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Alternative Design of Menara 17 Structure in Surabaya with Castellated Steel Beam

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ABSTRACT

Assigning structural material is one of critical step in designing, steel material often used because of it high ductility. Menara 17 is a 17 storey multipurpose building with reinforced concrete as an existing material structure. This research analyze Menara 17 alternative design with Special Moment Frame (SMF) steel structure to resist lateral load with high ductility and expected to withstand significant inelastic deformation. Castellated beam is used to increase it capacity to weight ratio and ease MEP installation through the opening. SNI and AISC is used for main design guide. From the analysis result, the alternative design has satisfy criteria of irregularity, P-delta effect, story drift, and Strong Column Weak Beam. Honeycomb castellated beam HCO 520.250.9.14 profile is used as main beam. King-cross column KC 800.400.16.30 profile is used as main column. Circular hollow section CHS 406,4.40 profile is used as diagonal bracing. Wide flange WF 200.200.8.12 profile is used as lateral bracing. Lateral bracing to beam is connected with shear connection. Beam to column is connected with endplate moment connection. Menara 17 design with steel structure is 59,17% less heavy than reinforced concrete structure.

Keywords: *Steel Structure; Lateral Load;High Ductility; Special Moment Frame; Castellated Beam.*

INTRODUCTION

Assigning structural material is one of critical step in designing. Lumber, reinforced concrete, and steel is an example of common structural material. Steel is often used as structural materal because of it high ductility [1]. Structural engineering aim to create a strong, long lasting, and affordable building from an architectural concept. This engineering contains structural component placement, verify structural member capacity to bear given load, connection designing, etc. Steel is a material that can bear many stresses, steel has high yielding stresses (hundreds of MPa), because of this advantages, structural component with steel material need relatively small section area, thus reducing overall structural weight, increasing workability, accelerating erection and construction duration, and giving more architectural accessibility.

Menara 17 is a 17-story multipurpose building, this building has reinforced concrete as an existing structure material, this research aim to create alternative design of Menara 17 with Special Momen Frame (SMF) steel structure. Castellated beam is used as beam component structure in this alternative design, castellated beam is used to increase capacity to weight ratio and ease Mechanical, Electrical, and Plumbing (MEP) installation because of castellated hole opening. Castellated beam is one of hot rolled wide flange profile modification, wide flange web is cut with zig-zag pattern, this will produce two section, upper tee and bottom tee, those

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two section then stacked and welded together, because of this process, castellated modification will increase section depth up to 50%. This research aim to design Menara 17 structure model with steel material which contain steel grade requirements, sections requirement, connections designing, and weight comparison between steel structure design and reinforced structure design.

Steel Wide Flange Beam

Beam that subjected with some load in gravity direction will create positive bending moment, this bending moment will create compression zone in upper side of the beam, and tension zone in bottom side of the beam. Compression zone will cause steel material to contract and tend to have deflection (laterally). Vice versa, tension zone will cause steel material to expand and tend to not have deflection, this will cause section to rotate (torsion), this phenomenon known as lateral torsional buckling. LTB can reduce beam flexuran capacity.

Seismic Resistant Building

Seismic resistant building concept expect building still standing after subjected with vibration and lateral load from seismic activity, seismic resistant building is allowed to have some damages but with limited condition as requied. In frequent small earthquake magnitude, main structural damages is prohibited. In infrequent medium earthquake magnitude, main structural is allowed to have some repairable minor damages. In infrequent high earthquake magnitude, main structural is allowed to have major damages without total collapse [2].

Special Moment Frame

Moment Frame is a frame portal which it structure component and constituent rigid connection resist lateral and overturning forces. Moment frame consist of Ordinary Moment Frame (OMF), Intermediate Moment Frame (IMF), and Special Moment Frame (SMF) [3]. Special Moment Frame (SMF) is expected to give deformation capacity through inelastic yielding from beam component and limited yielding at column panel zone from beam to column connection yielding. Column must be designed stronger than the beam but flexural yielding at the column base is allowed, this concept known as Strong Column Weak Beam (SCWB) [4].

Castellated Beam

Castellated beam s steel modification method proposed by H. E. Horton and Iron Work in 1910. Castellated open web expanded beam and girder is a beam modification with opening in the web, this modification is produced by cutting beam web with predefined opening geometry, this cutting process will produce two section, upper tee section and bottom tee section, these two section is stacked and connected with weld[5]. There are various geometry of castellated beam opening, such as hexagonal, honeycomb, octagonal, diamond, etc [6]. This alternative design use castellated beam with honeycomb opening as shown in figure 1.

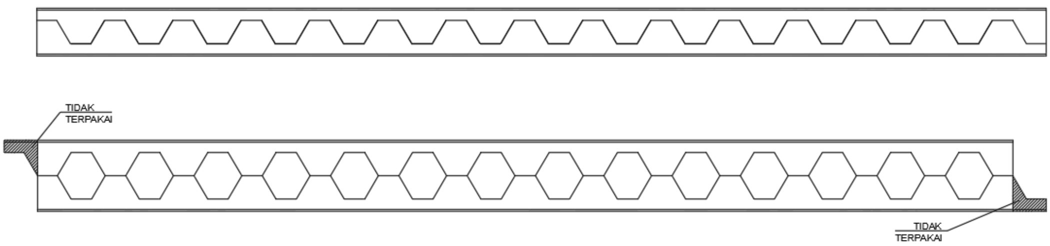

Figure 1: Castellated beam with honeycomb opening

MATERIALS AND METHODS

In this research ETABS program is used for modelling and analyzing, concrete slab, beam, column, lateral bracing, and diagonal bracing are the modelled structure component.

In outline, the flow of this research:

- Problem identification, in this case, what design is possible for the steel structure of Menara 17.
- Reviewing references and theories related to this steel designing.
- Sourcing secondary data such as existing architectural drawing of Menara 17.
- Determining preliminary design for structural modelling in ETABS.
- Determining loads.
- Checking structural irregularity, P-Delta effect, story drift, Strong Column Weak Beam criteria, structural component capacity, and connection designing. If this checking step is not meet the requirement then redesign is needed.
- If the requirement is satisfied then discussions and conclusions can be drawn.

RESULTS AND DISCUSSION

Structural Description

Menara 17 is 17 story building with 76,10 meter elevation at the top level, this alternative design use steel structure with BJ41, A325 bolt grade, K250 concrete grade, and BJTP24 reinforcing bar grade. Three dimensional modelling from ETABS is shown in figure 2.

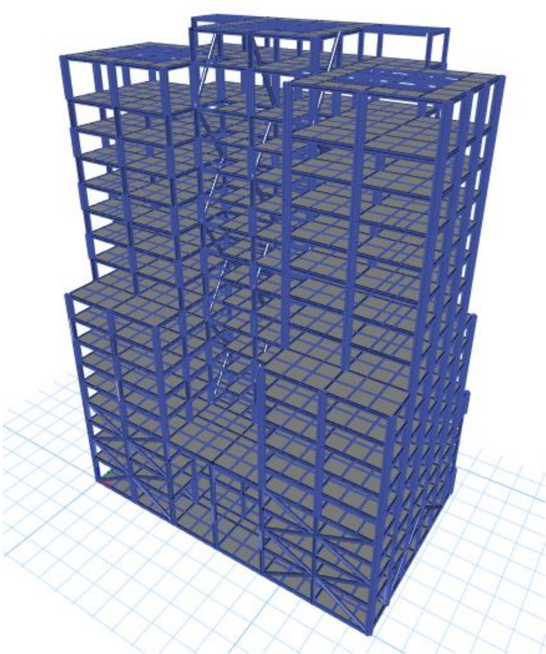

Figure 2: Alternative design 3D modelling.

Loading

Loading should be determined based on current regulation, minimum design loading based on SNI 1727:2020[7] and seismic loading based on SNI 1726:2019. This loading consist of dead load, super imposed dead load, live load, wind load, seismic load (response spectrum), and notional load, these load should be defined in ETABS, except for dead load and notional load,

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those two load can be generated automatically by ETABS program.

Portal Design Check

Modes of natural vibration must be considered and modal of mass participating ratio should exceed 90%, in this alternative design, modal of mass participating ratio is exceed 90% in 10th mode. Dynamic base shear exceed 100% of static base shear. This alternative design also meet requirement of torsional and vertical irregularities, story drift not exceed story drift limit, and P-Delta effect meet the required criteria.

Slab

Slab is made from concrete and have typical dimension of 2750 mm x 2500 mm, thus can be considered as two way slab, 19 mm of reinforced bar is used. From ETABS, slab have 75,369 KNm of slab momen, thus 0,018 of steel section area to concrete section area (ρ) is needed in both slab direction. 100 mm of reinforced bar spaces is satisfy ρ needs.

Castellated Beam

This alternative design have two type of beams, 8 meters span beam and 5 meters span beam, castellated beam have openings in its web, this opening introduce new criticals zone that need to be checked, this inspection procedure in based on AISC Design Guide 31 about castellated and cellular beam[8]. Castellated beam with honeycomb opening HCO 520.250.9.14 is used in this alternative design. Net shear force and net flexural force in each opening with composite action and haunch forces distribution is shown in table 1 and table 2.

Table 1: Net shear and flexural forces form each openings in 5 meters span beam

Opening No.	x_i (m)	V_{rnet} (kN)	M_{rnet} (kNm)
End	0	32,802	163,805
1	0,25	45,405	166,897
2	0,75	62,881	123,965
3	1,25	0	154,018
4	1,75	0	106,161
5	2,25	0	73,856
6	2,75	0	66,901
7	3,25	0	103,688
8	3,75	0	141,692
9	4,25	52,294	120,999
10	4,75	34,818	154,017
Oend	5	22,215	151,164

Table 2: Net shear and flexural forces form each openings in 8 meters span beam

Opening No.	x_i (m)	V_{rnet} (kN)	M_{rnet} (kNm)
End	0	12,419	171,605
1	0,25	23,403	174,845
2	0,75	36,959	138,506
3	1,25	0	179,765
4	1,75	0	136,059
5	2,25	0	124,802
6	2,75	0	115,272
7	3,25	0	98,506

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8	3,75	0	87,419
9	4,25	0	84,504
10	4,75	0	94,113
11	5,25	0	104,819
12	5,75	0	116,367
13	6,25	0,037	132,997
14	6,75	3,800	89,904
15	7,25	95,371	172,122
16	7,75	81,815	205,458
Oend	8	70,831	201,651

Flexural forces in each opening will be transferred as axial forces in upper tee and bottom tee section. In this composite castellated beam design, concrete slab is assumed to resist all compressive axial forces and bottom tee resist all tensile axial forces, this assumption will be correct if shear stud is sufficient, therefore this assumption needs to be checked.

This checking can be done with some iteration to calculate suitable effective concrete depth. Those depth is suitable if axial from iteration has less than 1% difference between previous iteration. Axial forces of tee section (T_1), effective concrete depth (X_c), and effective composite depth (d_{eff}) can be calculated with equations as follows and axial forces in each tee section is shown in table 3 and table 4

$$T_{1i} = \frac{M_{ri}}{d_{eff}}$$

$$X_c = \frac{T_{1i}}{0{,}85F'cb_{eff}}$$

$$d_{eff} = d_g - (H_{tee} - y_{cm}) + t_c - \frac{1}{2}X_c$$

where $F'c$ = concrete compressive strength (Mpa), b_{eff} = slab effective width (mm), d_g = castellated beam depth (mm), H_{tee} = tee section depth (mm), y_{cm} = tee section center of mass depth (mm), t_c = slab thickness (mm).

Table 3: Local axial forces form each openings in 5 meters span beam

Opening No.	x_i **(m)**	X_c **(mm)**	T_1 **(kN)**
End	0	11,336	240,898
1	0,25	11,551	245,485
2	0,75	8,561	181,937
3	1,25	10,653	226,392
4	1,75	7,325	155,666
5	2,25	5,088	108,119
6	2,75	4,607	97,903
7	3,25	7,154	152,020
8	3,75	9,794	208,142
9	4,25	8,355	177,558
10	4,75	10,653	226,391
Oend	5	10,454	222,163

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Table 4: Local axial forces form each openings in 8 meters span beam

Opening No.	x_i **(m)**	X_c **(mm)**	T_1 **(kN)**
End	0	11,880	252,471
1	0,25	12,106	257,280
2	0,75	9,573	203,429
3	1,25	12,450	264,587
4	1,75	9,402	199,810
5	2,25	8,620	183,173
6	2,75	7,958	169,104
7	3,25	6,794	144,385
8	3,75	6,026	128,062
9	4,25	5,825	123,773
10	4,75	6,490	137,915
11	5,25	7,232	153,687
12	5,75	8,034	170,720
13	6,25	9,189	195,283
14	6,75	6,198	131,719
15	7,25	11,916	253,238
16	7,75	14,248	302,803
Oend	8	13,981	297,134

Slab is resist all of compressive axial load in composite castellated beam, if slab and shear stud is able to withstand T_1 axial forces in each openings this section is considered as full composite system, vice versa, this section is considered as partial composite system. This composite action is shown in table 5 and table 6 with q_s is average shear stud capacity with equation as follow.

$$q_s = \frac{nQ_n}{\frac{1}{2}L}$$

where q_s = average shear stud capacity (kN/m’), n = number of shear stud, Q_n = nominal shear stud capacity (kN), L = beam span (m).

Table 5: Local axial forces with composite actions form each openings in 5 meters span beam

Opening No.	x_i **(m)**	T_1 **(kN)**	$q_s x_i$ **(kN)**	**Compsite Action**	T_0 **(kN)**	T_{comp} **(kN)**
End	0	240,898	0	N/A	N/A	N/A
1	0,25	245,485	138,637	PARTIAL	106,848	245,485
2	0,75	181,937	415,911	FULL	0	181,937
3	1,25	226,392	693,185	FULL	0	226,392
4	1,75	155,666	970,459	FULL	0	155,666
5	2,25	108,119	1247,733	FULL	0	108,119
6	2,75	97,903	1247,733	FULL	0	97,903
7	3,25	152,020	970,459	FULL	0	152,020
8	3,75	208,142	693,185	FULL	0	208,142
9	4,25	177,558	415,911	FULL	0	177,558
10	4,75	226,391	138,637	PARTIAL	87,754	226,391
Oend	5	222,163	0	N/A	N/A	N/A

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Table 6: Local axial forces with composite actions form each openings in 8 meters span beam

Opening No.	x_i **(m)**	T_1 **(kN)**	$q_s x_i$ **(kN)**	**Compsite Action**	T_0 **(kN)**	T_{comp} **(kN)**
End	0	252,471	0	N/A	N/A	N/A
1	0,25	257,280	138,637	PARTIAL	118,643	257,280
2	0,75	203,429	415,911	FULL	0	203,429
3	1,25	264,587	693,185	FULL	0	264,587
4	1,75	199,810	970,459	FULL	0	199,810
5	2,25	183,173	1247,733	FULL	0	183,173
6	2,75	169,104	1525,007	FULL	0	169,104
7	3,25	144,385	1802,281	FULL	0	144,385
8	3,75	128,062	2079,555	FULL	0	128,062
9	4,25	123,773	2079,555	FULL	0	123,773
10	4,75	137,915	1802,281	FULL	0	137,915
11	5,25	153,687	1525,007	FULL	0	153,687
12	5,75	170,720	1247,733	FULL	0	170,720
13	6,25	195,283	970,459	FULL	0	195,283
14	6,75	131,719	693,185	FULL	0	131,719
15	7,25	253,238	415,911	FULL	0	253,238
16	7,75	302,803	138,637	PARTIAL	164,166	302,803
Oend	8	297,134	0	N/A	N/A	N/A

Opening from castellated beam cause upper tee section and bottom tee section work as independent beam with fixed end, any shear forces from this independent beam will cause moment in each end, this moment is known as Vierendeel moment. This moment in each opening is shown in table 7 and table 8, Vierendeel moment can be calculated with equation as follow.

$$M_{vr} = V_{rnet}\frac{A_{tee}}{A_{net}} \times \frac{e}{2}$$

where M_{vr} = Vierendeel moment in inspected tee section (kNm), V_{rnet} = net shear force (kN), A_{tee} = inspected tee section area (m^2), A_{net} = upper and bottom tee section area (m^2), e = tee section span (m).

Table 7: Vierendeel moments form each openings in 5 meters span beam

Opening No.	x_i **(m)**	X_c **(mm)**	T_1 **(kN)**
End	0	32,802	1,230
1	0,25	45,405	1,703
2	0,75	62,881	2,358
3	1,25	0	0
4	1,75	0	0
5	2,25	0	0
6	2,75	0	0
7	3,25	0	0
8	3,75	0	0
9	4,25	52,294	1,961
10	4,75	34,818	1,306
Oend	5	22,215	0,833

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Table 8: Vierendeel moments form each openings in 8 meters span beam

Opening No.	x_i **(m)**	X_c **(mm)**	T_1 **(kN)**
End	0	12,419	0,466
1	0,25	23,403	0,878
2	0,75	36,959	1,386
3	1,25	0	0
4	1,75	0	0
5	2,25	0	0
6	2,75	0	0
7	3,25	0	0
8	3,75	0	0
9	4,25	0	0
10	4,75	0	0
11	5,25	0	0
12	5,75	0	0
13	6,25	0,037	0,001
14	6,75	3,800	0,143
15	7,25	95,371	3,576
16	7,75	70,831	2,656
Oend	8	12,419	0,466

Due to axial and flexural capacity combination, obtained interaction 0,96 ≤ 1 (o.k). Interaction due to flexural forces with buckling flexural web post capacity is 0,45 ≤ 1 (o.k). Interaction due to horizontal shear force with web post capacity is 0,60 ≤ 1 (o.k). Interaction due to vertical shear force with tee section shear capacity is 0,85 ≤ 1 (o.k). Interaction due to vertical shear force with gross section is 0,45 ≤ 1 (o.k), this castellated beam section have 2,339 mm of deflection.

King-cross Column

From ETABS, maximum force subjected to column is P_u = 13042,519 kN, M_{ux} = 222,746 kNm, M_{uy} = 121,14 kNm, and V_u = 42,479 kN. King-cross section KC 800.400.16.30 is used as column component. Interaction due to axial and flexural capacity combination is 0,94 ≤ 1 (o.k).

Bracing

This alternative design model have two type of bracing. Lateral bracing for high ductility system as specified by SNI 7860:2020 and diagonal bracing to resist lateral load together with Special Moment Frame (SMF). Diagonal bracing will resist lateral load in a form of axial load, from ETABS, maximum force subjected to diagonal bracing is P_{tu} = 6293,722 kN and P_{cu} = 7356,268 kN. Circular hollow section CHS 406,4.40 is used as diagonal bracing component. Interaction due to tensile axial force is 0,61 ≤ 1 (o.k) and interaction due to compressive axial force is 0,94 ≤ 1 (o.k).

Lateral bracing serve as bracing for main beam, this will reduce unbraced beam length, this length must not exceed high ductility limit for unbraced beam length, this limit can be calculated with equation as follow.

$$L_b = 0{,}095 \frac{r_y E}{R_y F_y}$$

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where L_b = unbraced beam length limit (mm), r_y = gyrration radius of y axis (mm), E = steel elasticity modulus (MPa), R_y = expected yielding ratio, F_y = steel yielding strength (MPa).

Lateral bracing is designed to be composite with slab, from ETABS, maximum force subjected to lateral bracing is V_u = 68,647 kN and M_u = 67,56 kNm. Interaction due to flexural force is 0,57 ≤ 1 (o.k) and interaction due to shear force is 0,25 ≤ 1 (o.k).

Lateral Bracing to Beam Connection

Shear connection is used in lateral bracing to beam connection, from ETABS, shear force in this connection is V_u = 68,647 kN, this connection use 150.100.4 connection plate and 2M16 bolt. Interaction due to block shear rupture is 0,75 ≤ 1 (o.k). Connection plate is welded in beam web with E70XXX electrode grade, 3 mm weld thickness, and 100 mm minimal weld length.

Diagonal Bracing Connection

Diagonal bracing is connected to nearby structure component such as beam and column by gusset plate with 30 mm thickness, diagonal bracing is welded into gusset plate, and gusset plate is welded into nearby structure component such as beam and column. From ETABS, axial force from diagonal bracing is N_{ut} = 6293,722 kN and N_{uc} = 7356,268 kN. Interaction due to tearout of gusset plate is 0,67 ≤ 1 (o.k). Interaction due to Whitmore section tension yielding is 0,85 ≤ 1 (o.k)[9]. Interaction due to gusset plate buckling is 0,991 ≤ 1 (o.k). Interaction due to gusset plate capacity to resist gusset plate internal force with uniform force method is 0,5 ≤ 1 (o.k)[10]. Weld connection is used with E70XXX electrode grade, 20 mm weld thickness, 2400 mm minimal weld length for diagonal bracing to the gusset plate, 1000 mm minimal weld length for gusset plate to the nearby structure component.

Column Splice Connection

Column splice connection is needed to connect two column, this column is limited by length (often 12 meters), this splice connection is connect column flange and web. From ETABS, shear and flexural forces in splice connection is V_u = 42,479 kN and M_u = 222,746 kNm. 400.200.8 connection plate and 12M16 is used to connect column flange. 100.200.8 connection plate and 2M16 is used to connect column web. Interaction due to block shear rupture in column flange connection plate is 0,98 ≤ 1 (o.k). Interaction due to block shear rupture in column web connection plate is 0,16 ≤ 1 (o.k).

Beam to Column Connection

Beam to column connection in Special Moment Frame (SMF) must use rigid moment connection, this alternative design use stiffened endplate eight bolt design (8ES) based on prequalified parameter limitation of endplate connection according to SNI 7972:2020[11].

1032.250.30 endplate and 18M30 is used, beam flange is groove welded to endplate with E70XXX electrode grade, 10 mm weld thickness, and 150 mm minimal weld length. Beam web is fillet welded with E70XXX electrode grade, 5 mm weld thickness, and 300 mm minimal weld length. Continuity plate and doubler plate is needed in this endplate connection design, due to king-cross section, conventional doubler plate cannot be used. Loads from the beam is distributed through inclined plate to king-cross flange, thus adjacent king-cross flange with load direction from the beam is serve as doubler plate[12]. Continuity plate and inclined plate have 30 mm of thickness and groove welded with E70XXX electrode grade, 10 mm weld thickness,

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600 mm minimal weld length for continuity plate, and 400 mm minimal weld length for inclined plate.

Base Plate

Base plate must be provided in steel structure, this base plate connects column base with corresponding pedestal, this will prevent steel column in direct contact with the soil, thus corrosion from soil humidity will be prevented. Base plate design procedure is based on AISC Design Guide 01 about base plate and anchor rod[13]. From ETABS, forces in base column is P_u = 16262,195 kN, V_u = 4011,385 kN, and M_u = 514,402 kNm. 1300.1300.150 of base plate and 19 mm X 300 mm ASTM F1554 anchor rod is used. From the analysis, this base plate design is considered as small moment base plate, thus detailed analysis for anchor rod is not necessary.

Strong Column Weak Beam

Steel Special Moment Frame (SMF) must have projected expected flexural column to beam capacity ratio exceed 1. This alternative design have 1,241 $>$ 1,0 SCWB ratio (o.k).

Structure Weight Comparison

Structure weight can be obtained in ETABS by using assembled joint masses analysis. Reinforced concrete structure weigh 20109,11 tons and steel structure weigh 11897,69 tons, thus steel structure have 59,17% less weight compared to reinforced concrete strucutre.

The result from the analysis, the required grade, structural components section, and its connection are:

- Steel structural component and connection plate use BJ41 steel grade, bolt use A325 grade, concrete use K250 grade, and reinforced bar use BJTP 24 grade.
- Slab use 180 mm of thickness with Ø19-100 of reinforced bars in longitudinal and transverse direction.
- Beam use hot rolled WF 350.250.9.14 modified into castellated beam with honeycomb opening in HCO 520.250.9.14 section.
- Column use 2 X WF 800.400.16.30 modified into king-cross KC 800.400.16.30 section.
- Diagonal bracing use circular hollow section CHS 406,4.40.
- Lateral bracing use hot rolled WF 200.200.8.12.
- Lateral bracing to beam connection use shear connection with 150.100.4 connection plate and 2M16 bolts.
- Diagonal bracing to gusset plate connection use welded connection with E70XXX electrode grade, 20 mm weld thickness and 2400 mm minimal weld length.
- Gusset plate to nearby structural component use welded connection with E70XXX electrode grade, 20 mm weld thickness and 1000 mm minimal weld length.
- Column flange splice connected with 400.200.8 connection plate and 12M16 bolts.
- Column web splice connected with 100.200.8 connection plate and 2M16 bolts.
- Beam to column connection use endplate moment connection with 1032.250.30 endplate and 18M30 bolts, *king-cross* flange is also serve as column doubler plate.
- Base plate use 1300.1300.150 plate and ASTM F1554 Grade 36 4 X 19 mm X 300 mm of anchor rod.

- Steel alternative design have 59,17% less weight compared to existing reinforced concrete.

This alternative design use non-composite vertical structural component, thus arise many downside, such as builiding is less convenience due to vibration and high fire hazard (can be solved with more responsive fire prevention). More detailed result can be obtained by checking structure performance, castellated beam and ordinary beam comparison, structure behavior when temperature factor is considered since almost all of structure components use steel material, etc.

CONCLUSION

Menara 17 can be modelled with steel structure alternative design, this design has satisfy criteria of irregularity, P-delta effect, story drift, Strong Column Weak Beam, and designed section can be used.

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