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STEEL ROOF REDESIGN USING HYBRID SINGLE BARREL VAULT: STRUCTURAL EFFICIENCY THROUGH LRFD AND VALUE ENGINEERING

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ABSTRACT

Roof structure is one of the important elements in a building, there are many types of roof structures such as Portal Frame. Using steel pipe as a portal frame will cause higher dimensions to resist the flexural moment, since it is categorized as a compression member. This research focuses on redesigning materials used with WF profile and modification of castellated beam to optimize the dimension of profile and reducing the cost of portal frame roof that uses steel pipe. By applying Value Engineering Method to analyze the cost and LRFD Method to analyze the structure, the most effective alternative design can be implemented at Rawamangun Station, LRT Jakarta Project, as the place of the research. Alternative materials considered are Hybrid Castellated with WF Solid, Full Castellated Beam and Full WF Solid. After going through many processes of analyzing, the alternative material chosen is Hybrid Castellated Beam with WF Solid based on the fulfilled structural aspects, workability considerations and optimum cost. It is obtained that the alternative model cost Rp. 7,110,683,434.77, while the existing design cost Rp. 17,588,537,219.38. So, the alternative model successfully reduces the cost of existing model up to 40.43% without changing the function of the building structure.

Keywords:roof structure, value engineering, castellated beam

ABSTRAK

Struktur atap merupakan salah satu komponen penting dalam sebuah bangunan. Terdapat berbagai jenis struktur atap, salah satunya adalah sistem portal frame. Namun, penggunaan pipa baja sebagai portal frame cenderung membutuhkan dimensi yang lebih besar untuk menahan momen lentur, karena elemen ini berfungsi terutama sebagai elemen tekan. Penelitian ini berfokus pada perencanaan ulang struktur atap dengan menggunakan profil WF (Wide Flange) dan modifikasi balok castellated untuk mengoptimalkan dimensi penampang dan mengurangi biaya sistem portal frame dari pipa baja. Dengan menerapkan metode Value Engineering untuk analisis biaya dan metode LRFD (Load and Resistance Factor Design) untuk analisis struktur, desain alternatif paling efektif dikembangkan dan dievaluasi untuk diterapkan pada Stasiun Rawamangun dalam proyek LRT Jakarta Fase 1B. Tiga alternatif desain yang dipertimbangkan adalah: kombinasi Castellated Beam dan WF Solid, Castellated Beam penuh, dan WF Solid penuh. Setelah melalui analisis menyeluruh, kombinasi Castellated Beam dan WF Solid dipilih sebagai alternatif paling optimal berdasarkan kinerja struktural, kemudahan konstruksi, dan efisiensi biaya. Total biaya model hybrid yang diusulkan adalah Rp7.110.683.434,77, dibandingkan dengan desain eksisting sebesar Rp17.588.537.219,38, sehingga menghasilkan penghematan biaya sebesar 40,43% tanpa mengorbankan fungsi struktural bangunan.

Kata Kunci: struktur atap, rekayasa nilai, balok castellated

1. INTRODUCTION

Towards the Vision of Golden Indonesia 2045, infrastructure development continues to be encouraged to improve public welfare through economic growth and equitable distribution of public services (Bappenas, 2023). One strategic priority is the public transportation sector, such as the Jakarta LRT Phase 1B (Velodrome–Manggarai) project implemented by PT Waskita – Nindya – LRS KSO based on DKI Jakarta Gubernatorial Regulation No. 154 of 2017. One of the key elements of this project is Rawamangun Station, which is the first station of the 6 km line. The station's steel roof plays a crucial role as both weather protection and a key visual element. Due to its long span without a central column (single span), the roof design demands strength, efficiency, and ease of construction. The existing design uses a gable frame made of 20.6-meter-long steel pipes. Although aesthetically pleasing, steel pipes have a lower moment of inertia than I/H profiles, making them less efficient at resisting bending and requiring large dimensions that lead to material waste and high costs.

This condition is the basis for the application of Value Engineering (VE) in this project, namely a systematic approach to increasing the value of a project by identifying more efficient design alternatives without reducing its main function.[2]. Based on the standards of SAVE International (2007), the VE stages include: (1) information, (2) function analysis, (3) creativity, (4) evaluation, (5) development, and (6) final recommendations.

One of the solutions analyzed was replacing the roof system with a vault, specifically a barrel vault, which has proven efficient for long spans without columns, such as at King's Cross Station in London and Liege-Guillemins in Belgium.[3] [4]Barrel vaults distribute loads more evenly, enhance aesthetics, and reduce the need for vertical elements in key circulation areas. However, due to project scale limitations, a double-barrel vault design like that at King's Cross could not be fully implemented. Therefore, this study proposes a hybrid single-barrel vault approach, combining solid WFs in high-moment zones and castellated beams in low-moment zones. This solution reduces structural weight, accelerates installation, and lowers costs without compromising strength. Furthermore, this segmentation supports efficient connections based on internal force distribution. With this approach, the roof design not only meets technical aspects but also the principles of sustainability and efficiency in modern public infrastructure.

Literature review

Castellated Beam

According to Fares et al. (2016) in AISC Steel Design Guide 31, Castellated Beams are steel beams with hexagonal holes in their bodies, so they are often called Honeycomb Beams, while Cellular Beams have circular holes.[5]. This hollow beam technology was first used in 1910 by the Chicago Bridge and Iron Works, and its use increased rapidly since the 1940s due to the limited steel cross-sections available in Europe. Although the fabrication process is similar, Castellated Beams are made by zigzag cutting using a computer-based automatic torch. Castellation is a zigzag cutting process on the body of a steel profile such as H, I, or U made of hot-rolled steel. After being cut according to a certain pattern, the two sections of the profile are shifted and rearranged to form a hexagonal hole. The remaining piece is discarded, and then the two sections are reconnected by welding to form a castellated profile.[5]

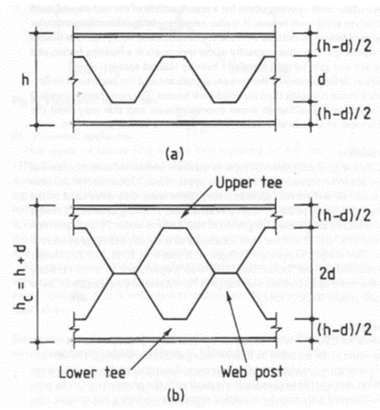


Figure 1 Castellation Process

Advantages and Disadvantages of Castellated Beam

Based on the nature of changes in geometric dimensions, castellated beams have several advantages and disadvantages, including:[5] [6] [7]:

1. Advantages:
- Flexural stiffness is increased, allowing longer spans.
- Capable of bearing greater moments due to high allowable stress.
- The moment of inertia and section modulus increase due to the height of the profile.
- High strength to weight ratio, suitable for light and long span structures.
- Aesthetic and functional, hexagonal (honeycomb) holes facilitate MEP installation.
- Reduces the need for columns and foundations, creating free and flexible spaces.
- Lighter structure, speeding up installation in the field.
2. Disadvantages
- The web shear capacity is limited, not suitable for short spans with large loads.
- Deflection analysis is more complex than solid profiles.
- Stress concentration occurs at the corners of the hole.
- Less strong against lateral forces, requires additional stiffeners at the ends of the beam.

Castellated Beams Potential Failures

1) Vierendeel or Shear Mechanism of Tees

   The failure mechanism in castellated beams occurs because shear forces are transferred through the holes in the web, following the distribution of bending moments along the beam. Critical points are at the four reentrant corners, which are prone to plastic hinge formation due to the high concentration of shear forces. As a result, the tee section can deform into a parallelogram shape.

2) Web Post Buckling due to Shear and Compression
Web post buckling in castellated beams is caused by shear forces or concentrated loads that exceed the holes in the web. Failure can occur through two modes: (1) bending due to the formation of plastic hinges, or (2) buckling of the web cross-section. The failure mode depends on the geometry and thickness of the web post (tee section). Prevention is done by selecting the appropriate opening angle to avoid shear failure, as well as adding stiffeners to prevent compression failure.
3) Welded Joint Rupture in a Web Post
Failure can occur when the spacing between the web openings is too close, resulting in a small effective web post width. This increases the stiffness of the beam web, causing greater shear forces and moments to be transferred to the welded joint.[8].If the shear stress exceeds the yield strength of the weld, the joint may fracture.[9]. Soltani et al. (2012) added that high stress concentration around the welded joint is the main cause of failure in this condition.[9].
4) Flexural Mechanism
Flexural failure is the main mode in castellated beams under pure flexural loading.[9]Although the presence of holes hinders the distribution of plastic strain, plastic deformation still occurs similar to that of a full WF profile. Yield points generally occur at the top and bottom of the tee section, which bears both tensile and compressive stresses.
5) Lateral – Torsional Buckling
Lateral torsional buckling in castellated beams is similar to that which occurs in WF beams without holes.[5].This failure occurs due to a lack of lateral support or torsional stiffness in the long-span beam. Because its properties are similar to those of a solid beam, the effect of holes in a castellated beam can be ignored in the analysis of lateral torsional buckling.

2. METHOD

Location of Research

The research was conducted at Rawamangun Station, part of the Jakarta LRT Phase 1B project on the Velodrome–Manggarai route, spanning 6.4 km and encompassing five stations. Up to the architectural stage, the only station that has been realized is Rawamangun Station at kilometer 0 (the first 1 km). This station consists of a concourse floor (2nd floor), a platform (3rd floor), a train track slab, and a steel roof structure.

Station Roof Structure Layout

Number of spans : 7 spans (P22B – P29B)
Length per span :
22B – 24B = 17.25 meters
24B – 25B = 14 meters
25B – 29B = 17.25 meters
Span width : 20.6 meters
North = 10.3 meters
South = 10.3 meters

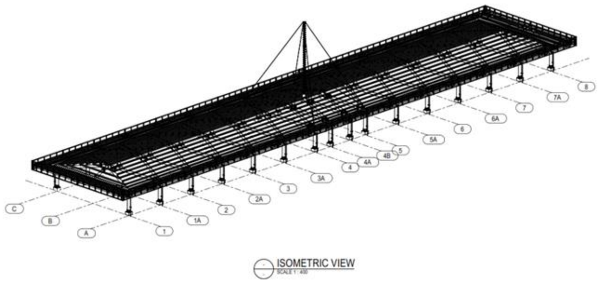


Figure2 Isometric View Existing Design

Based on the working drawings and specifications obtained as secondary data from the Jakarta LRT Phase 1B Velodrome – Manggarai Project, the steel material used in the existing structure is steel pipe with the following specifications.

Table1 Material Specification of Existing Design

NO	Components	Specification	Strength
1	Main Structural Components	JIS G3102 SS400 / ASTM A36	Fy min = 235 MPa Fu = 400 – 510 MPa
2	Main Structural Components of Pipe	ASTM A53 – GRADE B / A252 GRADE 2 JIS G3444 – STK400	Fy min = 235 MPa Fu = 400 MPa
3	Steel Plate Connection	JIS G3101 SS400 / ASTM A36	Fy min = 235 MPa Fu = 400 – 510 MPa
4	Bolt	ASTM A325 / JIS B1051 G 8.8	Fu = 800 – 830 MPa
5	Anchor Bolt	ASTM A307	Fu = 415 Mpa

The following is a flow chart of the compilation method used:

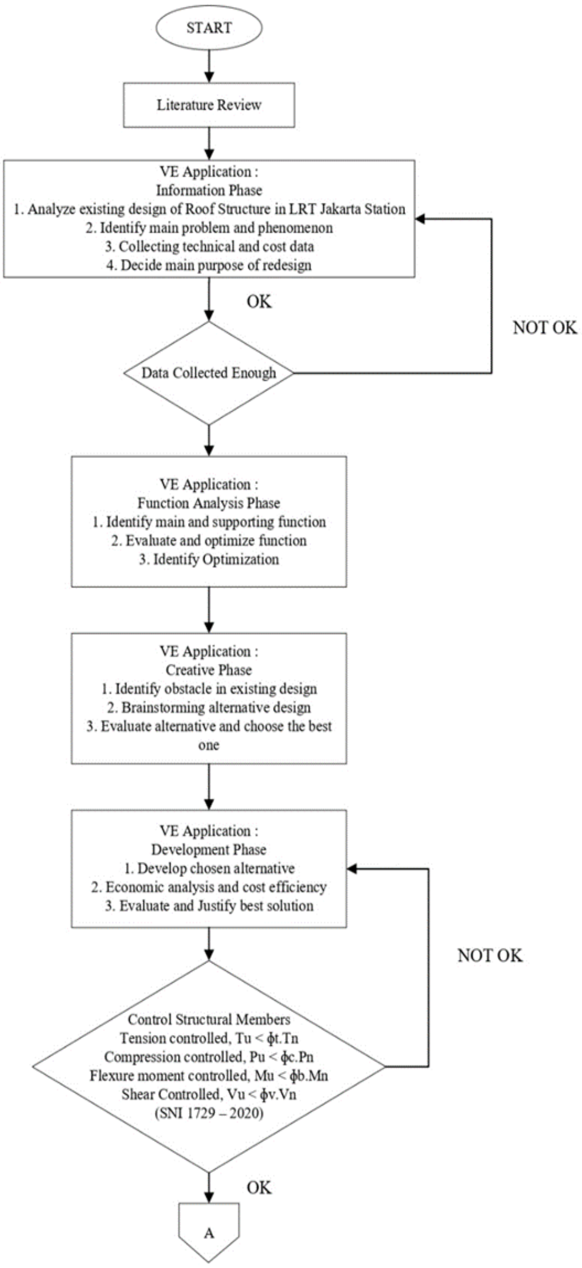


Figure 3 Flow Chart Part A

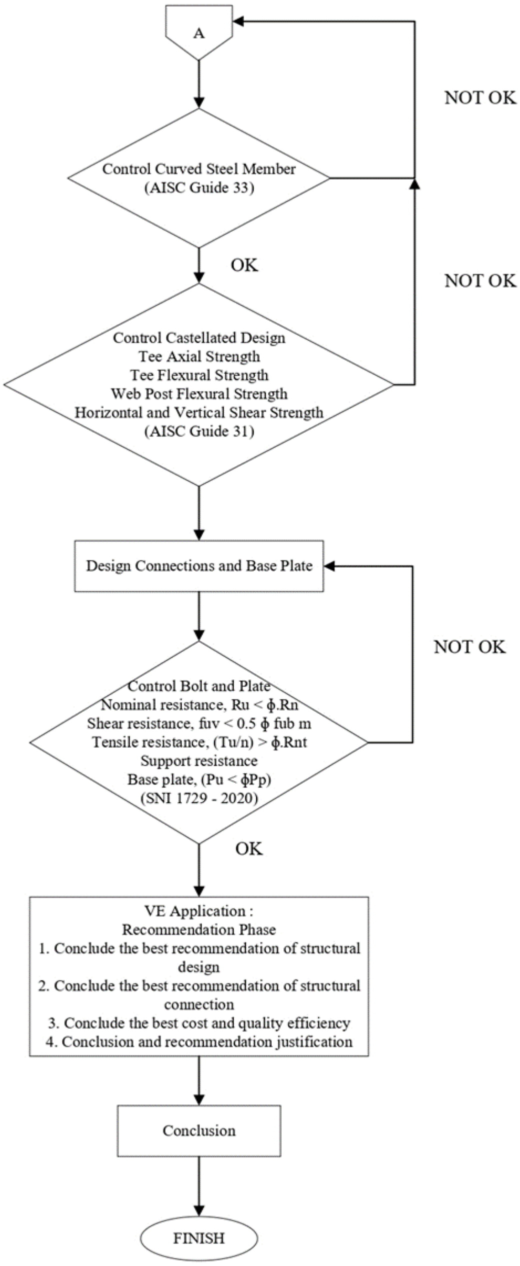


Figure5 Flow Chart Part B

Alternative Design

Several alternative designs for this problem can be developed based on the previously analyzed functions. These creative ideas include:

1. Hybrid Solid WF and Castellated Beam – Combining solid WF in areas of high moments and castellated beams in areas of low moments. This design is structurally efficient, uses lighter materials, and is easy to fabricate, but requires further connection analysis.
2. Full Castellated Beam – Uses a full castella with web stiffeners. This design is lightweight and material-efficient, but is less than optimal in high-moment areas and is at risk of shear failure if not properly calculated.
3. Full WF Solid – Uses a solid WF profile throughout the entire element. It offers high strength and stability, but its high weight and cost make it unsuitable for optimization purposes.

Of the three alternatives, the hybrid WF Solid & Castellated Beam design was chosen because it best aligns with the objectives of material efficiency, ease of installation, and connection optimization.

Structural Designing

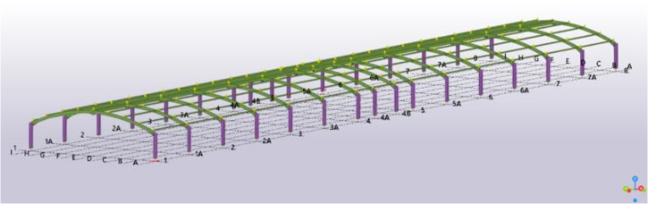


Figure 6 3D Model of Steel Roof Structure in Tekla Structures

Loading

1. Dead Load (SIDL)

Structural Steel γs	=	78.5	kN/m3
Purlin	=	38.4	kg/m2
Roof Finishing	=	10	kg/m2
Insulation	=	5	kg/m2
Ceiling	=	18	kg/m2
Solar Cell	=	30	kg/m2
Antenna	=	700	kg/point
Lightning Rod	=	40	kg/point

2. Live Load
   Live loads on the roof are generated by worker activities during equipment maintenance and other activities. The live load value is in accordance with SNI 1727-2020, which is 1 kN/m^2.[10]
3. Wind Load
   Wind Load
   Wind load from H direction
   Wind Load (Winward)
   (-) qh -100.80 kg/m2 -0.0009888kN/mm^2
   Wind Load (Leward)
   (-) qh -64.53 kg/m2 -0.0006330 kN/mm^2
   Wind Load from Z Direction
   Wind Load (Winward)
   (+) qz -35.52 kg/m2 -0.0003484kN/mm^2
   Wind Load (Leward)
   (+) qz -64.53 kg/m2 -0.0006330kN/mm^2
4. Seismic Load
   a) Acceleration parameters mapped 0.2 seconds and 1 second
      Ss = 0.565 g
      S1 = 0.23 g
   b) Design spectral acceleration parameters
      Short period (SDS) = 0.89 g
      Period 1 second (SD1) = 0.7 g

LRFD / Load Resistance Factor Design Method

According to Setiawan (2013) referring to SNI 1729-2020, there are two main methods in steel structure planning: Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD).[11]ASD has been used for about 100 years, but in the last 20 years, planning has shifted to LRFD, as it is considered more rational. LRFD is considered to meet standards if the design strength of each structural component equals or exceeds the required strength based on the load combination.

$Ru \leq \phi . Rn$

Ru =Ultimate strength / need to use LRFD load combination
Rn =Nominal power
ϕ = Resistance factor / reduction factor
ϕRn =The power of design

Tension Members

a) For yielding in the gross section:

$$Nominal\ Tensile\ Strength, \tag{1}$$

$$Pu \leq \phi Pn$$

$$Pn = Fy \,.\, Ag \tag{2}$$

$$\phi_t = 0.9\ (LRFD)$$

b) For a fracture in the net section:

$$Pn = Fu \,.\, Ae \tag{3}$$

$$\phi_t = 0.75\ (LRFD)$$

c) Effective net area

$$A_e = U \,.\, An \tag{4}$$

d) Effective net area (transmitted by bolt):

$$A_e = An \leq 0.85\ Ag \tag{5}$$

Ae = Effective net area mm^2
Ag = Gross area of member mm^2
An = Net area of the member mm^2
U = Shear lag factor
(1 – (x/L)) < 0.9
Fy = Specified minimum yield stress(Mpa)
Fu = Specified minimum tensile strength (Mpa)
Pn = Nominal axial strength (N)

Compression Members

$$Pn = Fcr \,.\, Ae \tag{6}$$

Ag	=	Gross area of member	mm^2
E	=	Modulus of elasticity	MPa
K	=	Effective length factor	
l	=	Unbraced length of member	mm^2
r	=	Radius of gyration	mm
λ_c	=	Column slenderness factor	

Flexural Members

1) Yield

$$M_n = M_p = Fy\,.Zx \tag{7}$$

Fy	=	Yield stress	MPa
Zx	=	Modulus of plasticity in x axis	mm3

2) Lateral – Torsional Buckling
   a) If Lb ≤ Lp , the lateral torsion buckling limit state does not apply.
   b) If Lp < Lb ≤ Lr

$$M_n = Cb\,[Mp - (Mp - 0.7.Fy.Sx)\,.(\frac{Lb - Lp}{Lr - Lp})\,] \leq Mp$$

   c) If Lb > Lr

$$M_n = Fcr\,.Sx \leq Mp$$

Shear Members

$$Vu \leq \phi v\,.Vn \tag{8}$$

The nominal shear strength (Vn) is determined by the following formula,

$$Vn = 0.6\,.Fy\,.Aw\,.Cv1 \tag{9}$$

Fy	=	Yield stress	MPa
Aw	=	Area of web, overall depth times web thickness	mm^2

Curved Steel Members in Elevation

The curve condition must meet the cross-sectional capacity, where the design strength (ρy) is adjusted to a reduced design strength (ρyd) using a specific equation to account for the reduction in strength due to the design condition.

$$\rho_{yd} = [\rho_y^2 - 3(\frac{\sigma_2}{2})^2]^{0.5} - \frac{\sigma_2}{2} \tag{10}$$

$$\frac{F_c}{A_g\,.\rho_{yd}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1 \tag{11}$$

σ_2	=	Out-of-plane bending stress	N/mm^2
ρ_y	=	Yield Strength	N/mm^2

$M_{cx} = \rho_{yd}\,.S_x$(plastic or compact section)

$M_{cx} = \rho_{yd}\,.Z_x$(slender section)

Out-of-plane buckling

$$\frac{Fc}{Pcy} + \frac{m_{LT}.M_{LT}}{M_b} \leq 1 \tag{12}$$

$$M_b = \rho_b.S_x \tag{13}$$

Beam-Column

a) When Pr/Pc is greater than 0.2

$$\frac{Pr}{Pc} + \frac{8}{9}\,(\frac{Mrx}{Mcx} + \frac{Mry}{Mcy}) \leq 1.0 \tag{14}$$

b) When Pr/Pc is less than 0.2

$$\frac{Pr}{2Pc} + (\frac{Mrx}{Mcx} + \frac{Mry}{Mcy}) \leq 1.0 \tag{15}$$

Bolt Connections

$$R_u \leq \phi\,.Rn \tag{16}$$

$R_n = m\,.r1\,.fub\,.Ab$(Shear Resistance)

$R_n = 0.75\,.fub\,.Ab$ (Tensile Resistance)

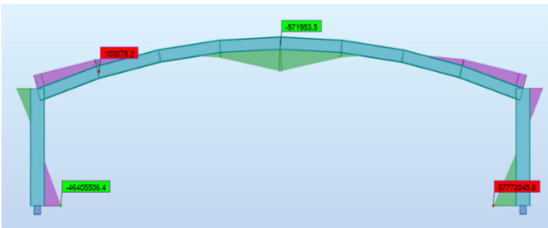
$R_n = 2.4\,.db\,.tp\,.fu$(Support Resistance)

r1	=	0.5 bolt without thread on the shear plane
r1	=	0.4 bolt with thread on the shear plane
fb	=	Bolt tensile strength (MPa)
Ab	=	Gross cross-sectional area of the bolt in the unthreaded area
m	=	Number of sliding planes
db	=	Bolt diameter in the unthreaded area
but	=	Plate thickness

Recommendation Phase

The design finalization phase aims to evaluate the analyzed alternatives to recommend the most efficient structural elements in terms of function, cost, and ease of implementation. In the redesign of the Rawamangun Station steel roof, the recommended design is a hybrid system: castellated beams in areas of small moments and solid WF in areas of large moments. The connection system chosen is simple and efficient in terms of fabrication. The evaluation is carried out by comparing the existing and alternative designs based on structural strength, material efficiency, ease of installation, and cost estimates. The results of this evaluation serve as the basis for cost calculations and the creation of final structural drawings.

3. RESULTS AND DISCUSSION



Section Control & Design of Flexible Structural

Figure7 Moment Occurred on Portal Alternative Design

Components

Component Name : Rafter Beam 210

Steel Properties

Profile Used: WF 525 x 350 x 12 x 19

Steel Quality: ASTM A36 (Fy = 240 MPa)

Internal Forces

Length	Eq 2-40	(Lb)	=10300 mm
Mmax		(Your)	=375058058Nmm
		(MAz)	=93764514.5Nmm
	Eq 2-41	(MBz)	=187529029Nmm
		(MCz)	=281293543.5Nmm
Vmax		(Vu)	=5794.15N

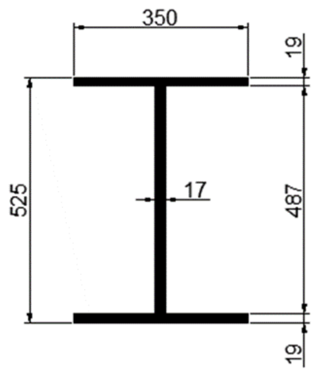


Figure8 Section Properties Rafter

1) Cross-sectional Stiffness Check (Section Local Buckling)
   Flange: COMPACT
   Web: COMPACT
2) Nominal Moment Check
   Mn1 (Yield Condition): 978.33×10^6 Nmm
   Mn2 (Lateral-Torsional Buckling): 1223.12×10^6 Nmm
   Mn3 (Condition 3 – Lb > Lr): 1451.93×10^6 Nmm
   Final Mn (used): 978.33×10^6 Nmm
3) **Factored Nominal Capacity**
   ϕMn: 880.50×10^6 Nmm
   Moment of Greatest Occurrence (Mu): 375.06×10^6 Nmm

Evaluation Results: ϕMn > Mu → PROFILE USED

Using the same approach as applied to the main rafter beam, by treating each member as a straight flexural element, the design checks were extended to the secondary structural components. Based on the internal forces and serviceability requirements, the appropriate section dimensions were determined as WF 200 × 150 × 6 × 9 mm for the secondary beams, and WF 100 × 100 × 6 × 8 mm for the purlins. These sections satisfy both strength and deflection criteria and are considered structurally adequate for use in the overall roof system.

Section Control & Design of Castellated Beam Geometry

The castellated beam geometry was adopted directly from the Gunung Garuda Steel catalogue, with the selected profile being C525 × 350, which is produced from a parent solid section of WF 350 × 350.

USED CASTELLATED BEAM PRFOILE

Height of castella	with	=	525	mm
Flange width	bf	=	350	mm
Thickness of flange	tf	=	19	mm
Thickness of web	tw	=	12	mm
Height of tee section	dt	=	85.5	mm
Depth of Castellation Hole	ds	=	354	mm
Depth of Between Flanges	dbflange	=	487	mm
Cutting angle	θ	=	45	

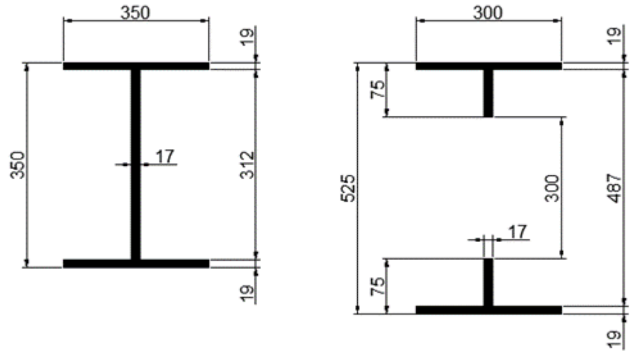


Figure9 Section Properties of Castella

Section Control & Failure Design of Castellated Beam Components

Component Name : Rafter Beam 153
Steel Properties
Material: ASTM A36
Fy = 240 MPa, Fu = 400 MPa
E = 200,000 MPa, G = 80,000 MPa
Fr (critical stress) = 70 Mpa

1) Voltage Check

Bending stress (fb) = 58.88 MPa
Shear stress (ft) = 13.56 MPa
Total stress (ftotal) = 72.44 MPa < All. 133.13 MPa → OK
Maximum shear stress of solid
(σv max) = 1.37 MPa < 64.49 MPa → OK

2) Cross-section Geometry

Tee area (AT) = 5,738 mm^2
Tee modulus of inertia (IT) = 859,634.67 mm^4
Zs tee = 13,644.99 mm^3
Distance between tee axes (d) = 363 mm
Total modulus of inertia (Ig) = 379,764,530.3 mm^4
Castella cross-section modulus (Zg) = 1,687,842.36 mm^3 → far > minimum Zg

3) Cross-Section and Slenderness Control

X-axis flange control (bf/tf) = 8.82 < 193.65 → OK
Y-axis web control (bs/tw) = 6.82 < 258.20 → OK
Flange Local Buckling: $\lambda = 8.82 < \lambda p = 10.97$ → COMPACT
Web Local Buckling: $\lambda = 37.82 < \lambda p = 108.54$ → COMPACT

Section Control & Design of Curved Structural Components

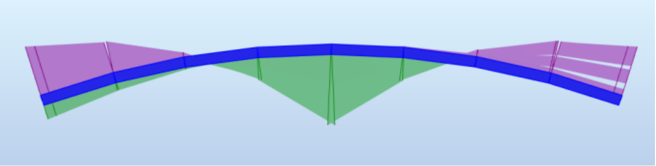


Figure 10 Moment on Curved Rafter

Component Name : Rafter Beam 210
Steel Properties
Material: ASTM A36
F_y = 240 MPa, F_u = 400 MPa
E = 200,000 MPa, G = 80,000 MPa
F_r (critical stress) = 70 Mpa
Profile: WF 525x350x12x19

1) Internal Loads and Geometry

Beam Span Length (L_b) = 4,129 mm
Maximum load:
M_{max} = 375,058,058 Nmm
V_{max} = 5,794.15 N
P_u (axial force) = 1,904.23 N
Column height (h_c) = 4,129 mm
Station horizontal span = 20,600 mm
Rise (arch height) = 1,816 mm

2) In-Plane Stability Check

Sway Check:
$5 \times h_c$ = 20,645 mm → L < 20,645 mm → STABLE
Span to rise ratio:
$0.25 \times L$ = 5,150 mm > 1,816 mm → OK, USE SWAY CHECK
Horizontal deflection:
Δ = 1 mm < $\Delta_{all} = h_c/1000$ = 4.129 mm → OK
Slenderness ratio used = 1 → STABILITY OK

3) Voltage Check

Direct stress (σ_1) = 133.88 N/mm²
Out-of-plane bending stress (σ_2) = 18.64 N/mm²
Total stress = $\sigma_1 + \sigma_2$ = 152.51 N/mm²

4) Flexural Capacity and Cross Section

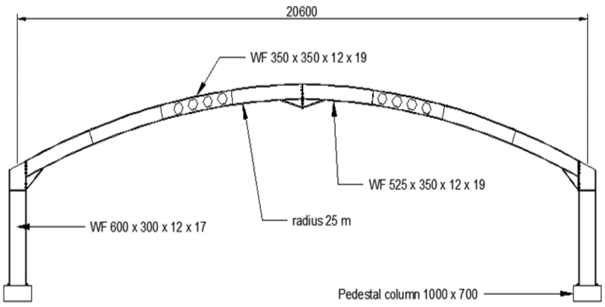
Design stress reduction (ρ_{yd}) = 230.14 MPa
Nominal moment $M_{nx} = \rho_{yd} \times S_x = 582.22 \times 10^6$ Nmm
Final flexural cap. = 0.645 < 1 → SECTION OK

5) Lateral Torsional Buckling (LTB)

Elastic critical moment (M_e) = 4,713,638,022 Nmm
Length between lateral restraints = 2,627.2 mm
Effective slenderness (λ_{LT}) = 32.55
Interpolation py = 230.30 MPa
Axial compressive capacity (P_{cy}) = 3,536,088.23 N
Moment distribution factor (β) = 0.02 → m_{LT} = 0.61
Final buckling moment (M_b) = 607.17×10^6 Nmm
Final capacity = 0.582 < 1 → STABLE OUT OF PLANE OK

Following the same analytical procedures used in the solid beam evaluation, the castellated beam is then assessed using its specific geometric properties as previously defined. The evaluation focuses on the structural response of the castellated profile under bending and curvature, particularly due to its perforated web which influences stiffness distribution and deformation patterns.

From this assessment, the curvature behavior of the castellated beam is found to remain within acceptable limits, ensuring that the structure maintains its integrity without exceeding stress or deflection thresholds. The perforation geometry contributes to a lighter profile while maintaining comparable flexural capacity.



Control of Flexural Structural Components - Axial (Beam-Column)

Figure 11 Final Curved of Castella

Component Name : Column 146
Steel Properties
Material: ASTM A36
F_y = 240 MPa, F_u = 400 MPa
E = 200,000 MPa, G = 80,000 MPa
F_r (critical stress) = 70 Mpa
Profile 600x300x12x17

1) Geometric Properties:

Ag: 16,992 mm²
Ix / Iy: 1.048×10^9 mm⁴ / 76.586×10^6 mm⁴
Zx / Zy: 3.934×10^6 mm³ / 785.376 mm³
rx / ry: 248.38 mm / 67.14 mm
J (Torsional Const.): 1,308,616 mm⁴

2) Internal Forces

Axial Force (P_u): 16,063 N
Moments:
Mux: 4.34×10^6 Nmm
Muy: 3.90×10^7 Nmm
Shear (V_u): 16.063 N

3) Moment Capacity (Flexural)

Mp-x = 944.2×10^6 Nmm
Mp-y = 188.5×10^6 Nmm
Lateral-Torsional Buckling (LTB):
L_b = 10,300 mm > L_r → Use Mn3
$\phi M_{n-x} = 837.3\times10^6$ Nmm > Mux → OK
$\phi M_{n-y} = 169.6\times10^6$ Nmm > Muy → OK
Beam-Column Interaction Check:
$P_u / \phi P_n = 0.0053 < 0.2$
Interaction Eq (Eq.2):

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1{,}0$$

0.2377<1
ϕM_n → Profile Adequate

The WF 600x300x12x17 column is declared safe in terms of axial strength, flexure, lateral-torsional buckling, and axial-flexure interaction. All components meet the LRFD (AISC) design requirements. This profile is suitable for use as a structural column.

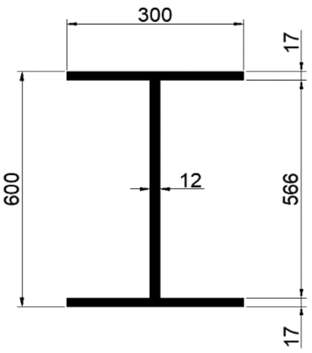


Figure12 Column Section Properties

Shear Control

1) Alternative Design 1
   Shear Stress (ϕ.Vn) =757382.4 N
   ϕ.Vn > Vu
   Shear ratio = (Vu/ϕ.Vn)
   = 0.443
2) Alternative Design 2
   Shear Stress (ϕ.Vn) =757382.4 N
   ϕ.Vn > Vu
   Shear ratio = (Vu/ϕ.Vn)
   = 0.338
3) Alternative Design 3
   Shear Stress (ϕ.Vn) =757382.4 N
   ϕ.Vn > Vu
   Shear ratio = (Vu/ϕ.Vn) = 0.814

Deflection Control

1) Alternative Design 1
   Deflection (Δ) = 81 mm
   Length (Lb) = 20600 mm
   Allowable
   Deflection (Δall) = L/240
   = 85.83 mm
   (Δ) < (Δall) OK
2) Alternative Design 2
   Deflection (Δ) = 59 mm
   Length (Lb) = 20600 mm
   Allowable
   Deflection (Δall) = L/240
   = 85.83 mm
   (Δ) < (Δall) OK
3) Alternative Design 3
   Deflection (Δ) = 167 mm
   Length (Lb) = 20600 mm
   Allowable
   Deflection (Δall) = L/240
   = 85.83 mm
   (Δ) > (Δall)
   PROFILE NOT OK

Hybrid Single Barrel Vault Connection Analysis

Table 2 Connection Used on Alternative Design

No	Connection	Profile	Plate	Bolt	Number of Bolts/Plate
1	Apex Haunch	Curved Rafter WF450x 300x11x 18	PL 25x300 PL 12x300	M24	12
2	Eaves Haunch	Curved Rafter WF450x 300x11x 18 Column WF600x 300x12x 17	PL 20x300 PL 12x145 PL 15x300	M24	12
3	Clip Angle	Curved Rafter WF450x 300x11x 18 Secondar y Beam WF200x 150x6x9	L100x 100x16	M20	6
4	Purlin	Purlin WF100x 100x6x8	PL 12x90	M16	4
5	Base Plate	Column WF600x 300x12x 17	PL 30x500 x900	M22	4

Total Design Weight

Total Weight Comparison of 1 Portal Structure

1) Total mass of existing portal design = 12.4 tons
2) Total mass of alternative portal design 1 = 5.89 tons
3) Total mass of alternative portal design 2 = 5.95 tons
4) Total mass of alternative portal design 3 = 5.62 tons

Comparison of Total Weight of the Overall Structural Model

1) Total mass of existing design = 335.35 tons
2) Total mass of alternative design 1 = 92.82 tons
3) Total mass of alternative design 2 = 93.91 tons
4) Total mass of alternative design 3 = 89.62 tons

Comparison Table of Alternative Designs

To facilitate a clear comparison, a summary table is presented below in table 3, showcasing the technical and economic aspects of each alternative. The results of this comparison will serve as the basis for recommending the most optimal design choice for implementation in this project.

Table 3 Comparison Table of Alternative Designs

No	Variable	Alternative Design 1	Alternative Design 2	Alternative Design 3
1	Cost	Rp7,110,683,434.77	Rp7,193,751,195.81	Rp6,865,655,588.48
2	Total Weight	92.82 Tons	92.91 Tons	89.61 Tons
3	Shear Ratio	0.443	0.338	0.814
4	Stress Ratio	0.612	0.584	0.612
5	Deflection Control	81 mm < 85 mm	59 mm < 85 mm	167 mm < 85 mm

4. CONCLUSION

Based on the Hybrid Single Barrel Vault redesign for the Jakarta LRT Phase 1B station roof using the LRFD method and Value Engineering, the most optimal design alternative is the combination of castellated beams and solid WF beams. This hybrid approach meets both structural performance and cost efficiency.

1) Design Selection:
   Among three alternatives, Alternative Design 1 is selected as the final recommendation. Although Alternative 3 had the lowest cost, it failed structurally due to excessive deflection and shear ratio. Between Alternative 1 and 2, both met structural requirements, but Alternative 1 was more economical.
2) Key Structural Elements:
   - Curved Rafter: WF 525x350x12x19 & Castella WF 350x350x12x19
   - Column: WF 600x300x12x17
   - Secondary Beam: WF 200x150x6x9
   - Purlin: WF 100x100x6x8
   - Connection Design includes:
   - Apex & Eaves Haunch (M24 bolts, PL 25–12 mm thick)
   - Clip Angle (L100x100x16, M20 bolts)
   - Purlin Plate (PL12x90, M16 bolts)
   - Base Plate (PL 30x500x900, M22 bolts)
3) Cost Efficiency:
   - Existing design: Rp. 17.59 billion
   - Alternative 1 (hybrid): Rp. 7.11 billion
   - Alternative 2 (solid): Rp. 7.19 billion
   - Alternative 3 (full castellated): Rp. 6.87 billion → rejected due to structural failure

   Alternative 1 offers a cost saving of Rp. 10.48 billion (40.43%) compared to the existing design, while still meeting structural and functional requirements.

REFERENCES

[1] JM Gere and BJ Goodno, “Mechanics of Materials,” 2012.
[2] G. Paus, G. Seran, and AH Pattiraja, “Application of Value Engineering on Building Elements (Stairs) of the UPTP Kupang Job Training Center,” 2024.
[3] D. Paunovic and J. Pawlowski, “The King's Cross station redevelopment of 6 Arup and King's Cross,” 2012.
[4] ZS Makowski, “ANALYSIS, DESIGN AND CONSTRUCTION OF BRACED BARREL VAULTS,” 2006.
[5] SS Fares, P. Eng, J. Coulson, and DW Dinehart, “AISC: Castellated and Cellular Beam Design 31 Steel Design Guide,” 2016.
[6] L. Amayreh and MP Saka, “FAILURE LOAD PREDICTION OF CASTELLATED BEAMS USING ARTIFICIAL NEURAL NETWORKS,” 2005.
[7] K. Bauer, M. Professor, and B. Zhang, “Design of castellated steel beams,” 2021.
[8] K. Kuchta and M. Maslak, “FAILURE MODES DETERMINING THE RESISTANCE AND THE STABILITY OF STEEL CELLULAR BEAMS,” 2015.
[9] MR Soltani, A. Bouchaïr, and M. Mimoune, “Nonlinear FE analysis of the ultimate behavior of steel castellated beams,” *J Constr Steel Res*, vol. 70, pp. 101–114, March. 2012.10.1016/j.jcsr.2011.10.016.
[10] SNI 1727 - 2020, “Minimum design loads and related criteria for buildings and other structures,” 2020.
[11] A. Setiawan, “STEEL STRUCTURE DESIGN USING LRFD METHOD,” 2013.