Study on Performance of Cable Stayed Bridge with Different Geometric Conditions

[1]Shaikh Azan [2]Prof.Sharat Chowka
[1]M. Tech Scholar,
[2]Assistant Professor Department of Civil Engineering, PDA College Of Engineering, Kalaburagi, Karnataka, India

Abstract: *The construction of cable-stayed bridges has been on the rise in recent years, driven by their aesthetic appeal and unique structural design. This research focuses on the analysis of cable-stayed bridges, The study employed the MIDAS CIVIL software to model and analyse the cable-stayed bridges of two types H-shape and A-D shape, with all the bridges having the same material and section properties. For the seismic analysis time-history analysis has been performed by utilizing data from the 1940 El Centro earthquake, allowing for the investigation of the dynamic behaviour of the bridges. The cable-stayed bridge design is characterized by the transmission of the deck's reaction forces to the pylons, which in turn transfer the load to the foundation. The study reviewed a range of evaluations, including axial forces, displacements, and bending moments, to understand the structural behaviour of the cable-stayed bridges. The results reveal that as the complexity of the cable-stayed bridge design rises, the structural behaviour becomes more complex. The findings of this research contribute to the understanding of the dynamic response of cable-stayed bridges, which is vital for their structural health monitoring and design optimization.*

Keywords: Cable stayed bridge, Unknown load optimization, Deck Width, Construction stage Analysis, Time History Analysis.

1. Introduction

Cable-stayed bridges have emerged as iconic structures, blending engineering with architectural elegance. These structures rely on a delicate balance of forces to maintain stability and functionality. Among the key components influencing their behavior, the design of the pylon stands as a crucial element, particularly under dynamic loads such as wind. The girder (deck), tower (pylon), and cables are the three primary subsystems that together make up the structural system of cable-stayed bridges. A precise balance of forces is necessary for these structures to continue being stable and useful. The design of a pylon is one of the main factors affecting its behavior, especially when dynamic stresses like wind are present. When the load is conveyed to a pylon and then to the piles, cables function as a tension-resistant structural element. This is the fundamental concept of cable-stayed bridges.. As compared to suspension bridges, the main factors are the attractive aesthetics, the shorter construction time, the effective use of materials for the building structure, the light appearance, and most significantly, the increased stiffness. Structures with these characteristics often have a long-life span, a high degree of stability, are light in weight, and have low structural damping.

To determine the most effective pylon design type, three scenarios are compared based on shear force and bending moment in terms of self-weight. The conclusions thus obtained are helpful in reducing the disadvantages of alternative pylon styles. [1] Farhan Farid Reshi he has done research on bridge using Staad pro to identify the dynamic behavior of cable bridge with respect to wind load in zone 2 and 5[2] Ahmed

M. Khaled F. studied the seismic performance performance of bridge with different pylon conditions and considering different deck width for same loading conditions using MIDAS Civil and determining the effective pylon design [3]. Umang A. Koyani, Kaushik C. Koradia [4] did a parametric study of a three span, two plane cable-stayed bridge with a box girder deck. The various parameters were considered for analysis of cable-stayed bridges; those are side span to main span ratio, upper strut height, cable system, number of cables per plane and cable diameter. MIDAS CIVIL analyzes the impact of the above parameters on the girder's maximum girder moment, deflection, shear force, and axial force. It was found that with the increase in side to main span ratio maximum moment is decreased up to a certain limit and then increases. With increase in number of cables maximum moment in girder decreases. Marko Justus Grabow [5] has given a detailed methodology that is to be followed in MIDAS CIVIL for modeling and analyzing the overall construction process of a cable-stayed bridge. An example of a construction stage analysis is provided in detail for the Second Jindo Bridge, Korea. Various analysis features on MIDAS CIVIL are also verified in the thesis Merin Mathews, Silina joseph intended to examine a three-span bridge and investigated the relationships between several factors, including as displacement, shear force, and bending moment.[6]. A study is carried out which focuses on the effect of the shape of the pylon on the seismic response of cable-stayed bridge. For this study, complete geometry, material properties, loads and boundary conditions of the Quincy Bayview Bridge are considered from the past published literature. A dynamic analysis was carried out in which several pylon configurations were used to create and assess the cable stayed bridge using the MIDAS CIVIL program. In the seismic analysis section, the nonlinear dynamic behavior of bridges was examined using data from the 1940 El Centro earthquake and time history analysis.[7] Generally, software like ETABS, Staad V8, SAP2000 and MIDAS CIVIL are used. In the project MIDAS CIVIL is used to analysis the bridges.

2. Objective of Study

The Objective of this study is to:

- To assess the effect of shear force, bending moment, and maximum deflection of the cable-stayed bridge
- To gain insight into how change in span and deck width affects loading performance.
- To investigate performance of bridge under Seismic loads.

3. Methodology

In this paper analysis of three span double plane cable stayed bridge is carried out. This study analyzes a three-span double plane cable stayed bridge using MIDAS CIVIL software to perform an effective structural linear analysis, where the moving load on the bridge is defined as IRC class AA. Software automatically finds critical position of this loads and gives the result.Various parameters and its effect on maximum moment, maximum torsional moment, and maximum axial force, maximum shear force and maximum deflection in the girder.

3.1 Modelling and Analysis of Bridge

1. Generating model of cable stayed bridge with different type of pylons in MIDAS CIVIL
2. Defining materials and section properties of cable stayed bridge.
3. Assigning load in the model like self-weight, pretension cable force, vehicle load and seismic load.
4. Assigning vehicle definition by selecting vehicle database, provide IRC Class A wheel loading and IRC Class 70 R wheel loading.
5. Assigning all load combinations and time history data of El Centro earthquake.

6. After all, perform analysis of cable stayed bridge.

MODEL INFORMATION

Details of H-Shape Pylon Bridge Model for carrying out the analysis: -

S. No.	Parameters	Model 1	Model 2
1	Type of stay cables	Parallel wires	Parallel wires
2	Longest span	220m	300m
3	Total Length	400m	520m
4	Height of Pylon	90m	90m
5	Clearance below Cable stayed and sea level	25m	25m
6	Thickness of R.C.C. Deck slab	250mm	300m
7	Total Number of Pylons	4	4
8	Total Number of Cables	80	80
9	Deck width	15m	18m
10	Number of Lanes	2	2
11	Loading	I.R.C. Class AA tracked vehicle	I.R.C. Class AA tracked vehicle
12	Support at Footing	Fixed	Fixed
13	Support near Abutments	Roller	Roller

3.2 Parameters Considered

a. Side span to main span ratio- 0.55, 0.60

b. Number of cables per plane- 20

c. Cable diameter - 25cm

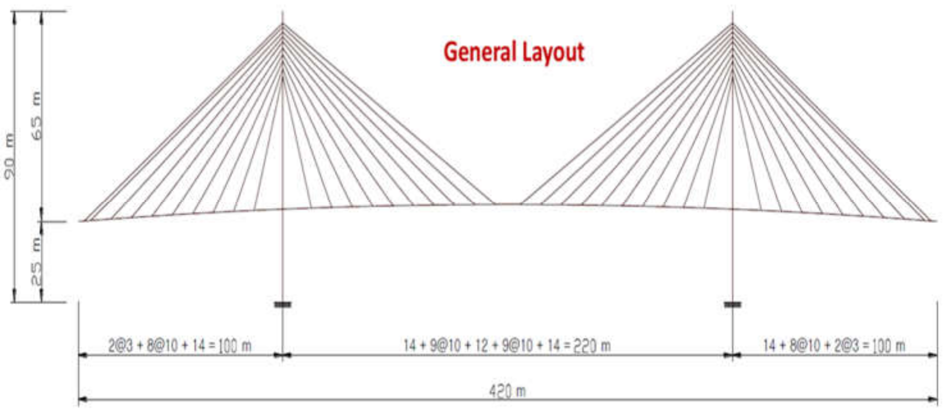


Figure 1. Bridge Layout

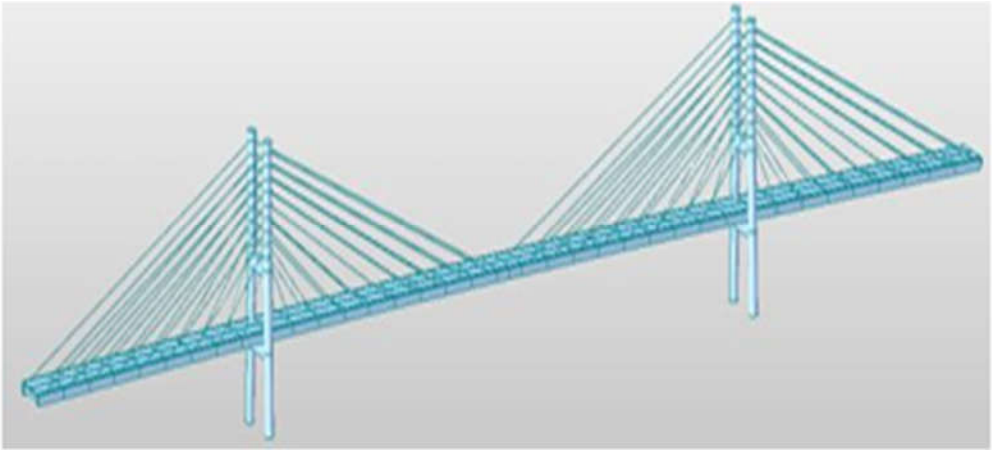


Figure 2. 3D view of bridge in MIDAS Civil

3.3 Unknown Load Factor Method (ULF)

The Unknown Load Factor Method in MIDAS Civil is a nonlinear analysis technique used to determine the critical load-carrying capacity of a structure. This method is used to find out the optimum post tensioning cable force for bridge using unit displacement. This function optimizes tensions of cables at the initial equilibrium position of a cable structure. The program can calculate the initial cable force by inputting the restrictions such as displacement, moment, etc. and satisfying the constraints.

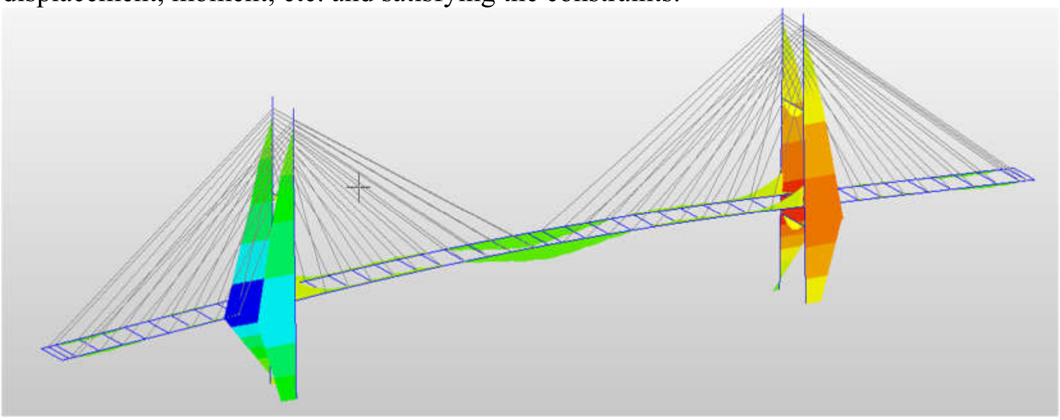


Figure 3. Bending Moment Prior Unknown-load factor optimization

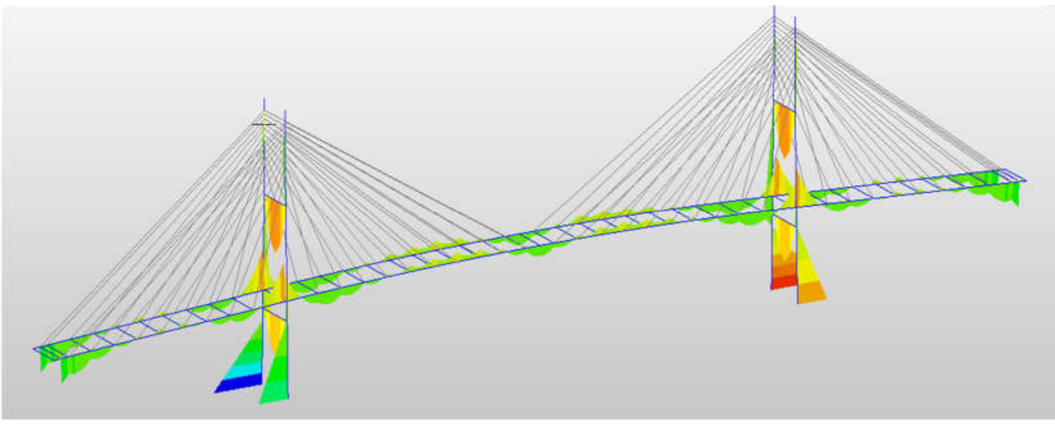


Figure 4. Bending Moment Post Unknown-load factor optimization

3.4 Construction Stage Analysis (Backward stage Analysis)

Comparably to the order of erection stages in the actual bridge construction, the structure is virtually disassembled stage by stage in the opposite way. Internal forces of the members are calculated in each erection stage of the backward analysis once girder segments or stay cable are released. The tension of a specific cable right before it is removed can be used to determine the cable's original stressing force when it was installed during the actual bridge construction. Not able to take into consideration time-dependent factors like as shrinkage and creep.

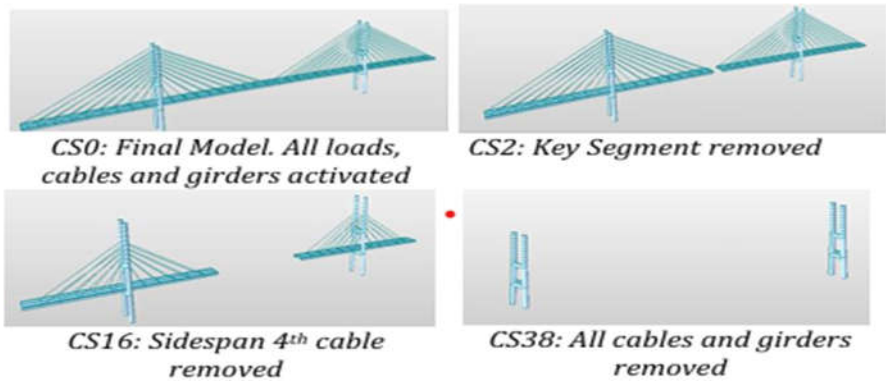
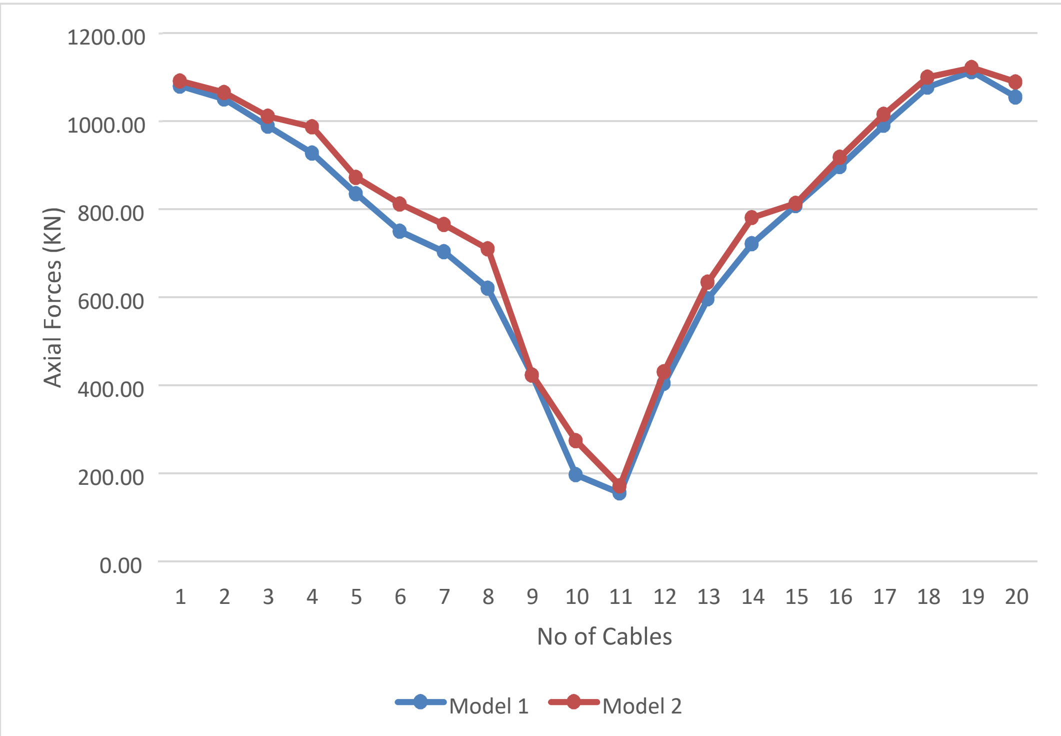


Figure 5. Backward Analysis Sequence

4. Results And Discussions

4.1 Axial cable Forces

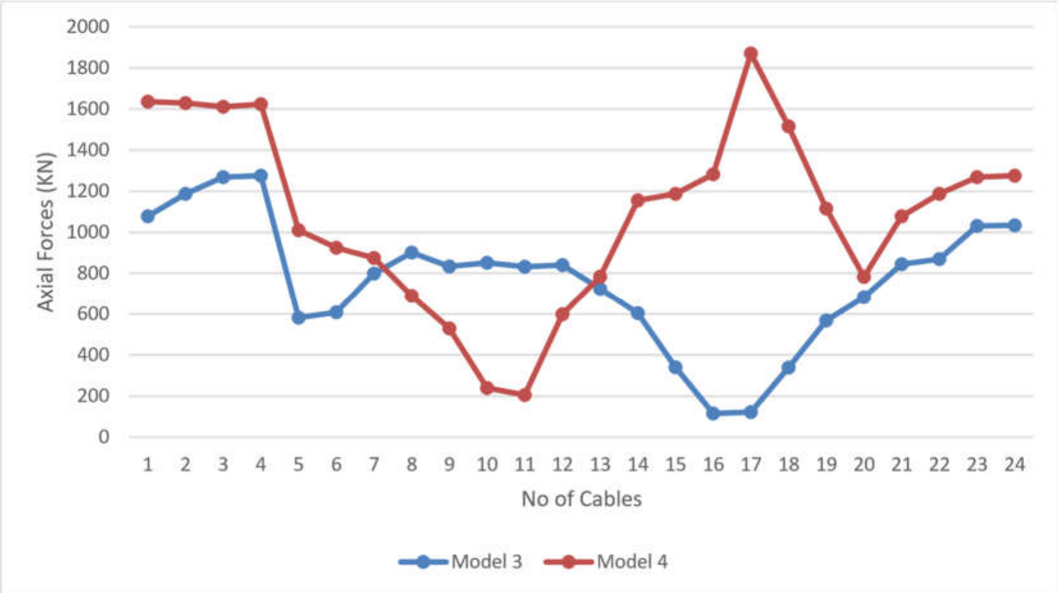
Axial Forces resulted from **H-Shape Pylon**



Graph 1. Comparison between Axial cable Forces

The Axial forces recorded in cables for both Model 1 and Model 2 show interesting trends and variations. Overall, the data indicates that the forces for most cables are higher in Model 2 than in Model 1, though the differences vary across the cables. As the cable numbers increase, the pretension forces generally rise in both models, with Cables 18 through 20 showing the highest pretension forces. Cable 19 stands out with a Axial force of **1112.97 KN** in Model 1 and **1121.90 KN** in Model 2, a marginal difference that suggests both models perform similarly for cables under higher loads.
In the first cable (Cable 1), the Axial forces for Model 1 and Model 2 are quite close, with values of **1079.79 KN** and **1091.72 KN**, respectively, suggesting a minor increase in Axial in Model 2. This slight increment is consistent across most cables, but a notable outlier is Cable 10, where the Axial force in Model 2 **274.22 KN** is significantly higher than in Model 1 **196.88 KN.** This sharp difference could indicate a potential issue with the calibration or design in Model 1, or an improvement in Model 2 ability to distribute forces more efficiently.

Axial Forces resulted from **A-D Shape Pylon**

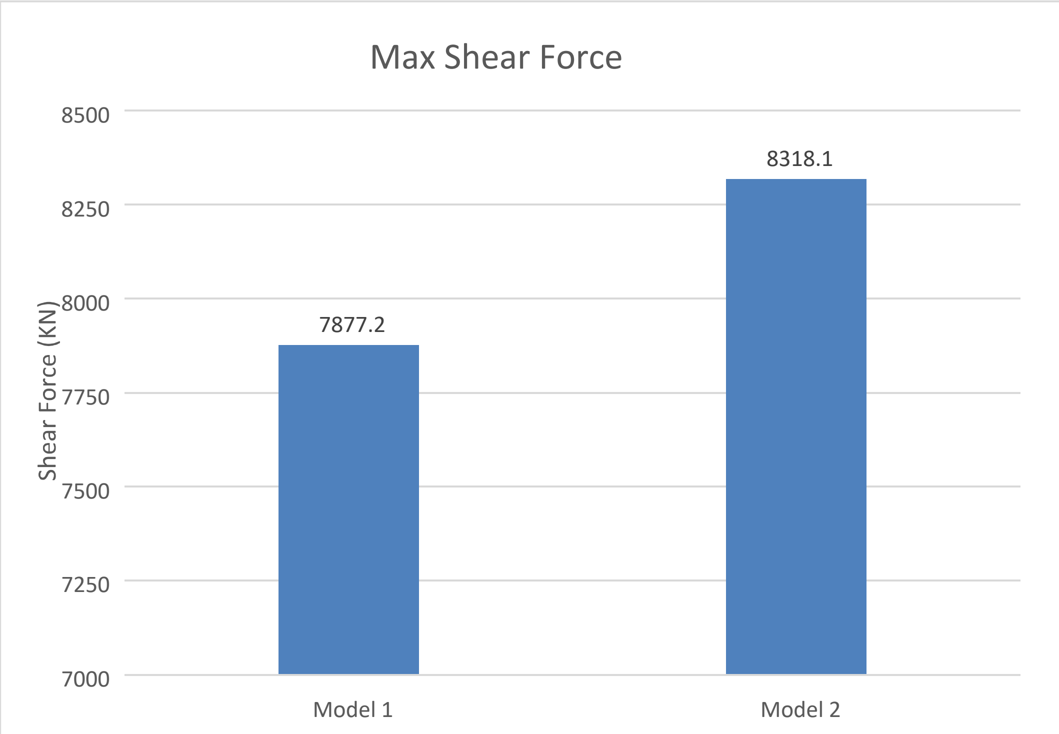


Graph 2. Comparison between Axial cable Forces

From results Model 3 generally shows higher performance values compared to Model 4 for most of the cables. For example, in cables 1 to 4, Model 3 has values ranging from approximately 1078 to 1275, while Model 4 ranges from 1611 to 1636. This indicates a consistent increase of higher values for Model 4 in these cases.
Notably, cables 15 to 18 show a large discrepancy between the two models, with Model 4 values exceeding Model 3 by a considerable margin. This may indicate that Model 4 has a different reaction mechanism or is more vulnerable to particular cable specifications. Overall, the observed variations could influence the selection criteria for these models based on specific application needs.

4.2 Shear Forces

Shear Forces resulted from **H-Shape Pylon-**

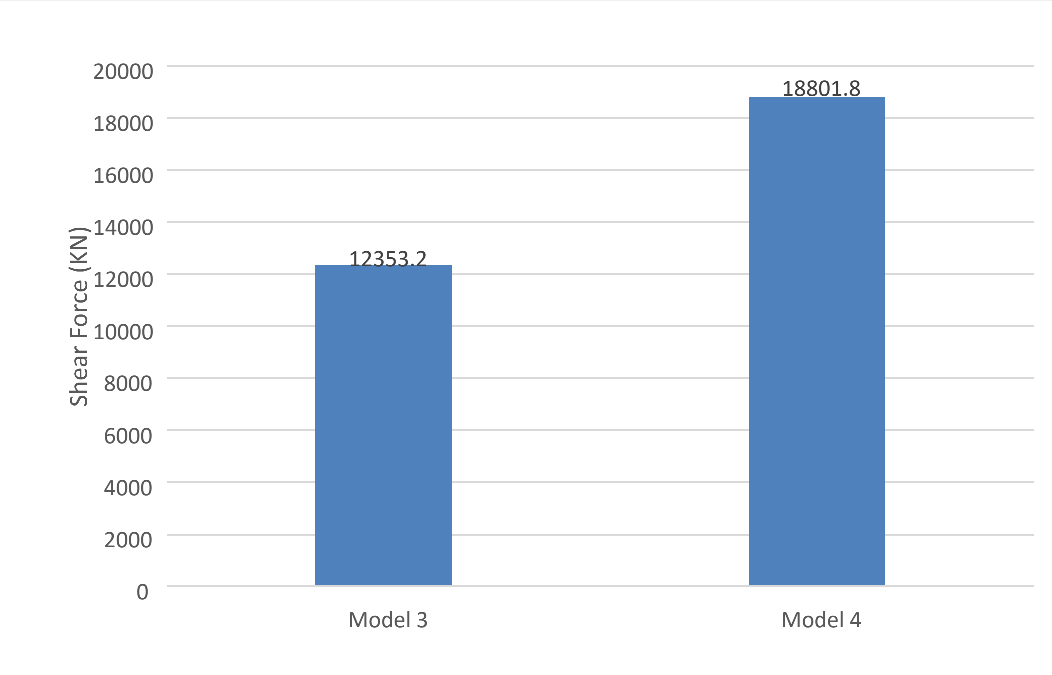


Graph 3. Comparison between Shear Forces

The higher shear force in Model 2 suggests that this model may be designed to handle larger loads or may be subject to different loading conditions compared to Model 1. This could be due to variations in the structural design, such as stiffer cables, tower height, or deck properties, all of which affect the distribution of shear forces across the bridge. The higher shear force in Model 2 might indicate that the forces in the cables are being transferred more effectively or that external loading conditions are more substantial. The difference in shear forces between the two models may also reflect changes in the distribution of live loads, wind loads, or dynamic factors like traffic. While Model 2 experiences a higher maximum shear force, the margin of **440.9 KN** is relatively small considering the total forces involved. This suggests that both models perform similarly under maximum shear force conditions, but Model 2 may offer enhanced resilience or capacity for additional load.

In conclusion, the higher shear force in Model 2 could imply a more robust design capable of handling greater loads, while the relatively small difference suggests both models are structurally sound under maximum load conditions. However, further investigation into the loading conditions and structural elements is necessary to fully understand the implications of the difference in maximum shear forces.

Shear Forces Resulted from **A-D Shape Pylon-**

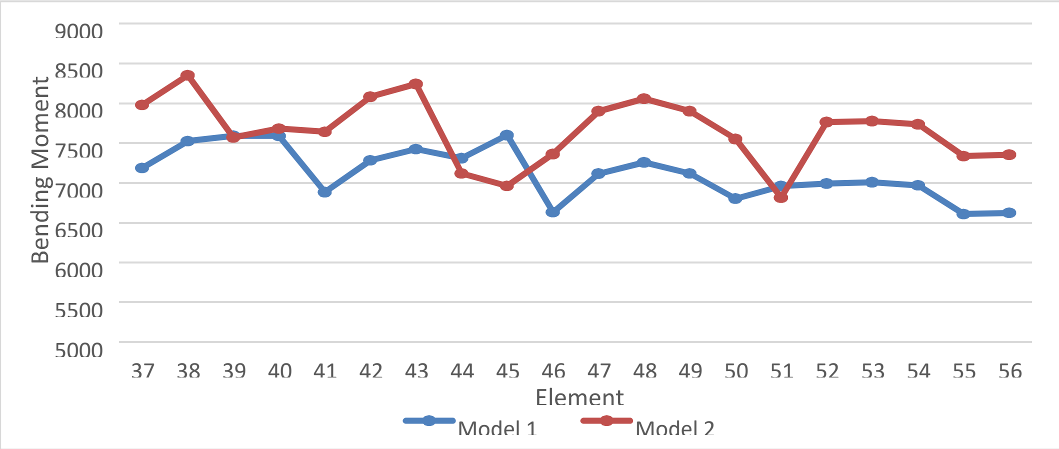


Graph 4. Comparison between Shear Forces

The results shows that Model 3 can resist a maximum shear force of 12,353.2 kN, whereas Model 4 can handle up to 18,801.8 kN, making its shear capacity 52% higher than Model 3. This significant difference could be due to several factors. Model 4 may utilize materials with higher shear strength or have a larger cross-sectional area in key load-bearing sections, improving its ability to resist shear forces. Additionally, Model 4 could feature better reinforcement techniques, such as more effective use of stirrups and ties, or an optimized design geometry to distribute shear forces more effectively. Due to these characteristics, Model 4 is better suited for applications that involve high shear forces, as it provides a greater safety and reduced risk of shear failure modes like cracking or sliding. On the other hand, Model 3 might still be appropriate for situations with lower shear demands or where weight and cost considerations are more important. In conclusion, Model 4 offers superior shear performance and reliability for high-load conditions, but further analysis of the materials, design specifications, and testing conditions is necessary to fully understand the reasons behind these performance differences.

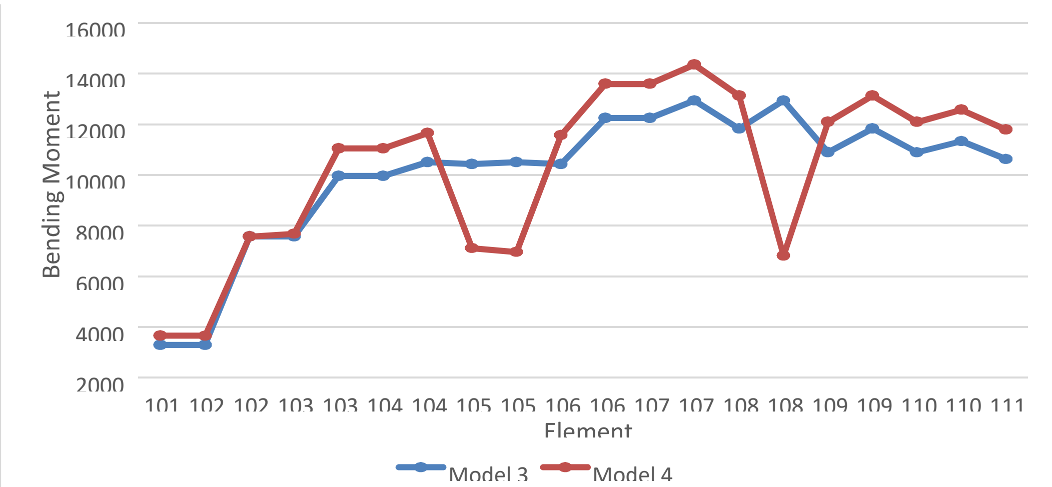
4.3 Bending Moment

Bending Moment Resulted from **H-Shape Pylon**



Graph 5. Variation between Bending Moment

The Results indicate variation in bending moments (in KNm) between two models (Model 1 and Model 2) across structural elements ranging from 37 to 56. Model 1 demonstrates a relatively stable trend, with bending moments consistently ranging between 6700 and 7500 KNm. There are minor fluctuations, but overall, Model 1 maintains a controlled performance. Notably, slight dips in bending moments are observed at elements 45 and 50, suggesting localized reductions in moment resistance, but the overall remains stable. A comparative analysis highlights thatModel 1 offers more predictable and steady behavior, potentially making it more suitable for applications requiring stability and less variability under load. Model 2, on the other hand, with its higher and more fluctuating bending moments, might be optimized for scenarios involving more dynamic or complex forces. Both models experience dips at elements 42, 45, and 50, indicating common points of lower moment resistance, possibly due to shared structural characteristics or design weaknesses. Overall, the more controlled performance of Model 1 suggests it may be better suited for applications demanding consistency, while Model 2, with its higher moment variability, might be designed to handle more variable or intense load conditions.

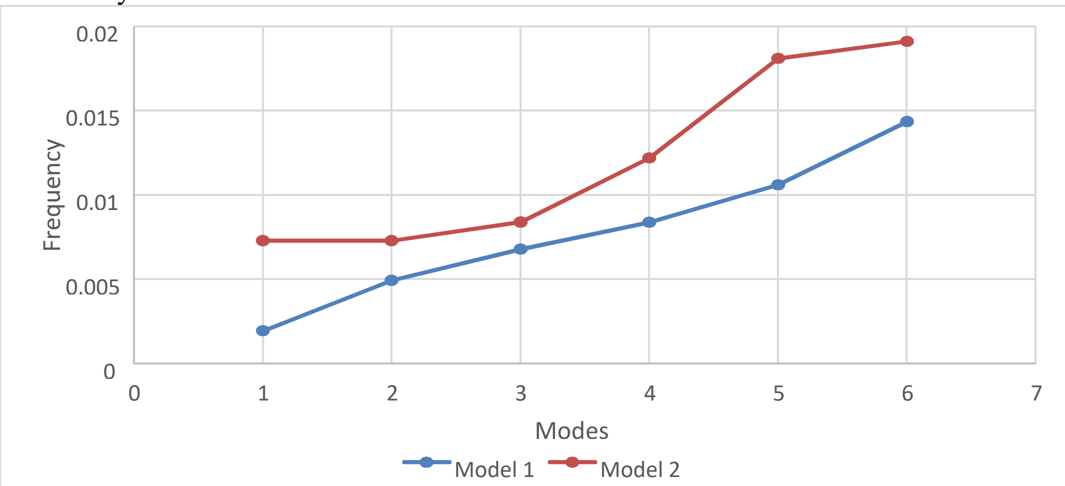


Graph 6. Variation between Bending Moment

In some cases, the bending moment values are nearly same between the two models. For example, in Element 102, both models show similar values Model 3: 7572.21 KNm, Model 4: 7571.35 KNm. This suggests that for certain loading conditions, both models behave similarly. Slight differences are seen in some Element. For example, in Element 105, Model 3 has a value of 10424.47KNm, while Model 4 shows a lower value of 7120 KNm, showing a reduction in the bending moment for Model 4 under this condition. Another difference is seen in Element 108, where Model 3 has a value of 12935.56, while Model 4 shows a much lower value of 6819. For some Element, the bending moment rapidly increases for Model 4. For example, in Element 106, Model 4 exhibits a value of 13595.60 KNm, which is significantly higher than Model 3's value of 12248.29 KNm. Similarly, Element 107 shows a large increase from 12935.56 in Model 3 to 14358.47 KNm in Model 4. In conclusion, Model 4 tends to produce higher bending moments in most categories, which could be due to differences in structural design parameters, load distribution, or material properties.

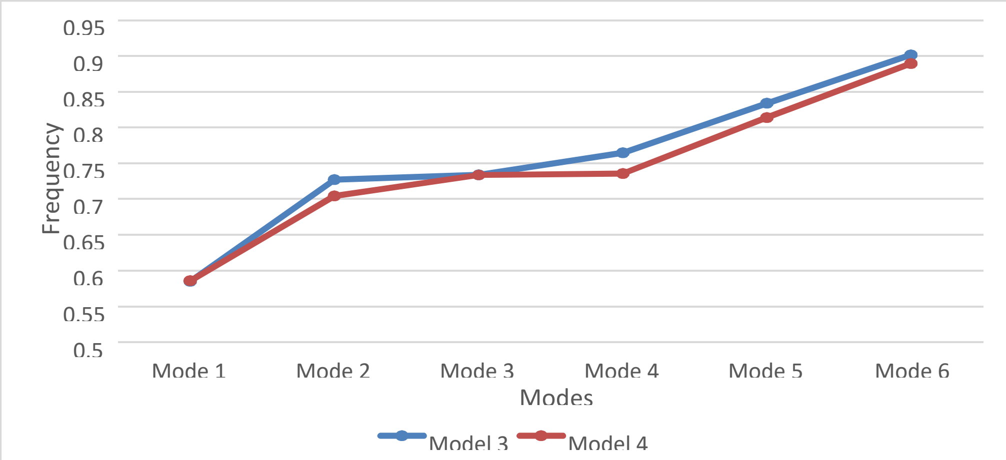
4.4 Time-History Analysis

Since cable stayed bridges have less structural dampening, their increased span raises many questions regarding how they will behave under dynamic loads like wind, earthquakes, and traffic from vehicles. Extreme loads are rarely applied to these bridges, unless there is a significant earthquake. To do the time history analysis, the El Centro, 1940 earthquake time history data is given. The earthquake was acting in all directions, allowing the bridge to be affected by its movements in different directions.



Graph 7. Frequency Mode for H shape Pylon

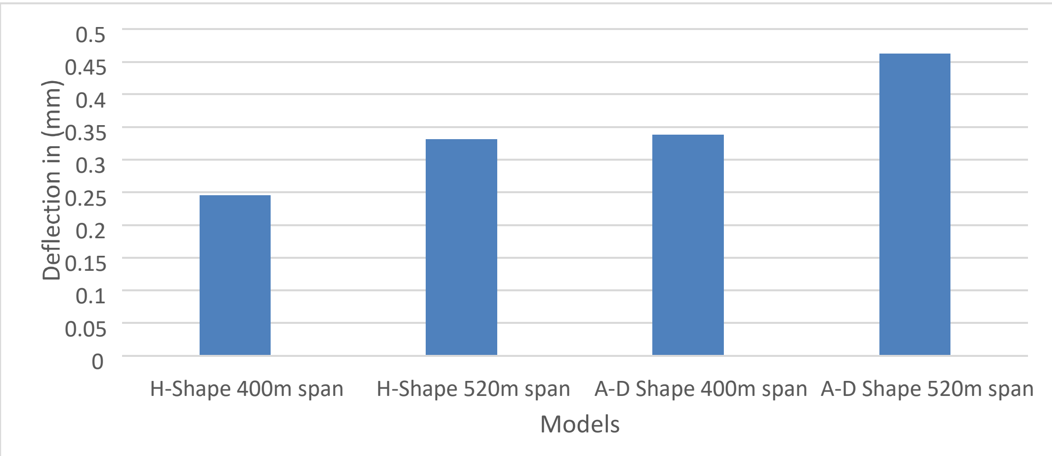
The comparison of natural frequencies and time periods between Model 1 and Model 2 reveals notable differences in their dynamic behaviours. Model 2 consistently exhibits higher natural frequencies and shorter time periods than Model 1 across all modes. This trend suggests that Model 2 is structurally stiffer or has a higher resistance to deformation.For instance, in Mode 1, Model 1 has a frequency of 0.001953 Hz and a time period of 512.007412 seconds, while Model 2 has a frequency of 0.007305 Hz and a time period of 136.894765 seconds. This difference indicates a significant variation in their dynamic responses.The implications of these differences are significant for applications requiring specific vibration characteristics. Model 1 might be preferable for scenarios where avoiding resonance is critical, whereas Model 2 could be advantageous in applications demanding a quicker dynamic response. In summary, across all modes, Model 2 consistently exhibits higher frequencies and shorter time periods compared to Model 1. This suggests that Model 2 operates at higher oscillation rates and faster cycles, which could be indicative of differences in system dynamics or parameters between the two models.



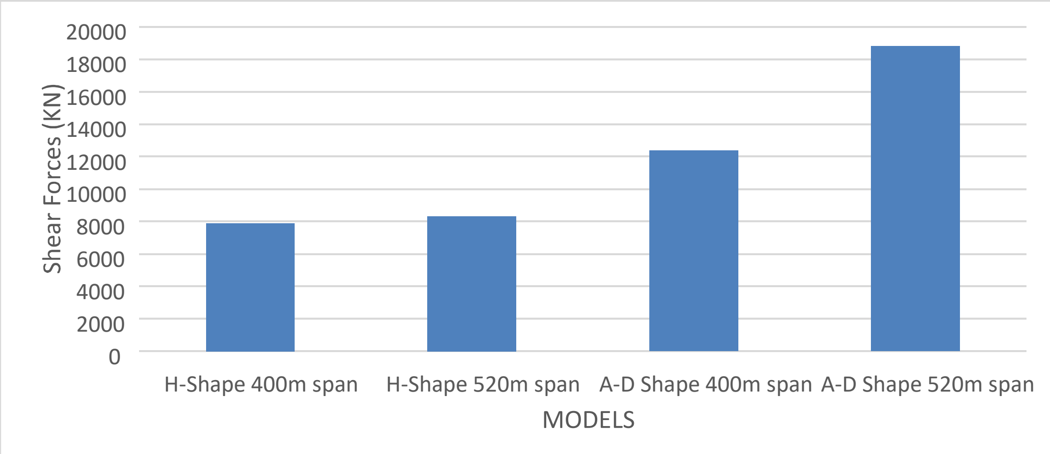
Graph 8. Frequency Mode for A-D shape Pylon

The frequencies and time periods of the two models Model 3 and Model 4 show very close results across all modes. For **Mode 1**, both models have nearly same frequencies (around 0.5855 Hz for Model 3 and 0.5863 Hz for Model 4) and corresponding time periods around 210 seconds, showing minimum variation. Similarly, **Mode 2** shows a slight difference in frequency, with Model 3 at 0.7275 Hz and Model 4 slightly lower at 0.7048 Hz, leading to a small difference in time periods 83.3 seconds and 81.6 seconds From **Mode 3 to Mode 6**, both models show consistent and small deviations in frequencies and time periods. For **Mode 3** shows identical time periods 61.27 sec for both models. **Mode 4** and **Mode 5** present more noticeable but still small variations in time periods, with Model 4 slightly differing from Model 3. Lastly, **Mode 6** also shows close values, but with Model 3 having a slightly higher frequency 0.9021 Hz compared to Model 4 0.8900 Hz. Overall, the results show that both models show close related frequencies and time periods.

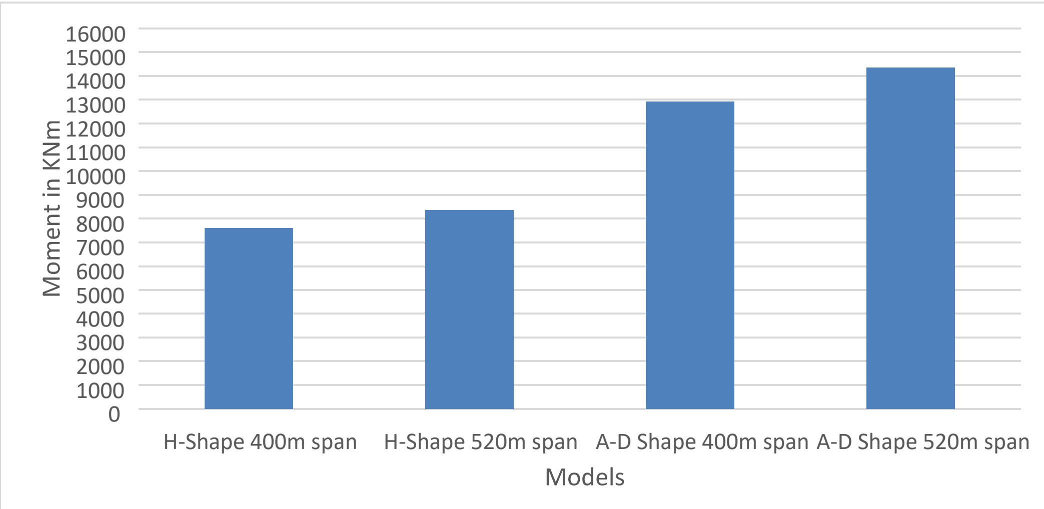
4.5. Comparison between Different Geometry



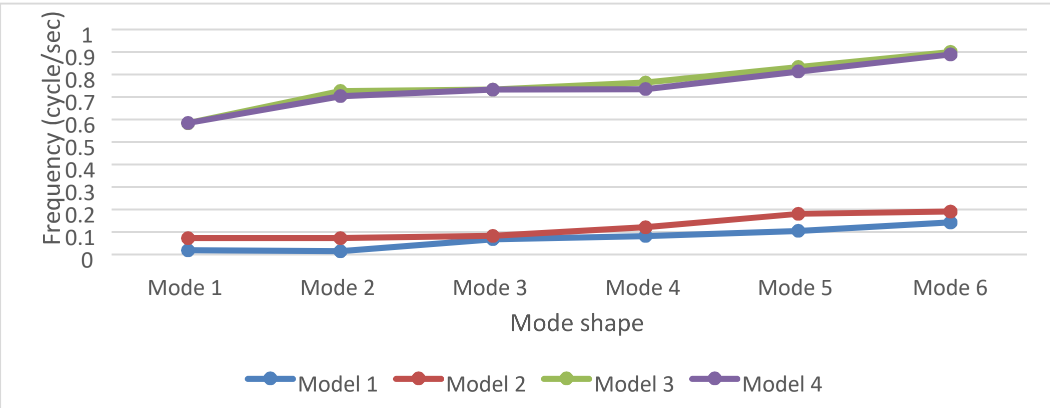
Graph 9. Variation in Displacement between H-Type and A-D Type Bridge



Graph 10. Variation in Shear Force between H-Type and A-D Type Bridge



Graph 11. Variation in Bending Moment between H-Type and A-D Type Bridge



Graph 12. Variation in Mode shape Frequency between H-Type and A-D Type Bridge

5. Conclusions

- The shear force increases by approximately 5.6% in H-Shape. when compared with A-D Shape, the shear force increases significantly by 52.2%. The A-D Shape exhibits much higher shear forces compared to the H-Shape for both span lengths. For the 400m span, the shear force in the A-D Shape is 56.8% higher than the H-Shape. And for the 520m span, the shear force in the A-D Shape is 125.9% higher than the H-Shape.
- The increase in shear force with increased span length the A-D Shape indicates a need for careful consideration of material strength and support structures when choosing this shape, especially for longer spans.
- Increasing the span length results in higher bending moments for both structural shapes. For the H-Shape, the bending moment increases by 9.9% for the A-D Shape, the increase is at 11.0%. When comparing between structural shapes the A-D Shape exhibits significantly higher bending moments compared to the H-Shape for both span lengths. For the 400m span, the A-D Shape has a bending moment that is 70.2% higher than the HShape. For the 520m span, A-D Shape having a bending moment 71.9% higher than the HShape.
- The higher bending moments associated with the A-D Shape suggest that this structural shape experiences more significant flexural stresses compared to the H-Shape, making it more demanding in terms of material strength and support.
- The H-Shape shows a noticeable increase in mode values when the span length is increased. The A-D Shape, on the other hand, displays minimum variation in mode values between the 400m and 520m spans. For example, in Mode 1, the change is only from 0.585514 to 0.586334, suggesting a relatively stable behaviour with respect to changes in span length.
- The A-D Shape exhibits much higher mode values across all modes compared to the HShape. For example, in Mode 6, the A-D Shape shows values around 0.89, while the HShape is around 0.19 for the 520m span. This suggests that the A-D Shape has a higher dynamic response and potentially greater susceptibility to vibrations.
- In summary, while the A-D Shape structure displays more consistent behaviour across span lengths, its higher mode values suggest a stronger dynamic response, which could lead to greater susceptibility to vibrations. The H-Shape, though showing lower mode values, is more affected by increases in span length, requiring careful consideration for longer spans.

REFERENCES

1. Analysis of the behavior of Cable stayed bridge with different types of Pylons

[1] *Priyanka Singh1*, Mirza Jahangir Baig1, Bhumika Pandey1, and Kartik Papreja1 E3SWebofConferences304,02006(2021)https://doi.org/10.1051/e3sconf/202130402006 ICECAE 202.*

2. Analysis and Design of Cable Stayed and Suspension Bridge Subjected to Wind Loading

[2] *Farhan Farid Reshi1, Ravinder KumarTomar1,SKSingh 2 IOPConf.Series:doi:10.1088/1755-1315/889/1/01205.*

3. Parametric Study of Cable Stayed Bridges

[3] *Umang A. Koyani, Kaushik C. Koradia, (2016) Journal of Emerging Technologies and Innovative Research, Volume 3, Issue 5, pg. 127-133. http://dx.doi.org/10.1155/2014/302707.*

4. Construction Stage Analysis of Cable-Stayed Bridges

[4] *Marko Justus Grabow, Technical University of Hamburg, Germany, 2004. http://dx.doi.org/10.1185/2013/302706.*

5. Seismic performance of cable-stayed bridges with different geometric conditions

[5] *Ahmed M. Fawzy, Khaled F. El-Kashif and Hany A. Abdalla. http://dx.doi.org/10.1155/2013/330706.*

6. Analysis and Design of Cable Stayed Bridge

[6] *Merin Mathews, Silna Joseph International Research Journal of Engineering and Technology (IRJET) e-ISSN: 2395-0056 Volume: 08 Issue: 06 | June 2021. http://dx.doi.org/11.1255/2013/302706.*

7. Comparison of Different Types of Pylon Shapes on Seismic Behaviour of Cable-Stayed Bridges

[7] *Govardhan Polepally, Venkata Dilip Kumar Pasupuleti and Archanaa Dongre https://doi.org/10.1007/978-981-15-1404-3_7.*

8. Dynamic analysis of cable stayed bridge with various patterns of pylon

[8] *Sreerag S Kallingal1*, and Priyanka Singh1 E3S Web of Conferences 304, 02005 (2021) https://doi.org/10.1051/e3sconf/202130402005 ICECAE 2021.*

9. Dynamic Analysis of Cable-Stayed Bridges Affected by Accidental Failure Mechanisms under Moving Loads

[9] *Fabrizio Greco, Paolo Lonetti, and Arturo Pascuzzo Hindawi Publishing Corporation Mathematical Problems in Engineering Volume 2013, Article ID 302706 http://dx.doi.org/10.1155/2013/302706.*