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3D FE modeling of cable-stayed bridge according to ICE code

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Abstract: This article presents the performance of cable-stayed bridge by representing a model using the ANSYS program under the influence of a concentrated load and then comparing them with the experimental results. One model was used in the analytical, and one was (harp type 1) used in the length of the girder (4,500 mm), height of the pylon (1,480 mm), and depth of the flange for the girder (40 mm). If cable-stayed bridges are constructed using the balanced cantilever method, the stability problem of the girder is more significant during the construction stage than at the initial state. In this study, to investigate the ultimate behavior of cable-stayed bridges, experimental and analytical studies were conducted for one model: Harp type 1. At the limit state, several plastic hinges occurred and the girders buckle along the entire span. Numerical analysis was conducted for the experimental model, and the results of which showed good agreement with the experimental results. The influence of such kind of effects on the analysis and the structural behavior of cable-stayed bridges has been examined in detail in the study. The dimensions of the model were also compared with institute civil engineering Code in terms of the height of the tower to the main space, and Back Span to the main span ratio. It was close to standard specifications. A finite element procedure for the nonlinear analysis of cable-stayed bridges is first set up, and then, a detailed static deflection analysis of such bridges is carried out.

Keywords: FE modelling, cable-stayed bridge, ICE code

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1 Introduction

Although three-dimensional finite element (FE) modeling is the most time-consuming and demanding, it remains the most general and comprehensive technique for static and dynamic studies, covering all aspects impacting structural response. The other approaches were adequate, but their breadth and applicability were limited.

The method has become an important part of engineering analysis and design due to recent advancements in computer technology. FE computer programs are currently used in all branches of engineering. Bridges' behavior has also been successfully simulated using the FE method [1].

Wang et al. [2] developed a 3D FE model for the Sutong cable-stayed bridge (SCB) using ANSYS throughout their research. A subspace iteration method is used to investigate the bridge's dynamic features. Spatial beam elements (BEAM 4), a uniaxial element with tension, compression, torsion, and bending characteristics, are used to model steel girders, cross beams, towers, and piers. And also, they used LINK10 to represent the stay cables.

Ernst [3] provided a simple and effective method of simulating equal cable stiffness. Despite the fact that Ernst's method improves the efficiency of cable modeling, it is limited to extracting the response of the long-span bridge structure. It is impossible to ascertain the natural frequencies and vibration mode forms. However, when the deck girders are only concentrated in long bridge construction, this is still a highly typical strategy.

Based on the FE model, a method for simulating the static wind load was presented [4]. In the study, the Sogne Suspension Bridge in Norway was chosen, and the model was created using ABAQUS software.

Kim et al. [5] created a cable-stayed bridge analysis program and conducted a case study on the limited state of cable-stayed bridges. The structural stability of cable-stayed bridges is investigated in this work, employing geometric nonlinear FE analysis and taking into account various geometric nonlinearities such as cable sag, girder and mast beam-column effect, and huge displacement impact.

Xianlong et al. [6] used a numerical model to examine the impact of blast loads. The actual bridge, the Shanghai Minpu II Bridge, was chosen as the benchmark. An 800 kg TNT explosive charge was assumed to be placed on top of the bridge deck. For blast load simulation, the ANSYS AUTODYN computer program (2007) was utilized. The stress concentration caused by the blast load was found to be limited to the explosion location, and cracks were found to be scattered in tiny areas.

The response and failure manner of a bridge deck subjected to blast loadings were studied by ref. [7]; to simulate blast loadings, a nonlinear FE model was used to generate a typical orthotropic deck for cable-stayed bridges.

2 Description of collecting experimental bridge models

This study's experimental model is a reduction model based on Wang's model [8], which is a two-dimensional model with two pylons (61 m per pylon) and three spans (total length of 610 m); the experimental model is built at a scale of (1/68). The length of both the girder and the height of the pylon are 4,500 and 1,480 mm, respectively, according to the law of similarity, as shown in Figure 1.

Arranged in a fan system, the cables are made of steel wires. Every model has 20 cables, which are made of seven-strand wires with a diameter of 6 mm. Every cable is anchored end to end at the deck. The deck for the Harp type 1 models, the smallest size available ready-made channel section (C75 × 40), is chosen where it consists of two flanges. Each one has dimensions (40 mm × 8 mm), and one web has 75 mm × 5 mm dimensions. In addition to the total length, the girder is 4,500 mm. Cables withstand all loads operating on the girder and carry them into the ground via the pylons. The bridge has two steel pylons with a height of 1,480 mm to support the vertical reactions. Every pylon consists of T-section (T50 × 100) where the flange has dimensions (40 mm × 8 mm) and the web has dimensions (75 mm × 5 mm). At the base level, the towers are supported by a fixed wall.

Lee et al. [9] detailed the test findings of cable-stayed bridge model. The geometrical arrangement of the bridge model in combination with boundary conditions is shown in Figure 2. In this work, the numerical evaluation is used to replicate the behavior of giving a model. The steel pylon's height was 1,480 mm, as well as the web and flange thicknesses were 6 and 8 mm, respectively. The girder's length is 4,500 mm, and his width is 40 mm

3 Material properties

Yield stress and the tensile strength of the steel used in the construction of the two pylons and girder portion are 240 and 500 MPa, respectively. Steel's elastic modulus is 200 GPa. Cable elastic modulus used to build the stays is 205 GPa, and the cross-sectional area for the strand used to build the stays is 197.82 mm^2.

4 Experimental model loading

An actuator applied one concentrated load at the girder bridge's mid-span at a distance of 300 mm from the termination point. Figure 3 shows a testing system with a capacity of 5,330 N. The mid-span was subjected to static tests in order to observe the model's elastic behavior.

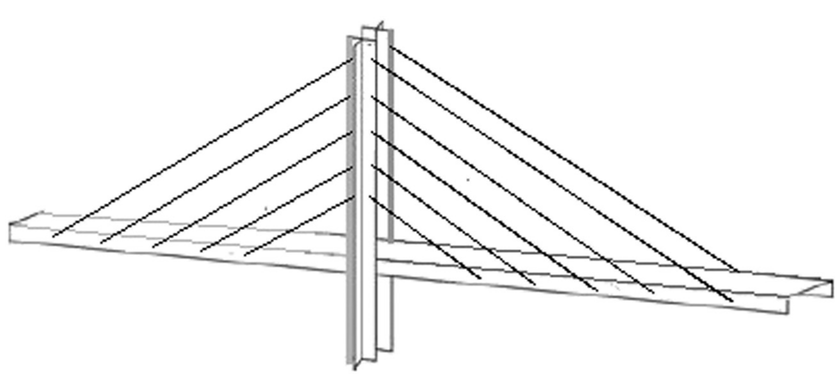

Figure 1: The concept of the model.

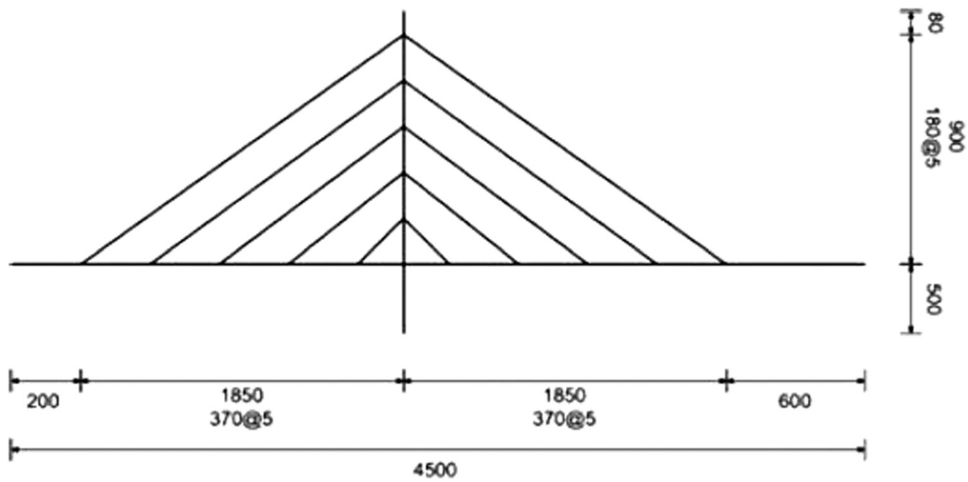

Figure 2: The geometrical configuration of the bridge model.

The displacements of the mid-span point were measured using linear variable differential transformers (LVDTs).

5 FE models

Pylon or tower, deck, and stay cables are the three primary components of cable-stayed bridges. Three-dimensional line elements are used to model these three components. A three-dimensional linear frame element is used to simulate both the deck and the pylon. The cables are also modeled using a three-dimensional non-linear cable element. The modeling of these components is detailed in the following sections.

A commercial FE program called ANSYS was used to simulate cable-stayed bridge models. Assumedly, the pylon and the girder were connected by full interaction in the middle, and each section of the pylon was linked to the girder. Figure 4 presents the FE model by ANSYS program, which is developed using solid elements for both pylon and girder sections and link elements for cables. In this model, the constrained load is used, as illustrated in Figure 5. As illustrated in Figure 6, the nodes' vertical translation degrees-of-freedom (DOF) are linked over the deck's breadth. Coupling is a method of assigning the same DOF value to a collection of nodes.

Since the model of the bridge consists of two sections, using 3D brick elements would not necessitate a time-consuming modeling process. Model 1 is built with eight-node brick elements to evaluate the performance of this modeling technique against solid models and test results. Solid components still depict the steel pylon and girder part of the bridge as in model 1. Brick elements are only utilized to portray the steel pylon and deck portion of the bridge.

In this study, each flange of the girder is divided into 46 elements and the web is divided into 138 rectangle elements, while each web was divided into 15 elements in every pylon and every flange in every pylon was

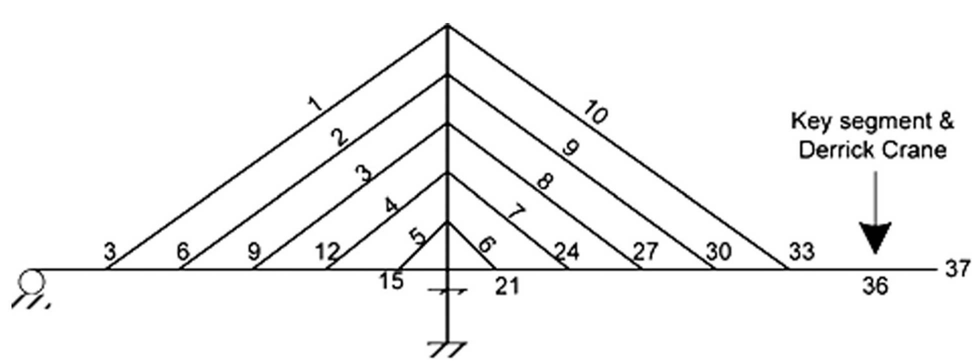

Figure 3: Bridge loading.

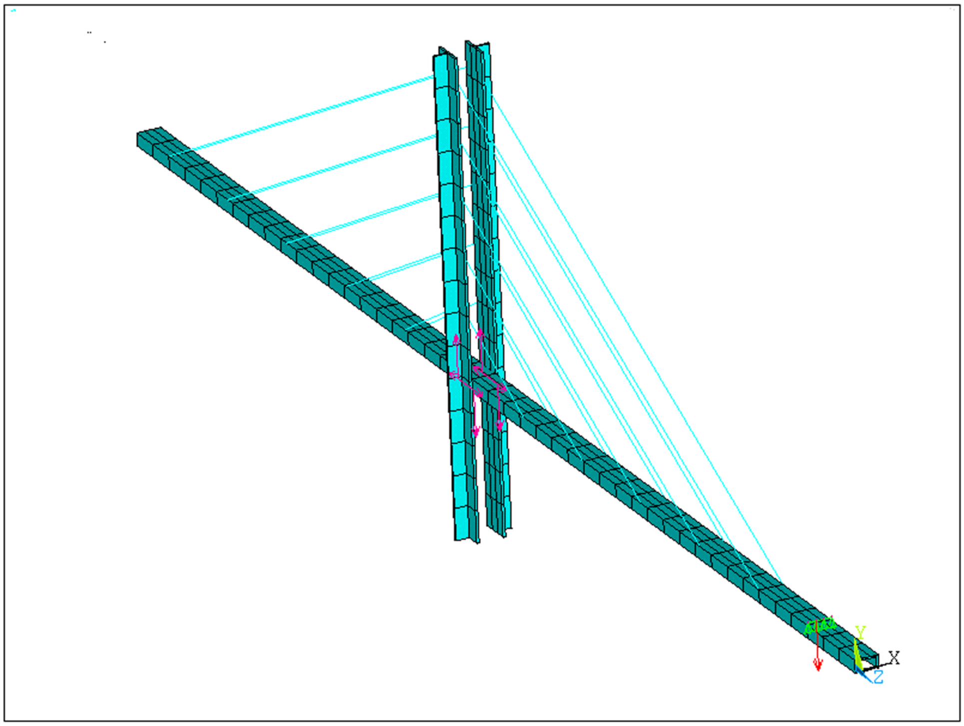

Figure 4: FE model by ANSYS program.

divided into 45 elements. Also, each cable was considered as one element, as the total cable became 20 elements; 185 solid elements are used to model the pylon and girder in three dimensions. They are made up of eight nodes, each with three DOFs. The link180 element is used to simulate a cable constraint between pylons and girders, which are utilized to transfer forces and moments in all of the previous models. The boundary conditions are

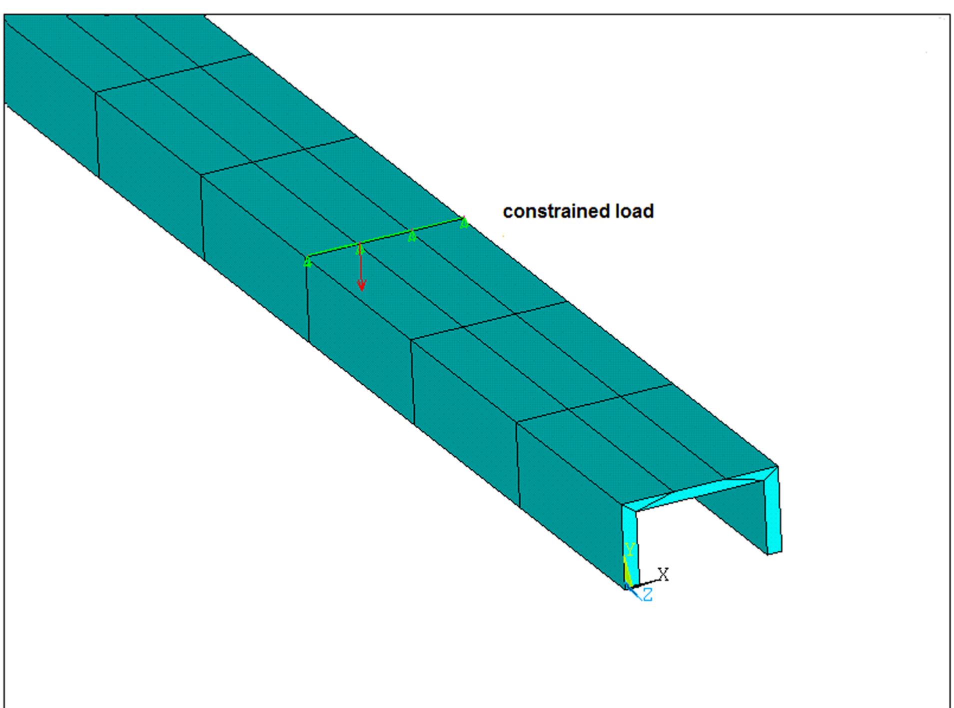

Figure 5: Point load is applied in FE model.

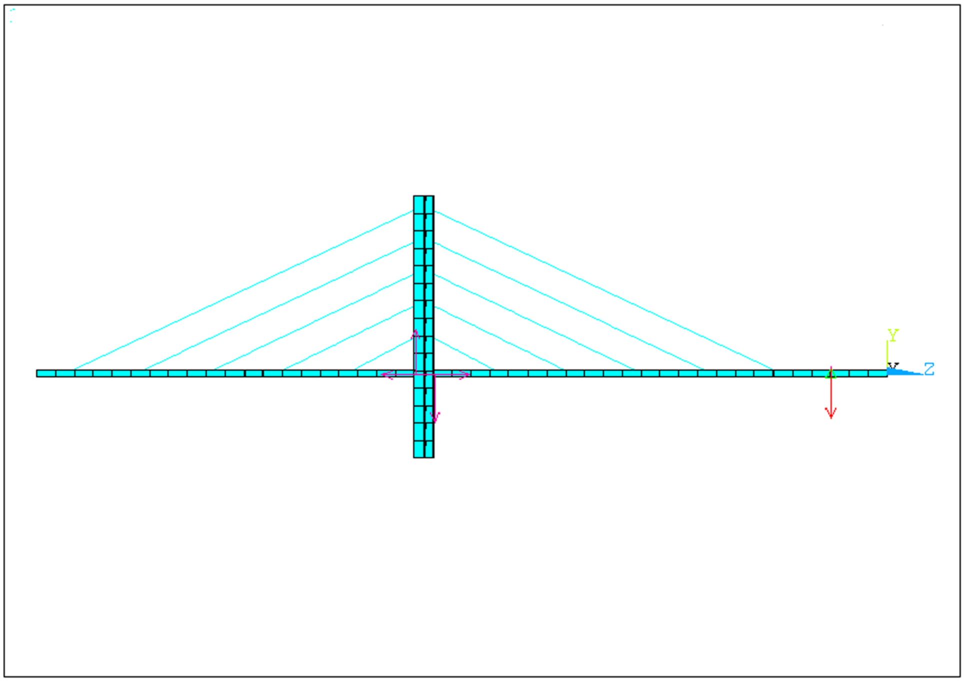

Figure 6: Coupling technique for making a group of nodes.

handled in a way that they replicate the test specimen's simple supported circumstances.

6 Ratio of back span to main span according to institute civil engineering (ICE) code

In the case of the model that was represented, the ratio of the backspace to the main space is 0.418, which is considered less than 0.5. Therefore, the model is considered within the limits of the specification set by the ICE code.

7 Pylon height according to ICE code

As shown in Figure 7, the ideal pylon height above the deck (H)-to-main span (L) ratio (H\L) is between 0.2 and 0.25.

In the case of the two models that were represented, the ratio of the height of the tower to the main span is 0.3, so it does not cross the limits of the ideal ratio that is recommended to be taken into account during the design of cable suspension bridges in the ICE code.

8 Numerical results and discussions

Table 1 and Figures 8–10 show the results of the finite element analysis (FEA) employing SM (standard model), DF (girder's flange depth), and P (constrained load) along with the results of the loading tests and the design values. The stress and vertical displacement obtained in

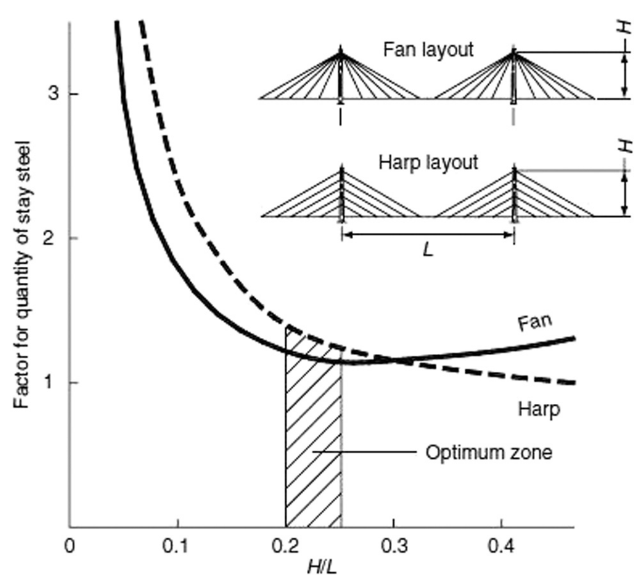

Figure 7: Optimum pylon height.

Table 1: Deflection results for FE models

Model Name	Constrained load (N)	Girder's flange depth (mm)	Mid-span deflection (analytical) results (mm)	Mid-span deflection (experimental) results (mm)
Harp type 1	5,330	40	224	245

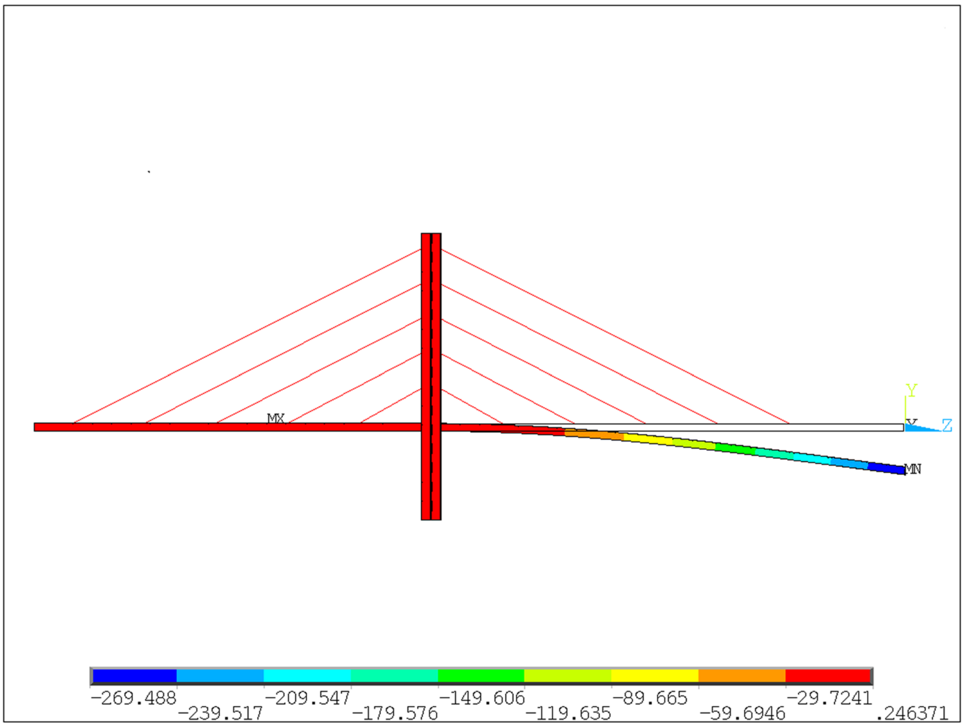

Figure 8: Deflection of numerical model Harp type 1.

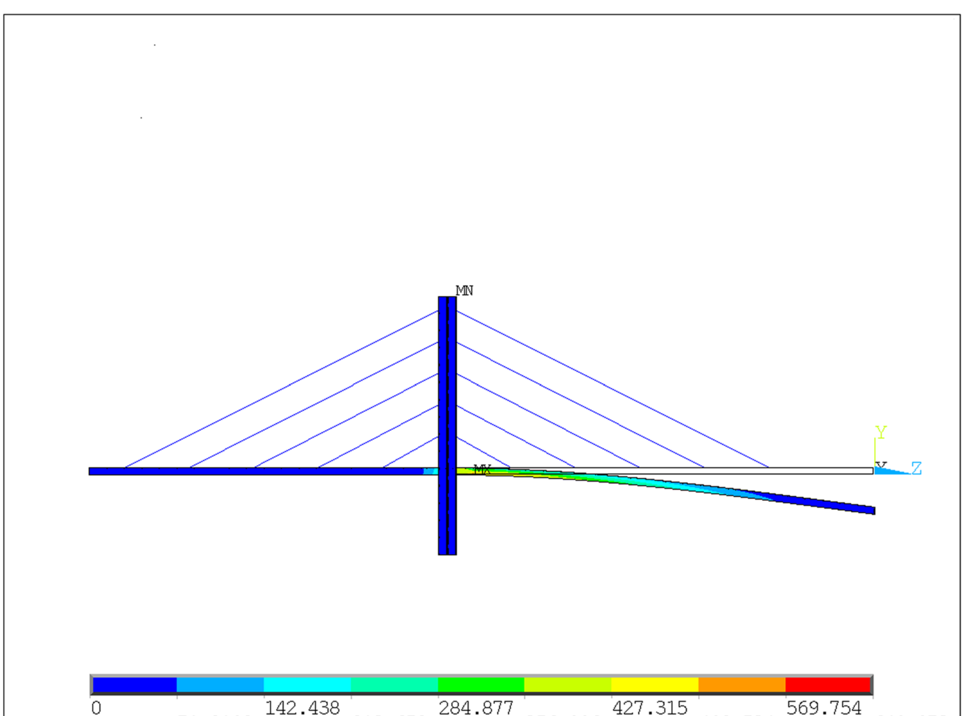

Figure 9: Stress of numerical model Harp type 1.

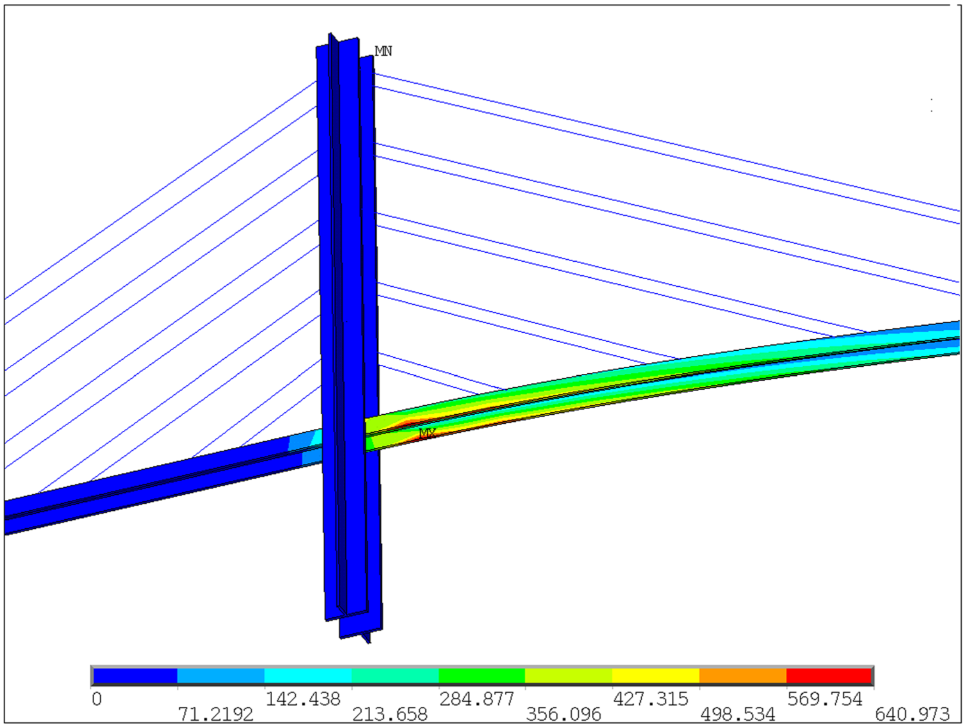

Figure 10: Maximum stress of numerical model Harp type 1.

the loading test are found to be overestimated by the design analysis.

A cable-stayed bridge's behavior may be determined by using displacements and stresses obtained from FE models. It is also possible to compare stress profiles with it. The cable-stayed bridge's flexural stiffness demonstrated linear elastic behavior in the elastic range of loading. The mid-span deflections in the studies were compared to the test findings. The mid-span deflection during the experiment was 245 mm, while it was 224 mm in an analysis done at a load of 5,330 N for model 1 by using ANSYS program. Figures following show the deflection results obtained from FE models with various parameters, which are summarized in Table 1.

Through the comparison between the experimental and theoretical results in the above table, we notice that there is an acceptable and slight error rate, which indicates that the theoretical analysis using the ANSYS program can be relied upon to obtain a rough impression and perception of the behavior of the analyzed model.

By analyzing Figure 8, we note that the cable-stayed bridge acted as a free-moving end, as the maximum and largest displacement was at the end of the free end. While in Figures 9 and 10, we also find the maximum stress we find in the near end of the pylon, and we conclude from that that the cable-stayed bridge acted as a cantilever.

9 Conclusion

This article develops a fundamental grasp of cable-stayed bridge systems. The results of fundamental behavioral patterns of cable-stayed systems for static loads are presented in this article. One type of cable-stayed bridge models is studied: harp type 1. The bridge's performance is assessed using the one measure.

The performance of a cable-stayed bridge with a long span is enhanced by adding additional depth to the flange of the girder, according to analytical results derived by the simulation program ANSYS, which employs the FE approach in the study.

The installation of additional slanted cables is beneficial as the main span lengthens. In contrast, adding cables to a short-span bridge does not greatly increase its performance. The highest stress is found near the junction of the tower and the girder, according to the findings.

The ideal number of cables is determined by the S/M ratio of the side span to the main span. Furthermore, the study reveals that the towers' height is determined by the main span's length rather than the number of cables in the system.

For the model where the dimensions are 4,900 mm for the main span and high of pylon 1,480 mm, the ratio

(H/M) is 0.3. Therefore, this ratio is not considered within the limits of the ideal region according to the code, Because, according to the ICE code, the ideal ratio of tower height to main span (H/M) for the two models is between 0.2 and 0.25, as detailed in Chapter 3, the optimal ratio of tower height to main span (H/M) for the proposed model is between 0.2 and 0.25. The optimal H/M ratio does not change much as the number of cables increases. Because the cable angle is excessively severe when the tower is short, the bridge does not operate efficiently. Since the towers are much too flexible to sustain the span when they are tall, the bridge's performance suffers.

Also, in the case of the two models that were represented, the ratio of the backspace to the main space is 0.418, which is considered less than 0.5. Therefore, the two models are considered within the limits of the specification set by the ICE code.

As engineers and even the general public get a better understanding of cable-stayed bridges, more of them will be built in the future. The current limit will almost certainly be pushed as the trend toward a larger primary span continues. With cable-stayed bridges, the greatest possible main span is presently projected to be around 1,200 m. Composite materials plus improved computer technology will undoubtedly enable softer systems to be built at a lower cost and in a shorter amount of time.

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