Cable Stayed Bridge

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ABSTRACT

This project report is all about the analysis of the cable-stayed bridges for attaining the cable forces, girder moments and shear forces of the girder and the pylons. The methods adopted for attaining the cable forces is the strain energy principle. This strain energy principle was adopted from a journal of ASCE and the same procedure is followed from that journal. This journal was incorporated with some assumptions. The same assumptions were considered for making this report. The obtained cable forces which are done by strain energy principle are cross checked with the STAAD results. As the strain energy principle is a procedure which comprises of a laborious iterative process, a "C" programming is made for this procedure, which performs the analysis according to our requirement of the no. of cables, girder span, pier portion, loading conditions, tower dimensions, stiffness of all elements, position of cables. Thus, the difference between these values of the cable forces obtained from the strain energy method and that obtained from the STAAD gave not much difference, the STAAD.PRO analysis is performed for the model chosen for designing. The model chosen for designing is performed for analysis and later on, carried out with designing of the pylon dimensions, steel girder and the deck slab along with the concrete pylon.

1. Introduction

Cable stayed bridges have good stability, optimum use of structural materials, aesthetic, relatively low design and maintenance costs, and efficient structural characteristics. Therefore, this type of bridges are becoming more and more popular and are usually preferred for long span crossings compared to cable stayed bridges

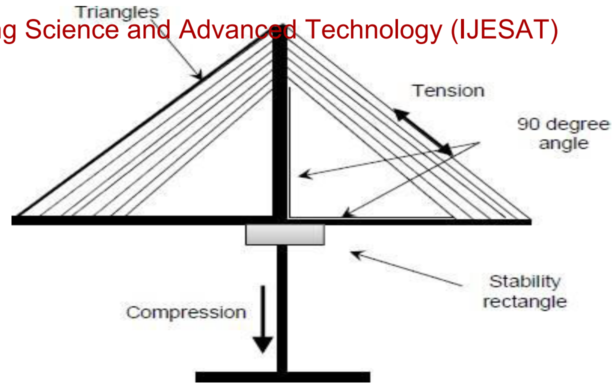

Figure 1.2 Illustration of Typical Cable Stayed Bridg

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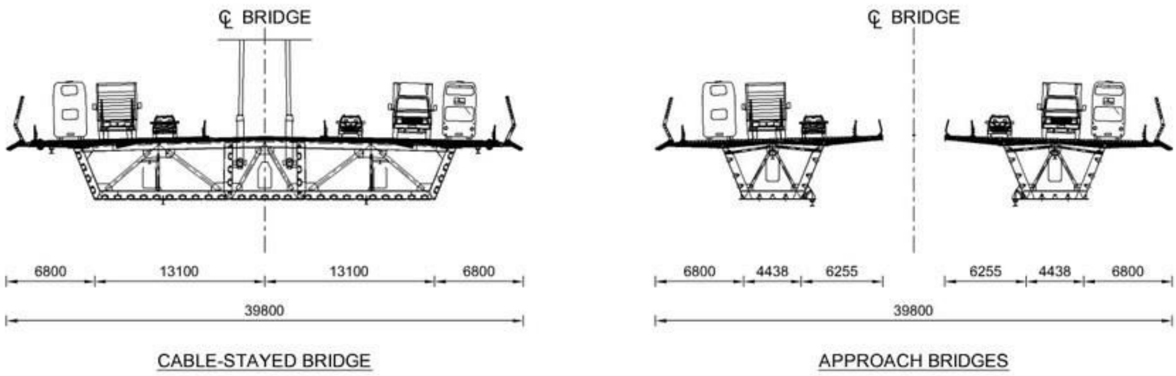

Figure 1.3 Typical Cross Section of the Deck

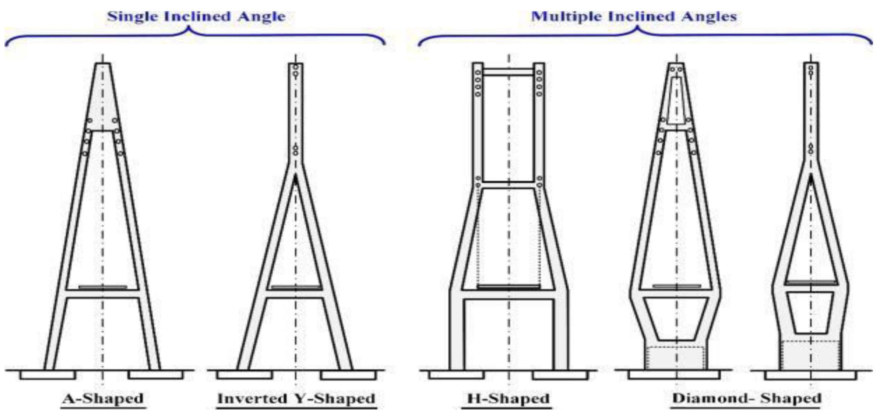

Figure 1.4 Types of Pylons

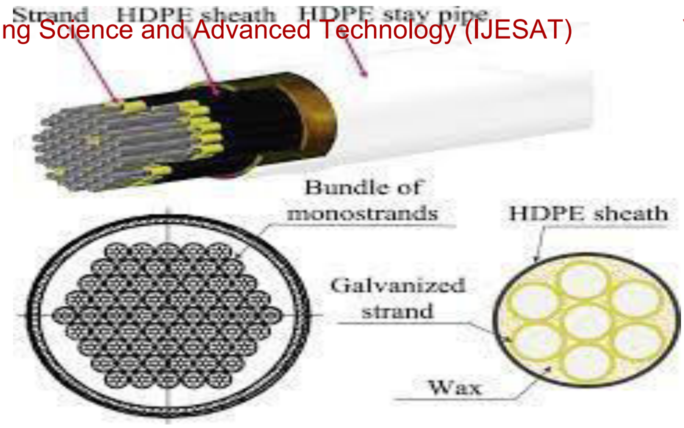

Figure 1.5 Cross Section of the cable

Methods of Analysis for Attaining the Cable Forces:

- Non-linear analysis of cable – stayed bridges
- Linear analysis of cable stayed bridges
- STAAD Analysis using STAAD.PRO software
- Strain energy method of analysis

2. LITERATURE REVIEW

N.K Paul, (2011) In this review, it is exhibited that, utilization of super elastic shape memory alloy bars consolidating with steel reinforcement with some rate in T-Beam concrete bridge longitudinal girder works successfully exceptionally well. The load carrying capacity can be increased. The failure mechanism of a reinforced concrete girder is demonstrated great utilizing FEA, and the failure load anticipated is near the failure load measured during trial testing. The whole load distortion reaction of the model created coordinates well with the reaction from trial result. This gave trust in the utilization of ANSYS 11.0 and the model created.

R.Shreedhar Spurti Namadapur,(2012) A straightforward span T-beam extension was analyzed by utilizing I.R.C. determinations and loading (dead load and live load) as a one dimensional structure. Finite Element analysis of a three-dimensional structure was done using Staad pro programming. Both models were subjected to I.R.C. Loadings to convey most outrageous bending moment. The results were broke down and it was found that the results got from the limited component model are lesser than the results got from one dimensional examination, which suggests that the results got from I.R.C. loadings are traditionalist and FEM gives practical design.

Amit Saxena, (2013) Dead load bending moment and Shear forces for T-Beam girder are lesser than two cell Box Girder Bridge. Which empower designer to have lesser heavier region for TBar Support than Box Brace for 25 m span. Moment of resistance of steel for both has been evaluated and conclusions drawn that T-Beam Girder has more noteworthy utmost with respect to 25 m span. Cost of concrete for T-Beam Girder is under two cell Box Girder as sum required by T-Beam Girder.

Mahesh Pokhrel, (2013) General design and analysis of a run of the mill T-Girder RCC Bridge has been completed with Assessment of reaction and design theories as per three worldwide codes to be specific IRC, AASHTO and Euro code. Among of all, the Euro code gave most moderate design. It might be because of the utilization of qualities load utilized with no component. Euro code is compensated for extensive variety of pertinence and scope so it can be referred for the design of bridges. In which truck loading is utilized for reaction in the superstructure and in which non-direct conduct of pier and abutment is not considered. Considering nonlinearity is one of the suggestions for the future work for more practical outcome.

M.G Kalyan Shetti,(2013) This review is done for four path and six Path scaffolds of traverses 15m, 20m, 35m, 30m, 35m utilizing IRC class A loading by differing various longitudinal girder. From the perceptions, it can infer that-load figure acquired by Courbon's technique is steady for all ranges and this demonstrates the impact of variety of traverse is not considered. In which need to revise the condition of load variable given by Courbon's hypothesis.

Madda, (2013) The review is done for two lane and four lane bridges, for two lane bridges, all the different span gave sensible outcome with the exception of 35m in light of the fact that its redirection/traverse proportion is (2.51x10-3)very near allowable limit (2.66x10-3). Furthermore, prompt serviceability issues in future. Also, four path spans, avoidance/traverse proportion are inside allowable confine up to 30m for all the blend of longitudinal support. However, for 35m traverse of 3 longitudinal girder (2.66x10-3) and 5 longitudinal girder (2.19x10-3) there is no minor distinction amongst real and passable esteem. Subsequently it is very conceivable that they may prompt serviceability issue.

Rajamoori Arun Kumar, (2014) Bending moment and shear force for PSC T-Beam Girder are lesser then RCC T-Beam girder bridge. Which allow designer to have lesser heavier section for PSC T-Beam Girder then RCC T-Beam Girder for 24m span. Momentof resistance of PSC T- Beam Girder is more as compare to RCC T-Beam Girder for 24 mspan. Cost of concrete for PSC T-Beam Girder is less then RCC T-Beam Girder.

3. STRAIN ENERGY METHOD

The method adopted by us is the strain energy method which was adopted from a journal from ASCE named "Energy Analysis Of Cable-Stayed Bridges" by Hassan I. A. Hegab1 2.1 Method of analysis Consider the system shown in Fig.2. 1 where the stay cables 1, 2, ...,N can be replaced by a system of elastic supports with stiffnessesk1,k_2,…….k_N. If the support provided by the pier is released, the loaded girder may thus be reduced to the base system shown in Fig. 2(a). Now, the potential energy U for the girder base system can be expressed as

$$u = \int_0^L \int_0^K MdKdx - \int_0^L \int_0^y pdxdy + \sum_{i=1} \int_0^{y_i} W_i dy_i$$

$$= EI_G \int_0^L \int_0^{y^{11}} y^{11} dy^{11} dx - \int_0^L \int_0^y pdxdy + \sum_{i=1}^N \int_0^{y_i} W_i dy_i \ldots\ldots\ldots\ldots\ldots (1)$$

in which M = EIG-y" = bending moment in the girder; K = y" = d2y/dx2 = curvature of the girder; EIG = its bending stiffness; L = span length(between the two end abutments); y = vertical deflection of the girder at a point distance x from the left end support; Wi = Fi sin 0i = vertical component of the axial force Fi in cable i due to the application of the live load P on the girder; □i = slope of cable i (to the horizontal); y_i = girder deflection at a point distance xi from the left end abutment, where cable i is anchored to the girder; and N = total number of stay cables. The girder deflection y at any point distance x from the left end abutment, fig. 2.2(a), may be expressed as

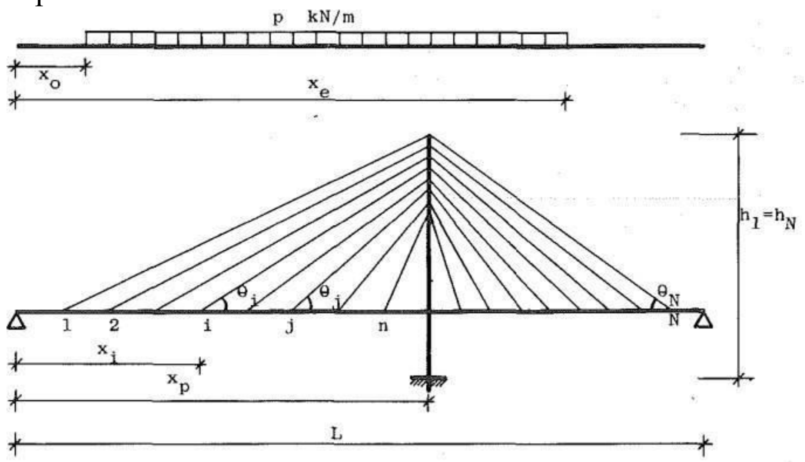

Figure 3.1 bridge Configuration & Live Load

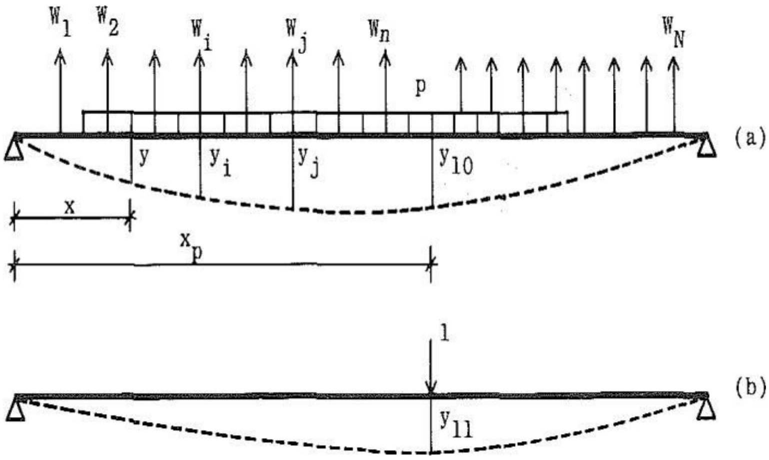

Figure.3.2 Base System of The Girder

Figure.3.2(a) Deflection Curve Due to live load & Cable Forces

Figure.3.2(b) Deflection Curve due to Unit girder reaction at pier support

$y = \sum_{j=1}^{\infty} a_j \sin\frac{j\pi x}{L}$.................. (2)

In which a_j= Fourier coefficient. For a stationary potential energy, $dU/da_m = 0$, and thus, one can write

$$EI_G.\int_0^L y^{11}\frac{\partial y^{11}}{\partial a_m}dx - \int_0^L p\frac{\partial y}{\partial a_m}dx + \sum_{i=1}^{N} W_i\frac{\partial y_i}{\partial a_m} \text{...} \quad (3)$$

In which, $y^{11} = -\sum_{j=1}^{\infty}\left(\frac{j\pi}{L}\right)^2 a_j \sin\frac{j\pi x}{L}$; $\frac{\partial y^{11}}{\partial a_m} = -\left(\frac{m\pi}{L}\right)^2 \sin\frac{m\pi x}{L}$; $\frac{\partial y}{\partial a_m} = \sin\frac{m\pi x}{L}$; *and* $\frac{\partial y_i}{\partial a_m} = \sin\frac{m\pi x_i}{L}$......

For a uniform live load P extending from x_0 to x_e, Fig. 1.1, a Fourier coefficient a_m can be obtained by substitution from Eqs. 4 into Eq. 3, and after manipulation, as

$$a_m = \frac{\left(\frac{p.L}{m\pi}\right).\left[\cos\frac{m\pi\, x_0}{L} - \cos\frac{m\pi\, x_e}{L}\right] - \sum_{i=1}^{N} W_i \sin\frac{m\pi\, x_i}{L}}{EI_G.\left(\frac{m\pi}{L}\right)^4 \frac{L}{2}} \ldots\ldots(5)$$

Using Eq. 5, the deflection curve, Fig. 2(a), can be obtained if the vertical components Wi of the cable forces Fi are known. However, the cable forces Fi are not known and an iterative procedure should be developed in order to solve the problem. The girder deflection y10 at the pier point for the base system, Fig. 2.2(a), can thus be written in the form

$$y_{10} = \sum_{j=1}^{\infty} a_j \sin\frac{j\pi x_p}{L} \ldots\ldots\ldots\ldots(6)$$

in which x_p= horizontal distance from the pier to the left support. Similarly, a Fourier coefficient b_m for the deflection curve due to unit girder reaction at the pier position, Fig. 2.2(b), can be obtained as

$$b_m = \frac{\sin\frac{m\pi\, x_p}{L}}{EI_G.\left(\frac{m\pi}{L}\right)^4 \frac{L}{2}} \ldots\ldots\ldots\ldots(7)$$

The girder deflection y11 at the pier point due to unit girder reaction at the pier position, Fig. 2.2(b), may thus be written as

$$y_{11} = \sum_{j=1}^{\infty} b_j \sin\frac{j\pi\, x_p}{L} \ldots\ldots\ldots\ldots(8)$$

Thus, the girder reaction R_1 at the pier support can be obtained, using Eqs. 6 and 8, as

$$R_1 = \frac{-y_{10}}{y_{11}} \ldots\ldots(9)$$

The final deflection Y of the girder, fig.1, becomes

$$Y = \sum_{j=1}^{\infty} a_j \sin\frac{j\pi x}{L} + R_1.\sum_{j=1}^{\infty} b_j.\sin\frac{j\pi x}{L} \ldots\ldots.(10)$$

The final value of the tension Fi in cable i, Fig.3, is given by

$$F_i = k_i(Y_i \sin\theta_i \pm \Delta_i \cos\theta_i) \ldots\ldots.(11)$$

in which f_c, = stiffness of cable i; A= horizontal displacement of the pylon at the point of attachment of cable i; Y_i= final girder deflection at the point where cable i is anchored; and i = 1, 2, ..., N.

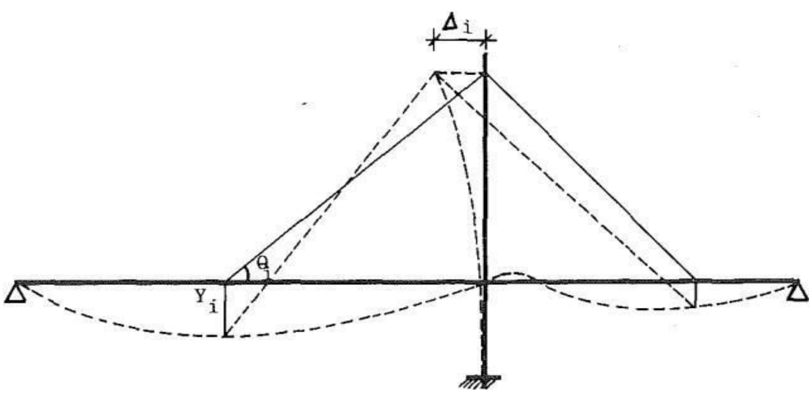

Figure 3.3 Final Movements of Two Ends of Cable i

Thus, the vertical and horizontal components Wi, and Qi of the cable tension Fi are given by

$W_i = F_i \sin\theta i$.........(12a)

$Q_i = F_i \cos\theta i$.........(12b)

Again, Wi and Qi are determined in terms of the vertical deflection Yi of the girder and the horizontal displacement A, of the pylon, see Eq.11. The procedure for solving the problem can be summarized as follows:

i. Eq. 8 is used to find the girder flexibility coefficient y_u for the pier point, Fig. 2.2(b).
ii. Guess some initial values for the cable tensions Fi, e.g., Fi = 0.
iii. The horizontal components Q, of the cable tensions Fi are determined from Eq. 12(b). Then, using Eq. 19, the horizontal displacements Δi, of the pylon can be determined, see below.
iv. Using Eq. 12(a), the vertical components Wi of the cable tensions Fi are determined.
v. The Fourier coefficients a_m are then determined using Eq. 5.
vi. The girder deflection y10 at the pier point for the base system, Fig. 2.2(a), can be determined using E.
vii. Eq. 9 is then used to determine the girder reaction R1at the pier support. viii. The final girder deflections V, are then obtained using Eq. 10. ix. Improved values for the cable tensions F; can then be obtained using Eq. 11.
x. Steps 3 through 9 are repeated until convergence is obtained. Convergence may be achieved if the difference in the value of the tension Fi in any cable i, determined from two successive iterations, does not exceed a small percentage (say 1%).

The axial compressive force in the pylon F_p, due to live load, is equal to the sum of the vertical components W_i of the forces F_i in all cables attached to the pylon, i.e.

$$F_P = \sum_{i=1}^{N} W_i\text{.........(13)}$$

The axial compressive load in the pier Rp, due to live load, is equal to the axial force in the pylon F_p plus the girder reaction R_i at the pier position, i.e.

$$R_P = F_P + R_1\text{.........(14)}$$

METHODOLOGY

STAAD.PRO SOFTWARE

In this study A typical tee beam bridge is considering having the component longitudinal girder, continuous deck slab and cross beam, the cross girders are provided a lateral rigidity to the bridge deck. The particular bridge model is taken then that model is analysis and design on the STAAD.PRO and also analysis and design with manually. Result are comparing between STAAD.PRO and manually with considering the class A and class AA tracked loadings.

THE STEPS IN MODELLING THE STRUCTURE ON STAAD.PRO ARE AS FOLLOWS

Figure 5.1: Model generation in the software

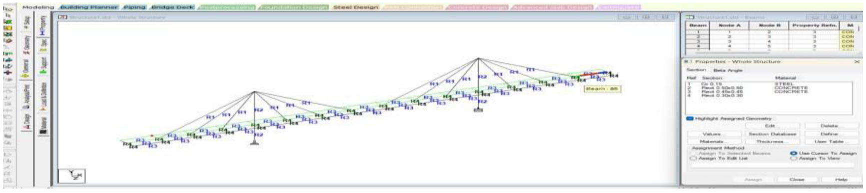

Figure 5.2: Assigning properties to the model

Three kinds of forces operate on any bridge: the dead load, the live load, and the dynamic load.

1) **<u>Dead load:</u>** refers to the weight of the bridge itself. Like any other structure, a bridge has a tendency to collapse simply because of the gravitational forces acting on the materials of which the bridge is made.
2) **<u>Plate load:</u>** refers to traffic that moves across the bridge as well as normal environmental factors such as changes in temperature, precipitation, and winds. The relatively low deck stiffness compared to other (non-suspension) types of bridges makes it more difficult to carry heavy rail traffic where high concentrated LL occur. Loads calculation considerations will follow the AASHTO, BS 5400, and the Iraqi code.

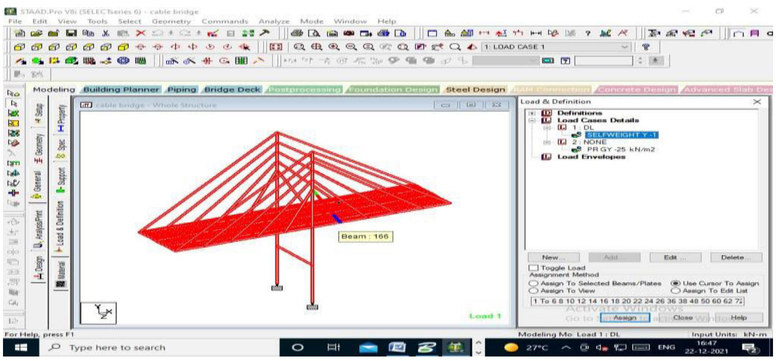

Figure 5.3: Assigning loads & load combinations

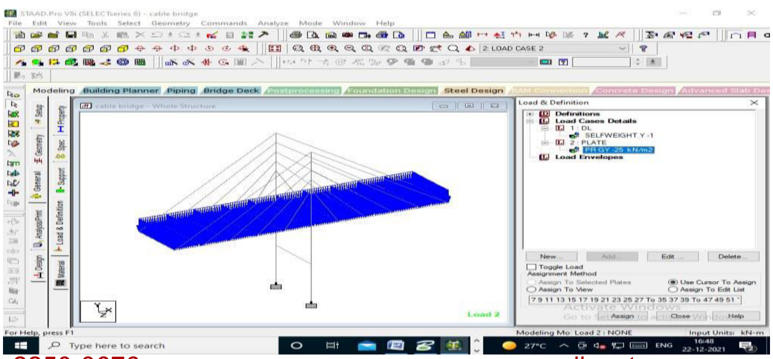

Figure 5.4: Applied loads 22

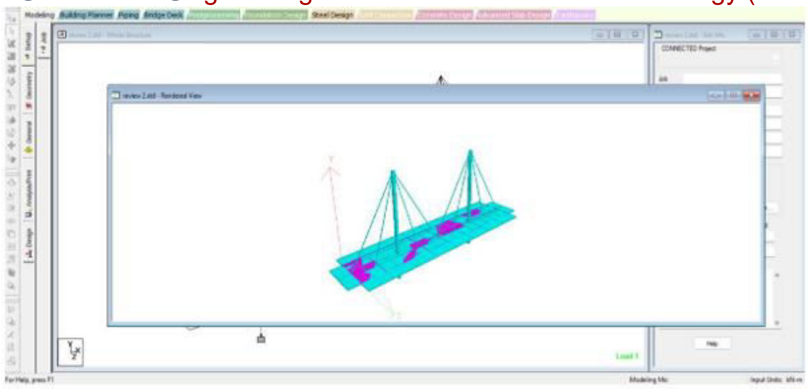

Figure 5.5: 3D view of the bridge

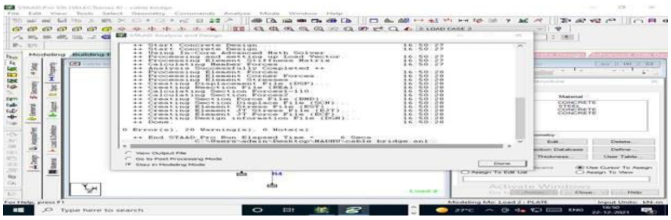

Figure 5.6: Analysis of the Bridge

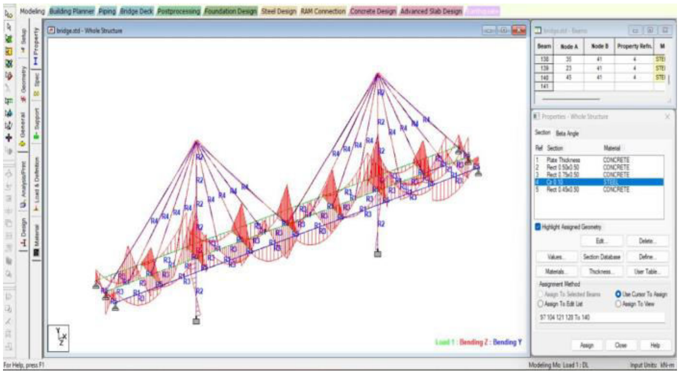

Figure 5.7: Bending moment in the bridge (My)

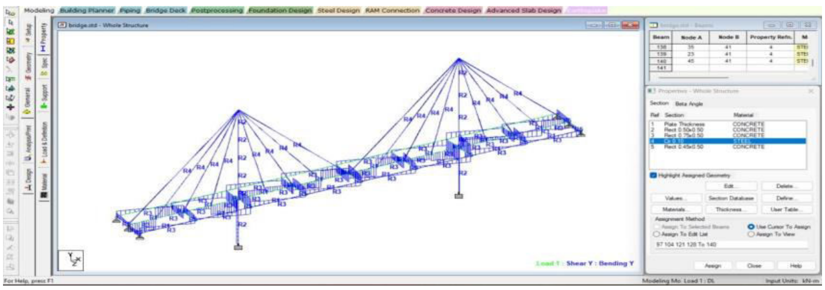

Figure 5.8: Shear Force induced in the bridge (Fy)

CONCLUSION

In conclusion, the analysis and design of the cable stayed bridge using STAAD Pro have been successfully carried out, taking in to account the specified parameters. Through meticulous analysis and precise design calculations, the bridge slab has been engineered to withstand anticipated loads and environmental conditions, ensuring safety and durability for its intended lifespan. The comprehensive approach in analysis and design using STAAD Pro ensures the efficiency and reliability of the cable stayed bridge structure. Furthermore, the utilization of STAAD Pro facilitated through analysis and optimization of the cable stayed bridge slab, ensuring its structural soundness under varying loads and conditions. By adhering to industry standards and incorporating innovative design techniques, the bridge slab has been engineered to effectively distribute stresses and enhance overall performance, thereby contributing to its longevity and safety.

In order to provide appropriate design service as a bridge designer, it is essential to develop a comprehensive understanding and knowledge on typical bridge types, common bridge construction materials and cross sections and available construction technologies. These knowledge's combined with the well-established engineering toolbox is valuable especially in the early conceptual design stage to determine the most appropriate bridge design proposal for each site. Having a well-established knowledge on the above-mentioned area can greatly reduce the time of conceptual design phase, improve the feasibility of the conceptual design, potentially lower the project cost, as well as optimize the final output of the project.

From the analysis it can be observed that bending moment and shear force obtained. A 2D model can be effectively used for analysis purpose for all the loading condition mentioned in IRC: 6 and Directorate of bridges & structures (2004), "Code of practice for the design of substructures and foundations of bridges" Indian Railway Standard. Further research is needed to verify the use of 2D model for different parameters such as dynamic analysis, soil structure interaction

REFERANCES

i. IS 456-1978, design aids for reinforced concrete, SP16

ii. Hassan I. A. Hegab, "Energy analysis of cable stayed bridges" Journal of Structural engineering, ASCE, Vol. 112, Jan., 1986, pp.1182-1195

iii. SK. Duggal, "Design of steel structures", third edition by the McGrawHill Companies, Tenth reprint 2011

iv. N. Krishna Raju, " Design of bridges", Oxford Publication pvt.ltd, fourth Edition, 2010

v. IS 875 (part -3) -1987

vi. OlfatSarhangZadeh, "Comparison Between Three Types of Cable Stayed Bridges Using Structural Optimization" (Thesis Format: Integrated Articles) 2010

vii. https://noppa.aalto.fi/noppa/kurssi/rak- 11.3001/materiaali/Rak-11_3001_lecture_07_2.pdf

viii. M. Hassan. Optimum Design of Cable-Stayed Bridges. PhD thesis, The University of Western Ontario, 2010.

ix. John. Wilson and Wayne Gravelle. Modelling of A Cable-Stayed Bridge For Dynamic Analysis. Journal of Earthquake Engineering and Structural Dynamics, 20:707–771, 1991.