https://doi.org/10.46335/IJIES.2022.7.5.7 **e-ISSN: 2456-3463**
Vol. 7, No. 5, 2022, PP. 30 -38

International Journal of Innovations in Engineering and Science, www.ijies.net

Design and Optimization of Mechanical Gantry Structure for Over Head Electrical Line (220KVD/C)

Ashish Mahure[1], Nitin Sawarkar[2], Swapnil Choudhary[3], Bharat Chede[4], Yogesh Menghare[5]

[1,2,3,4,5]*Department of Mechanical Engineering, Wainganga College of Engineering and Management, Dongargaon, Wardha Road, Nagpur*

asheeshmahure@gmail.com

***Received on**: 15 April, 2022* ***Revised on**: 21 May, 2022,* ***Published on**: 23 May, 2022*

Abstract -*A Gantry Tower Structure, often known as a power tower Structure, is a tall structure that supports an overhead power line It is usually made of steel lattice. Any type of failure in the transmission line system that causes a disruption in the energy supply will consequently result in economic losses. This paper is on the failure Gantry Tower Structure, Failure Structure implementation by using CAD, Analytical. This 220 kvD/C Gantry type tower & Beam was failed after installation in Power Grid Substation at Bikaner (Rajasthan) dt on 24-OCT-2020. The failure can be costly. This is one of the most serious issues confronting the global electrical utility industry. While previous research has focused on the behavior and failure of a single tower, the research presented in this research paper is the first to consider the re-design and progression of failure of a transmission structure segment. To accomplish this, a unique CAD/numerical model was developed in this research. The formulation and validation of this CAD/numerical model are described in several portions of the thesis, which will be explored in detail in this article.*

Keywords: *Failure of Gantry Tower Structure**, CAD-**Model & Analytical comparison*

I- INTRODUCTION

Gantry structures are mainly used for guiding the power conductor from last tower near substation to the electrical equipment's in a substation. This structure consists of a number of columns and Girder beams, which depend on number of circuits of the line.A **gantry** is an over head bridge-like structure supporting equipment such as a crane, signals, or cameras. A Structural design of Gantry Structure (also known as a mechanical pylon structure) is a tall structure (usually a steel lattice structure) used to support an overhead power line. In electrical grids, they are used to carry high voltage transmission lines that transport bulk electric power from generating stations, transmission and distribution lines that transport power from substations to electric customers. A Structural design of Gantry Structure plays a very important role in power distribution network and is often subject to massive load. Design of lattice gantry structure often based a linear response to various loading.A Structural design of gantry has to carry the heavy transmission conductors at a sufficient safe height from the ground. In addition to that, all gantries have to sustain all kinds of natural calamities. So Gantry structure design is an important engineering job where civil, mechanical, and electrical engineering concepts are equally applicable.

B-AnalyticalAnalysisofBeam&GantryTowerStructure

1.0 DESIGNCRITERIA

1.1 SCOPE

This document covers design Calculations of 220KV Gantry. Beam Tower Structure Type (T2), Foundation Boltand Base Plate

S.NO	STRUCTUREDESCRIPTION
1	220KVTOWERT2

1.2 UNITS OF MEASUREMENTS

Units of measurements used in analysis shall be of SI Units

1.3 CODES AN DREFERENCES

The following Codes and standards have been referred:

S.No	CODE	DESCRIPTIONS
1	IS800:1984	Code of Practice for General Construction in Steel
2	IS802:1995(Part1/Sec1)	Code of Practice Use of Structural Steel in OHT Line Towers, Part1 Materials, Loadsand Permissible Stresses (Section 1- Materia ls and Loads)
3	IS802:1995(Part1/Sec2)	Code of Practice Use of Structural Steel in OHT Line Towers, Part1 Materials, Loads and Permissible Stresses (Section 2- Permissible Stresses)
4	IS802:1978(PartII)	Code of Practice Use of Structural Steelin OHT Line Towers, Part Fabrication, Galvanizing, Inspection and Packing
5	IS808:2004	Dimesnions for Hot Rolled Steel Beam, Column, Channel and Angle Sections
6	IS875(PartItoV)	Code of Practice for Design Loads for Buildings & Structures
7	IS1893:1984	Criterial for Earthquake Resistant Design of Structures

1.4PLANTSITEINFORMATION

Location of site :BIKANER(RAJSTHAN)

1.5WINDPARAMETERS

Basic Wind Speed	Vb	:47m/sec		(AsperIS875,Part-3:1987,Appendix-Acl-5.2)
Risk Coefficient	k1	:1.07		(AsperIS875,Part-3:1987,Table-1)
Terrain, Height Factor	k2	:1.03	(20mheight)	(AsperIS875,Part-3:1987,Table-2)
Topography Factor Design	k3	:1.18		(AsperIS875,Part-3:1987,Ann.C)
Wind Speed Design Wind Pressure	Vz	:61.13m/sec	(Vb*k1*k2*k3)	
	Pz	:2.29kN/Sqm	$(0.6*Vz^2/980.6)$	

1.6 FACTOR OFSAFETY:

A) For Structures

:Normal Conditions :2.00

SCF :1.50

https://doi.org/10.46335/IJIES.2022.7.5.7 e-ISSN: 2456-3463
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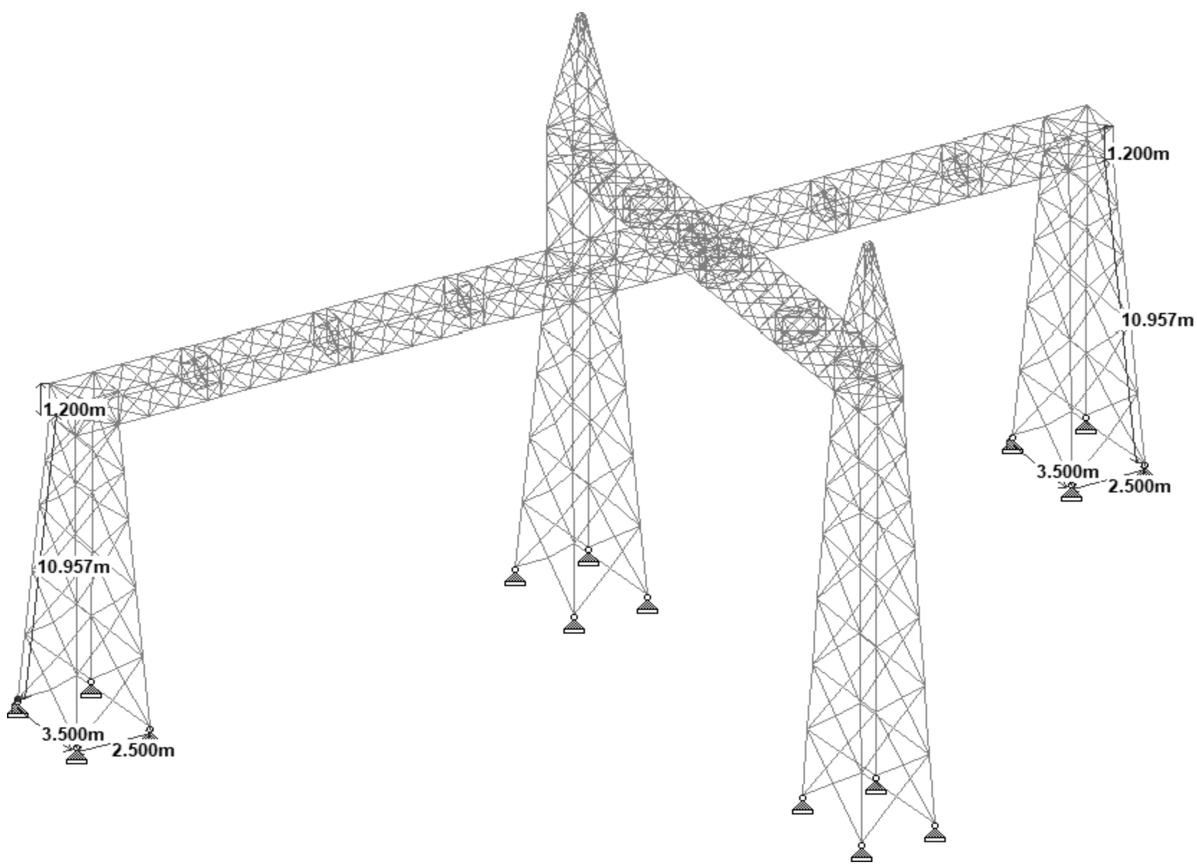
DESCRIPTION OF STRUCTURES:

2.0) STAAD MODEL SKETCH:



2.1) TOWER TYPE-T1:

Height of Beam from PL	=	11.5	m
Base Dimesion at Plinth Level	=	**2.5X3.5**	m
Base Dimesion at Girder Level	=	**1.5X1.5**	m

2.2) GIRDERDETAILS:

GIRDER-BEAM-B2

Clear Span of Girder	=	**15.50**	m
Width of Girder	=	**1.50**	m

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3.0)WIND LOAD CALCULATIONON TOWER: T1

3.1)WIND LADO N Transverse FACE (T-WIND):

Design Wind Pressure	=	2.29 kN/m^2		
Length of Tower at Top of Girder (B/B)	=	1.50 m	x	1.50
Length of Tower at Plinth Level (B/B) Transverseface	=	3.50 m	x	2.50 m
Height of Tower from Girder Topto Peak	=	4.400 m		
Height of Girder-1	=	1.20 m		
Height of Tower from PL to Girder Bottom	=	10.90 m		
Slope of Tower below Girder	=	0.092 Rad		
	=	5.257 Deg.		
		0.0917		

Panel No.	Member	Width of Panel at Top(m)	Width of Panel at Bottom(m)	Panel Height(m)	Length of Member(m)	No. of Member	Member Size			Exposed Area(m^2)	Total Exposed Area(m^2)	Total Boundary Area(m^2)	CG Height (m)	Solidity Ratio	Drag Factor	Total Wind (kN)	No. of Nodes	Wind Transferedon Each Node (kN)
6	MainLeg	1.50	1.50	1.200	1.200	2	110	x110x	10	0.264	0.784	1.80	0.600	0.436	2.430	4.361	8	1.185
	HORIZONTALBRACING	1.50	1.50	0.000	1.500	2	90	x90x	12	0.270								
	InclinedBracing	1.50	1.50	1.200	1.921	2	65	x65x	6	0.250								
5	MainLeg	1.50	1.82	1.737	3.000	2	110	x110x	10	0.660	0.972	2.88	0.841	0.338	2.300	5.117		
	InclinedBracing	1.50	1.82	1.737	2.397	2	65	x65x	6	0.312								
4	MainLeg	1.82	2.13	1.737	1.737	2	110	x110x	10	0.382	0.724	3.43	0.845	0.211	2.800	4.639	8	1.430
	InclinedBracing	1.82	2.13	1.737	2.626	2	65	x65x	6	0.341								
3	MainLeg	2.13	2.59	2.482	2.482	2	110	x110x	10	0.546	0.990	5.86	1.201	0.170	3.000	6.804		
	InclinedBracing	2.13	2.59	2.482	3.418	2	65	x65x	6	0.444								
2	MainLeg	2.59	3.04	2.500	2.500	2	110	x110x	10	0.550	1.039	7.04	1.216	0.148	3.200	7.610	8	1.945
	InclinedBracing	2.59	3.04	2.500	3.758	2	65	x65x	6	0.489								
1	MainLeg	3.04	3.50	2.500	2.500	2	110	x110x	10	0.550	1.084	8.18	1.221	0.133	3.200	7.947		
	InclinedBracing	3.04	3.50	2.500	4.111	2	65	x65x	6	0.534								

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3.2)WIND LOAD CALCULATION ON TOWER: T1
3.2)WIND LADONL ON GITUDINAL FACE (L-WIND):

Design Wind Pressure	=	2.29 kN/m^2		
Length of Tower at Top of Girder (B/B)	=	1.50 m	x	1.50
Length of Tower at Plinth Level (B/B) Transverseface	=	2.50 m	x	3.50 m
Height of Girder-1	=	1.20 m		
Height of Tower from PL to Girder Bottom	=	15.40 m		
Slope of Tower below Girder	=	0.032 Rad		
	=	1.860 Deg.		
		0.0325		

Panel No.	Member	Width of Panel at Top(m)	Width of Panel at Bottom(m)	Panel Height(m)	Length of Member(m)	No. of Member	Member Size	Exposed Area (m^2)	Total Exposed Area(m^2)	Total Boundary Area(m^2)	CG Height(m)	Solidity Ratio	Drag Factor	Total Wind(kN)	No. of Nodes	Wind Transferred on Each Node (kN)
6	MainLeg	1.50	1.50	1.200	1.200	2	110 x110x 10	0.264	0.784	1.80	0.600	0.436	2.600	4.666	8	1.114
	HORIZONTALBRACING	1.50	1.50	0.000	1.500	2	90 x90x 12	0.270								
	InclinedBracing	1.50	1.50	1.200	1.921	2	65 x65x 6	0.250								
5	MainLeg	1.50	1.66	1.737	1.737	2	110 x110x 10	0.382	0.687	2.74	0.854	0.251	2.700	4.249	8	1.639
	InclinedBracing	1.50	1.61	1.737	2.347	2	65 x65x 6	0.305								
4	MainLeg	1.66	1.82	1.737	1.737	2	110 x110x 10	0.382	0.701	3.02	0.855	0.233	2.800	4.498		
	InclinedBracing	1.66	1.82	1.737	2.456	2	65 x65x 6	0.319								
3	MainLeg	1.82	2.04	2.482	2.482	2	110 x110x 10	0.546	0.990	4.79	1.217	0.207	3.800	8.611		
	InclinedBracing	1.82	2.04	2.482	3.412	2	65 x65x 6	0.444								
2	MainLeg	2.04	2.27	2.500	2.500	2	110 x110x 10	0.550	0.979	5.40	1.228	0.182	2.850	6.389	8	1.900
	InclinedBracing	2.04	2.27	2.500	3.300	2	65 x65x 6	0.429								
1	MainLeg	2.27	2.50	2.500	2.500	2	110 x110x 10	0.550	0.999	5.97	1.230	0.168	3.850	8.808		
	InclinedBracing	2.27	2.50	2.500	3.454	2	65 x65x 6	0.449								

https://doi.org/10.46335/IJIES.2022.7.5.7 e-ISSN: 2456-3463
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(DESIGN OF FOUNDATION BOLT & BASE PALTE FOR TOWER-T2)

DESIGN OF FOUNDATION BOLT:

Provide foundation bolt perlegnos 4 & 40 mm
dia Area of bolt = 1257 mm^2
Max. Up lift Load perleg: = 213480 N
Max. shear force perleg: = 37323 N
Max. compression: = 200205 N

Uplift force(max. Tension/bolt = 213480 / 4 = 53370 N
Maximum Shear Stressper Bolt: = 37323 / 4 = 9330.8 N

Allowable Tension in Bolt: = 120 N/mm^2 (ReferTable8.1ofIS:800-1984)
Allowable Bond strength of concrete: = 0.8 N/mm^2 (ReferTable21ofIS:456:2000)
Shear Stress for Bolt: = 80 N/mm^2 (ReferTable8.1ofIS:800-1984)

Area of bolt required(A) = 444.750 mm^2

Dia of bolt required: = 23.803 = 24 < 40 safe

Edge distance required, = 1.5 x 40 = 60.0 mm

Tension capacity of bolt Tdb= Tnb/ γm, γ_{mb} = 1.25

$T_{nb} = 0.9 f_{ub} A_n < f_{yb} A_{sb} (\gamma_{mb}/\gamma_{mo})$, f_{yb} = 240 N/mm^2

γ_{mo} = 1.1 f_{ub} = 400 N/mm^2

Shank are a of the bolt (Asb) = 1257 mm^2

Net tensile area at the bottom of threads(An) = 980 mm^2

$0.9 f_{ub} A_n$ = 352.86 KN

$f_{yb} A_{sb} (\gamma_{mb}/\gamma_{mo})$ = 342.72 KN

T_{nb} = 342.7 KN

Tension capacity of single bolt Tdb = 274.2 KN

Tension/bolt(Tb) = 53.4 KN

HenceO.K

Shear capacity of bolt:

$V_{nsb} = f_u A_n / \sqrt{3}$ = 226.4 KN , V_{dsb} = 181.09 KN

Bearing capacity of the bolt:

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Vnpb= **2.5 kb dtfu** Kb =Min(e/3do,p/3do-0.25, fub/fu, 1)

= 22960 = 0.02

e= 71.4

V_{dpb}= 18.37 KN fu= 410

do= 42mm

There fore Shear capacity of single Bol= 18.4 KN p= 110mm

HenceO.K

Embedded Length of bolt required: = 53370X 1 / Pi 40 x 0. 8 = 531 mm

Provide foundation bolt per legnos 4& 40mm diaad 1500mmHe ncesafe embedmentlength

DESIGN OF BASE PLATE

Max. Compression perleg: = 200.21KN

Max. Tension perleg: = 213.48KN

Referring to Clause 34.40of IS:456-2000 permissible bearing stress on concreteis 0.45 fck Hence permissible bearing stress on concreteis.45fck

= 0.45x 20= 9.00N/mm²

Provide M.S. Base Plate of size 350x 350x 28mm perleg

Bearing capacity of base plate: = 1.6< 9.00 Henceok

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Maximum bearing pressure on plate:	=	213480 /	122500	=	1.743	N/mm^2		
B/B dist. Of bott. Leg from C.G. of plate	=	75	mm		(IS808:1989,Table-5.1)			
Lever of base plate in axis A-A	=	75	mm					
Bottomleg	=	110	x	110	x	10mm		
Length of base plate in axis A-A	=	350/2 +	350	/2				
	=	175	+	175				
	=	$\sqrt{[(175)^2 + (175)^2]}$						
	=	247.487	/ 4					
	=	61.87	mm					
Hence moment on the base plate Md	=	6671.25	N-mm					
Assuming width of the plate b	=	1.00	mm					
f_{ck}	=	25	N/mm^2					
fu	=	500	N/mm^2					
fy	=	250	N/mm^2					
Υmo	=	1.1						
Thickness of base plate required	=	$\sqrt{(6Md\gamma mo/1.2*bfy)}$						
	=	12.11mm<		28 mm		Henceok		
Provide M.S. Stiffener Plate of size		350	x	300	x	10mmperleg		
Edge distance of boltc entre	=	60.0	<	60	mm			
Boltto Bolt distance alongx-x	=	350	-	120	=	230	mm	
Boltto Bolt distance alongz-z	=	350	-	120	=	230	mm	

Moment at the face of the column flange-(for compression on column)

= 1.63432653061224*(230/4+61.8718433538229)*((60^2)/2=

= 351166.6271N-mm

Moment at face of stiffener (due to bolt tension)-

= 53370*(230-110)/2

= 3202200mmN-mm

Height of plate required = $\sqrt{(6M_d\gamma_{mo}/1.2*bfy)}$

15.32 mm

Hence, Provide stiffner plate heightas 350 mm

Henceok

REACTIONFORFDNANDBASEPLATEDESIGN-

CONCLUSIONS

The paper was successfully achieved its objectives that is the problems which are occurred in the Gantry tower are now overdesigned and checked by using all possible parameter which are occur red while installation as well as giving its service life without any catastrophic failure. And it is possible because of team work and support of senior person in this project. It is validate by using FEM based modeling technique, for this CAD is designing such away that the proposed model will not be failed in the future and its fatigue life will be more.

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