RESEARCH ARTICLE OPEN ACCESS

Design and Analysis of Cable Stayed Bridge

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Abstract:

Cable stayed bridge were firstly developed in Germany during post-world war years, due to shortage of materials like steel. With the introduction of high strength steel, orthotropic type decks, Development of welding techniques & progress in structural analysis and above mentioned shortage of steel in post-world war years the successful introduction of cable stayed bridges is possible. Cable stayed bridges were highly statically indeterminate structures and in order to find out exact solution of these highly indeterminate systems and analyse the cable stayed bridge the electronic computers played a very vital role.

Keywords — **Cable stayed bridges, High strength steel, Orthotropic type decks, Suspension bridge.**

I. INTRODUCTION

The various structural component of a typical cable stayed bridge are the following:-

1) Tower or Pylon
2) Deck system
3) Cable system supporting the deck

Scope & objective:

A brief history of cable stayed bridge is illustrated along with the structural components of it such as Tower or pylon, Deck systems, cable system supporting the deck.

A systematic approach is discussed for the preliminary design of cable stayed bridge which will fix the outline proportions of the structure to give response to the static and dynamic analysis of the structure.

II. LITERATURE

Various literatures are available on cable stayed bridge in recent years; a brief scenario of the same is presented below,

Kao and Kou (2010) analyzed a symmetrical, fan-shaped cable-stayed bridge under sudden loss of cable as it is most critical phenomenon in the analysis of cable stayed bridge.

Wolff and Starossek *(2008)* studied the collapse behaviour of a 3D cable-stayed bridge model and found out that the initial failure (loss) of the three cables around the pylon can trigger a zipper-type collapse associated with a large vertical deformation within the bridge deck.

Jenkins and Gersten *(2001)* reports in FTA report that about 58% of terrorist attacks targeted the transportation sector including bridge structures. **Mahoney** (2007) analyzed typical highway bridges under blast loads.

*(**Huang et al**., 2011)* studied the significant damage and collapse of several bridges which were occurred as the result of severe past earthquake events. Therefore, he recommends different guidelines for response to seismic actions is considered in the design of bridges. For example, Xiaoyudong bridge in China was damaged during the May 12th 2008 Wenchuan earthquake with the magnitude of 8.0.

*(**Kawashima et al**., 2011, **Hoshikuma**, 2011).*they studies the strong earthquake in Japan in Fukushima which created significant damage in several bridges caused by strong ground motion as well as tsunami inundation and soil liquefaction

The dynamic response of cable stayed bridges is more critical due to effect of earthquake and wind loadings as compared to other types of bridges. However with increasing span length and increasing slenderness of the stiffening girder the great attention is paid not only to dynamic response of bridges under earthquake and wind loading but also to the dynamic traffic loading.

The literature survey indicates that most of the research is done on seismic effects, wind effects, dynamic loadings on cable stayed bridges but actual design and analysis of cable stayed bridge is not done and validation of results on various analytical software's will give us the confidence of bridge design as that of a professional structural designer.

III. METHODOLOGY

***Preliminary Design Approach*:**

a.Back span to main span ration while fixing the basic arrangement of cable stayed bridge should be such that it should be always less than 0.5 in order to highlight the main span of cable stayed bridge. When stiffness of bridge is taken into account the optimum length of back span should be 0.4 to 0.45 of main span.

b. The spacing of stay anchors of cable stayed bridge along the deck should be incompatible with the longitudinal girder and size of stay should be limited such that breaking load is less than 25-30 MN.

c. stay oscillations can occur due to various effects such as Vortex shadding, wake induced vibrations, cable galloping, parametric instability, Rattling etc should be damped by incorporating internal and external damping mechanism.

d. pylon height of cable stayed bridge determines the overall stiffness of the structure, as the stay angle (α)increases, the required stay size will decreses & height of pylon will increase. However weight of stay cable & deflection of deck become minimum when ($\frac{1}{sin\alpha x \cos\alpha}$) is minimum. Therefore the most effective stay is that one with angle α=45°.

e. for the design of deck, tuning of loads in stays, to reduce the moments in the deck, under the applied Dead Load to small moments between stays, however reducing the dead load moments in the deck to purely local effects will not provide the optimal solution.

f. For the Static analysis the common approach is to model either a half or the entire structure as a space frame. The pylon, deck and the stays will usually be represented within the space frame model by 'bar' elements. The stays can be represented with a small inertia and a modified modulus of elasticity that will mimic the sag behavior of the stay.

g. Dynamic analysis is the determination of the frequencies and the modes of vibration of the structure. This information is utilized for the following aspects of the design such as the seismic analysis of the structure; response of the structure in turbulent steady flow wind, the physiological effect of vibrations.

Problem Statement:

Type of girder:-'**N**' type girder, Span of girder:-12 mt.,Spacing of cross girder:-2 mt.,Clear walking width between main girders:-2.8 mt. Let us provide **'N'** type truss within panel 2 mt. each, height of truss 2.5 mt. & cross girder at the six panel pts.; Total span of pedestrian bridge over national highway in urban city like Mumbai where population is very huge is 64 mt. i.e. 32 mt on both side.

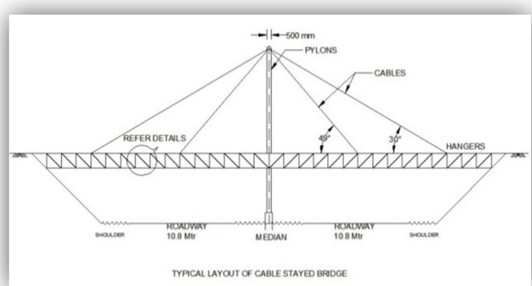

Fig.1 showing typical sketch of cable stayed Bridge

Design Calculations:-

Load calculation:

Total load = L.L. (KN) +D.L. (KN)
= 5.5 + 5.1 = 10.6 KN/ m^2
Consider 1 m width of R.C.C.
Maximum B.M. = $(wl^2)/8 = (10600 \times (2)^2)/8$
= 5300 Nm…………………… (1)
Moment of resistance
$=\sigma ct \times (1/6) \times (bd)^2$… …………………………… (2)
5300= $\sigma ct \times (1/6) \times (1000\times230)^2$
σct = 0.6011 N/mm ^{2}< 10.00 N/ mm^2
………… Which is ok.

Maximum shear force = wl/2 = 10600/2 =10600 N
Average shear stress = Va = 10600/(1000×230) = 0.0588
Maximum shear stress = Vm = 1.5 × 0.0588
= 0.0883 N/mm^2<0.8 N/mm^2

Moment of inertia of R.C.C. slab = $(bd^3)/12$
= $(1000 \times 180^3)/12$
= 486.6 x 10^6 mm^4
Maximum deflection = $(5/384) \times (WL^4/EI)$
= $(5/384) \times (10600 \times 12000^3)/ (2 \times 10^4 \times 486.6 \times 10^6)$
= 2.453 mm (negligible)

Crossbeams:

Clear walking width between main girders = 2.8 m,Say, length of cross girders = 0.3 m
Total length of cross beam = 2.8+ (2× (0.3/2)) = 3.1m,
Let us try ISMC 200 @ 221 N/m, for crossbeams various check for the shear force,shear stresses, permissible & maximum deflection are safe

Load calculations:- (ILD diagrams for N type Deck girder bridge.)

For P1 = (area of I.L.D.) × (intensity)
= (0.5 × 12 × 1.2) × (10.56 + 7.5) = 130 KN (**C**)

For P3 = (area of I.L.D.) × (intensity)
= (0.50 × 12 × 1.066) x (10.56+7.5) = 115.51 KN (**T**)

For P2:-= (area of I.L.D.) × (intensity)
= {(0.5 × 12 × 1.067) × (10.56 + 7.5)
= 115.620 KN (**T**)

For P4 = (area of I.L.D.) × (intensity)
= (0.5 × 12 × (5/6)) × (10.56 + 7.5)
= 90.3 KN (**C**)

Load coming on each member,
P1 =130 KN (C),
P2 = 115.620 KN (T),
P3 = 115.51 KN (T),

P4 = 90.3 KN (**C**).

Design of P1:-

Force in member P1 = 130 KN (C),
Area of c/s required = $(130\times10^3) / (80) = 1625\ mm^2$
Area of angle section $=817\ mm^2$,
provide 2 ISA (65×45×8) b/b with a gusset plate of 8 mm thk, i.e. is safe.

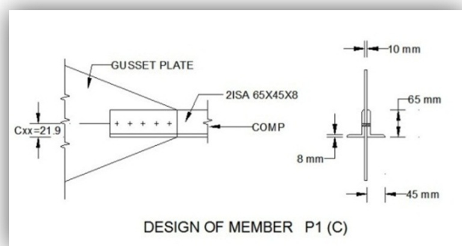

Fig.2 showing design of member P1(C)

Design of member P2/P3:-

Force in memberP2/ P3 = 115.620 KN (T),
Area req. = $p/ (0.6Fy) = (115.620\times10^3) / (0.6 \times 250)$
$=770.8\ mm^2$
Cross sectional area required = $1.3 \times 770.8 = 1002.04\ mm^2$,
provide ISA (90 × 60 × 8) which is safe.

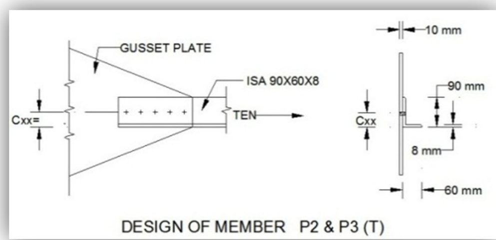

Fig.3 showing design of member P2 & P3 (T)

Design of member P4:-

Force in member P4 =90.3 KN (C),
Area of c/s = $(90.3 \times 10^3) / (80) = 1128\ mm^2$,
Select 2ISA (65×45× 6) having Area = $1250\ mm^2$i.e. Safe

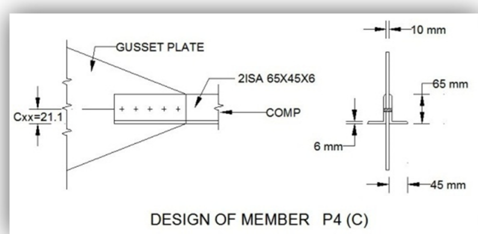

Fig.4 showing design of member P4(C)

Design of cable forces:-

Loads:
R.C.C. = (2.8/2) ×0.18 × 1 × 25 = 6.30 KN/m,
F.F. = (2.8/2) × 0.05 × 1 × 18 = 1.26 KN/m,
S.W. = 10.6 KN/m,
L.L. = 7.5 KN/m (assumed for Normal crowd)

But while calculating cable forces as it is most critical component part of cable stayed bridge, we can find it with below mentioned formula, i.e. for effective span of over 30 mt, the intensity of load should be determined ,

$P = (P' - 263 + (4800/L) \times((165 - W)/15)$
$= (500 - 263 + (4800/64) \times((165 - 2.8)/15)$
$= 162 \times 10.813$
$= 1751.70\ kg/m^2 = 17.51\ KN/ m^2$

Total weight of the structure = (6.3+1.26+10.6+17.51)
= 35.67 KN/m = (35.67 KN/m) x 64m
=2282.88 KN

Or,

f = (35.67KN/m) x 0.096 = 3.424 KN/m,
Where 0.096 is the vertical seismic coefficient

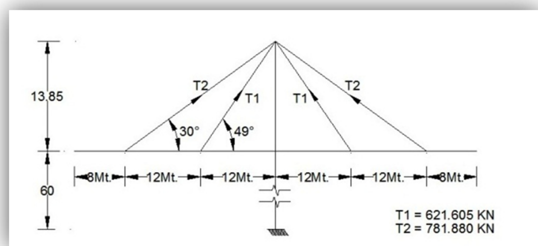

Fig.5 showing schematic sketch of Tension in the Cables

T1 sin α = (35.67+3.424) x 12,
T1 sin (49) = 469.13 KN
T1 = 621.605 KN
(T1=Tension in the first cable which is inclined at 49 degrees)

T2 sin 30 = (35.67 + 3.424) x 10,
T2 = 390.94 / (sin 30)
T2 = 781.88 KN
(T2=Tension in the second cable which is inclined at 30degree)

T1 = 621.605 KN
T2 = 781.88 KN

Design of Hanger at T1 & T2:-

For T1:-T1 = 621.605×10^3 N
Area = $P/ (0.6 \times fy) = (621.605 \times 10^3) / (0.6 \times 250)$
$= 4144.03\ mm^2$
Cross sectional area = $1.3 \times 4144.03 = 5387.24\ mm^2$
2ISA (200×150×20) connected b/b by gusset plate of 10mm thickness plate, which is safe and ok.

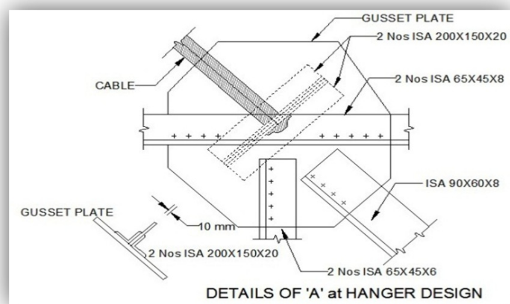

Fig.6 showing details of Hanger for T1 & T2

Design of cable:-

For T1 = 621.605 KN

Using 7 mm Φ height tensile wires initially stressed to 1200 N/mm^2

Force in each = (38.48 × 1200) × 1000 = 46 KN

Number of wires = (621.605) / (46) = 13.514 ≈ 14 nos.

Having @ giving strength; 14 × 38.5 × 1600

= 862.4 KN > 621.605 KN …………………hence safe & OK

Or, Double strand (ASTM- A 426);2 wires having 15.24 mm

The cross sectional area (A) = 2 × (140) = 280 mm^2

Ultimate tensile force (Afpu) = (260.7 x 2) for each cable,

Nominal wt (g) = 1.102 (kg/m)

For T2 = 781.88 KN; Using 7 Φ mm, initial stress = 1200 KN/m^2

Force in each cable; (38.48 × 1200) / (1000) = 46 KN,

Number of wires = (781.88) / (46) = 16.99 mm ≈ 18

18 nos. of 7 mm wire gives ultimate strength = (18 × 38.48 × 1600) = 1108.224 KN

Or, Super strand Euro norm EU- 138 @ 15.77 mm dia.

The cross sectional area (A) = (150 × 2) =300 mm^2

Ultimate tensile strength (Afpu) = (265 × 2)

Nominal weight = 1.019 kg/m; Total vertically downward load on pylon

= 469.13 + (2 ×621.605 × sin 49) + (2 ×781.88 ×sin 30)

= 2189.272 KN

Design of pylon:-

2 ISHB – 450 ;

Area = 22228 mm^2;

Pcap = 59.219 × 22228 = 1316.319 KN

Total load carried by two pylon = 1316.319 × 2

= 2632.638 > 2189.272 KN …. safe & OK

Design of gusseted base:-

Design of base plate

A req. = P / W = (1094.636 × 10^3) / 4

= 273659 mm^2

Sq. plate = √ 273659 = 523.12 mm ≈ 550 mm

Take plate having size (750 × 750),

Assume 10 mm thickness gusset plate,

with a gusset angle (150 × 75 × 10); 3)

Thickness (t):- t = [t'- thickness of angle]

= [c × √ [(3wa) / σbs] – thickness of angle]

Where, c = (15 + 75) -10 = 80 mm

Wa = (1094.636 × 10^3) / (750 × 750)

= 1.9460,

t = [80 × √ [(3 × 1.9460) / (185)] –10 ;

t= 4.2113 ≈ 6 nos.

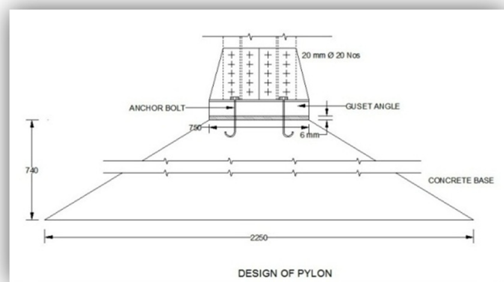

Fig.7 showing details of Pylon & Gusseted Base

***Design of concrete base*:-**

1) A req. = (1.15 P) / B.C
   . = (1.15 × (1094.636 × 10^3) / (0.25)
   = 5035325.6 mm^2
2) B.C. = √A = √ 5035325.6
   = 2243.95 ≈ 2250 mm
3) T = (0.5 × Bc- Bb) = (0.5 × [2250-750]) = 750

Fastening

Load on fastening = (outstand) / (width) × load on column

= (750-450) / (750) × 1094.636 = 437.854 KN

Assuming 20 mm Φ rivets,

Ps = σs × (π/4) × $Φ^2$ = 100 × (π/4) × 20^2 = 31415,

Pb = σb × Φ × t = 300 × 20 × 12 = 72000;

Take rivet value maximum from above two i.e. 72000;

= (load on fastening) / (rivet value)

= (1094.636 ×10^2) / (72000)= 15.20 ≈ say 20 nos.

Design of crossbeam:-

RA+ RB = 469.130 KN

S.F. = (363.575× 10^3) N,

B.M. = (wl^2) / 8 = (469.13 × 3.1^2) / 8 = 563.542 × 10^6 Nmm

Z= M / σmax = 3.4154× 10^6 mm^3,

Trial section (with cover plate)

Z = 1.5 × Z = 1.5 ×3.4154× 10^6 mm^3 = 5123.109× 10^3 mm^3;

ISMB 500 (250 x 32)

Z= 5622.9× 10^3 mm^3

D = 500 + (32 x 2) = 564 mm;

tf = 17.2 mm; tw = 10.2 mm;

d1 = (500-(17.2 × 2)) = 465.6 mm;

leff = 3100 mm

ryy = 59.9 mm
(tf / tw) = (17.2 / 10.2) =1.68…… (Should not greater than 2)

OR,
d1/ tw = (465.6) / 10.2 = 45.647 ..(Should not greater than 85)

Use table no 6.6.2 of IS code (page 58)
D / tf = (564)/ (17.2) = 32.79
l / ryy = (3100)/(59.9) = 51.75

D/tf		30	32.79	35
l/ryy	50	155	155	155
	51.75		154.65	
	55	152	152	152

Check for bending:-
Zcal = M/ σcal = $(563.542 \times 10^6)/(154.65)$
= 3643.983×10^3
………………………… (which is not greater than 5622.9×10^3)

Check for shear:-
= (S.F.)/ (D × tw) …………(should not be greater than 100)
= $(363.575 \times 10^3)/(564 \times 10.2)$
= 63.19 …..(should not be greater than 100)

Hence safe & O.K.

Check for deflection:-
δ max = K (Pl^3)/ (E x Ixx) …..< (l/325
= (3100)/(325) = 9.53mm)

= $\{(5/384)\} \times \{(727.15 \times 10^3 \times 3100^3) / (2\times10^5 \times (158564.4 \times 10^4))\}$ = 0.8894 mm < 9.53 mm ……………hence safe & O.K.

(PROVIDE THE CROSSBEAM AT THE TOP OF THE PYLON & UNDER THE N TYPE TRUSS BRIDGE ABOVE THE 6 m FROM THE GROUND)

Design steel rocker bearing:-
Vertical reaction = 469.13 KN ≈ 500 KN
Horizontal reaction takes it as 50 KN
Assuming following permissible stresses according to IRC 83-1982.
Permissible compression stresses in concrete block = 4 N/mm^2
Permissible bending stresses in steel plate = 160 Nmm^2
Permissible shearing stresses in steel plate = 185 N/ mm^2;
Permissible shear stresses in steel = 105 N/ mm^2

Details of design:-

1) Bed plate = Area of bed plate
= $(500 \times 10^3) / (4)$
= 125×10^3 mm^3

Provide bed plate of size (500 x 500 × 80) mm ;
Top plate of overall size = (500 × 400 × 80) mm

2) Rocker diameter

Let R = radius of the rocker surface in contact with the flat surface of bottom plate;
Vertical design load per unit length= $(170R^2 \times \sigma u^3) / E^2$
But, design load/ unit length = $(500 \times 10^3) / 500$
= 1000 N/mm;
Hence 1000 = $[(170 \times R^2 \times 500^3) / ((2 \times 10^5)^2)]$
$R^2 = 1882.35$; R = 43.38mm
Provide radius of 250 mm for rocker surface

3) Thickness of base of plate:-

Maximum B.M. @ central axis of base plate
= $(250 \times 10^3 \times 100) = 250 \times 10^5$ Nmm
If t = thickness of base plate required;
Section modulus = Z = $(bt^2) / 6$ = M/σ1
t = [√ (6M) / (b× σt)]
= $[\sqrt{(6 \times 2.5 \times 10^7) / (500 \times 160)}]$ = 43.30 mm

Provide an overall thickness of 62 mm for central portion of the base plate
Check for Bearing stress
Assuming 50 percent contact area between top & bottom plates
Bearing stress
= $(500 \times 10^3) / (500 \times 200)$ = 5 N/mm^2 < 185 N / mm^2
Hence bearing stresses are within safe permissible limit.

Analysis using STAAD Pro

Introduction

The following is the fundamental considerartions for the effective use of STAAD-PRO (i.e. Structural Analysis & Design Program software) for the analysis of structures. It must be mentioned however that since STAAD is a computer program, blind faith should not be placed in STAAD or any other engineering program.

It is therefore strongly recommended that until at least one years' experience of continually using STAAD is obtained, and for important structures parallel hand calculations for the analysis and design of the structure be done as well.

Modelling of Cable Stayed Bridge

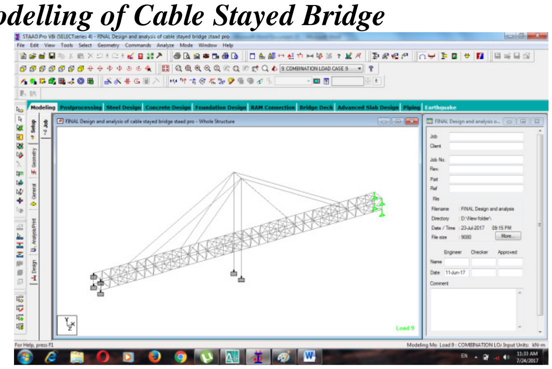

Fig.8 Three dimensional modelling of cable stayed bridge

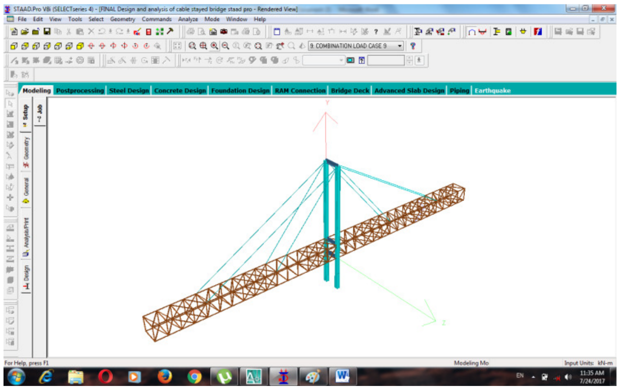
Fig no.9 showing 3d rendered view of cable stayed bridge

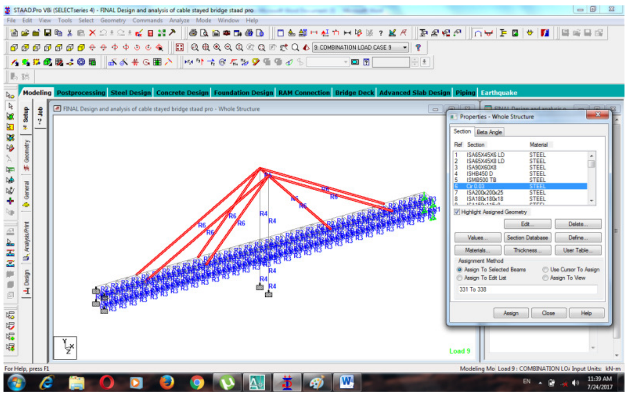
Fig no.10 Assigning Properties of Cable Stayed Bridge

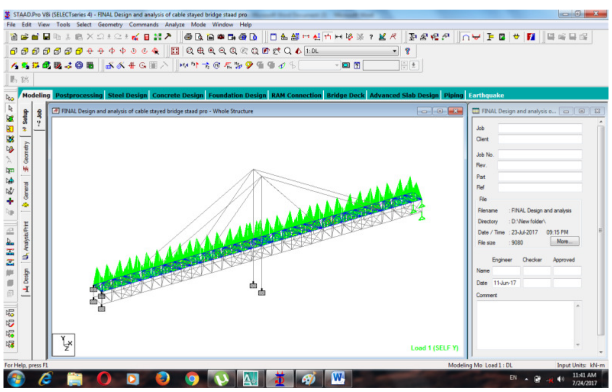
Fig no.11 Dead load of Cable Stayed Bridge

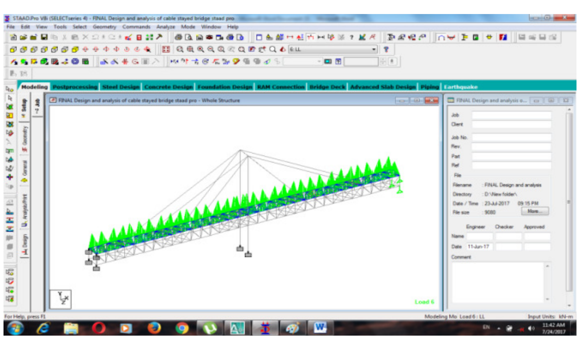
Fig no.12 Live load onCable Stayed Bridge

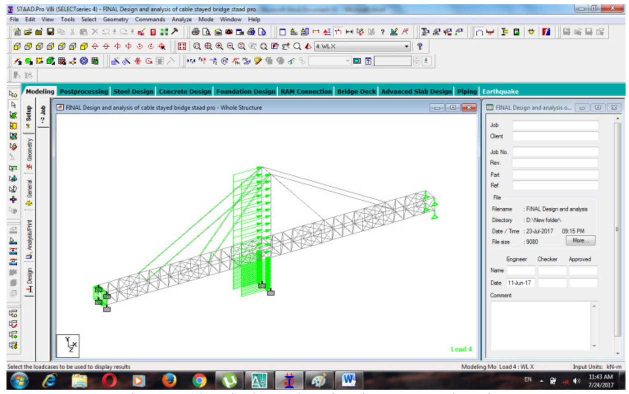
Fig. no. 13 Wind Load acting in +ve X Direction

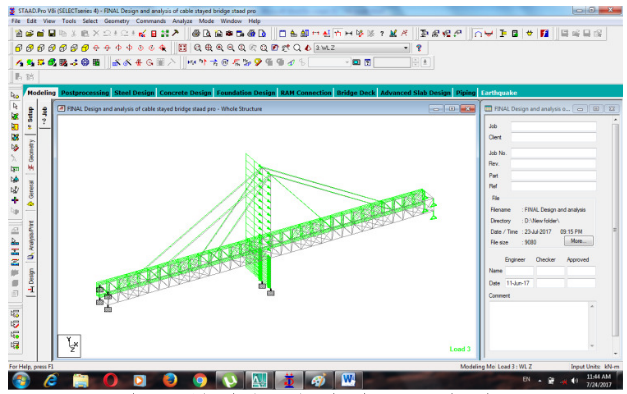
Fig. no. 14 Wind Load acting in +ve Z Direction

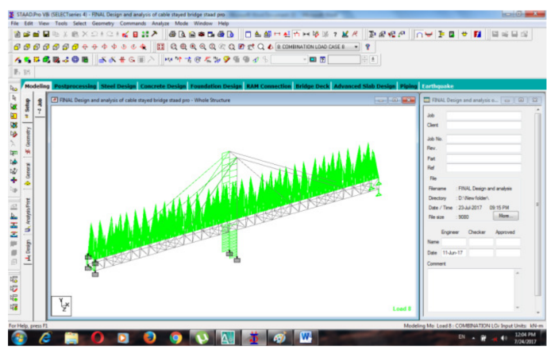
Fig. no. 15 DL+LL+WL @ X Axis combination of load

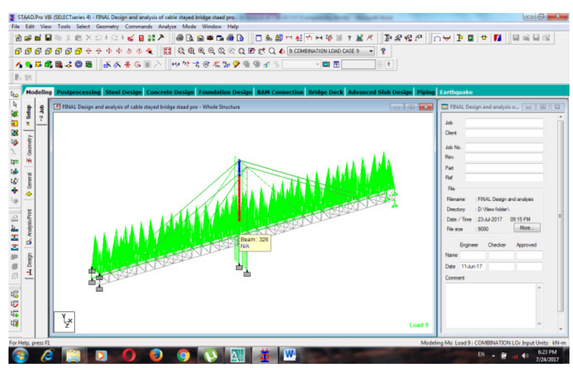
Fig. no. 16 DL+LL+WL @Z Axis combination of load

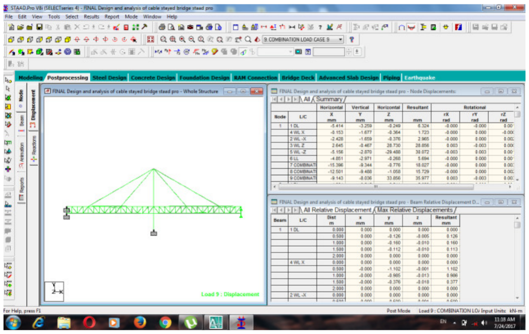

Fig no. 17 Deflection of cable stayed bridge by considering nodal displacement and relative displacement

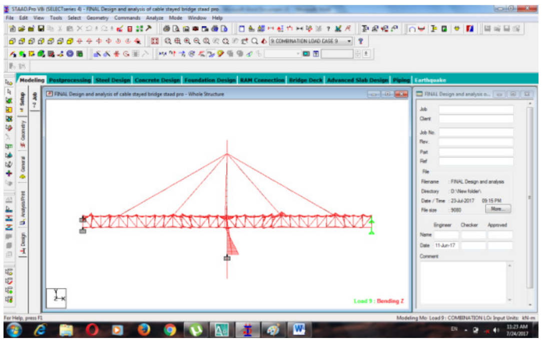

Fig. no. 18 Bending of cable stayed bridge along the z axis

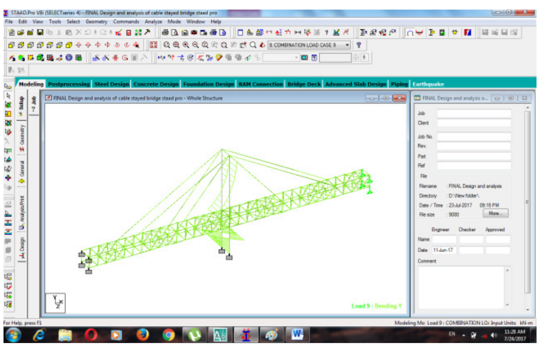

Fig. no. 19 Bending of csb along the y axis

Results and Discussions:

Design of cable stayed pedestrian bridge is done manually, various check for the allowable and maximum deflection. Shear force, bending moment, bearing stresses and shearing stresses in case of rivets are applied and found satisfactory and analyze the same bridge from computer aided software. Compare the results obtained from both i.e. computer outputs and manual calculations and validate the design.

Conclusion:

Actually, most of the research is done on seismic effects, wind effects, dynamic traffic loadings on cable stayed bridges but the actual design and analysis of cable stayed bridge is not done and validation of results on software will give us the confidence of bridge design as that of a professional structural designer.

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