to prevent such happening, the steel structure can be protected by applying and installing insulation.
3. Steel are susceptible towards buckling. To prevent buckling, additional structures may be added but it may increase its cost.
4. The strength of steel may be reduced if the steel structure is exposed to a large number of stress reversals or a variation of tensile stress.
5. Under certain conditions such as fatigue-type loadings, low temperature or even triaxial stress condition may lead to steel losing its ductility and eventually leads to brittle fracture.

2.2 Castellated Beam

According to Boyer (1964), castellated beam is a type of expanded beam in which a steel profile is being expanded and creates a regular pattern of holes in the web of the profile. The name castellated means “built like a castle, having battlements, or regular holes in the walls, like a castle”.

Castellated beams are usually custom designed according to the design from a professional engineer in which a specific area that needs a strong and yet economical steel member. According to AISC, the fabrication of castellated beams is helped with computer operated cutting torch to cut a zigzag pattern along the web of a wide-flange section. After the cutting the section into the appropriate pattern, the waste of both ends of the beam is removed and the two sections are welded back together to form the castellated section. And lastly, a full or partial penetration butt weld is being made from either side of the web.

2.2.1 Classifications of Castellated Beam

According to Rugge (2017), the classifications of castellated beam are:

1. Hexagonal Openings

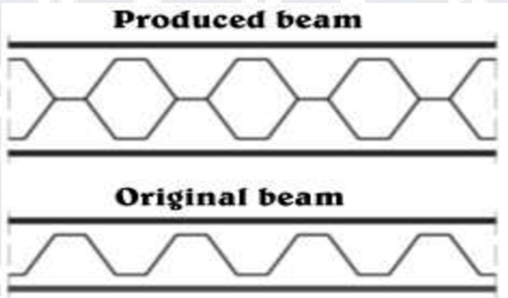

Figure 2.1 Hexagonal castellated beam. Source: Rugge & Pasnur (2017)

2. Circular Openings

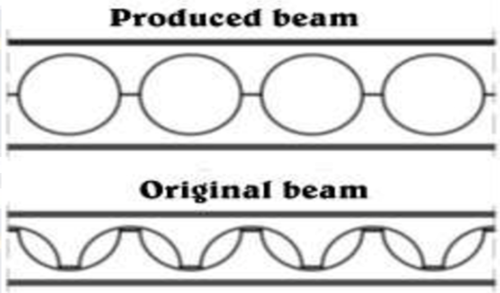

Figure 2.2 Circular castellated beam. Source: Rugge & Pasnur (2017)

3. Sinusoidal Openings

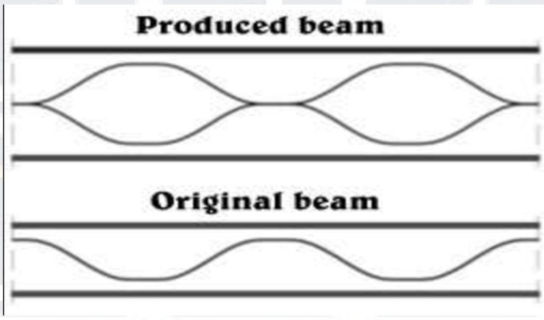

Figure 2.3 Sinusoidal castellated beam. Source: Rugge & Pasnur (2017)

4. Rectangular Openings

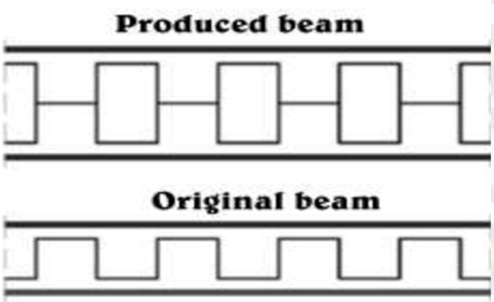

Figure 2.4 Rectangular castellated beam. Source: Rugge & Pasnur (2017)

While according to Jamadar (2015), castellated beam can be classified as:

1. Hexagonal Castellated Beam

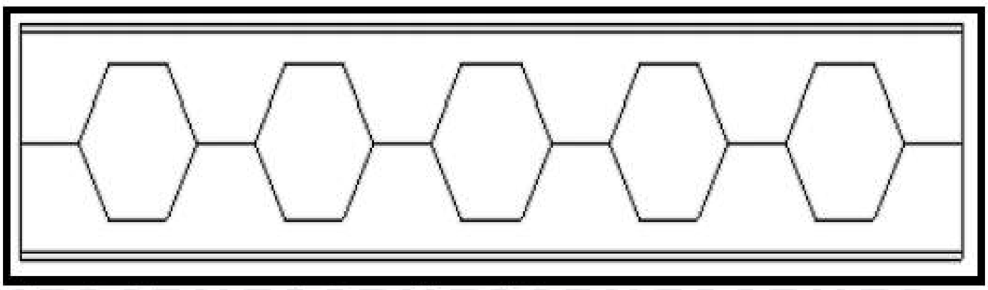

Figure 2.5 Hexagonal castellated beam. Source: Jamadar & Khumbar (2015)

2. Cellular Beam

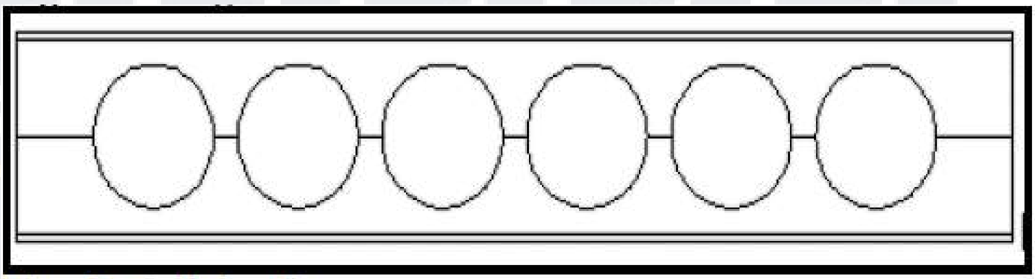

Figure 2.6 Cellular castellated beam. Source: Jamadar & Kumbhar (2015)

3. Castellated Beam with Diamond Shaped Opening

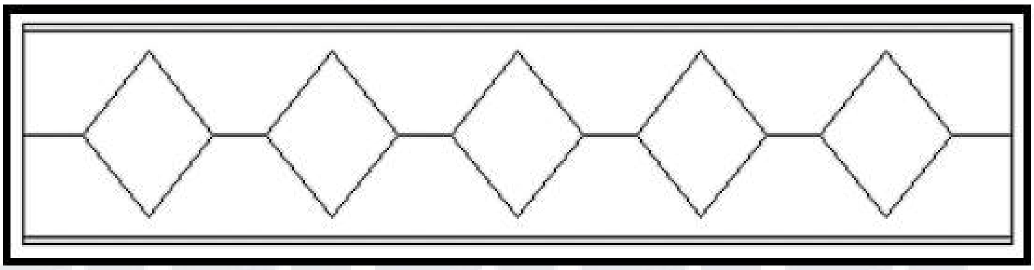

Figure 2.7 Diamond shaped opening castellated beam. Source: Jamadar & Kumbhar (2015)

2.2.2 Terminology of Castellated Beam

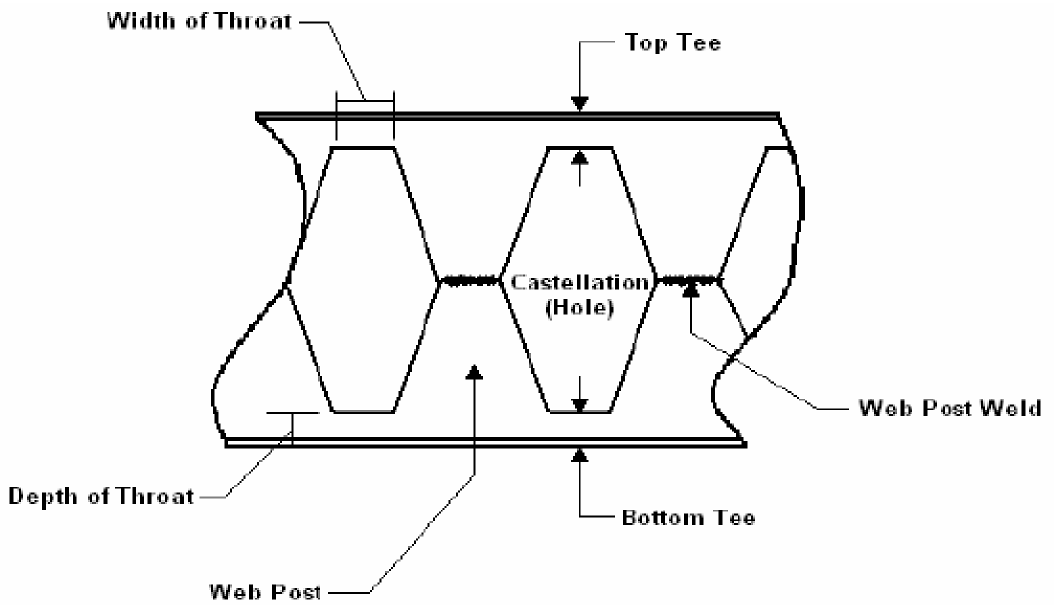

Figure 2.8 Terminology of castellated beam. Source: Wakchaure, *et al.* (2012)

The parts of a castellated beam according to Fig. 2.8 are:

1. Throat Width: the width of the opening in the form of a polygon.
2. Throat Depth: the height between the flange and the opening.
3. Web Post: the solid cross section of a castellated beam

2.2.3 Advantages of Castellated Beam

According to Hayati (2009), the advantages of castellated beam are:

1. Higher moment of inertia due to higher web profile hence making the profile much stronger and rigid.
2. The moment of the structure produced is large, while the minimum allowed stress is meager.
3. The material is light yet strong and can be easily assembled.

While according to Zirakian & Shokati (2006), castellated beam has the following advantages:

1. The increase of profile's height creates higher moment of inertia, ductility, stiffness and flexibility of the profile.
2. The decrease of the profile's weight will have the overall weight of the structure much lighter.
3. The opening on the castellated beam can be used for mechanical, electrical and plumbing purposes.

2.2.4 Disadvantages of Castellated Beam

According to Hayati (2006), the disadvantages of castellated beam are:

1. The increase of tension on the end of the profile.
2. Castellated beams are not suitable on designing short length with huge loads.
3. The structural analysis is much complicated than conventional profiles.

While according to Verweij (2010), castellated beam has the following disadvantages:

1. The structure is not suitable for withholding concentrated load.
2. It is needed to have much complex calculation on the strength of the profile.
3. The capacity on withholding axial loads is reduced.
4. The increase of cost production.

2.3 Failures on Castellated Beam

In a study conducted by Kerdal & Nethercot (1984), there are six failure modes assuming that there are adequate lateral support and applied loading producing both moment and shear:

1. Vierendeel mechanism.
2. Bending failure.
3. Lateral torsional buckling.
4. Rupture of the welded joint.
5. Web post buckling due to shear.
6. Web post buckling due to compression.

Since the presence of web opening inflicts such failure modes, the following limit states should be investigated and check thoroughly when designing castellated beams:

1. Compactness and local buckling
2. Overall beam flexural strength
3. Vierendeel bending of tees
4. Web post buckling
5. Axial tension/compression
6. Horizontal shear
7. Vertical shear
8. Lateral-torsional buckling

2.3.1 Failure by Formation of Vierendeel Mechanism

This failure occurs due to the presence of a high magnitude of shear force working on the beam. It forms plastic hinges at corners of the openings in which the openings are deformed in a manner of a parallelogram, though some studies report that these plastic hinges are also formed at corners of the other openings in the span and the distortion pattern varies. When shear force is applied to the castellated beam, the beam receives shear force and bending moment. The location of the failure occurs at the span which receives the maximum shear force.

2.3.2 Failure by Bending Moment

This failure is occurred by bending moment whereas the upper and below tee section of the profile yields. In this case, calculating the castellated beam's capability of withstand the maximum moment is the same as calculating for the conventional beam.

2.3.3 Failure by Lateral Torsional Buckling

This failure is similar to conventional profile's lateral torsional buckling. Lateral torsional buckling is usually linked to the span of the beam, from one support to another. But even so, lateral torsional buckling does not bring any significant effect towards the structure.

2.3.4 Failure by Rupture of Welded Joint

This failure happens at welded joint on the web post. This failure happens because of the horizontal shear force is more than the yield force of the welded joint. In other words, the welded joint cannot withhold the horizontal shear force applied to it.

2.3.5 Failure by Web Post Buckling Due to Shear

This failure happens at the web post. The reason for this buckling is shear force that is acting on the web post. Shear force affects the bending moment of the profile. Hence, if ever the shear force is too much to handle and caused bending moment to act irregularly, the upper tee section needs to support tensile stress while the lower tee section needs to support compressive stress and it costs the beam to buckle to the web post.

2.3.6 Failure by Web Post Buckling Due to Compression

The main cause of this failure is concentrated load applied on the beam. Too much load caused the web post to buckle. This failure may be solved by adding more stiffeners.

2.4 Structural Design and Analysis

In designing the structure, LRFD (Load and Resistance Factor Design) will be used as the limit state of design. The proposed strength of every component of the structure must not be lower than the strength needed to withhold the loads working on the structure. According to SNI 1729:2015, the design with LRFD matches the specification needed if:

$$R_u \leq \phi R_n \qquad (1)$$

With R_u is the strength needed for withholding the load combinations and ϕR_n is the strength of the designed structure with ϕ is the safety factor.

According to SNI 1727:2013, the types of load combinations are:

1. 1,4D
2. 1,2D + 1,6L + 0,5 (L_r or R)
3. 1,2D + 1,6 (L_r or R) + (1,0L or 0,5W)
4. 1,2D + 1,0W + 1,0L + 0,5 (L_r or R)
5. 1,2D + 1,0E + L
6. 0,9D + 1,0W
7. 0,9D + 1,0E

With

D = Dead load

L = Live load

L_r = Live roof load

R = Rain load

W = Wind load

E = Quake load

2.4.1 Dead Load

Dead load is defined as the weight of the construction material itself, such as: floor slab, stairs, columns, beams, walls, floor ceramics, roofs, etc. While for roofs, dead loads can be: roof tiles, purlin, ceilings, hangers, maintenance bridge, mechanical-electrical installations, roof insulations, etc.

2.4.2 Live Roof Load

Live Roof Load is defined as the weight of an active person with its materials working on the roof in case of installation or maintenance and also an active object such as: plants or moving accessories.

According to SNI 1727:2013, the formula of reducing the live roof load is:

$$L_r = L_o R_1 R_2 \text{ with } 0.58 \leq L_r \leq 0.96 \tag{2}$$

With R_1 and R_2 is defined as

$$R_1 = 1 \text{ if } A_T \leq 18.58\ m^2$$

$$R_1 = 1.2 - 0.011A_T \text{ if } 18.58 < A_T < 55.74\ m^2$$

$$R_1 = 0.6 \text{ if } A_T \geq 55.74\ m^2$$

$$R_1 = 1 \text{ if } F \leq 4$$

$$R_1 = 1.2 - 0.05F \text{ if } 4 < F < 12$$

$$R_1 = 0.6 \text{ if } F \geq 12$$

In which

A_T = Tributary area (m^2)

F = 0.12 x slope; slope is defined in percentage

L_r = Reduced live roof load (N/m^2)

L_o = Live roof load without reduction (N/m^2)

Table 2.1 Live roof load specifications

Usage	Distributed Load (kN/m^2)	Concentrated Load (kN/m^2)
Roof		
Flat, conjoined, curved roof	0.96	
Roof garden	4.79	
Roof for other purposes	Same as the other	
Roof for other residential purposes		
Canopy and awning		
Workshop construction with rigid frame	0.24 (cannot be reduced)	
Roof fulcrum	0.24 (cannot be reduced and must according to the tributary area of the roof to the structure)	0.89
All other constructions	0.96	
Main components of roof structure in which is related to floor works		
Singular panel points from lower member of the truss or every point on the main structure that supports roof of workshops, storages or garages		8.9
All other components of the main structure		1.33
Workers doing maintenance on rooftop		1.33

Source: SNI 1727:2013

2.4.3 Wind Load

A structure must be able to withstand wind load. SPBAU (*Sistem Penahan Beban Angin Utama*) is a standard code in which SNI 1727:2013 follows. The rule mentions that the minimum wind load design for enclosed buildings or semi-enclosed buildings can be no lower than 0.77 kN/m^2 times the area of the building's wall and 0.38 kN/m^2 times the area of the roof projected vertically, perpendicular towards the wind direction.

According to SNI 1727:2013, the parameters of wind loads are:

1. Wind speed (V): this data may be acquired from BMKG/Meteorology, Climatology, and Geophysical Agency. Though the latest data is not available, we may assume the wind speed from the standard Design Wind Speeds for Asia-Pacific Region (HB 212-2002). According to the journal, Indonesia is on region number 1. The wind speed of each region is explained on Table 2.2.

Table 2.2 Wind speed for Asia-Pacific region

Risk Category	Reset Period (Years)	Wind Speed (m/s)
I	300	38.3
II	700	40.9
III	1700	43.4
IV	1700	43.4

Source: "Design Wind Speeds for Asia-Pacific Region", John Holmes, HB 212-2002

2. Wind direction factor (K_d): wind direction factor decreases the probability of maximum wind speed from any direction and decreases the probability of maximum pressure coefficient of wind speed from any direction. The factor K_d is defined in Table 2.3.

Table 2.3 Wind direction factors by structure type

Structure Type	**Wind Direction Factor**
Buildings	
Truss members	0.85
Cladding	0.85
Curved roof	0.85
Chimney, tanks and similar structure	
Rectangular	0.90
Hexagonal	0.95
Ellipse	0.95
Solid walls and billboards	0.85
Billboard grille	0.85
Truss tower	
Triangular, square, rectangular	0.85
Other types of cross section	0.95

Source: SNI 1727:2013

3. Exposure: Every wind direction must be considered. The exposure of resisting winds must be based on topography, vegetation, and the facility that is going to be build. Exposure is categorized into 3 classes: B, C, D.

4. Topography factor (K_{zt}): Wind speed is affected by topography, where winds on hills and cliffs will cause a dramatical change. The factor can be calculated with the following formula:

$$K_{zt} = (1 + K_1 K_2 K_3) \quad (3)$$

As for flat lands

$K_{zt} = 1$

5. Wind-blown effect factor (G): The factor of wind-blown effect on a rigid structure is 0.85.

6. Enclosure: Enclosure affects the internal pressure coefficient. Structures must be classified whether it is enclosed fully, partially enclosed or open fully.

7. Internal pressure coefficient (GC_{pi}): internal pressure coefficient is classified according to the enclosure of the structure. Table 2.4 explains the coefficient with each type of enclosure.

Table 2.4 Internal pressure coefficient of each enclosure type

Enclosure Type	**GC_{pi}**
Enclosed fully	0.00
Partially enclosed	+ 0.55
	- 0.55
Open fully	+ 0.18
	- 0.18

Source: SNI 1727:2013

8. Exposed wind speed pressure coefficient (K_z or K_h): this coefficient is classified according to the height from level soil.

Table 2.5 Exposed wind speed pressure coefficient

Height from level soil (m)	**Exposure**		
	B	**C**	**D**
0 – 4.6	0.57	0.85	1.03
6.1	0.62	0.90	1.08
7.6	0.66	0.94	1.12
9.1	0.70	0.98	1.16
12.2	0.76	1.04	1.22
15.2	0.81	1.09	1.27
18	0.85	1.13	1.31
21.3	0.89	1.17	1.34
24.4	0.93	1.21	1.38
27.4	0.96	1.24	1.40
30.50	0.99	1.26	1.43

Source: SNI 1727:2013

9. Wind speed pressure (q): wind speed pressure can be calculated with this formula

$$q_h = 0.613 K_h K_{zt} K_d V^2 \quad (4)$$

where V is in m/s

10. External pressure coefficient (C_p): the external pressure of an enclosed structure with gable roof affects the wind load.

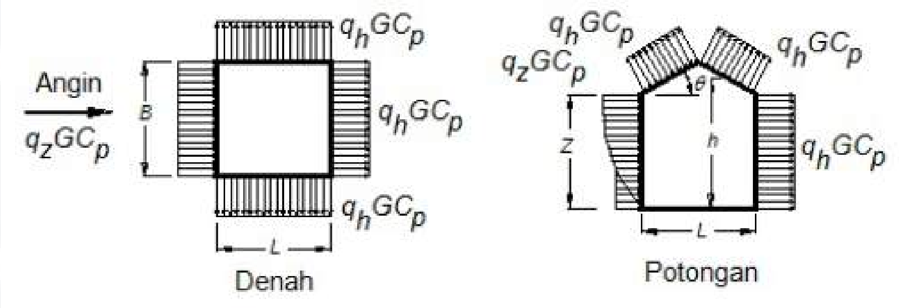

Figure 2.9 External pressure coefficient for gable roof. Source: SNI 1727:2013

11. Wind pressure (p)

$$p = qGC_p - q_i(GC_{pi}) \quad (5)$$

With

p = wind pressure (N/m^2)

q = q_h of roof with the height of h

q_i = q_h of positive internal pressure

2.5 Steel Properties

Steel beam may happen instability due to local buckling on flange or web, flexural buckling, torsional buckling and flexural torsional buckling. According to SNI 1729:2015, there are 2 properties of steel components:

1. Shear

The structural component that withstand shear force is classified as slender and non-slender. Non-slender structural component has a ratio of width thickness less than the restricted slender ratio (λ_r). If the width thickness ratio is larger than λ_r, hence the structure is a slender element.

Limitation of ratio λ_r for flanges of WF and Tee-section (*b/t*):

$$\lambda_r = 0.56\sqrt{E/F_y} \tag{6}$$

Limitation of ratio λ_r for web of WF (h/t_w):

$$\lambda_r = 1.49\sqrt{E/F_y} \tag{7}$$

Limitation of ratio λ_r for stem Tee-section (*d/t*):

$$\lambda_r = 0.75\sqrt{E/F_y} \tag{8}$$

2. Bending

The structural component that withstand bending force is classified as compact element and non-compact element as well as slender element. The section is accounted as a compact element whereas the flanges are joined with the web and the width-thickness ratio is less than λ_p. If the width-thickness ratio is larger than λ_p but less than λ_r, the section is accounted as a non-compact element. If the width-thickness ratio is larger than λ_r, the section is accounted as a slender element

Limitation of ratio λ_p for flanges of WF and Tee-section (*b/t*):

$$\lambda_p = 0.38\sqrt{E/F_y} \tag{9}$$

Limitation of ratio λ_r for flanges of WF and Tee-section (*b/t*):

$$\lambda_r = 1.0\sqrt{E/F_y} \tag{10}$$

Limitation of ratio λ_p for web of WF (h/t_w):

$$\lambda_p = 3.76\sqrt{E/F_y} \tag{11}$$

Limitation of ratio λ_r for web of WF (h/t_w):

$$\lambda_r = 5.70\sqrt{E/F_y} \quad (12)$$

Limitation of ratio λ_p for stem of Tee-section (h/t_w):

$$\lambda_p = 0.84\sqrt{E/F_y} \quad (13)$$

Limitation of ratio λ_r for stem of Tee-section (h/t_w):

$$\lambda_r = 1.52\sqrt{E/F_y} \quad (14)$$

With

E = elastic modulus of steel (MPa)

F_y = yield stress of steel (MPa)

λ_r = Limitation of ratio for slender element

λ_p = Limitation of ratio for non-compact element

2.6 Castellated Beam Design

The design for castellated beam follows "Castellated and Cellular Beam Design" by AISC.

2.6.1 Design Criteria

The design criteria of beams with web openings or castellated beams follows the journal by the ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete. The committee has outlined the criteria of the dimensions of the castellated beam, in which:

- The opening depth (h_o) must not exceed 0.7d.
- The depth of the tee (d_t) must not be less than 0.15d.
- The ratio of the opening length to the depth of the steel portion of a tee (v) must not exceed 12.

- The clear spacing (S) must be greater than or equal to the opening depth (h_o).

Hence with the given values of e, b and d_t in which are designated from the depth of the root beam section and a trial opening size that can be modified, the design of the castellated beam with hexagonal opening consists of:

1. Half height of castellated opening

$$h = d - d_t \tag{15}$$

2. Opening depth

$$h_o = 2h \tag{16}$$

3. Depth of expanded beam

$$d_g = h_o + 2d_t \tag{17}$$

4. Angle of hexagonal cut

$$\theta = \tan^{-1}\left(\frac{h}{b}\right) \tag{18}$$

5. Spacing of openings

$$S = 2e + 2b \tag{19}$$

with

S = Spacing of openings (mm)

b = horizontal length (mm)

d = depth of the steel profile (mm)

d_g = depth of the expanded beam (mm)

d_t = depth of tee (mm)

e = length of tee section (mm)

h = half height of castellated opening (mm)

h_o = opening depth (mm)

θ = angle of hexagonal cut (°)

2.6.2 Beam Net Section Analysis

1. Combined area of top and bottom tees

$$A_{net} = 2A_{tee} \tag{20}$$

2. Distance between centroids of top and bottom tees

$$D_{effec} = d_g - 2(d_t - \bar{y}_{tee}) \tag{21}$$

3. Net moment of inertia about x-axis

$$I_{x-net} = 2I_{x-tee} + 2A_{tee}\left(\frac{d_{effec}}{2}\right)^2 \tag{22}$$

4. Net elastic section modulus about x-axis

$$S_{x-net} = \frac{I_{x-net}}{\left(\frac{d_g}{2}\right)} \tag{23}$$

5. Net plastic section modulus about x-axis

$$Z_{x-net} = 2A_{tee}\left(\frac{d_{effec}}{2}\right) \tag{24}$$

with

A_{tee} = area of tee section (mm^2)

$\bar{y}_{tee}$ = distance from either top or bottom fiber to centroid of tee (mm)

2.6.3 Beam Gross Section Properties

1. Gross area of castellated tee

$$A_{gross} = A_{net} + h_o t_w \tag{25}$$

2. Gross moment of inertia about x-axis

$$I_{x-gross} = I_{x-net} + \left(\frac{t_w {h_o}^3}{12}\right) \tag{26}$$

4. Gross elastic section modulus about x-axis

$$S_{x-gross} = \frac{I_{x-gross}}{\left(\frac{d_g}{2}\right)} \tag{27}$$

5. Gross plastic section modulus about x-axis

$$Z_{x-gross} = Z_{x-net} + 2t_w h\left(\frac{h}{2}\right) \tag{28}$$

2.7 Vierendeel Bending Analysis

According to AISC Design Guide 31, vierendeel bending is caused by axial force and Vierendeel moment transferred throughout the beam altogether on both upper and bottom tee. The axial strength and Vierendeel moment can be calculated with the following formula:

1. Required axial compressive strength

$$P_r = \frac{M_r}{d_{effec}} \tag{29}$$

2. Vierendeel required flexural strength (Vierendeel moment)

$$M_{vr} = V_r\left(\frac{A_{tee}}{A_{net}}\right)\left(\frac{e}{2}\right) \tag{30}$$

with:

M_r = required flexural strength (N-mm)

V_r = required shear strength (N)

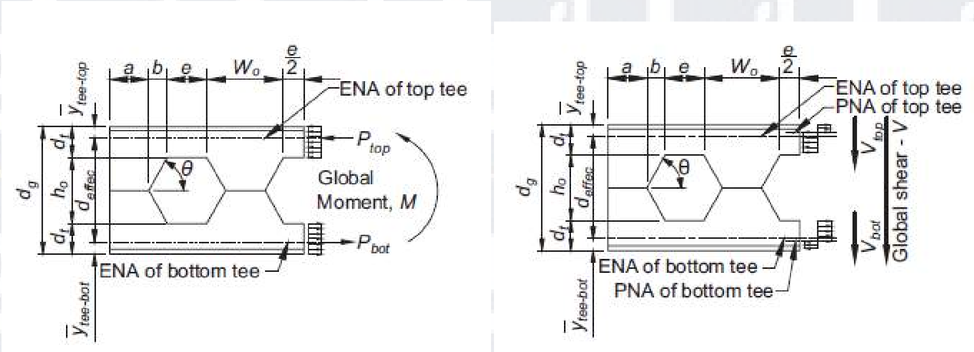

Figure 2.10 Terminologies used for calculating axial forces and Vierendeel moment in noncomposite beams. Source: AISC Design Guide 31

2.7.1 Calculation of Axial and Flexural Strength of Top and Bottom Tees

In order to reduce the number of calculations, it is tolerable to have tension force on the bottom tee use as a compression force. The nominal compressive strength, P_n, is obtained from the lowest value between flexural buckling and flexural-torsional buckling. Assuming the design uses:

1. $K_x = 0.65$ (assumes translation and rotation are fixed at the ends of the tee section)
2. $K_y = 1.0$
3. $L = e$ (length of the laterally unbraced member)
4. $L_c = K_xL$ or K_yL
5. $E = 200{,}000$ MPa
6. $G = 77{,}200$ MPa

According to SNI 1729:2015, compressive strength for flexural buckling of members without slender elements can be calculated with the following formula:

$$P_n = F_{cr}A_g \tag{31}$$

The critical stress, F_{cr}, is determined with the following formula with different conditions, in which:

1. If $\frac{L_c}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$ or $\frac{F_y}{F_e} \leq 2.25$

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y \tag{32}$$

2. If $\frac{L_c}{r} > 4.71\sqrt{\frac{E}{F_y}}$ or $\frac{F_y}{F_e} > 2.25$

$$F_{cr} = 0.877 F_e \tag{33}$$

in which the elastic buckling stress, F_e, is calculated with the following formula:

$$F_e = \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} \tag{34}$$

As for calculating the compressive strength for flexural-torsional buckling, it is needed to determine the nominal compressive strength with this formula:

$$P_n = F_{cr} A_g \tag{35}$$

The formula of critical stress, F_{cr}, can be determined through Equation (32) and (33). The torsional or flexural-torsional elastic buckling stress, F_e, is determined with the following formula:

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{\left(F_{ey} + F_{ez}\right)^2}}\right] \tag{36}$$

with

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{L_{cy}}{r_y}\right)^2} \tag{37}$$

$$F_{ez} = \left[\frac{\pi^2 E C_w}{(L_{cz})^2} + GJ\right]\frac{1}{A_g \bar{r}_o^2} \tag{38}$$

$$H = 1 - \frac{x_o^2 + y_o^2}{\bar{r}_o^2} \tag{39}$$

$$\bar{r}_o^2 = x_o^2 + y_o^2 - \frac{I_x + I_y}{A_g} \tag{40}$$

C_w = Warping constant (mm^6)

F_e = Elastical critical buckling stress (MPa)

G = Shear modulus of elasticity (G_{steel} = 77,200 MPa)

H = Flexural constant

K_x = Effective length factor with respect to x-axis

K_y = Effective length factor with respect to y-axis

L_c = Effective length of member for buckling (mm)

P_n = Nominal compressive strength (N)

$\bar{r}_o$ = Polar radius of gyration about the shear center (mm)

x_o, y_o = Coordinates of the shear center with respect to the centroid (mm)

2.7.2 Calculation of Nominal Flexural Strength, M_n

In order to support the Vierendeel design moment, the flexural strength of top and bottom tee sections must be calculated and compared to the known flexural strength. The design assumption is:

$L_b = e$

The nominal flexural strength is obtained from the lowest value according to the obtained value of plastic bending moment and yield moment.

$$M_n = M_p \tag{41}$$

$$M_n = M_y \tag{42}$$

with

$$M_y = F_y S_{x-tee} \tag{43}$$

L_b = Distance between lateral braces (mm)

M_n = Nominal flexural strength (N-mm)

M_p = Plastic bending moment (N-mm)

M_y = yield moment (N-mm)

$S_{x\text{-}tee}$ = Section modulus of tee about x-axis (mm^3)

The lateral torsional buckling on element of the structure is calculated with the following formula and conditions:

If $L_b \leq L_p$, the lateral-torsional buckling does not apply on this condition.

If $L_p < L_b \leq L_r$,

$$M_n = M_p - \left(M_p - M_y\right)\left(\frac{L_b - L_p}{L_r - L_p}\right) \tag{44}$$

If $L_b > L_r$,

$$M_n = M_c \tag{45}$$

For tee stems

$$M_n = M_{cr} \leq M_y \tag{46}$$

with

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \tag{47}$$

$$L_r = 1.95\left(\frac{E}{F_y}\right)\frac{\sqrt{I_y J}}{S_x}\sqrt{2.36\left(\frac{F_y}{E}\right)\frac{dS_x}{J} + 1} \tag{48}$$

$$M_{cr} = \frac{1.95E}{L_b}\sqrt{I_y J}\left(B + \sqrt{1+B^2}\right) \tag{49}$$

$$B = -2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}} \tag{50}$$

The flange local buckling on element of the structure is calculated with the following formula and conditions:

If the section has a compact flange, the limit state of flange local buckling does not apply

If the section has a noncompact flange in flexural compression:

$$M_n = \left[M_p - \left(M_p - 0.7F_y S_{xc}\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \tag{51}$$

If the section has a slender flange in flexural compression:

$$M_n = \frac{0.7ES_{xc}}{\left(\frac{b_f}{2t_f}\right)^2} \tag{52}$$

with

$$\lambda = \frac{b_f}{2t_f} \tag{53}$$

S_{xc} = elastic selection modulus referred to the compressions flange (mm^3)

$\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange

$\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange

The nominal flexural strength for local buckling in flexural compression is calculated with the following formula:

$$M_n = F_{cr} S_x \tag{54}$$

with

S_x = elastic section modulus about the x-axis (mm^3)

The critical stress for the formula (54) is calculated with the following formula and conditions:

If $\frac{d}{t_w} \leq 0.84\sqrt{\frac{E}{Fy}}$

$$F_{cr} = F_y \tag{55}$$

If $0.84\sqrt{\frac{E}{Fy}} < \frac{d}{t_w} \leq 1.52\sqrt{\frac{E}{Fy}}$

$$F_{cr} = \left(1.43 - 0.515\frac{d}{t_w}\sqrt{\frac{Fy}{E}}\right)F_y \tag{56}$$

If $\frac{d}{t_w} > 1.52\sqrt{\frac{E}{Fy}}$

$$F_{cr} = \frac{1.52E}{\left(\frac{d}{t_w}\right)^2} \tag{57}$$

with

$d = d_t$

2.7.3 Calculation of Combined Flexural and Axial Forces on Top and Bottom Tees

According to AISC Specifications, the interation of flexure and axial forces in top and bottom tees constrained to bend about a geometric axis to either x and/or y axis is limited. The formulas and conditions below are used to check the tees on combined axial land flexural loads:

If $\frac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \tag{58}$$

If $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \tag{59}$$

with

$d = d_t$

P_c = available axial strength (N)

P_r = required axial strength (N)

M_c = available flexural strength (N)

M_r = required flexural strength (N)

2.8 Web Post Buckling

The cause of web post buckling is due to the horizontal shear force passing through the web post. The main factor of the failure is dependent on the geometry as well as the thickness of the web post. According to researches done by Aglan and Redwood in 1974, the buckling capacity of the web post is calculated using equations that have been developed through destructive testing.

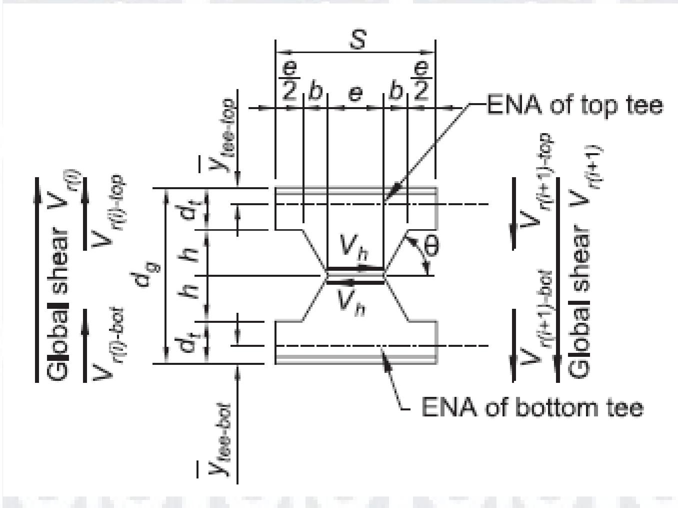

Figure 2.11 Terminologies used for calculating web post buckling. Source: AISC Design Guide 31

The calculation to determinate the horizontal shear on the web post uses the following formula:

$$V_{rh} = \left|\frac{M_{r(i+1)} - M_{r(i)}}{d_{effec}}\right| = \left|T_{r(i)} - T_{r(i+1)}\right| \quad (60)$$

The required flexural strength in the web post of top tee is calculated with the following formula:

$$M_{rh} = V_{rh} h_{top} \quad (61)$$

The required flexural strength in the web post of bottom tee is calculated with the following formula:

$$M_{rh} = V_{rh} h_{bot} \quad (62)$$

Plastic bending moment is determined with the following formula:

$$M_p = 0.25 t_w (e + 2b)^2 F_y \quad (63)$$

The critical moment for lateral buckling is calculated with the following formula and condition in which θ = 60°:

If $e/t_w = 10$

$$\frac{M_{ocr}}{M_p} = 0.587(0.917)^{\frac{2h}{e}} \leq 0.493 \quad (64)$$

If $e/t_w = 20$

$$\frac{M_{ocr}}{M_p} = 1.96(0.699)^{\frac{2h}{e}} \quad (65)$$

If $e/t_w = 30$

$$\frac{M_{ocr}}{M_p} = 2.55(0.574)^{\frac{2h}{e}} \quad (66)$$

It is to be noted that the value of M_{ocr}/M_p is limited to 0.493. With the condition of θ = 60°, it is needed to interpolate between equation (64) and (66) based on the actual value of e/t_w.

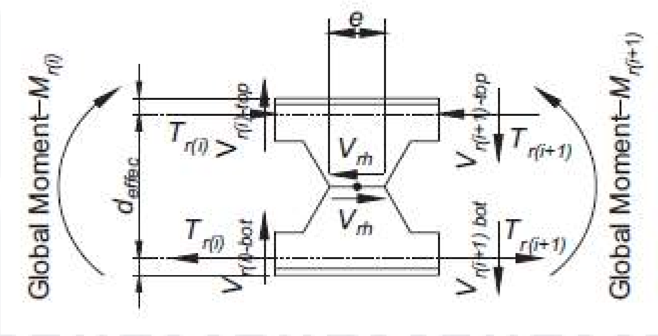

Figure 2.12 Terminologies used for calculating web post horizontal shear in noncomposite castellated beam. Source: AISC Design Guide 31

The resistance factor of LRFD as θ = 60°, ϕ_b = 0.90.

The available flexural strength of the web post is calculated with the following formula with different design combinations:

$$\phi M_n = \phi_b \left(\frac{M_{ocr}}{M_p}\right) M_p \tag{67}$$

with

M_{ocr} = Critical moment for lateral buckling (N-mm)

M_{rh} = Required flexural strength using load combinations (N-mm)

$T_{r(i)}$ = Required axial force in tee at opening (i) (N)

$T_{r(i+1)}$ = Required axial force in tee at opening ($i+1$) (N)

V_{rh} = Horizontal shear force (N)

2.9 Calculation of Horizontal and Vertical Shear Strength

The nominal horizontal shear strength can be calculated with the following formula:

$$V_n = 0.6F_yA_w \tag{68}$$

$$A_w = et_w \tag{69}$$

$$V_c = \phi_v V_n \tag{70}$$

with

ϕ_v = 1.00

A_w = Area of web post (mm^2)

V_c = Available horizontal shear strength (N)

V_n = Nominal horizontal shear strength (N)

The nominal vertical shear strength can be calculated with the following formula:

$$V_c = \phi_v V_n \tag{71}$$

The resistance factor is determined with the conditions below

a. If $\frac{h}{t_w} \leq 2.24\sqrt{\frac{E}{F_y}}$, then ϕ_v = 1.00

b. If $\frac{h}{t_w} > 2.24\sqrt{\frac{E}{F_y}}$, then ϕ_v = 0.90

2.9.1 Calculation of Available Vertical Shear Strength at Gross Section

The available vertical shear strength at gross section is calculated with the following formula:

$$V_n = 0.6F_yd_gt_wC_{v1} \tag{72}$$

with

If $\frac{h}{t_w} \leq 1.10\sqrt{\frac{k_v E}{F_y}}$, then $C_{v1} = 1.0$

If $\frac{h}{t_w} > 1.10\sqrt{\frac{k_v E}{F_y}}$, then

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \tag{73}$$

$$\frac{h}{t_w} = \frac{d_g - k_{top} - k_{bot}}{t_w} \tag{74}$$

2.9.2 Calculation of Available Vertical Shear Strength at Net Section

The available vertical shear strength at gross section is calculated with the following formula:

$$V_n = 0.6F_y\left(d_{t-top} + d_{t-bot}\right)t_w C_{v2} \tag{75}$$

with

If $\frac{h}{t_w} \leq 1.10\sqrt{\frac{k_v E}{F_y}}$, then $C_{v2} = 1.0$

If $1.10\sqrt{\frac{k_v E}{F_y}} < \frac{h}{t_w} \leq 1.37\sqrt{\frac{k_v E}{F_y}}$, then

$$C_{v2} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \tag{76}$$

If $\frac{h}{t_w} > 1.37\sqrt{\frac{k_v E}{F_y}}$, then

$$C_{v2} = \frac{1.51k_v E}{\left(h/t_w\right)^2 F_y} \tag{77}$$

$$\frac{h}{t_w} = \frac{d_t}{t_w} \tag{78}$$

2.10 Structural Analysis of Conventional Steel Profile

For the analysis of conventional steel profile, flexural strength and shear strength is put into consideration on calculating the efficiency of the profile.

2.10.1 Flexural Strength

The calculation of yield stress is determined with the following formula:

$$M_n = F_y Z_x \tag{79}$$

For the scetion in which the flange and the web of the steel profile is noncompact, the flexural strength is determined with the following formula:

$$M_n = R_{pc} M_{yc} \tag{80}$$

$$M_{yc} = F_y S_{xc} \tag{81}$$

R_{pc} is determined with the following conditions:

1. $\frac{I_{yc}}{I_y} > 0.23$

If $\frac{h_c}{t_w} \leq \lambda_{pw}$

$$R_{pc} = \frac{M_p}{M_{yc}} \tag{82}$$

If $\frac{h_c}{t_w} > \lambda_{pw}$

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \leq \frac{M_p}{M_{yc}} \tag{83}$$

2. $\frac{I_{yc}}{I_y} \leq 0.23$; hence $R_{pc} = 1.0$

with

M_{yc} = yield stress of compressed flange (N-mm)

R_{pc} = plasticity factor of web

S_{xc} = elasticity modulus of compressed flange (mm^3)

I_{yc} = moment of inertia of compressed flange with respect to y-axis (mm^4)

2.10.2 Shear Strength

The calculation of shear strength of the conventional steel profile is similar to the formula of vertical shear strength of gross section.

2.11 Calculation of Deflection

A structure is consider worthy to be used if the state of the structure is deemed safe for use. Deflection of the structure must be restricted in order to prove that the structure is safe. According to SNI 03-1729:2002, the allowable deflection of gable frame is calculated with the following formula:

$$f = \frac{L}{240} \tag{84}$$

with

L = the span of the frame (mm)

f = the limited deflection of the frame (mm)

The deflection for castellated beams can be approximated by using 90% of the moment inertia at the net section and treating it as a prismatic section (Hosain et al., 1974; Altfillisch et al., 1957). The formula of the deflection of castellated beam is:

$$\Delta = \frac{5qL^2}{384EI_{x-net}(0.90)} \tag{85}$$

with

E = modulus elasticity of steel (200,000 MPa)

$I_{x\text{-}net}$ = moment of inertia about x-axis of net section (mm^4)

q = distributed load (kN/m)

Δ = the deflection of the castellated beam (mm)

2.12 Previous Researches

M.R. Wakchaure and A.V. Sagade (2012) conducted a research about the height of the opening on castellated beam. The conclusion of this research is that the castellated beam is being used efficiently if the height of the opening is limited to 0.6*h* and also being used efficiently at longer spans which is controlled by deflection.

Masita Nur Hayati (2013) conducted a research about the width of the opening towards bending on castellated beam. The conclusion of this research is that the width of the opening is not effective on withstanding moment and bending while the height of the opening is much more effective on withstanding moment and bending as well as deflection.

Hansel Fatah Khorasani (2014) conducted a research about the comparison analysis between conventional beam with castellated beam on roof rafters. With the research based on SNI and ASCE, the result of the research shows that castellated beam is much efficient compared to conventional beam in terms of volume of steel being used for the structure, with the efficiency of the main rafter is 7.38% while for the secondary rafter is 16.03%.

Seetha D. (2016) conducted a research about the flexural behavior of rolled steel I-beams with castellated beams. The research concluded that the usage of castellated beam is much more advantageous and economic compared to using conventional beams.

Benny Rudiyanto (2018) conducted a research about the efficiency between conventional beam with castellated beam with circular opening. The result of the research shows that castellated beam with circular opening is much efficient compared to conventional beam in terms of volume of steel being used for the structure, with the efficiency of the beam is 23.29%.