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Analysis and Design of 220 kV Transmission Line Tower (A conventional method of analysis and Indian Code based Design)

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Abstract: *Transmission line tower constitute about 28 to 42 percent of the cost of the transmission power line project. The increasing demand for electricity can be made more economical by developing different light weight configuration of transmission line tower. In this study an attempt is made to model, analyse and design a 220KV transmission line tower using manual calculations. The tower is designed in wind zone – V with base width 1/5thof total height of the tower. This objective is made by choosing a 220 KV single circuit transmission line carried by square base self-supporting tower with a view to optimize the existing geometry and then analysis of the tower has been carried out as a 2-D structure. Structure is made determinate by excluding the horizontal members and axial forces are calculated using method of joints and design is carried out as per IS 800:2007.*

Keywords: *Transmission towers, Geometry of tower, Self-supporting tower, Configuration oftower, unit load, method of joint, deflection, design.*

I. INTRODUCTION:

India has a large population residing all over the country and the electricity supply need of this population creates requirement of large transmission and distribution system. Also, the disposition of the primary resources for electrical power generation viz., coal, hydro potential is quite uneven, thus again adding to the transmission requirements. Transmission line is an integrated system consisting of conductor subsystem, ground wire subsystem and one subsystem for each category of support structure. Mechanical support of transmission line represents a significant portion of the cost of the line and they play an important role in the reliable power transmission. They are designed and constructed in wide variety of shapes, types, sizes, configuration and materials. The supporting structure types used in transmission line generally fall into one of the three categories: lattice, pole and guyed.

The supports of high voltage transmission lines are normally steel lattice towers. The cost of tower constitutes about quarter to half of the cost of transmission line and hence optimum tower design will bring in substantial savings. The selection of an optimum outline together with right type of bracing system contributes to a large extent in developing an economical design of transmission line tower. The height of tower is fixed by the user and the structural designer has the task of designing the general configuration and member and joint details. In this paper, the sag tension calculation is carried for conductor and ground wire using parabolic equation. Then different loading format including normal condition, top conductor broken, earth wire broken condition is evaluated.The wind loading is calculated on the longitudinal face of the towers and then two dimensional analysis of the tower is carried outand accordingly the design is completed for different members.

II. Transmission Line Components:

2.1. Transmission Line Tower

The following parameters for transmission line and its components are assumed from I.S. 802: Part 1: Sec: 1:1995, I.S. 5613: Part 2: Sec: 1:1989.

- Transmission Line Voltage: 220 kV (A. / C.)
- Right of Way (recommended): 35, 000 mm
- Angle of Line Deviation: 0 to 2 degrees
- Terrain Type Considered: Plain
- No. of Circuits: Single Circuit
- Tower Configuration: Vertical Conductor Configuration
- Bracing Pattern: Pratt system
- Cross Arm: Pointed

- Terrain Category: 2 (Normal cross country lines with very few obstacles)
- Return Period: 50 years
- Wind Zone: 5
- Basic Wind Speed: 50 m/s
- Design Wind Pressure: 793 N/sqm
- Tower Type: Self-Supporting, Type "A"
- Tower Geometry: Square Base Tower
- Inclination of the tower legs: 70 (with vertical)
- Shielding Angle: 30°
- Insulator Type: I String
- Number of Insulator Discs: 15
- Size of Insulator Disc: 255 × 145 mm (Skirt Diameter)
- Length of Insulator String: 2,500 mm
- Creep Effect: Not Considered

2.2. Conductor

A substance or a material which allows the electric current to pass through its body when it is subjected to a difference of electric potential is known as Conductor. The properties of the conductor considered here are tabulated in Table1.

Table 1: Conductor mechanical and electrical properties.

Conductor material	ACSR
Code name	Panther
Conductor size	30/7/3.00 mm
Area of the conductor (for all strands), A	2.6155 cm^2
Overall diameter of the conductor (d)	21 mm
Weight of the conductor (w)	0.973 kg/m
Bearing strength of the conductor (UTS)	9130 kg
Coefficient of linear expansion (α)	17.73×10^{-6}/°C
Modulus of elasticity Final (E_1)	0.787×10^{6}kgf/cm^2
Modulus of elasticity Initial (E_2)	0.626×10^{6}kgf/cm^2

2.3. Earthwire

The earthwire is used for protection against direct lightning strokes and the high voltage surges resulting there from. There will be one or two earthwire depending upon the shielding angle or protection angle. The earthwire considered for transmission line has the following properties as mentioned in Table 2.

Table 2: Earthwire mechanical and electrical properties

Material of earthwire	Galvanized steel
No of earthwire	one
Stranding/wire diameter	7/3.15mm
Total sectional area	54.55mm^2
Overall diameter	9.45 mm
Approximate weight	428kg/km
Calculated D.C. resistance at 20˚C	3.375ohm/km
Mini UTS	5710 kg
Modulus of elasticity	19361 kg/mm^2
Coefficient of linear expansion	11.50×10^{-6}/˚C
Maximum allowable temperature	53˚C

2.4. Insulator Strings

Insulators are devices used in the electrical system to support the conductors or to support the conductors carrying at given voltages.The insulators separate thecurrent carrying conductors of a transmission line from their support structures to prevent the flow of current through the structure to ground and to provide necessary mechanical support to the conductors at a safer height above the ground level.

III. Sag tension for conductor and ground wire:

Indian standard codes of practice for use of structural steel in over-head transmission line towers have prescribed following conditions for the sagtension calculations for the conductor and the ground wire:

- Maximum temperature (75°C for ASCR and 53°C for ground wire) with design wind pressure (0% and 36%).
- Every day temperature (32°C) and design wind pressure (100%, 75% and 0%).
- Minimum temperature (0°C) with design wind pressure (0% and 36%).

IS 802: part 1:sec 1: 1995 states that conductor/ ground wire tension at every day temperature and without external load should not exceed 25 % (up to 220 kV) for conductors and 20% for ground wires oftheir ultimate tensile strength. Sag tensions are calculated by using the parabolic equations as discussed in the I.S. 5613: Part 2: Sec: 1: 1989 for both the conductor and ground wire. In this paper, the consideration of the sag of ground wire as 90% the sag of the conductor at 0°C and 100% wind condition. The sag tension values are mentioned in the Table 3.

3.1. Parabolic Equation

$$F_2^2 (F_2 - (K - \alpha.t.E)) = \frac{L^2.\delta^2.q_2^2.E}{24} \qquad (1)$$

Take, $$K = F_1 - \frac{L^2.\delta^2.q_0^2.E}{24 F_1^2} \qquad (2)$$

Table 3. Sag tension for conductor (ASCR)

Temperature variation°C	0		32			75
Wind variation %	0	0. 36	0	0.75	1.0	0
Tension (kg)	2282.5	3733.0	3246.24	3108.30	3416.27	2367.23

All tension values are giving F.O.S < 4.

So, we consider the minimum tension(tension for F.O.S = 4.) to find the maximum sagging in all condition. So, sagging = $\frac{wl^2}{8T_2} = \frac{0.973\times320^2}{8\times2282.5}$ = 5.46m

By increasing 4% of calculated sag we get= 5.46 × 4%= 5.70 m.

IV. Height of tower:

h1 = minimum permissible ground clearance = 7.1 m (cl=13.1, IS: 5613.2.1)

h2 = sag (maximum) = 5.46 × 1.04 (increase by 4%) = 5.7 m

h3 = minimum clearance between two conductor = 4.9× 2 m. = 9.8m (cl=7.3.1.1, IS: 5613.2.1)

h4 = vertical distance between earth and top conductor = 7.4 (cl=13.2, IS: 5613.2.1)

Total **H = (h1 + h2 + h3 + h4)**= 30 m

The dimension of the tower is shown in the Fig 1

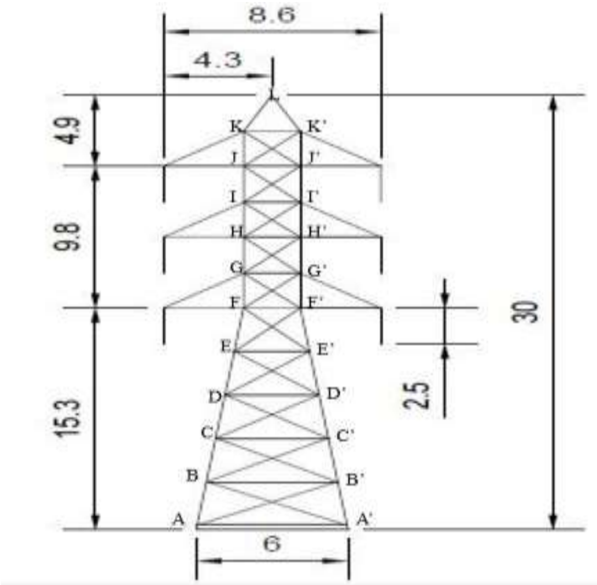

Figure 1: Dimensions of tower in meter

V. Wind loads on tower:

Wind loads on all the towers are calculated separately by developing excel programs by following Indian Standards. For finding the drag coefficients for the members of triangular tower, the solidity ratio is derived from Table 30 –IS-875 (part 3)-1987 in the similar fashion as prescribed in the IS- 826 (part-1/sec 1)-1995.

5.1. Design Wind Pressure

To calculate design wind pressure on conductor, ground wire, insulator and panels: $P_d=0.6\times V_d^2$ (3)
Where, P_d = design wind pressure in N/m^2
V_d = design wind speed in m/s
To calculate design wind pressure, $V_d = V_R\times K_1\times K_2$ (4)
V_R = 10min wind speed (or) reduced wind speed
$V_R = V_b/k_0$ (5)
V_b = basic wind speed
K_0 =1.375 [conversion factor]
K_1 = risk coefficient
K_2 = terrain roughness coefficient.

5.2. Wind Loads on Conductor/Ground Wire

To calculate wind loads on conductor and ground wire, $F_{wc}= P_d\times C_{dc}\times L\times D\times G_C$ (6)
Fwc = wind load on conductor
Pd = design wind pressure
Cdc = drag coefficient for ground wire=1.2 drag coefficient for conductor = 1.0
L = wind span
d = diameter of conductor/ground wire
Gc = gust response.

5.3. Wind Load on Insulator

To calculate wind load on insulator, $F_w=P_d\times C_{di}\times A_I\times G_I$ (7)
where, A_I = 50% area of insulator projected parallel to the longitudinal axis of string
G_I = gust response factor for insulator
C_{di} = drag coefficient, to be taken as 1.2150 mm

5.4.Wind Load on Panels

The lateral force due to wind acting at every panel joint is found as a product of intensity of wind and the exposed area of members of the tower consist of the projected area of the windward force plus fifty percent of plant of the leeward force.The sizes of the members taken are as

Assuming-
Main leg: ISA 200×200×25 single angle back to back section.
For diagonal bracing: ISA 100×100×8 single angle
Horizontal bracing: ISA 130×130×10 single angle
Cross arm bracing: ISA 90×90×12 single angle
To calculate wind load on panels, $\mathbf{F_w=P_d\times C_{dt}\times A_e\times G_T}$
C_{dt} = drag coefficient for panel considered against which the wind is blowing
A_e = effective area of the panel
G_T = gust response factor for towers

The wind load on panels for various conditions:

1. *Normal operating conditions.*
2. *Top most power conductor in broken wire condition.*
3. *Ground wire in broken condition*.

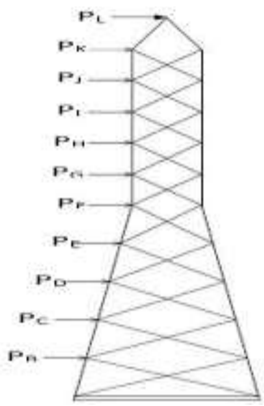

Figure 2: Wind Loads in Tower Panels

The Wind force acting at different panels are shown in Figure 2 and the values are tabulated in Table 4

Table 4. Lateral Force in Each Panel

Lateral force	Under the various condition		
	Condition 1 (kN)	Condition 2 (kN)	Condition 3 (kN)
P_L	11.17	11.17	5.79
P_K	3.965	3.965	3.965
P_J	19.19	9.98	19.19
P_I	3.89	3.89	3.89
P_H	18.37	18.37	18.37
P_G	3.72	3.72	3.72
P_F	20.32	20.32	20.32
P_E	4.47	4.47	4.47
P_D	4.83	4.83	4.83
P_C	5.21	5.21	5.21
P_B	5.57	5.57	5.57

VI. Stresses in the members of the tower under various conditions:

The transmission line tower as shown in the above figure is highly indeterminate. The stresses in the various members may be found by approximate method. The tower is reduced to a determinate plane frame by neglecting the horizontal and secondary members. The stresses are determined for the following conditions:

1. Normal operation conditions.
2. Due to lateral forces under the topmost power conductor broken condition.
3. Due to longitudinal forces under the topmost power conductor broken condition.
4. Due to lateral forces under the ground wire in broken condition.
5. Due to longitudinal forces under the ground wire in broken condition.

The axial forces in column and diagonal members are determined by the method of joints and by horizontal equilibrium and these values are shown in the table 5.

Table 5. Stresses in different members of the Tower

Member	Conditions			Member	Conditions		
	Normal(kN)	Top conductor broken (kN)	Earth wire broken (kN)		Normal (kN)	Top conductor broken (kN)	Earth wire broken (kN)
AB	189.81	214.45	245.67	FG'	26.33	45.27	34.32
AB'	11.83	23.10	10.29	F'G'	-117.98	-140.38	-183.80
A'B'	-189.81	-214.45	-245.67	F'G	-26.33	-45.27	-34.32
A'B	-11.83	-23.10	-10.29	GH	80.75	96.78	134.65
BC	178.63	203.37	236.20	GH'	24.14	43.51	32.31
BC'	10.36	22.04	8.52	G'H'	-80.75	-96.78	-134.65
B'C'	-178.63	-203.37	-236.20	G'H	-24.14	-43.51	-32.31
B'C	-10.36	-22.04	-8.52	HI	50.43	60.10	92.40
CD	167.63	192.53	227.56	HI'	16.88	35.82	24.87
CD'	9.18	21.45	6.88	H'I'	-50.43	-60.10	-92.40
C'D'	-167.63	-192.53	-227.56	H'I	-16.88	-35.82	-24.87
C'D	-9.18	-21.45	-6.88	IJ	27.41	30.70	57.45
DE	156.50	181.63	219.85	IJ'	14.34	33.71	22.52
DE'	8.40	21.65	5.35	I'J'	-27.41	-30.70	-57.45
D'E'	-156.50	-181.63	-219.85	I'J'	-14.34	-33.71	-22.52
D'E	-8.40	-21.65	-5.35	JK	11.61	11.65	29.73
EF	144.55	170.04	213.35	JK'	7.00	7.00	14.99
EF'	8.28	23.24	3.81	J'K'	-11.61	-11.65	-29.73
E'F'	-144.55	-170.04	-213.35	J'K	-7.00	-7.00	-14.99
E'F	-8.28	-23.24	-3.81	KL	7.12	7.16	20.57
FG	117.98	140.38	183.80	K'L	-7.12	-7.16	-20.57

Note: The positive value shown indicates tension and the negative value shown indicates compression for the various members of the tower and greater value under different conditions are highlighted.

VII. Deflection of the members of the tower:

To determine the vertical deflection at joint B, first removed the externally applied load system and then applied a unit load only in a vertical direction at joint B as shown in Figure 3. After that the method of joint resolution is used as earlier to determine the magnitude and the unknown member forces. Similarly, the horizontal deflection at B can be determined by applying a unit load at B in the horizontal direction. The horizontal and vertical deflection is calculated under all three conditions as mentioned above for the entire panel joints (i.e. at joints B', C, C', D, D', E, E', F, F', G, G', H, H', I, I', J, J', K, K', L) and tabulated in as shown in Table 6.

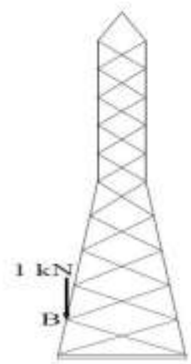

Figure 3**:** Unit load apply at panel jointB.

Table 6: Deflection at all panel joints

Panel Joint	Maximum Deflection		Panel Joint	Maximum Deflection	
	Horizontal	Vertical		Horizontal	Vertical
B	0.682	-0.346	G'	9.652	1.276
B'	0.682	0.346	H	13.236	-1.456
C	1.621	-0.632	H'	13.236	1.456
C'	1.621	0.632	I	16.874	-1.574
D	2.882	-0.851	I'	16.874	1.574
D'	2.882	0.851	J	20.846	-1.651
E	4.533	-0.992	J'	20.846	1.651
E'	4.533	0.992	K	24.661	-1.689
F	6.685	-1.041	K'	24.661	1.689
F'	6.685	1.041	L	28.508	0
G	9.652	-1.276			

VIII. Design of the members of the tower:

In X-type bracings the member which is under tension, due to lateral load acting in one direction, undergoes compressive force, when the direction of the lateral load is changed and vice-versa (as in the member force calculations).

Hence, such members are to be designed to resist both tensile and compressive forces.

The Members used in the Towers are standard Indian Angles of:

1. Main leg: ISA 200×200×25 single angle back to back section.
2. Diagonal and Cross arm bracing: ISA 100×100×8 single angle.
3. Horizontal bracing: ISA 130×130×10 single angle.

The gusset plate is of 20 mm thickness and connection of gusset plate with angles is shown in Figure 4.

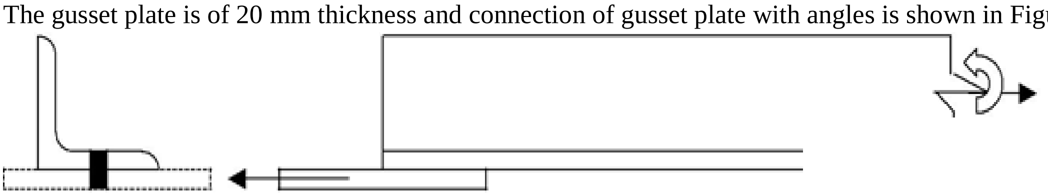

Figure 2: Angles Eccentrically Loaded through Gussets

7.1 Design of tension member by limit state method (IS 800:2007)

Tension members are linear members in which axial forces act to cause elongation (stretch). Such members can sustain loads up to the ultimate load, at which stage they may fail by rupture at a critical section. The design strength of the tension member shall be minimum of T_{dg}, T_{dn} and T_{db}.

7.1.1 Strength Due To Yielding of Gross Section

The design strength in the member under axial tension is given by:

$T_{dg} = f_y\ A_g/\ \gamma_{mo}$, where

γ_{mo} = the partial safety factor for failure in tension by yielding. The value of γ_{mo} according to IS800:2007 is 1.10.

7.1.2 Design Strength Due To Rupture of Critical Section

Tension rupture of the plate at the net cross-section is given by:

$T_{dn} = 0.9 f_u A_n\ /\ \gamma_{ml}$, where

γ_{ml} = the partial safety factor against ultimate tension failure by rupture ($\gamma_{ml} = 1.25$)

Single Angle Tension Member:

The strength of an angle connected by one leg as governed by tearing at the net section is given by,

$T_{dn} = 0.9 f_u\ A_{nc}\ /\gamma_{ml} + \beta A_{go}\ f_y\ /\gamma_{m0}$, where

β accounts for the end fastener restraint effect and is given by

$\beta = 1.4 - 0.076\ (w/t)\ (f_u/f_y)\ (b_s/L_c) \leq (f_u\ \gamma_{m0}\ /\ f_y\ \gamma_{m1}) \geq 0.7$

The b_s (shear lag distance) is calculatedas shown in the Figure 5

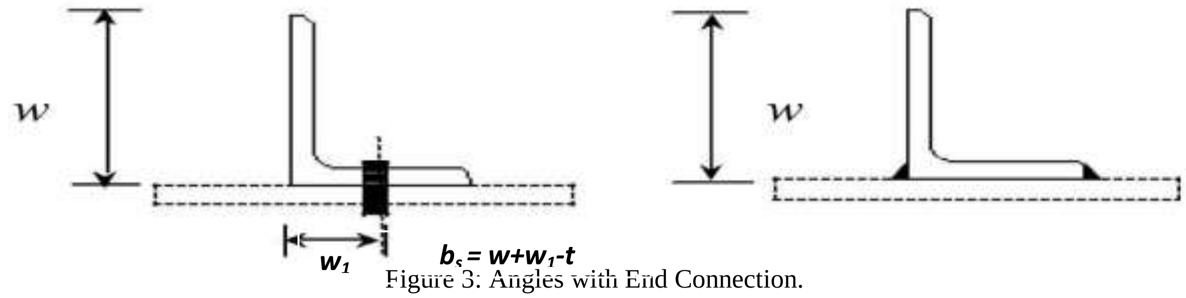

Figure 3: Angles with End Connection.

7.1.3Design Strength Due To Block Shear

A tension member may fail along end connection due to block shear as shown in Figure 6. The corresponding design strength can be evaluated using the following equations. The block shear strength T_{db}, at an end connection is taken as the smaller of

$T_{db1} = \quad (\ A_{vg} f_y\ /(\surd 3\gamma_{m0}) + 0.9 f_u A_{tn}\ /\gamma_{m1}\ )$ or, $T_{db2} = \quad (0.9 f_u A_{vn}\ /(\surd 3\gamma_{m1}) + f_y A_{tg}\ /\gamma_{m0}\ )$

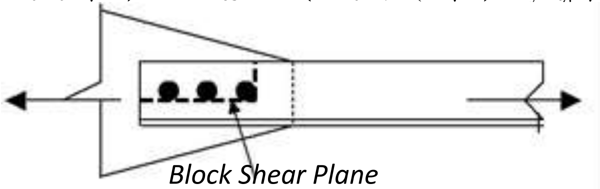

Figure 4: Block Shear Failure.

7.2Design charts tension member

The charts have been prepared based on IS 800:2007 for Tension members. The procedure is shown below.

Assumed material properties:

f_y = 250 MPa, f_u = 400 MPa, f_{ub} = 410 MPa

7.2.1Design chart for Main leg ISA 200×200×25

Tensile Strength of Single Angle ISA 200 X 200 X 25 (As per IS 800:2007) with single row bolted connection as shown in Figure 7 (9nos. 20mm dia.).

The no of bolts considered for the design of tension members for end connections is based on minimum no. of bolts required for the full strength of the angle for Block shear.

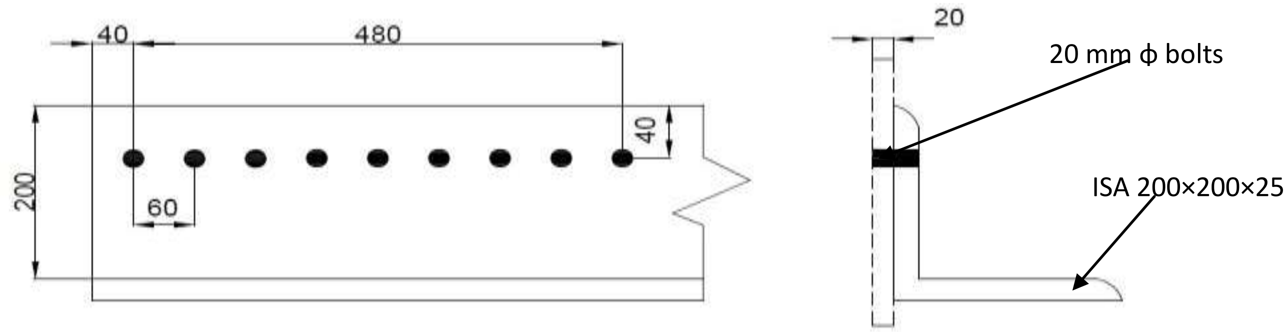

Figure 7: Design Details of leg member (All Dimensions are in mm).

Design strength due to yielding of gross section

$T_{dg} = f_y A_g/ \gamma_{mo}$

A_g= 9380 mm^2 (from steel table), $\gamma_{m0} = 1.1$

T_{dg}= 250×9380/1.1 = 2131.818 kN

7.2.1.1 Design Strength due to rupture of critical section

e = 40 mm, p = 60 mm

$T_{dn}= 0.9f_u A_{nc} /\gamma_{ml} + \beta A_{go} f_y /\gamma_{m0}$

$A_{nc} = (200 – 22 – 25/2) \times 25 = 4137.5\ mm^2$

$A_{go} = (200 – 25/2) \times 25 = 4687.5\ mm^2$

$\beta = 1.4 – 0.076 (w/t) (f_u/f_y) (b_s/L_c) \leq (f_u \gamma_{m0} / f_y \gamma_{m1}) \geq 0.7$

$L_c = 60\times 8 = 480$, $b_s = 200+140-25 = 315$

$\beta = 1.4 – 0.076(200/25) (250/410) ((315)/480) = 1.156$ (> 0.7)

$1.156 < (f_u \gamma_{m0} / f_y \gamma_{m1}) = (410\times 1.1)/ (250\times 1.25) = 1.44$

Therefore, T_{dn}= (0.9×4137.5×410)/1.25 + (1.15×4687.5×250)/1.1 = 2446.5 kN

7.2.1.2 Design strength due to block shear

The block shear strength T_{db}, at an end connection is taken as the smaller of

$T_{db1} = (A_{vg}f_y /(\sqrt{3}\gamma_{m0}) + 0.9f_uA_{tn} /\gamma_{m1})$ or, $T_{db2} = (0.9f_uA_{vn} /(\sqrt{3}\gamma_{m1}) + f_yA_{tg} /\gamma_{m0})$

$A_{vg} = (40+60\times 8) \times 25 = 13000\ mm^2$

$A_{vn} = (40+60\times 8-22\times 7.5) \times 25 = 8325\ mm^2$

$A_{tg} = (60\times 25) = 1500\ mm^2$

$A_{tn} = (60-0.5\times 22) \times 25 = 1225\ mm^2$

$T_{db1} = ((13000\times 250)/ (\sqrt{3}\times 1.1)) + ((0.9\times 1225\times 410)/1.25) = 2067$ kN

Or, $T_{db2} = ((0.9\times 8325\times 410)/ (\sqrt{3}\times 1.25)) + ((1500\times 250)/1.1) = 1759$ kN

Therefore, the block shear strength is T_{db} = 1759 kN

Now, Strength of the single angle Tension member should be least of the above three values (i.e. 2131.81 kN, 2446.5 kN and 1759 kN) which is equal to 1759 kN.

As per our calculation we get that the maximum tension force is in the leg member of the ground panel which is 245.67 kN i.e. factored load = 245.67×1.5 = 368.5 kN is lesser than the above three values. Therefore our design is safe for maximum tension.

7.2.2 Design chart for Diagonal member ISA 100×100×8

Tensile Strength of Single Angle ISA 100 X 100 X 8 (As per IS 800:2007) with single row bolted connection as shown in Figure 8 (3nos 16mm dia.).

The no of bolts considered for the design of tension members for end connections is based on minimum no. of bolts required for the full strength of the angle for Block shear.

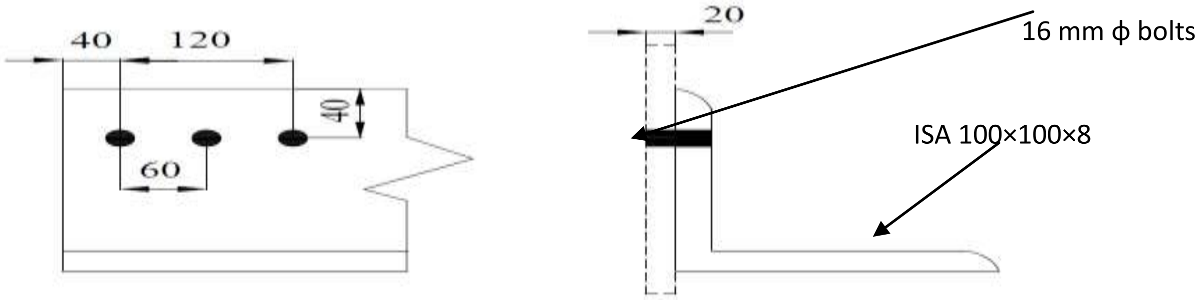

Figure 8: Design Details of diagonal member (All dimensions are in mm).

7.2.2.1 Design strength due to yielding of gross section
$T_{dg} = f_y A_g / \gamma_{mo}$
A_g= 1539 mm^2 (from steel table), γ_{m0} = 1.1
T_{dg}= 250×1539/1.1 = 349.7 kN

7.2.2.2 Design Strength due to rupture of critical section
e = 40 mm, p = 60 mm
$T_{dn}= 0.9f_u A_{nc} /\gamma_{ml} + \beta A_{go} f_y /\gamma_{m0}$
A_{nc} = (100 – 18 – 8/2) ×8 = 624 mm^2
A_{go} = (100 – 8/2) ×8 = 768 mm^2
$\beta = 1.4 – 0.076 (w/t) (f_u/f_y) (b_s/L_c) \leq (f_u \gamma_{m0} / f_y \gamma_{m1}) \geq 0.7$
L_c = 60×2 = 120, b_s = 100+44-8 = 136
β = 1.4 – 0.076(100/8) (250/410) ((136)/480) = 0.74 (> 0.7)
0.74 < ($f_u \gamma_{m0}$ / $f_y \gamma_{m1}$) = (410×1.1)/ (250×1.25) = 1.44
Therefore, T_{dn}= (0.9×624×410)/1.25 + (0.74×768×250)/1.1 = 313 kN

7.2.2.3 Design strength due to block shear
The block shear strength T_{db}, at an end connection is taken as the smaller of
$T_{db1} = (A_{vg}f_y /(\sqrt{3}\gamma_{m0}) + 0.9f_uA_{tn} /\gamma_{m1})$ or, $T_{db2} = (0.9f_uA_{vn} /(\sqrt{3}\gamma_{m1}) + f_yA_{tg} /\gamma_{m0})$
A_{vg} = (40+60×2) ×8 = 1280 mm^2
A_{vn} = (40+60×2-18×2.5) ×8 = 920 mm^2
A_{tg} = (40+18) ×8 = 464 mm^2
A_{tn} = (40+18-0.5×18) ×8 = 392 mm^2
T_{db1} = ((1280×250)/ (√3×1.1)) + ((0.9×392×410)/1.25) = 283.6 kN
Or, T_{db2} = ((0.9×920×410)/ (√3×1.25)) + ((464×250)/1.1) = 262 kN
Therefore, the block shear strength is T_{db} = 262 kN
Now, Strength of the single angle Tension member should be least of the above three values (i.e. 349.7 kN, 313kN and 262 kN) which is equal to 262 kN.

As per the calculation, the maximum tension force is obtained in the diagonal member of the sixth panel which is45.27 kN i.e. factored load = 45.27×1.5 = 67.9 kNis lesser than the above three values. Therefore the design is safe for maximum tension.

7.3 Design of compression member by limit state method (IS 800:2007)
7.3.1.Design chart for Main leg ISA 200×200×25
Length = 3.02 m
K = 0.85
f_y = 250 MPa
A = 9380 mm^2
r_{min} = 60.5 mm
KL/ r_{min} = (0.85×3020)/60.5 = 42.43
From Table 10 of IS 800:2007, the member belongs to buckling class c.
Therefore, from Table 9(c) of IS 800:2007 the values of f_{cd} are found using KL/r = 42.43 and f_y = 250 MPa.

Here,
For, KL/r = 40 => f_{cd} = 198 MPa
KL/r = 50 => f_{cd} = 183 MPa
Therefore, for KL/r = 42.43
f_{cd} = 194.35 N/mm^2
Thus, strength of the angle as column
$P_d = A \times f_{cd} = 9380 \times 194.35 = 1823050$ N = 1823 kN
Working load = 1823/1.5 = 1215 kN > 245.67 kN
Therefore, the section is safe for maximum compressive force.

IX. CONCLUSIONS

The transmission line tower is a statically indeterminate structure and the manual analysis of such a structure is very complex. A rigorous analysis considering three dimensional space actions is quite difficult. The development and application of computer analysis opened up a new and practically unlimited possibilities for the exact solution of these statically indeterminate structures with precise statically analysis of their three dimensional performance. However the adopted method of analysis presented in this paper considering linear behavior with two dimensional approaches gives satisfactory results which should be further verified with advanced software like STAAD Pro, Ansys etc. As per the design concern all section we consider are found safe against worst condition. In summary, the study presented here would certainly useful for Design Engineers basically for the new learners for better understanding the behaviors and the method of analysis and design of the transmission tower as per Indian Standard Codes of practice in a very simple and easy manner.

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