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Optimized Design of Steel Transmission Line Tower by Limit State Methodology

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ABSTRACT: *For the transfer of high voltage power from the generating source to longer distances, "Steel Transmission Line Towers" are used to carry the transmission lines. The behaviour of the tower when subjected to wind was studied by analyzing the tower for wind load in combination with longitudinal loads due to Broken -Wire condition and Vertical Forces. Understanding the behaviour of tower and Optimum Design of members to withstand the forces are the main objectives of the present study. A 132KV Double Circuit, 4-legged, Free-Standing and Intermediate-Tangent 3-Dimensional tower was selected. After the analysis of the tower by using STAAD PRO v6i software, the tower was designed manually as well as Optimum designed using STAAD PRO v6i by fallowing "Limit state method" as per IS: 800-2007, and was compared with conventional "working stress method".*

Keywords: *Broken-Wire condition, Optimum Design, Steel Transmission Line Tower, STAAD Pro and Wind Force.*

I. INTRODUCTION

1.1 Increase in power stations and transmission line structures (structures used for transferring power from power generating stations to the load centers) was observed in recent past due to increase in population, machines and industrial growth which lead to demand for Extra High Voltage (EHV) in all developed and developing countries.

The EHV has given rise to the need for usage of relatively large transmission line structures such as Steel Transmission Line Towers. Transmission Line Towers constitute about 28 to 42 percent of the total cost of the Transmission Lines [1].

The tall three dimensional structures with relatively small cross-section having a large ratio between the height and the maximum width are known as *Towers*. Towers can also be termed as *Pylons*. The towers used for carrying transmission lines (wires or cables) are called T*ransmission Towers* or T*ransmission Line Towers*.

Towers are subjected to mechanical forces like dead load and broken wire loads as well as environmental forces such as wind, floods and earthquake. So, the towers should be designed properly i.e.; Optimum Design should be achieved by the designer and correct detailing should be provided to the site engineer.

Optimum design can be achieved through appropriate selection of height-width ratio; insulation strength (no: of insulator discs with adequate clearances); conductors & ground wires with required steel reinforcement; span, configuration & weight of tower; analysis & design method to provide strength & stability of the structure; and effective cost analysis are met. Necessary maintenance should be provided for proper functioning.

Working stress method does not consider the strength of the member beyond proportionality limit and after local yielding [2], resulting in heavier sections. Hence optimum design cannot be achieved with working stress method. Limit state methodology is a rational method [3] which overcomes the drawbacks of working stress method and provides optimum sections. Limit state method is fallowed in this thesis. Manual design does not lead to optimum sections because often higher sections than required may be selected. So with the help of computers, programs developed based on fuzzy logic of optimization which resulted saving in tower weight of 6% [4] and reliability based optimization in which the weight of the optimal tower accounting for reliability as a constraint for both 110 and 220 kV tangent towers is only 3-4% heavier than the tower designed using the conventional method [5] or other techniques must be used to obtain an optimum section which the structural engineers may not be able to fallow practically. Fortunately the latest design soft wares are capable of arriving at optimum section fallowing the desired code of designer such as STAAD.

Tower structure with least weight is directly associated in reduction of the foundation cost [6]. Configuration of the structure of the tower plays a vital role in its performance especially while considering eccentric loading conditions. The bottom tier members have major role in performance of the tower in taking axial forces and the members supporting the cables are likely to have localized role. The vertical members are more prominent in taking the loads of the tower than the horizontal and diagonal members. The members supporting the cables at higher elevation are likely to have larger influence on the behavior of the tower structure. The effect of twisting moment of the intact structure is not significant. [7]. The three legged tower members are subjected more force and deflection when compared with four legged tower, but requires less area as well as resulted in less weight[8]. For towers up to 50m height, Y bracing resulted in less joint displacement and is economical [9].

The triangular tower is found to have little higher amount of axial forces in the leg members in comparison with the square tower [10].

II. TOWER CONFIGURATION

2.1 The tower studied in this thesis is a Four-Legged square shaped 132 KV Double Circuit Steel Transmission Line Tower. The tower has 6 conductors and 1 ground wire. Six cross-arms are provided to carry the conductors and clamp of the tower carries the ground wire. The tower is a Free-standing/Self-supported single cantilever structure fixed at the base meaning that no guys are used to support the tower. The tower is assumed as an intermediate, Tangent tower (Angle of deviation with respect to adjacent towers = 0-2 deg.). X-X bracing system is adapted. The insulators are suspension type insulators. All the members are to be provided with steel angle sections. Figure.1 depicts the 132 KV tower with above configurations.

Figure 1: 132 KV GNT-TDK DC TOWER

2.2 The other basic details required for calculation loads are as follows:

- Site : Guntur (wind zone : 5)
- Type of land : Exposed terrain and less obstruction
- Conductor : 30/3.00mm Al + 7/3.00mm Steel ACSR conductor
  Overall diameter = 21mm; Maximum Working Tension = 3640Kg;
  Unit Weight = 9.77 N/m [11]
- Ground Wire : 7/4.06mm Steel Strand
  Overall Diameter = 12.2mm; Maximum Working Tension = 2960 Kg;
  Unit Weight = 7.52 N/m [12]
- Temperature : 32^oc (Maximum Temperature = 60^oc and Minimum Temperature = 20^oc)
- Area : $36m^2$ (6m X 6m) $\Rightarrow$ Base width = 6m.
- Height : 21m (Refer section 2.3)
- Span : 330m

2.3 Determination of Height and Top hammer width:

2.3.1 The factors governing the height of the tower are:

1. Minimum permissible ground clearance (h1)

2. Maximum sag (h2)
3. Vertical spacing between conductors (h3)
4. Vertical clearance between ground wire and top conductor (h4)

Thus the total height of the tower is given by

H = h1 + h2 + h3 + h4 (m)

- For 132KV DC tower clearance above the lowest point of the conductor as 6.10m = h1. [13]
- The conductors are subjected to sagging during hot climate. The tower height should be determined by considering the maximum sag (sag at peak hot climate). By considering the temperature and the external forces acting on the conductor (horizontal force due to wind, vertical force due to weight of conductor and ice formation) the amount of sag is calculated by catenary method. For a maximum temperature of 60^{o}c, sag can be assumed as 6m = h2 [1].
- Based upon the value of sag the vertical spacing required between the conductors is calculated by Swedish Empirical formula Vertical Spacing between the top most and lowest conductors [1] is $6.5\sqrt{S} + 0.7E$.
  Where S = Sag in cm E = Line Voltage in KV. From the formula h3 = 5.9m.
- Considering the shielding angle (angle which the line joining the ground wire and the outer most conductor makes with the vertical) required for interruption of direct lightning strikes at the ground and the minimum mid span clearance between the ground wire and the top power conductor h4= 3m. H = 21m.

2.3.2 The top hammer width of the tower is one-third (1/3) times the base width based upon the condition that "The intersection of the tower legs should be above the CG (resultant) of the entire loads so that the resultant load is carried both braces and leg members. Hence top hammer width = (1/3)*6 = 2m.

III. LOAD CALCULATION

3.1 Classification of Load and Load combinations

Before load calculation, the loads acting and load combinations can be classified as shown below:
The loads acting on the tower are:
a) Vertical Loads: These loads include weight of the tower, weight of Conductors and workman.
b) Transverse loads: These loads include wind force.
c) Longitudinal Loads: These loads include forces induced in conductors under broken wire conditions

The load combinations are:
a) Normal Condition (Vertical and Transverse Loads)
b) Ground Wire Broken Condition (Vertical ,Transverse and Longitudinal Load at clamp)
c) Top Conductor Broken Condition (Vertical, Transverse and Longitudinal load at top cross-arm)

3.2 Calculation *of Wind Pressure* [14, 15]

- Reliability level = 1 (as 132 KV < 400 KV)
- Basic wind Speed V_b = 50 m/s (as Wind Zone = 5)
- V_R (Meteorological Reference wind speed)=V_b/{ k_o (code)}=50/ 1.375
=36.3636 m/sec
- Design Wind Speed $V_d = V_R \times K_1 x\, k_2$
  From table 2 of IS: 802 K1=1.00(Risk Coefficient), K2 = 1.08(for Open Terrain)
  V_d = 36.36 x 1 x 1.08 = 39.2688 m/sec = 40 m/sec
- Design Wind Pressure $P_d = 0.6\, V_d^2 = 0.6 \times (40)^2 = 960$ N/mm^2 = assume 1000 N/mm^2

Therefore Wind Pressure on Tower = 1000 N/mm^2 = 100 Kg/mm^2

Transverse Load at G.W Level **[1, 14]**

a) Under Normal Condition:
1) Due to Wind on G.W = 2/3 x d x Span x Wind Pressure
= 2/3 x 0.0122 x 330 x 1000
= 269 Kg
2) Due to Deviation = 2T Sin θ/2 = 2 x 2960 x Sin 2/2 = 107Kg
3) Due to wind on clamp = 1 Kg
Total Load = 269 + 107 + 1 = 377Kg = 3.770 KN

b) Broken Wire Condition: (269/2) + (107/2) +1 = 189Kg = 1.89 KN

Transverse Load at Conductor Level **[1, 14]**

a) Normal Condition
1) Due to Wind on conductor = 2/3 x d x span x wind pressure

= 2/3 x 0.021 x 330 x 100 = 462 kg

2) Due to Deviation = 2T Sin θ/2 = 2 x 3640 x Sin 2/2 = 127.5 Kg

3) Due to wind on insulator string

For 132 KV, length of suspension insulator string = 168 cm [16, 17]

No: of Porcelain discs (255 x 146 mm) = 9 discs

Diameter of each disc = 25.5cm

Projected area of insulator string = 168 x 25.5 = 0.4284 m^2

Effective area for wind load assumed = 50 % = 0.2142 m^2

Computed Wind load on insulator string for wind pressure of 100 Kg/mm^2

= 100 x 0.2142 = 21.42 Kg = 25 Kg (assume.)

4) Equivalent load at conductor level due to wind on tower = (1/3)* 4620 N

= 160 Kg

Total Transverse load = 462+127.5+25+160 = 775 Kg = 7.750 KN

b) Broken Wire Condition = (462/2) + (127.5/2) + (25) + 160 = 480 Kg = 4.80 KN

***Longitudinal Loads* [1, 14]**

1) Normal Condition = 0

2) Broken Wire Condition

Ground wire = 2960 Kg

Conductor = 60% X 3640 = 2184 Kg (50% by code+ 10% by practical cons.)

***Vertical Loads* [1, 14]**

a) Conductor – Normal condition

Weight of one span = 0.977 * 350 = 322 Kg (0.977 kg/m = unit wt of conductor ACSR)

Weight due to weight span = 25% of wt of one span = (25/100)*322 = 80.5 Kg

Weight of two men with tools = 140 Kg

Weight of the insulator string = 91 Kg

Total = 322 + 80.5 + 140 + 91 = 634 Kg

b) Conductor Broken Wire Condition

Weight of half span = (322 * 50 %) = 161 Kg

Weight of weight span = (50% * 80.5) = 40.25 Kg

Weight of two men with tools = 140 Kg

Weight of insulator string = 91 Kg

Total = 432.25 Kg = 433 Kg

***Torsional Load* [1, 14]**

2T X 2 = 2184 X (1+2.50)

4T = 2184 X 3.50

T = ((2184 * 3.50)/4) = 1911 Kg

a)Ground Wire

Weight of on span = 0.752 * 330 = 248 Kg

Weight of weight span = 62 Kg = ((25/100) * 248)

Weight of two men with tools = 140 Kg

Total = 450 Kg

b) Under Broken Wire Condition Vertical Load = (50% * 248) + (50%* 62) + 140

= 124 + 31 +140 = 295 Kg

3.3 Calculated Load and Load combinations

Finally the load combinations for 132Kv Dc tower are

Load Combination 1: Normal Condition : At Cross Arms Fx =7.76 KN, Fy = -6.34 KN
At Clamp Fx = 3.78KN, Fy = -4.5KN

Load Combination 2: Ground Wire Broken : At Cross Arms Fx =7.76 KN, Fy = -6.34 KN
At Clamp Fx = 1.9KN, Fy = -6.34KN, Fz = -29.6 KN

Load Combination 3(Top – Left Conductor Broken): At Top-Left Cross Arm
Fx = 4.8KN, Fy=-4.34KN, Fz = -21.84 KN
: At other Cross Arms
FX=7.76KN, Fy = -6.34KN
: At Clamp
Fx = 3.78KN, Fy = -4.5KN

IV. ANALYSIS

4.1 The analysis has been carried out using STAAD PRO v6i software which is based on stiffness method. The members are modeled as one dimensional member. As tower is a space- truss, the internal force in the members is axial force only i.e.; either Tension or Compression.

4.2 The geometry and different load combinations assigned are shown in the figures below.

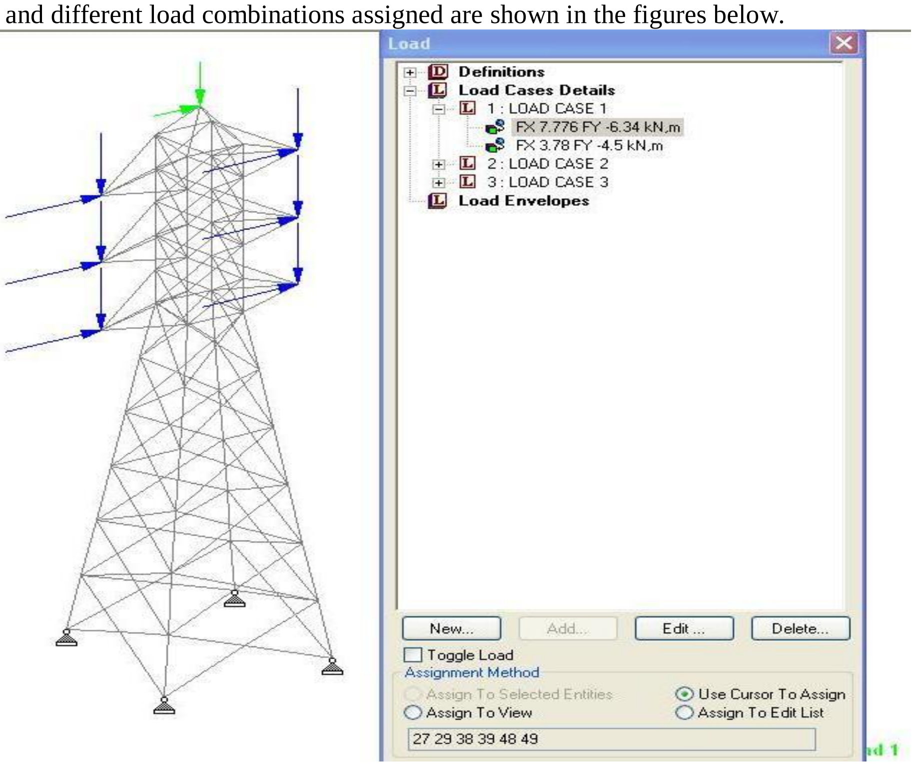

Figure 2: Load Combination 1

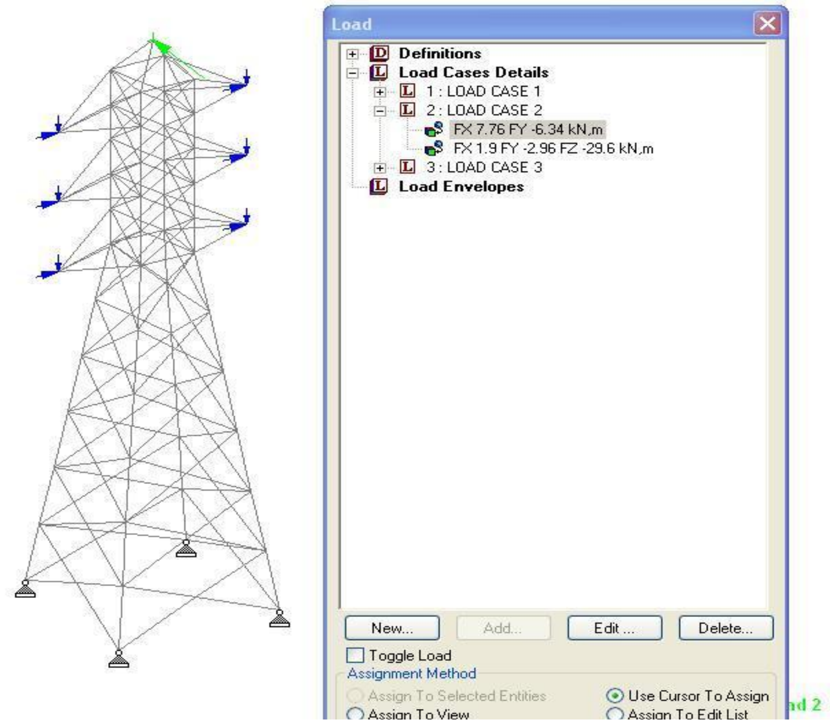

Figure3: Load Combination2

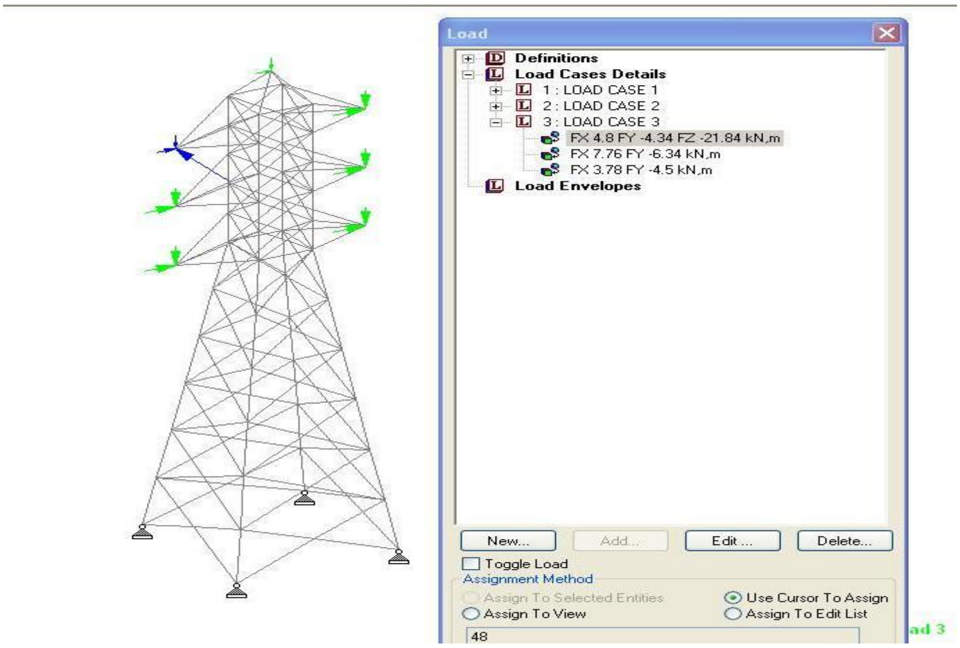

Figure 4: Load Combination 3

4.3 The member forces are given in Table 1

Table 1: Member Forces

COLUMN MEMEBRS	MAX. COMPRESSIVE FORCE (KN)	MAX. TENSILE FORCE(KN)
INCLINED COLUMN MEMBERS		
1		67.01
7		64.53
11		61.28
15	4.10	
19	17.56	50.33
23	30.33	39.94
4	**146.27**	
8	142.79	
12	140.21	
16	136.67	
20	131.51	
24	123.25	
172		**124.83**
168		121.92
164		119.34
160		115.81
156		110.65
152		102.39
173	88.65	
169	86.18	
165	82.93	
161	78.47	
157	71.98	
153	61.58	
VERICAL COLUMN MEMBERS		
25	39.06	19.32
26	32.20	21.03
27	34.17	
28	29.46	
29	12.30	
42	**103.33**	
43	76.13	
44	55.37	

45	40.08	
46	13.75	
128		**71.69**
129		68.44
130		43.46
131		25.56
132		20.94
145	51.80	
146		29.51
117	9.88	6.65
148	16.42	14.93
149		19.49
HORIZONTAL MEMBERS (HAMMER MEMBERS)		
53		8.63
30	16.84	
87	2.13	
133	**19.40**	
86	3.84	
41		**12.53**
100	1.66	
144	17.49	
CROSS ARM MEMBERS		
113	15.79	
48		6.51
47	9.88	
114		13.29
115		10.31
49	9.88	
116		7.02
50		6.86
117	9.88	
51	**19.57**	
118		**23.13**
52	14.22	
122	7.43	
123		13.29
56		4.40
57		6.51
124	4.09	
125		9.46
59	1.07	
58	1.50	
61		7.26
60	2.16	
127		6.51
126	1.50	
CLAMP MEMBERS		
120		**15.22**
121		13.26
54	15.29	
55	**17.25**	
BRACING MEMBERS		
170		2.67
171	2.67	
174		2.67
175	2.67	
176		3.10
177	3.10	
166		3.10
167	3.10	
162		3.70
195	3.70	

163	3.70	
194		3.70
158		4.60
159	4.60	
193	4.60	
192		4.60
154		6.08
155	6.08	
191	6.08	
190		6.08
150		8.88
189	8.88	
151	8.88	
188		8.88
134		**30.46**
231		15
135	15	
230		**30.46**
136	8.10	
137	16.99	
233		16.99
232		8.10
138	8.21	
139		6.58
235	6.58	
234		8.21
140		13.18
141	17.80	
237		17.80
236	13.18	
142	12.56	
143	14.92	
239		14.92
238		12.56
178		6.56
3	6.56	
179	6.56	
2		6.56
5		7.62
6	7.62	
181	7.62	
180		7.62
9		9.10
10	9.10	
13		11.30
14	11.30	
183	11.30	
182		11.30
17		14.93
18	14.93	
185	14.93	
184		14.93
21		21.82
22	21.82	
187	21.82	
186		21.82
220		28.74
221	10.48	
32		10.48
31		28.74
222	**28.74**	
223	28.23	
34		28.23
224		16.43
225		15.56
36	15.56	

35	16.43	
226		22.51
227	12.82	
38		12.82
37		22.51
106	2.91	
203		2.91
69		2.91
213	2.91	
108	3.38	
63		3.37
197		3.37
215	3.38	
104	4.03	
65		4.03
199		4.03
211	4.03	
112	5.01	
62	4.36	
202		5.01
219	5.01	
111	6.61	
73		6.61
207		6.61
218	6.61	
103	9.66	
72		9.66
206		9.66
210	9.66	
88	7.46	
89	17.49	
251	17.49	
250	7.46	
252		12.11
253		7.25
91		7.25
90		12.21
254		7.32
255	11.59	
94	11.59	
93		7.32
256		8.75
257	9.75	
96	9.75	
95		8.75
259		9.59
98	8.30	5.28
258	8.30	
99		9.59
66	3.75	
105		3.75
212		3.75
200	3.75	
62	4.36	
107		4.36
214		4.36
196	4.36	
64	5.20	
101		5.20
208		5.20
198	5.20	
67	6.46	
109		6.46
216		6.46
201	6.46	
110		8.54

70	8.54	
204	8.54	
217		8.54
102		12.48
71	12.48	
205	12.48	
209		12.48
75	17.17	
74		19.37
240	19.37	
241		17.17
243	16.61	
242		19.69
76	19.69	
77		16.61
79		18.83
245	17.90	
80		17.90
224		20.91
247		15.40
246	20.91	
81		20.91
82		15.40
84		14.08
249	10.11	
85		10.11
248	14.08	

4.4 The displacements or the deformations of tower under the action of loads convey the behavior or response of tower. The deflections for the load combinations are presented below.

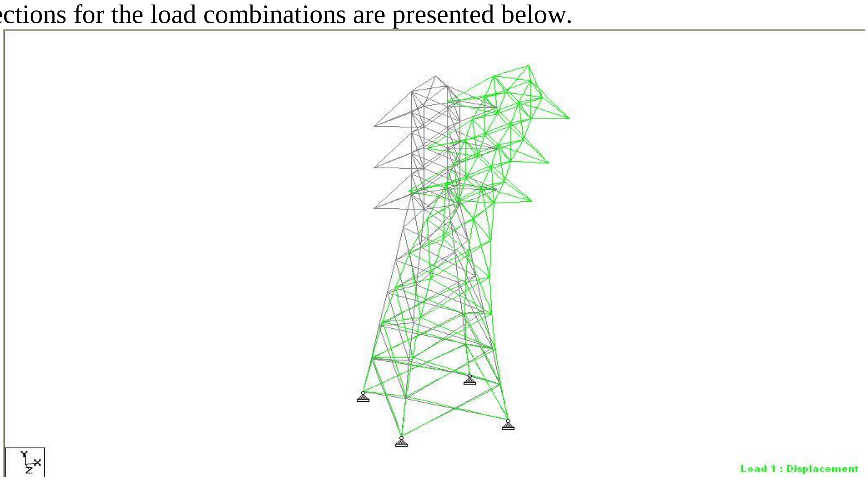

Figure 5: Load combination 1

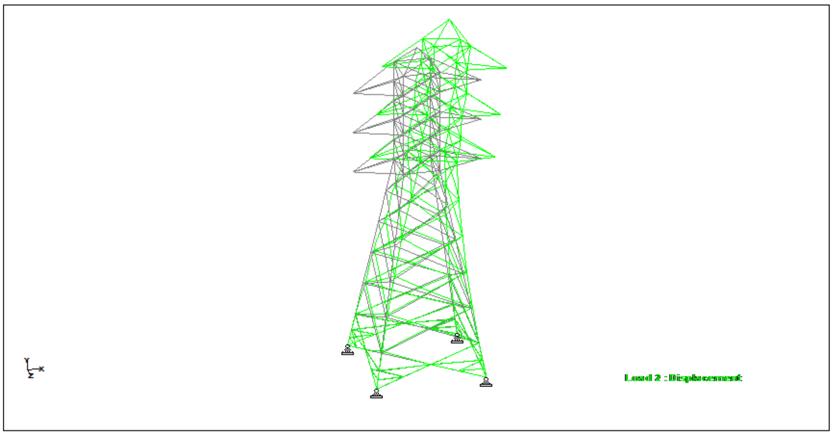

Figure 6: Load combination 2

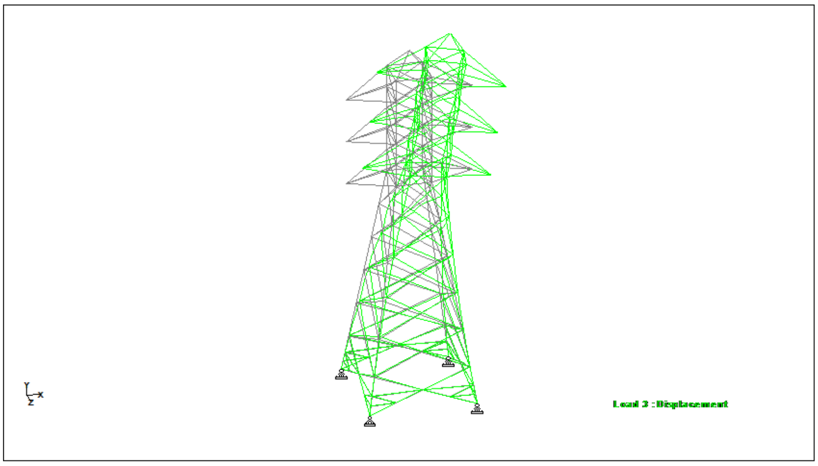

Figure 7: Load Combination 3

4.5 Maximum X- displacement = 63.1698 mm at clamp node under load combination 1. Maximum Y displacement = 15.9 mm at top cross arm nodes under load combination 1. Maximum Z- displacement =9.3mm at clamp node under load combination 2.

4.6 The maximum deflection for small angle tower is H/100 [1]. So for 21m tower, maximum deflection is 210mm. As 63.1698mm < 210mm, the deflection was well in control.

V. LIMIT STATE DESIGN OF MEMBERS [1, 2, 3 and 18]

5.1. MANUAL DESIGN

Single-Angle sections are provided for all members. The column members are assumed as continuous single angle members. So k = 1.0. [14]

5.1.1 INCLINED COLUMN MEMBERS *(loaded concentrically)*

a) For Compression

Maximum compressive force = 146.27KN, Factored Force = 1.5 X 146.27 = 219.405 KN
Maximum unsupported length of the member L = 2.54m
Initially assume KL/r as 100 and buckling class curve for single angle is 'c'.
Hence $f_{cd} = 107$ N/mm^2
Area required = $[219.405 * (10^3)] / [107] = 2050.51 = 2050$ mm^2
Provide 130 X 130 X 12 @23.5 kg/m; Area = 2990 mm$^{2;}$ $r_v = 25.6$ mm
$KL / r_v = (1 \times 2.54\times10^3) / 25.6 = 99.2 < 120$ ***Ok***
K=1 as the leg members are continuous members and the member is a single angle.
$\lambda = \sqrt{(f_y/f_{cc})}$, $f_{cc} = \pi^2 E / (KL/r)^2$
$= \sqrt{(99.2^2 \times 250) / (3.14^2 \times 2 \times 10^5)} = 1.116$
$Ø = 0.5 [1 + \alpha(\lambda-0.2)+ \lambda^2] = 0.5 [1+ (0.49 \times (1.116-0.2))+ 1.116^2] = 1.347$
$F_{cd} = (f_y / \gamma_{mo}) / [ø + \sqrt{(ø^2- \lambda^2)}] \leq f_y / \gamma_{mo}$
$= (250 / 1.1) / [1.347 + \sqrt{(1.347^2- 1.116^2)}] = 108.15$ N/mm^2
Where $f_y / \gamma_{mo} = 250/1.1 = 227.27$ N/mm^2
Therefore $F_{cd} = 108.15$ N/mm$^2 < 227.27$ N/mm^2
$P_d = A_e f_{cd} = 2990 \times 108.15 =$ *323.3 KN > 219.405KN* ***Hence Ok***

b) For Tension

Force acting = 124.83 KN; Factored Force = 1.5*124.83 = 187.245 KN
$KL / r_v = (1 \times 2.54\times10^3) / 25.6 = 99.2 < 400$ ***Ok***
Assume two bolts at each end of diameter 20mm. Bolt hole diameter = 20+22 = 22mm
Design strength due to rupture
$T_{dn} = 0.9 f_u A_n / \gamma_{m0} = [0.9*\{2990-(2*22*12)\}*410]/1.25 = 726.0$ KN
Design strength due to yielding
$T_{dg} = (2990*250)/1.1 = 679.54$ KN
Therefore *679.54 KN > 187.245 KN* ***Hence Ok***

5.1.2 VERTCIAL COLUMN MEMBERS *(loaded concentrically)*

a) For Compression

Maximum compressive force = 103.33 KN

Factored Force = 1.5 X 103.33 = 154.95 KN
Unsupported Length = 1.5 m
Initially assume KL/r as 100 and buckling class curve for single angle is 'c'.
Hence f_{cd} = 107 N/mm^2
Area required = [154.95* (10^3)] / [107] = 1448.13 mm^2
Provide 80 X 80X12 @14 kg/m; Area = 1780 mm^2, r_v = 15.4 mm
KL / r_v = (1 x 1.5x10^3) / 15.4 = *97.4 < 120* ***Ok***
$\lambda = \sqrt{(f_y/f_{cc})}$, $f_{cc} = \pi^2 E / (KL/r)^2$
$= \sqrt{(97.4^2 \times 250) / (3.14^2 \times 2 \times 10^5)} = 1.09$
$\text{Ø} = 0.5 [1 + \alpha(\lambda-0.2)+ \lambda^2] = 0.5 [1+ (0.49 \times (1.09-0.2))+ 1.09^2] = 1.3121$
$F_{cd} = (f_y / \gamma_{mo}) / [\phi + \sqrt{(\phi^2 - \lambda^2)}] = \chi f_y / \gamma_{mo} \leq f_y / \gamma_{mo}$
$= (250 / 1.1) / [1.3121 + \sqrt{(1.3121^2 - 1.09^2)}] = 111.27$ N/mm$^2 \leq 229.22$ N/mm^2
$P_d = A_e f_{cd}$ = 1780 X 111.27 = *198.06 KN >154.95KN* ***Hence Ok***

b) For Tension

Force = 71.69 KN
Factored Force = 1.5*71.69 = 107.535 KN
Kl/r_v = 97.4 < 400 ***Ok***
Design strength due to rupture
T_{dn} = [0.9*{1780-(2*22*12)}*410] / 1.25 = 369.5 KN
Design strength due to yielding
T_{dg} = (1780*250)/1.1 = 404.54 KN
Therefore *369.5KN > 71.69 KN* ***Hence Ok***

5.1.3 BRACING MEMBERS *(loaded through one leg)*

a) For Compression

Maximum compressive force = 30.46 KN
Factored Force = 1.5 * 30.46 = 45.69KN
Unsupported Length = 3.28 m
Initially assume KL/r as 100 and buckling class curve for single angle is 'c'.
Hence f_{cd} = 107 N/mm^2.
Area required = [45.69* (10^3)] / [107] = 427 mm^2
Provide 70X70X5 @ 5.3kg/m; Area = 677mm^2, r_v = 13.6 mm
L / r_v = (3.28x103) / 13.6 = 241.17 >120 and <200 $\therefore$ K =1
KL/r_v = 241.17<250
Then $(b_1+b_2)/2t = (70+70)/(2*5) = 14$
And $\varepsilon = 1.0$ and $\varepsilon (3.14^2 E/250)^{0.5} = 88.86$
And$\lambda_v = (L/r_v) / \varepsilon (\pi^2 E/250)^{0.5}$= 241.17/88.86 = 2.714
And $\lambda_\phi = [(b_1+ b_2)/2t] / \varepsilon (\pi^2 E/250)^{0.5}$= 14/88.86 = 0.15
And $\lambda_e = (k_1+k_2 \lambda_v^2+k_3 \lambda_\phi^2)^{0.5} = [0.20+ (0.35*2.714^2)+(20*0.15^2)]^{0.5} = 1.79$
Where k_1 =0.20, k_2 = 0.35 and k_3 = 20 assuming that number of bolts in end connection is greater than or equal to 2.
$\text{Ø} = 0.5 [1 + \alpha(\lambda_e-0.2)+ \lambda_e^2] = 0.5 [1+ (0.49 \times (1.79-0.2))+ 1.79^2] = 1.99$
And $\chi = 1 / [\phi + \sqrt{(\phi^2-\lambda_e^2)}] = 1/\sqrt{(1.99^2 - 1.79^2)} = 1.15$
And $f_{cd} = \chi f_y / \gamma_{mo}$ = (1.15*250) / 1.1 = 261.36 N/mm^2
$P_d = A_e \times f_{cd}$ = 677*261.36 = 176.94*KN >45.69KN* ***Hence Ok***
Design of end connection
For shear plane out of bolt threads
$V_{nsb} = V_{nsb} = (f_{ub} /\sqrt{3}) * (n_n A_{nb} + n_s A_{sb}) = (400/ \sqrt{3}) * (1* 314) = 72515$ N
V_{dsb} = 72515/1.25 = 58012N
$e/3d_0$, $(p/3d_0 - 0.25)$, f_{ub}/f_u,1.0 = 40/(3*21.5), (60/(3*21.5)-0.25), 400/410,1.0
(e= 40, p= 60, d_0 = 22, f_{ub} = 400,f_u = 410)
K_h = least of [$e/3d_0$, $(p/3d_0 - 0.25)$, f_{ub}/f_u,1.0 = 40/(3*21.5), (60/(3*21.5)-0.25), 400/410,1.0]
K_h = 0.620
$V_{npb} = 2.5 k_b d t f_u$ = 2.5 * 0.620 * 20 * 5 * 410 = 63550N
V_{dpb} = 63550/1.25 = 50840N
No.of Bolts required = (45.69*1000) / least of (50840 N, 58012N)
= (45690/ 50840) = 0.89 ≈ 3
Minimum number of bolts to be provided as per IS: 800-2007 is 2.

So, provide 3 black bolts of 20 mm diameter of 60 mm spacing at end connection to gusset of 14mm thick. Edge distance = 40mm.

b) For Tension

Force = 28.74 KN
Factored Force = 1.5*28.74 = 43.11 KN
$Kl/r_v = 241.17 < 400$ ***Ok***
$V_{nsb} = (400/\sqrt{3})*(314) = 72515$ N
$V_{dsb} = 72515/1.25 = 58041$N
$e/3d_0$, $(p/3d_0 – 0.25)$, f_{ub}/f_u,1.0 = 40/(3*21.5), (60/(3*21.5)-0.25), 400/410,1.0
($e= 40$, $p= 60$, $d_0 = 22$, $f_{ub} = 400$, $f_u = 410$)
K_h = least of [$e/3d_0$, $(p/3d_0 – 0.25)$, f_{ub}/f_u,1.0 = 40/(3*21.5), (60/(3*21.5)-0.25), 400/410,1.0]
$K_h = 0.620$
$V_{npb} = 2.5\ k_b d t f_u = 2.5 * 0.620 * 20 * 5 * 410 = 63550$N
$V_{dpb} = 63550/1.25 = 50840$N
No.of Bolts required = (45.69*1000) / least of (58041 N, 50840N)
= (43110/ 50840) = 0.89 ≈ *3 bolts*
Minimum number of bolts to be provided as per IS: 800-2007 is 2.
So, provide 3 black bolts of 20 mm diameter of spacing 60mm at end connection to gusset of 14mm thick. Edge distance = 40mm.
Design strength due to rupture
L_c =2*60 = 120 mm
$B_s = W + W_l – L = 70+40-5 = 105$mm
$\beta = 1.4 – [\ 0.076\ (w/t)\ (f_y/\ f_u)\ (b_s/\ L_c)] \leq (\ f_y\ \gamma_{mo} /\ \ f_y\ \gamma_{m1})$
$= 1.4- [0.076* (70/5)* (250/410) * (105/120)]$
$= 0.83 \leq (\ f_y\ \gamma_{mo} /\ \ f_y\ \gamma_{m1})$
$= 0.83 \leq$ *1.44 and ≥ 0.7* ***ok***
T_{dn} = [{0.9*(70-2.5-22)*5*410} / 1.25] + [{0.83*(70-2.5)*5*250}/1.1]
= 67158+63664.77= 130.8 KN
Design strength due to yielding
T_{dg} = (677*250)/1.1 = 153.86 KN
Design Strength due to Block Shear
A_{vg} = (2*60+40)*5 = 800 mm^2
A_{vn} = (160-2.5*22)*5 = 525 mm^2
A_{tg}= (40*5) = 200mm^2
A_{tn} = (40-0.5*22)*5 = 145mm^2
$T_{db1} = [(A_{vg} * f_y)/(\gamma_{mo}*\sqrt{3})]+[(0.9*A_{tn}*f_u)/(\gamma_{m1})]$
$= [\ (800*250)/(1.1* \sqrt{3})]+[(0.9*145*410)/(1.1)] = 153.54$KN
$T_{db2} = [(0.9*A_{vn}*f_u)/(\gamma_{m1} * \sqrt{3}))]+\ [(A_{tg} * f_y)/(\gamma_{mo})]$
$= [(0.9*525*410)/(1.25*\sqrt{3})] + [\ (200*250)/1.1] = 134.93$ KN
T_{db} = 134.93 KN
Therefore 134.93 *KN > 43.11KN* ***Hence Ok***

5.1.4 CROSS ARM MEBERS *(loaded through one leg)*

a) For Compression

Maximum compressive force = 19.57 KN
Factored Force = 1.5 * 19.57 = 29.355KN
Unsupported Length = 3m
Initially assume KL/r as 100 and buckling class curve for single angle is ‘c’.
Hence $f_{cd} = 107\ N/mm^2$.
Area required = $[29.355* (10^3)] / [107] = 274.345\ mm^2$
Provide 65 X 65X 4 @ 4kg/m Area = 504mm², r_v = 12.6 mm
For cross- arm members K =1.
$KL / r_v = (1x\ 3x10^3) / 12.6 = 238.09 <250$ ***Hence Ok***
Then $(b_1+b_2)/2t = (65+65) /(2*4) = 16.25$
And $\varepsilon = 1.0$ and $\varepsilon\ (3.14^2E/250)^{0.5} = 88.86$
And $\lambda_v = (L/r_v) / \varepsilon\ (\pi^2E/250)^{0.5}= 238.09/88.86 = 2.67$
And $\lambda_ø = [(b_1+ b_2)/2t] / \varepsilon\ (\pi^2E/250)^{0.5}= 16.25/88.86 = 0.18$
And $\lambda_e= (k_1+k_2\ \lambda_v^2+k_3\ \lambda_ø^2)^{0.5} = [0.20+(0.35*2.67^2)+(20*0.18^2)]^{0.5} = 1.82$

$\varnothing = 0.5\ [1 + \alpha(\lambda\text{-}0.2) + \lambda^2] = 0.5\ [1 + (0.49 \times (1.82\text{-}0.2)) + 1.82^2] = 2.55$
And $\chi = 1 / [\varnothing + \sqrt{(\varnothing^2 - \lambda^2)}] = 1/\sqrt{(2.55^2 - 1.82^2)} = 0.55$
And $f_{cd} = \chi\ f_y / \gamma_{mo} = (0.55*250) / 1.1 = 125\ N/mm^2$
$P_d = A_e \times f_{cd} = 504*125 =$ *63 KN >25KN* ***Hence Ok***
Design of end connection
$V_{nsb} = V_{nsb} = (f_{ub}/\sqrt{3}) * (n_n A_{nb} + n_s A_{sb}) = 400/\sqrt{3} * (1*314) = 72515N$
$V_{dsb} = 72515/1.25 = 58012$ N
$e/3d_0$, $(p/3d_0 - 0.25)$, f_{ub}/f_u,1.0 = 40/(3*22), (60/(3*22)-0.25), 400/410,1.0
(e= 40, p= 60, d_0 = 22, f_{ub} = 400,f_u = 410)
K_h = least of [$e/3d_0$, $(p/3d_0 - 0.25)$, f_{ub}/f_u,1.0 = 40/(3*22), (60/(3*22)-0.25), 400/410,1.0]
K_h = 0.620
$V_{npb} = 2.5\ k_b d t f_u = 2.5 * 0.620 * 20 * 4 * 410 = 50840N$
$V_{dpb} = 50840/1.25 = 40672N$
No.of Bolts required = (25*1000) / least of (58012 N , 40672N)
= (25000/ 40672) = 0.614 ≈ 3 bolts
Minimum number of bolts to be provided as per IS: 800-2007 is 2.
So, provide 3 black bolts of 20 mm diameter of spacing 60mm at end connection to gusset of 14mm thick. Edge distance = 40mm.

b) For Tension
Force = 23.13 KN
Factored Force = 1.5*23.13 = 34.695 KN
$Kl/r_v = 238.09 < 400$ ***Ok***
$V_{nsb} = (400/\sqrt{3})*(314) = 72515$ N
$V_{dsb} = 72515/1.25 = 58012$ N
$e/3d_0$, $(p/3d_0 - 0.25)$, f_{ub}/f_u,1.0 = 40/(3*22), (60/(3*22)-0.25), 400/410,1.0
(e= 40, p= 64, d_0 = 22, f_{ub} = 400,f_u = 410)
K_h = least of [$e/3d_0$, $(p/3d_0 - 0.25)$, f_{ub}/f_u,1.0 = 40/(3*22), (64/(3*22)-0.25), 400/410,1.0]
K_h = 0.620
$V_{npb} = 2.5\ k_b d t f_u = 2.5 * 0.620 * 20 * 4 * 410 = 50840N$
$V_{dpb} = 50840/1.25 = 40672N$
No.of Bolts required = (34.695*1000) / least of (58012 N , 40672N)
= (34695/ 40672) = 0.85 ≈ 3bolts
Minimum number of bolts to be provided as per IS: 800-2007 is 2.
So, provide 3 black bolts of 20 mm diameter of spacing 60mm at end connection to gusset of 14mm thick. Edge distance = 40mm.
Design strength due to rupture
L_c =2*60=120mm
$B_s = W + W_l - L = 65+35-4 = 96mm$
$\beta = 1.4 - [\ 0.076\ (w/t)\ (f_y/f_u)\ (b_s/L_c) \leq (f_y\ \gamma_{mo} / f_y\ \gamma_{m1})$
= 1.4 –[0.076* (65/4)*(250/410)*(96/120)
= 0.79 ≤ *1.44 and ≥ 0.7* ***Ok***
$T_{dg} = (504*250)/1.1 = 114.54KN$
$T_{dn} = [\{0.9*(65\text{-}2\text{-}22)*4*410\} / 1.25] + [\ \{0.79*(65\text{-}2)*4*250\}/1.1]$
= 93.6 KN
Design strength due to Yielding
$T_{dg} = (504*250)/1.1 = 114.54KN$
Design Strength due to Block Shear
$A_{vg} = (2*60+40)*4 = 640\ mm^2$
$A_{vn} = (160\text{-}2.5*22)*4 = 420mm^2$
$A_{tg} = (35*4) = 140mm^2$
$A_{tn} = (35\text{-}0.5*22)*4 = 96mm^2$
$T_{db1} = [(A_{vg} * f_y)/(\gamma_{mo}*\sqrt{3})] + [(0.9*A_{tn}*f_u)/(\gamma_{m1})]$
$= [\ (640*250)/(1.1*\sqrt{3})] + [(0.9*96*410)/(1.1)] = 116.17KN$
$T_{db2} = [(0.9*A_{vn}*f_u)/(\gamma_{m1} * \sqrt{3}))] + [(A_{tg} * f_y)/(\gamma_{mo})]$
$= [(0.9*420*410)/(1.25*\sqrt{3})] + [\ (140*250)/1.1] = 103.\ 39$ KN
T_{db} = 103.39KN
Therefore 93.6KN > *30KN* ***Hence Ok***

5.1.5 HORIZONTAL MEMBERS *(loaded through one leg)*

a) For Compression

Force = 19.40 KN
Factored Force = 1.5*19.40 = 29.KN
Unsupported Length = 2m
Initially assume KL/r as 100 and buckling class curve for single angle is 'c'.
Hence f_{cd} = 107 N/mm^2.
Area required = (29*1000) / 107 = 271mm^2
Provide 50X50X4 @3kg/m Area = 388mm^2, r_v =9.7
l/r_v = 2000/9.7 = 206.18 >120 and <250
$\therefore$ Kl/ r_v = 46.2 + 0.612 (l/r_v) = 172.38 < 250 ***Ok***
Then $(b_1+b_2)/2t$ = (50+50)/(2*4) = 12.5
And ε = 1.0 and $\varepsilon\,(3.14^2E/250)^{0.5}$ = 88.86
$\lambda_v = (KL/r_v) / \varepsilon\,(\pi^2E/250)^{0.5}$=206.18/88.86 = 2.32
$\lambda_ø = [(b_1+ b_2)/2t] / \varepsilon\,(\pi^2E/250)^{0.5}$= 12.5/88.86 = 0.14
$\lambda_e= (k_1+k_2\,\lambda_v^2+k_3\,\lambda_ø^2)^{0.5}$ = $[0.20+(0.35*2.32^2)+(20*0.14^2)]^{0.5}$ = 1.57
$Ø = 0.5\,[1 + \alpha(\lambda-0.2)+ \lambda^2]$ = 0.5 [1+ (0.49 x (1.57-0.2))+ 1.57^2] = 2.06
And $\chi= 1 / [ø + \sqrt{(ø^2- \lambda^2)}]$ = $1/\sqrt{(2.06^2 – 1.6^2)}$ = 0.77
And $f_{cd} = \chi\, f_y / \gamma_{mo}$ = (0.77*250) / 1.1 = 175 N/mm^2
$P_d = A_e$ x f_{cd} = 388*175 = 17028 = *67.9 KN >29KN* ***Hence Ok***
Design of end connection
$V_{nsb} = V_{nsb} = (f_{ub} /\sqrt{3}) * (n_n\,A_{nb} + n_s\,A_{sb})$ = 400/ $\sqrt{3}$ * (1* 314) = 72515 N
V_{dsb} = 72515/1.25 = 58012 N
$e/3d_0$, $(p/3d_0 – 0.25)$, f_{ub}/f_u,1.0 = 40/(3*22), (60/(3*22)-0.25), 400/410,1.0
(e= 40, p= 60, d_0 = 22, f_{ub} = 400,f_u = 410)
K_h = least of [$e/3d_0$, $(p/3d_0 – 0.25)$, f_{ub}/f_u,1.0 = 40/(3*22), (60/(3*22)-0.25), 400/410,1.0]
K_h = 0.620
$V_{npb} = 2.5\,k_b dtf_u$ = 2.5 * 0.620 * 20 * 4 * 410 = 50840N
V_{dpb} = 50840/1.25 = 40672N
No.of Bolts required = (29*1000) / least of (58012 N , 40672N)
= (29000/ 40672) = 0.71≈ 3 bolts
Minimum number of bolts to be provided as per IS: 800-2007 is 2.
So, provide 3 black bolts of 20 mm diameter of spacing 50mm at end connection to gusset of 14mm thick. Edge distance = 40mm.

b) For Tension

Force = 12.53 KN
Factored Force = 1.5*12.53 = 18.79 KN = 20 KN
Kl/r_v = 172.38 < 400 ***Hence Ok***
$V_{nsb} = (400/\sqrt{3})*(314)$ = 72515 N
V_{dsb} = 72515/1.25 = 58012 N
$e/3d_0$, $(p/3d_0 – 0.25)$, f_{ub}/f_u,1.0 = 40/(3*22), (60/(3*22)-0.25), 400/410,1.0
(e= 40, p= 60, d_0 = 22, f_{ub} = 400,f_u = 410)
K_h = least of [$e/3d_0$, $(p/3d_0 – 0.25)$, f_{ub}/f_u, 1.0 = 40/ (3*22), (60/(3*22)-0.25), 400/410,1.0]
K_h = 0.620
$V_{npb} = 2.5\,k_b dtf_u$ = 2.5 * 0.620 * 20 * 4 * 410 = 50840N
V_{dpb} = 50840/1.25 = 40672N
No.of Bolts required = (20*1000) / least of (58012.6 N , 40672N)
= (20000/ 40672) = 0.4917 ≈ 3 bolts
Minimum number of bolts to be provided as per IS: 800-2007 is 2.
So, provide 3 black bolts of 20 mm diameter of spacing 50mm at end connection to gusset of 14mm thick. Edge distance = 40mm.
Design strength due to rupture
L_c =2*50 = 100mm
$B_s = W + W_l – L$ = 50+50-4 = 96mm
$\beta = 1.4 – [\, 0.076\, (w/t)\, (f_y/ f_u)\, (b_s/ L_c) \le (f_y\,\gamma_{mo} /\ f_y\,\gamma_{m1})$
= 0.84≤ *1.44and > 0.7* ***Ok***
T_{dn} = [{0.9*(50-2-22)*4*410} / 1.25] + [{0.91*(50-2)*4*250}/1.1]
= 69.3 KN

Design strength due to yeilding
T_{dg} = (388*250)/1.1 = 88.18KN
Design Strength due to Block Shear
A_{vg} = (2*50+40)*4 = 560 mm^2
A_{vn} = (140-2.5*22)*4 = 340mm^2
A_{tg}= (28*4) = 112mm^2
A_{tn} = (28-0.5*22)*4 = 68mm^2
$T_{db1} = [(A_{vg} * f_y)/(\gamma_{mo}*\sqrt{3})]+[(0.9*A_{tn}*f_u)/(\gamma_{m1})]$
= [(560*250)/(1.1* √3)]+[(0.9*68*410)/(1.1)] = 96.2KN
$T_{db2} = [(0.9*A_{vn}*f_u)/(\gamma_{m1} * \sqrt{3}))]+ [(A_{tg} * f_y)/(\gamma_{mo})]$
= [(0.9*340*410)/(1.25*√3)] + [(112*250)/1.1] = 83.3 KN
T_{db} = 83.3 KN
Therefore *83.3 KN >20 KN* ***Hence Ok***

5.1.6 Clamp Members *(loaded through one leg)*
a) For Compression
Force = 17.25 KN
Factored Force = 1.5*17.25 = 26 KN
Initially assume KL/r as 100 and buckling class curve for single angle is 'c'.
Hence f_{cd} = 107 N/mm^2.
Area required = (26*1000) / 107 = 242.9mm^2
Provide 45X45X4
Area = 347mm^2, r_v =8.7
Kl/r_v = (0.85 *2000)/8.7 = 195.84 <250 ***Ok***
Then $(b_1+b_2)/2t$ = (45+45)/(2*3) = 15
And ε = 1.0 and $\varepsilon (3.14^2E/250)^{0.5}$ = 88.86
And$\lambda_v = (L/r_v) / \varepsilon (\pi^2E/250)^{0.5}$=230/88.86 = 2.58
And $\lambda_\phi = [(b_1+ b_2)/2t] / \varepsilon (\pi^2E/250)^{0.5}$= 15/88.86 = 0.168 = 0.16
And $\lambda_e= (k_1+k_2 \lambda_v^2+k_3 \lambda_\phi^2)^{0.5} = [0.20+(0.35*2.58^2)+(20*0.16^2)]^{0.5}$ = 1.79
$Ø = 0.5 [1 + \alpha(\lambda-0.2)+ \lambda^2] = 0.5 [1+ (0.49 x (1.74-0.2))+ 1.74^2]$ = 2.39
And $\chi= 1 / [\phi + \sqrt{(\phi^2- \lambda^2)}] = 1/\sqrt{(2.39^2 – 1.79^2)}$ = 0.63
And $f_{cd} = \chi f_y / \gamma_{mo}$ = (0.63*250) / 1.1 = 143.18 N/mm^2
$P_d = A_e$ x f_{cd} = 347*143.18 = 50*KN >26KN* ***Hence Ok***
Design of end connection
$V_{nsb} = V_{nsb} = (f_{ub} /\sqrt{3}) * (n_n A_{nb} + n_s A_{sb})$ = (400/ √3) * (1* 314) = 72515 N
V_{dsb} = 72515/1.25 = 58012N
$e/3d_0$, $(p/3d_0 – 0.25)$, f_{ub}/f_u,1.0 = 40/(3*22), (60/(3*22)-0.25), 400/410,1.0
(e= 40, p= 60, d_0 = 22, f_{ub} = 400,f_u = 410)
K_h = least of [$e/3d_0$, $(p/3d_0 – 0.25)$, f_{ub}/f_u,1.0 = 40/(3*22), (60/(3*22)-0.25), 400/410,1.0]
K_h = 0.620
$V_{npb} = 2.5\ k_b dtf_u$ = 2.5 * 0.620 * 20 * 4 * 410 = 50840N
V_{dpb} = 50840/1.25 = 40672N
No.of Bolts required = (26*1000) / least of (58012 N , 40672N)
= (26000/ 40672) = 0.63 ≈ 3bolts
Minimum number of bolts to be provided as per IS: 800-2007 is 2.
So, provide 3 black bolts of 20 mm diameter of spacing 60mm at end connection to gusset of 14mm thick. Edge distance = 40mm.
b) For Tension
Maximum Force = 15.22 KN
Factored Force = 1.5*15.22=22.83KN
Kl/r_v = 195.84 < 400 ***Ok***
V_{nsb} = (400/√3)*(314) = 72515 N
V_{dsb} = 72515/1.25 = 58012 N
$e/3d_0$, $(p/3d_0 – 0.25)$, f_{ub}/f_u,1.0 = 40/(3*22), (60/(3*22)-0.25), 400/410,1.0
(e= 40, p= 60, d_0 = 21.5, f_{ub} = 400,f_u = 410)
K_h = least of [$e/3d_0$, $(p/3d_0 – 0.25)$, f_{ub}/f_u,1.0 = 40/(3*22), (60/(3*22)-0.25), 400/410,1.0]
K_h = 0.620
$V_{npb} = 2.5\ k_b dtf_u$ = 2.5 * 0.620 * 20 * 4 * 410 = 50840N
V_{dpb} = 50840/1.25 = 40672N

No.of Bolts required = (22.83*1000) / least of (58041 N, 40672N)
= (22830/ 40672) = 0.56 ≈ 3 bolts
Minimum number of bolts to be provided as per IS: 800-2007 is 2.
So, provide 3 black bolts of 20 mm diameter of spacing 60mm at end connection to gusset of 14mm thick. Edge distance = 40mm.

Design strength due to rupture
L_c =2*60 = 120mm
$B_s = W + W_l - L = 45+25-4 = 67$mm
$\beta = 1.4 - [\ 0.076\ (w/t)\ (f_y/\ f_u)\ (b_s/\ L_c) \leq (\ f_y\ \gamma_{mo}\ /\ \ f_y\ \gamma_{m1})$
 = 1.11 *<1.44 and ≥ 0.7* ***Ok***
T_{dn} = [{0.9*(45-2-22)*4*410} / 1.25] + [{1.11*(45-2)*4*250}/1.1]
 = 57.6 KN

Design strength due to yielding
T_{dg} = (347*250)/1.1 = 78.86KN

Design Strength due to Block Shear
A_{vg} = (2*60+40)*4 = 640 mm^2
A_{vn} = (160-2.5*22)*4 = 420mm^2
A_{tg}= (25*4) = 100mm^2
A_{tn} = (25-0.5*22)*4 = 56mm^2
$T_{db1} = [(A_{vg} * f_y)/(\gamma_{mo}*\sqrt{3})]+[(0.9*A_{tn}*f_u)/(\gamma_{m1})]$
 = [(640*250)/(1.1* √3)]+[(0.9*56*410)/(1.1)] = 102.6KN
$T_{db2} = [(0.9*A_{vn}*f_u)/(\gamma_{m1} * \sqrt{3}))]+ [(A_{tg} * f_y)/(\gamma_{mo})]$
 = [(0.9*420*410)/(1.25*√3)] + [(100*250)/1.1] = 94.2 KN
T_{db} = 94.2 KN
Therefore *57.6 KN >22.83 KN* ***Hence Ok***

5.2 OPTIMUM DESIGN BY STAAD-PRO V6i

5.2.1 The tower was designed by STAAD for obtaining optimum weight of tower so as to meet the objective of optimum design. IS: 800 (LSM) is selected for design. The type of section chosen was Angle section for all members. The parameters such as diameter of bolt, edge distance, pitch, yield strength, ultimate strength, and slenderness ratios were defined. Then the option "SELECT OPTIMIZED" was selected for arriving at optimum sections. Then the option "STEEL TAKE OFF" was selected for obtaining the details of sections assigned to the members and their weights. The details of the allotted steel angle sections for the members in the output of STAAD design were tabulated in the Table2.

5.2.2 Considering the sections in Table 2, the corresponding lengths and weights are tabulated in Table 3. The total weight of the tower is also calculated. **Total Weight of Tower = 2.5221 Metric Tonnes**. As per reference [1] the weight of 132 KV DC tower for a span of 320 m span the weight of tower is 2.8 Metric Tons and is stated that 20% reduction can be possible with computer aided design. So, for a span of 330 m, as in our study, 2.5221 Metric Tons is an Optimum Weight of the tower. Design by IS: 800(1984) was also done by using STAAD and the weight of tower was found to be **2.8341 Metric Tonnes** which indicates 12% saving by LSM. Hence the objective of Optimum Design has been met.

Table 2: Member- Sections Details (STAAD PRO)

1 ISA60X40X5	2 ISA40X40X3	3 ISA100X100X 6	4 ISA130X130X8	5 ISA40X40X3	6 ISA90X90X6
7 ISA40X40X6	8 ISA130X130 X8	9 ISA65X65X5	10 ISA150X150X1 0	11 ISA40X40X6	12 ISA130X130X 8
13 ISA35X35X3	14 ISA70X70X5	15 ISA45X45X5	16 ISA130X130X8	17 ISA45X30X3	18 ISA65X65X5
19 ISA45X45X5	20 ISA130X130 X8	21 ISA45X45X3	22 ISA60X60X5	23 ISA75X75X5	24 ISA130X130X 8
25 ISA50X50X4	26 ISA50X50X3	27 ISA50X50X5	28 ISA50X50X4	29 ISA45X45X3	30 ISA60X60X5
31 ISA40X40X4	32 ISA40X40X3	33 ISA45X30X3	34 ISA50X50X5	35 ISA45X30X3	36 ISA40X40X3
37 ISA45X45X3	38 ISA50X50X3	39 ISA50X50X3	40 ISA50X50X3	41 ISA30X30X5	42 ISA90X90X6
43 ISA80X80X6	44 ISA75X75X5	45 ISA65X65X5	46 ISA50X50X3	47 ISA80X80X6	48 ISA45X45X3
49 ISA80X80X6	50 ISA45X45X3	51 ISA80X80X6	52 ISA90X90X6	53 ISA30X30X3	54 ISA60X60X5
55 ISA60X60X5	56 ISA80X80X6	57 ISA90X90X6	58 ISA80X80X6	59 ISA90X90X6	60 ISA80X80X6
61 ISA45X45X3	62 ISA90X90X6	63 ISA40X40X3	64 ISA80X80X6	65 ISA40X40X3	66 ISA100X100X 6
67 ISA70X70X5	68 ISA35X35X3	69 ISA40X40X3	70 ISA65X65X5	71 ISA60X60X5	72 ISA30X30X3
73 ISA30X30X3	74 ISA45X45X3	75 ISA40X40X3	76 ISA50X50X3	77 ISA40X40X3	78 ISA30X30X3
79 ISA40X40X3	80 ISA45X45X3	81 ISA50X50X3	84 ISA30X30X3	85 ISA40X40X3	86 ISA60X60X5
87 ISA60X60X5	88 ISA40X40X3	89 ISA50X50X3	90 ISA40X40X3	91 ISA40X40X3	92 ISA30X30X3
93 ISA40X40X3	94 ISA45X45X3	95 ISA40X40X3	96 ISA45X45X3	97 ISA30X30X3	98 ISA45X45X3
99 ISA45X30X3	100 ISA60X60X5	101 ISA80X80X6	102 ISA60X60X5	103 ISA60X60X5	104 ISA80X80X6
105 ISA100X100X6	106 ISA100X100 X6	107 ISA90X90X6	108 ISA90X90X6	109 ISA70X70X5	110 ISA65X65X5
111 ISA65X65X5	112 ISA70X70X5	113 ISA80X80X6	114 ISA90X90X6	115 ISA80X80X6	116 ISA90X90X6
117 ISA80X80X6	118 ISA90X90X6	120 ISA60X60X5	121 ISA60X60X5	122 ISA80X80X6	123 ISA45X45X3
124 ISA80X80X6	125 ISA45X45X3	126 ISA80X80X6	128 ISA40X40X5	129 ISA35X35X6	130 ISA40X25X5
131 ISA45X45X3	132 ISA45X45X3	133 ISA60X60X5	134 ISA30X30X5	135 ISA25X25X4	136 ISA40X40X3
137 ISA50X50X3	138 ISA40X40X3	139 ISA40X40X3	140 ISA40X40X3	141 ISA40X40X3	142 ISA40X40X3
143 ISA45X45X3	144 ISA35X35X3	145 ISA65X65X5	146 ISA50X50X4	147 ISA45X45X4	148 ISA45X45X3
149 ISA45X45X3	150 ISA60X60X5	151 ISA60X60X5	152 ISA60X40X5	153 ISA100X75X6	154 ISA65X65X5
155 ISA50X50X3	156 ISA45X45X6	157 ISA90X90X6	158 ISA70X70X5	159 ISA60X60X5	160 ISA65X45X5
161 ISA100X100X6	162 ISA80X80X6	163 ISA70X70X5	164 ISA70X45X5	165 ISA100X100X6	166 ISA75X75X5
167 ISA75X75X5	168 ISA70X45X5	169 ISA100X100X 6	170 ISA100X100X6	171 ISA100X100X 6	172 ISA70X45X5
173 ISA100X100X6	174 ISA90X90X6	175 ISA90X90X6	176 ISA90X90X6	177 ISA90X90X6	178 ISA45X45X3
179 ISA90X90X6	180 ISA35X35X3	181 ISA75X75X5	183 ISA60X60X5	184 ISA45X30X3	185 ISA50X50X4
186 ISA45X45X3	187 ISA50X50X4	188 ISA45X45X3	189 ISA45X45X3	190 ISA50X50X3	191 ISA65X65X5
192 ISA60X60X5	193 ISA70X70X5	194 ISA70X70X5	195 ISA80X80X6	196 ISA75X75X5	197 ISA35X35X3
198 ISA70X70X5	199 ISA35X35X3	200 ISA90X90X6	201 ISA60X60X5	202 ISA30X30X3	203 ISA45X45X3
204 ISA50X50X3	205 ISA45X45X3	206 ISA30X30X3	207 ISA25X25X3	208 ISA70X70X5	209 ISA45X45X3
210 ISA45X45X3	211 ISA70X70X5	212 ISA90X90X6	213 ISA90X90X6	214 ISA75X75X5	215 ISA75X75X5
216 ISA60X60X5	217 ISA50X50X3	218 ISA50X50X3	219 ISA60X60X5	220 ISA40X40X4	221 ISA40X40X3
222 ISA45X30X3	223 ISA50X50X5	224 ISA45X30X3	225 ISA40X40X3	226 ISA45X45X3	227 ISA50X50X3
228 ISA50X50X3	229 ISA50X50X3	230 ISA30X30X5	31 ISA25X25X4	232 ISA40X40X3	233 ISA50X50X3
234 ISA40X40X3	235 ISA40X40X3	236 ISA40X40X3	237 ISA40X40X3	238 ISA40X40X3	239 ISA45X45X3
240 ISA45X45X3	241 ISA40X40X3	242 ISA50X50X3	243 ISA40X40X3	244 ISA40X40X3	245 ISA45X45X3
246 ISA50X50X3	247 ISA40X40X3	248 ISA30X30X3	249 ISA40X40X3	250 ISA40X40X3	251 ISA50X50X3
252 ISA40X40X3	253 ISA40X40X3	254 ISA40X40X3	255 ISA45X45X3	256 ISA40X40X3	257 ISA45X45X3
258 ISA45X45X3	259 ISA45X30X3	260 ISA 45X30X3			

Table 3: Section – Weight Details (STAAD PRO)

PROFILE	LENGTH(METER)	WEIGHT(KN)
ST ISA60X40X5	5.09	0.186
ST ISA40X40X3	59.51	1.070
ST ISA100X100X6	29.88	2.679
ST ISA130X130X8	15.26	2.371
ST ISA90X90X6	57.97	4.663
ST ISA40X40X6	5.09	0.175
ST ISA65X65X5	21.08	1.012
ST ISA150X150X10	5.01	1.118
ST ISA35X35X3	14.40	0.224
ST ISA70X70X5	26.14	1.359
ST ISA45X45X5	5.09	0.167
ST ISA45X30X3	11.42	0.191
ST ISA45X45X3	60.86	1.234
ST ISA60X60X5	42.16	1.862
ST ISA75X75X5	19.72	1.102
ST ISA50X50X4	7.71	0.230
ST ISA50X50X3	29.21	0.662
ST ISA50X50X5	4.00	0.147
ST ISA40X40X4	2.50	0.059
ST ISA30X30X5	4.50	0.096
ST ISA80X80X6	47.31	3.376
ST ISA30X30X3	20.14	0.268
ST ISA40X40X5	1.50	0.044
ST ISA35X35X6	1.50	0.044
ST ISA40X25X5	1.50	0.035
ST ISA25X25X4	2.50	0.035
ST ISA45X45X4	1.50	0.040
ST ISA100X75X6	2.54	0.198
ST ISA45X45X6	2.54	0.099
ST ISA65X45X5	2.54	0.103
ST ISA70X45X5	7.63	0.324
ST ISA40X25X3	2.02	0.029
ST ISA25X25X3	1.74	0.019
TOTAL		**25.221**

Total Weight of Tower (LSD) = 2.5221 Metric Tons.

VI. CONCLUSION

The present study envisages the static analysis of the Four-Legged Square shaped 132 KV Double Circuit Steel Transmission Line Tower for all possible load combinations including wind. Maximum X-displacement = 63.1698 mm at clamp node under load combination 1. Maximum Y-displacement = 15.9 mm at top cross arm nodes under load combination 1. Maximum Z- displacement =9.3mm at clamp node under load combination 2.

The maximum displacements i.e.; X- displacement = 63.1698 mm was caused due to Dead and Wind load (Load Combination1). Hence, the Ground Wire Broken condition (Load Combination 2) and Conductor Broken condition (Load Combination 3) had less affect on the chosen tower of 21m height, horizontal length of cross arm of 3m and 2 m hammer width. It can be stated that with increase in height, length of cross arm members and hammer width, the twisting action due to conductor broken condition and bending action of Ground Wire broken condition may affect the structure but bending action due to wind load plays vital role. Hence Wind load is major load for towers.

As per the knowledge of the author, the available literature/research works as per Indian standards on towers till date have been focused on working stress method which does not yield optimum section as strength beyond proportionality limit and local yielding was neglected. There is an urgent need to arrive at an optimum design procedure of towers, as these structures are frequently constructed. Limit State methodology is a rational method which provides optimum design i.e.; optimum sections.

Hence the tower was designed by Limit State Methodology as per IS: 800-2007 rather than the conventional working stress methodology, both manually and by STAAD PRO V6i, for obtaining optimized design. Design by IS: 800-1984(WSM) was also carried out in STAAD. The weight of the tower by LSM was 2.5221Metric Tonnes and by WSM was 2.8341 Metric Tonnes resulting in 12% saving by LSM. Thus, the objectives of understanding the behaviour of tower under wind load in combination with other loads and obtaining optimized design by Limit State methodology, in this study, were realized.

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