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

**Ashish Mahure1, Nitin Sawarkar2, Swapnil Choudhary3, Bharat Chede4, Yogesh Menghare5**

*1,2,3,4,5Department 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*

# INTRODUCTION

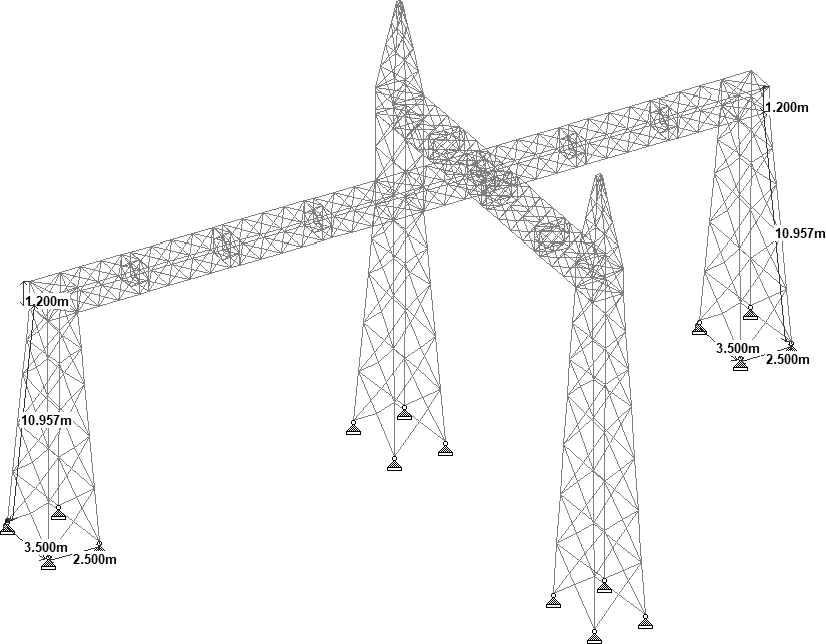
Gantry structures are mainly used for guiding the power conductor from last tower near substation to the electrical equipment’sin a substation. This structure consists of a number of columns and Girder beams, which depend on number of circuits of the line.A **gantry** is an over head bridge-like structure 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](https://www.electrical4u.com/electrical-power-transmission-system-and-network/) 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. **DESIGNCRITERIA**   2. **SCOPE**   This document covers design Calculations of 220KV Gantry. Beam Tower Structure Type (T2), Foundation Boltand Base Plate | | | | |
| **S.NO** | **STRUCTUREDESCRIPTION** | |  | |
| 1 | 220KVTOWERT2 | |
| * 1. **UNITS OF MEASUREMENTS**   Units of measurements used in analysis shall be of SI Units   * 1. **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** | |  | :47m/sec  :1.07  :1.03 (20mheight) |  |
| Basic Wind Speed | | Vb | (AsperIS875,Part-3:1987,Appendix-Acl-5.2) |
| Risk Coefficient | | k1 | (AsperIS875,Part-3:1987,Table-1) |
| Terrain, Height Factor | | k2 | (AsperIS875,Part-3:1987,Table-2) |
| Topography Factor Design Wind Speed Design Wind Pressure   * 1. **FACTOR OFSAFETY:**      1. For Structures :Normal Conditions   SCF | | k3 | :1.18 | (AsperIS875,Part-3:1987,Ann.C) |
| Vz | :61.13m/sec (Vb\*k1\*k2\*k3) |  |
| Pz | :2.29kN/Sqm (0.6\*Vz2/980.6) |  |
|  | :2.00 |  |
|  | :1.50 |  |

**DESCRIPTION OF STRUCTURES:**



* 1. **STAAD MODEL SKETCH:**
  2. **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 |

* 1. **GIRDERDETAILS:**

|  |  |  |  |
| --- | --- | --- | --- |
| **GIRDER-BEAM-B2** |  | | |
| Clear Span of Girder | = | **15.50** | m |
| Width of Girder | = | **1.50** | m |

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| **3.0)WIND LOAD CALCULATIONON TOWER: T1** | | | | | | | | | | | | | | | | | | |
| **3.1)WIND LADO N Transverse FACE (T-WIND):** | | | | | | | | | | | | | | | | | | |
| Design Wind Pressure | |  |  |  |  |  |  |  |  |  | = | 2.29 | kN/m2 |  |  |  |  |  |
| 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(m2)** | **Total Exposed Area(m2)** | **Total Boundary Area(m2)** | **CG**  **Height(m)** | **Solidity Ratio** | **Drag Factor** | **Total Wind (kN)** | **No. of Nodes** | **Wind Transfer redon 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 |

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| **3.2)WIND LOAD CALCULATION ON TOWER: T1** | | | | | | | | | | | | | | | | | | |
| **3.2)WIND LADONL ON GITUDINAL FACE (L-WIND):** | | | | | | | | | | | | | | | | | | |
| Design Wind Pressure | |  |  |  |  |  |  |  |  |  | = | 2.29 | kN/m2 |  |  |  |  |  |
| 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 (m2)** | **Total Exposed Area(m2)** | **Total Boundary Area(m2)** | **CG**  **Height(m)** | **Solidity Ratio** | **Drag Factor** | **Total Wind(kN)** | **No. of Nodes** | **Wind Transfer redon 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 |
| 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 | 8 | 1.639 |
| 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 |

**(DESIGN OF FOUNDATION BOLT & BASE PALTE FOR TOWER-T2)**

**DESIGN OF FOUNDATION BOLT:**

Provide foundation bolt perlegnos 4 & 40 mm dia Area of bolt = 1257mm2

Max. Up lift Load perleg: = 213480N

Max. shear force perleg: = 37323N

Max. compression: = 200205N

Uplift force(max. Tension/bolt = 213480 / 4 = 53370N

Maximum Shear Stressper Bolt: = 37323 / 4 = 9330.8N

Allowable Tension in Bolt: =

Allowable Bond strength of concrete: =

Shear Stress for Bolt: =

120N/mm2 (ReferTable8.1ofIS:800-1984)

0.8N/mm2 (ReferTable21ofIS:456:2000)

80N/mm2 (ReferTable8.1ofIS:800-1984)

|  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| Area of bolt required(A) = |  | 444.750mm2 | | |  | | | | |
| 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/ m, |  | | γmb | | | = | 1.25 |  | |
| Tnb=0.9fubAn<fybAsb(γmb/γmo) , |  | | fyb | | | = | 240 | N/mm2 | |
| γmo | =1.1 | | fub | | | = | 400 | N/mm2 | |
| Shank are a of the bolt (Asb) | | | = | | 1257 | mm2 | | | |
| Net tensile area at the bottom of threads(An) | | | = | | 980 | mm2 | | | |
| 0.9fubAn= | 352.86 | | | KN |  |  | |  |  |
| fybAsb(γmb/γmo) = | 342.72 | | | KN |  |  | |  |  |
| Tnb = | 342.7 | | | KN |  |  | |  |  |
| **Tension capacity of single bolt Tdb** = | 274.2 | | | KN |  |  | |  |  |
| Tension/bolt(Tb) = | 53.4 | | | KN |  |  | |  |  |
|  | HenceO.K | | |  |  |  | |  |  |
| **Shear capacity of bolt:** |  | | |  |  |  | |  |  |
| Vnsb= fuAn/ √3 = | 226.4 | | | KN | , | Vdsb= | | 181.09 | KN |

## Bearing capacity of the bolt:

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

= 22960

Vdpb= 18.37 KN

= 0.02

e= 71.4

fu= 410

do= 42mm

There fore Shear capacity of single Bol= 18.4 KN

HenceO.K

p= 110mm

Embedded Length of bolt required: = 53370X 1

Pi 40

x 0.8

= 531 mm

Provide foundation bolt per legnos 4& 40mm

**DESIGN OF BASE PLATE**

diaad

1500mmHencesafe

embedmentlength

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 |

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
| Maximum bearing pressure on plate: | = | 213480 | / | 122500 | = | 1.743 N/mm2 |
| 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 |  |  |
|  | = | √[(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 |  |  |  |
| fck | = | 25 | N/mm² |  |  |  |
| fu | = | 500 | N/mm² |  |  |  |
| fy | = | 250 | N/mm² |  |  |  |
| ϒmo | = | 1.1 |  |  |  |  |
| Thickness of base plate required = √(6Mdγmo/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 | = | √(6Mdγmo/1.2\*bfy) | | |  |  |
| Hence, Provide stiffner plate heightas |  | 15.32 mm  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.

## REFERENCES

1. *Failure Test report “MD” (30 -60 Dev./D.E.-00) NT +9M body extension 220 kV multi – circuit transmission tower design.*
2. *IS802Part1Sec11995Codeofpracticeforuseofstructuralsteelinoverheadtransmissionlinetowers,Part1.*
3. *Ch. Sudheer (2013), “Analysis And Design Of 220kv Transmission Line Tower In Different Zones I &VWithDifferentBaseWidths–AComparativeStudy“InternationalJournalOfTechnologyEnhancementsAnd Emerging Engineering Research, Vol1, Issue435Issn2347-4289*
4. *C. Preeti1 and K. Jagan Mohan (2013), “Analysis of Transmission Towers with Different Configurations” Jordan Journal of Civil Engineering, Volume7, No. 4.*
5. *Yoganan tham. C Helen Santhi.M (2013), “Dynamic Analysis of Transmission Towers International Journal of Advanced Information Science and Technology (IJAIST)”Vol.20,No.20.*
6. *Umesh S. Salunkhe and Yuwaraj M. Ghugal (2013),” Analysis And Design Of Three Legged 400k V Double CircuitSteelTransmissionLineTowersInternationalJournalofCivilEngineeringandTechnology” Vol.04, Issue 3.*
7. *Srikanth L. and Neelima Satyam D (2014), “Dynamic Analysis Of Transmission Line Towers International Journal of Civil, Environmental, Structural, Construction and Architectural Engineering Vol:8,No:4.*
8. *Dharmender Panth, IIT-BHU,Varanasi, U.P., India, Reasons For Failure Of Transmission Lines AndTheirPreventionStrategies, Volume-2, Issue-1, Jan.-2014*
9. *Siti Aisyah Kamarudin, Fathoni Usman, Intan Nor Zuliana Baharuddin “Review on analysis and design of lattice steel structure of overhead transmission tower” College of Graduate Studies, Universiti TenagaNasional,Kajang,Malaysia,InstituteofEnergyInfrastructure,UniversitiTenagaNasional,Kajang, Malaysia*
10. *Vinay RB, Ranjith A, and Bharath A (2014). Optimization of transmission line towers : P-Delta analysis. International Journal of Innovative Research in Science, Engineering and Technology, 3(7):14563–14569.*
11. *Usman Fand Megat Asyraf MIR (2011). Simulation of progressive collapse for transmission tower.Inthe 1st TNB ICT Technical Conference, College of Information Technology, Universiti Tenaga Nasional, Kajang, Malaysia.*
12. *Rao NP, Knight GS, Mohan SJ, and Lakshmanan N (2012). Studies on failure of transmission line tower sintesting. Engineering Structures, 35:55-70.*
13. *Rao NP and Kalyanaraman V(2001). Non-linear behavior flattice panel of angle towers. Journal of Constructional Steel Research, 57(12):1337-1357.*
14. *Preeti C and Mohan JK (2013). Analysis of transmission towers with different configurations. Jordan Journal of Civil Engineering,7(4): 450–460.*
15. *Mc Clure Gand Lapointe M (2003). Modeling the structural dynamic response of over head transmission lines. Computers and Structures, 81(8): 825-834.*
16. *Fu X and Li HN (2016). Dynamic analysis of transmission tower-line system subjected to wind and rain loads. Journal of Wind Engineering and Industrial Aerodynamics, 157: 95–103.*
17. *Baran E, Akis T, Sen G, and Draisawi A (2016). Experimental and numerical analysis of a bolted connection in steel transmission towers. Journal of Constructional Steel Research,121:253-260.*
18. *Albermani F, Kitipornchai S, and Chan RWK (2009). Failure analysis of transmission towers. Engineering Failure Analysis, 16(6): 1922-1928*