International Journal of Engineering Research & Technology (IJERT)
ISSN: 2278-0181
Vol. 3 Issue 10, October- 2014

Structural Behaviors of Long-Span Cable-Stayed Bridge With A-Shaped Tower Due to Three Wind Speeds

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***Abstract*—In our country, bridges play an important role because topography and terrains are steep and so many rivers flow. Among several types of bridge, cable-stayed bridges are the most popular bridge type for long-span bridges. Cable-Stayed bridges are very competitive economical for medium and long-span. In bridge engineering, it is customary to identify bridges as short span, medium-span and long span, depending on the span lengths. This study investigates on the structural behaviors of long-span cable-stayed bridge with A-shaped tower due to three wind speeds. The values of three wind speeds are 110 mph, 130 mph and 155 mph. These values are maximum values of basic wind speed according to Saffir-Simpson Hurricane Wind Scale based on a hurricane's sustained wind speed. So, superstructure of long-span cable-stayed bridge is analyzed and designed by using SAP 2000 version 14 software and specifications of American Association of State Highway and Transportation Officials and Japan Road Association. It is analyzed and designed under linear static condition and moving load conditions. Moreover, it is especially emphasized in the wind effects of long-span cable-stayed bridge with A-shaped tower with the values of wind speed 110 mph, 130 mph and 155 mph. This study provides the results of cable tension forces, bridge forces, truss girder displacement, support reactions and so on.**

Keywords— structural behaviors; cable-stayed bridge; A-shaped tower; three wind speeds.

I. INTRODUCTION

Cable-stayed bridges have become very popular over the last forty years because of their economy, structural efficiency and aesthetics. Stay cables are critical structural components in cable-stayed bridges. Nowadays, cable-stayed bridges are increasingly built because they can span distance far longer than any other kinds of bridges. Cable-stayed bridges offer outstanding architectural appearances due to its small diameter cables, minimum overhead structure and wide choice of design methods. A typical cable-stayed bridge is a continuous girder with one or two towers erected above piers in the middle of the span. From these piers, cables are attached diagonally to the girder to provide additional support. Cable-stayed bridges have a simple and elegant look. In this study, 2400 ft main span superstructure of cable-stayed bridge is analyzed and designed by using SAP software. In general cable-stayed bridges, the superstructure can be supported by one or two planes or lines of cables. To achieve the increase span length and more slender girders for future bridges, accurate procedures need to be developed that can lead to a thorough understanding and a realistic prediction of the structural response to not only wind and earthquake loads but also traffic loads.

II. STRUCTURAL CONFIGURATIONS AND STRUCTURAL STRATEGIES

A. Structural Configurations

The structural systems can be varied by changing tower shapes and the cable arrangements. The configurations of cable-stayed bridge are stiffening girder, cable system, towers and foundations. The stiffening girder is supported by straight inclined cables which are anchored at the towers. These pylons are placed on the main pier so that the cable force can be transferred down to the foundation system. Three basic cable arrangements are radial system, harp system, fan system. The proposed bridge is double plane fan type long-span cable-stayed bridge with A-shaped tower. The role of tower or pylon is also very important in cable-stayed bridge. The towers are the most visible elements of a cable-stayed bridge. The primary function of the pylon is to transmit the force arising from anchoring the stay and these forces will dominate the design of the pylon. Many varied types of the pylon are H-frame, A -frame, inverted frame or λ-frame and so on. In this cable-stayed bridge, A-shaped tower is used.

B. Structural Strategies

The proposed bridge is a long-span cable-stayed bridge with double plane fan type and its span type is also three spans type. New parallel wire strands are used for proposed bridge. New PWS cable with an outer diameter of 17 mm has a metallic cross-section of 16202mm^2 corresponding to a void ratio of 0.33. Its tensile strength is 1770 MPa and the size range is from 7 No. 7mm wires to 421 No. 7mm wires. Long-span cable-stayed bridge is designed and analyzed by using SAP-2000 Software. The cable-stayed bridge with A-shaped tower is analyzed under wind load condition by using tropical cyclone categories and analytical results are investigated in detail. Designing procedure and checking process will be carried out according to the specifications of American Association of State Highway and Transportation Officials. According to Saffir-Simpson Hurricane Wind Scale, three wind speeds are maximum values and they are 110 mph , 130 mph and 155 mph.

III. DESIGN DATA PREPARATION AND MODELING

For required design data preparation, material properties and design data are essentially taken into account. To become a safe and successful long-span cable stayed bridge, it is very important that modeling of proposed bridge is carried out within the defined structural specifications and strategies

A. *Design Data Preparation*

Design data used in proposed bridge are described in Table I.

TABLE I. DESIGN DATA USED IN PROPOSED BRIDGE

Name	Description
Bridge type	Cable-Stayed Bridge
Total length	4800 ft
Span arrangement	3-spans arrangement
Main span	2400 ft
Side span	1200 ft (each)
Pylon height	740 ft
Pylon type	A-type
Cable system	Fan type
Main cable plane	Double plane
Number of cables (each main span)	40@60 ft
Number of cables (each side span)	20@60 ft
Cable diameter	6 ft
Girder type	Warren truss type
Girder height	30 ft
Girder width	56 ft
Traffic	HS 20-44, four lanes

The following material properties are used in the proposed bridge with A-shaped tower model.

For cable,

- Modulus of Elasticity = 200 GPa
- Tensile strength = 1770 MPa
- Poisson ratio, v = 0.33
- Equivalent Density, γ = 82 kN/m^3

For structural steel, according to ASTM, A 992-50 steel,

- Modulus of Elasticity = 29000 ksi
- Tensile Strength, f_u = 65 ksi
- Yield Strength, f_y = 50 ksi
- Poisson ratio, v = 0.3
- Thermal coefficient = 6.5×10^{-6} °F

For concrete,

- Modulus of Elasticity = 3150 ksi
- Concrete Strength, f_c' = 4 ksi
- Poisson ratio, v = 0.2
- Thermal coefficient = 5.5×10^{-6} °F

B. *Modeling*

Elevation view and 3D view of proposed bridge can be seen in Fig 1, and 2.

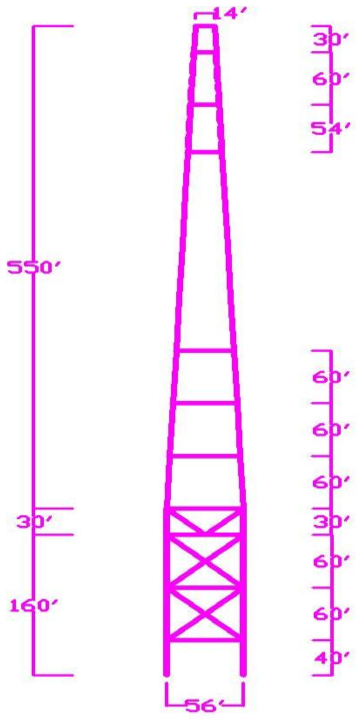

Fig. 1. Elevation view of proposed bridge.

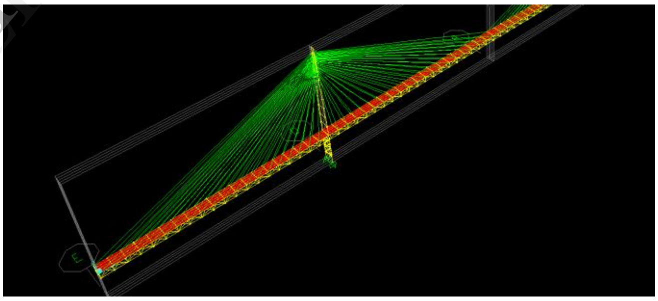

Fig. 2. 3D view of proposed bridge.

IV. DESIGN LOAD CONSIDERATIONS AND ALLOWABLE LIMITS

The applied loads on the proposed bridge model are dead loads, moving load and wind load. After then, structural responses of proposed bridge are investigated under these loads.

A. *Design Load Considerations*

1) Dead Load: In the dead load due to self weight and structural materials, the following loads are considered.

a) For girder and tower: unit weight = 490 lb/ft^3

b) For concrete slab: unit weight = 150 lb/ft^3

c) For Guardrail: unit weight = 200 lb/ft

d) For Asphalt: 2 in thick = 18 lb/in^2

2) Moving Load: Trucks are moving forward both in lane 1 and lane 2 whereas, other trucks are moving backward in lane 3 and lane 4. Moving load case is illustrated in Fig 3.

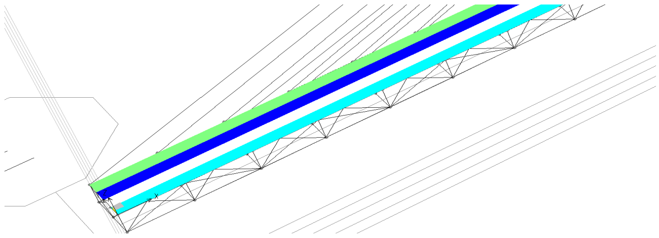

Fig. 3. Moving load case.

Fig. 4.

3) Wind Load: In wind load consideration, three kinds of wind speeds according to Saffir-Simpson Hurricane Wind Scale as displayed in Table II. These wind speeds are the maximum limit of hurricane category 2 (110 mph), category 3 (130 mph) and category 4 (155 mph).

TABLE II. SAFFIR-SIMPSON HURRICANE WIND SCALE

Scale Number (Category)	Sustained Wind (mph)
1	74 - 95
2	96 - 110
3	111 - 130
4	131 - 155
5	> 155

From maximum values of wind speeds, design wind speeds can be calculated in Table III according to (1).

$$V = k_1 V_{10} \qquad (1)$$

In which,

V = Design wind speed (m/s)

k_1 = Modified coefficient according to the change of wind height

V_{10} = basic wind speed appearing once in a hundred years

TABLE III. DESIGN WIND SPEED, V

Structural Component		k_1	V_{10} (m/s)	V (m/s)
For 110 mph	Girder	0.36	49.17	17.7
	Tower (windward)	2.489	49.17	122.38
	Tower (leeward)	2.489	49.17	122.38
For 130 mph	Girder	0.36	58.12	20.92
	Tower (windward)	2.489	58.12	144.66
	Tower (leeward)	2.489	58.12	144.66
For 155 mph	Girder	0.36	69.29	24.94
	Tower (windward)	2.489	69.29	172.46
	Tower (leeward)	2.489	69.29	172.46

By using (2), the values of wind load for different structural components can be calculated in Table V. from their respective design wind speed.

$$P = \frac{1}{2} \rho V^2 C_d G A_n \qquad (2)$$

In which,

P = Wind load (N/m)

ρ = Air density (kg/m^3) = 1.2 kg/m^3

V = Design wind speed (m/s)

C_d = Drag force coefficient

G = Gust response factor = 1.31

A_n = Effective project area (m^2/m)

TABLE IV. WIND LOAD, P

Structural Component		Factor	ρ (kg/m^3)	V^2	C_d	G	A_n (m^2/m)	P (kN/m)
For 110 mph	Girder	0.5	1.2	18^2	1.91	1.31	9.14	4.3
	Tower (wind-ward)	0.5	1.2	122^2	1.60	1.31	0.46	8.7
	Tower (lee-ward)	0.5	1.2	122^2	0.8	1.31	0.46	4.3
For 130 mph	Girder	0.5	1.2	21^2	1.91	1.31	9.14	6.0
	Tower (wind-ward)	0.5	1.2	145^2	1.60	1.31	0.46	12
	Tower (lee-ward)	0.5	1.2	145^2	0.8	1.31	0.46	6.1
For 155 mph	Girder	0.5	1.2	25^2	1.91	1.31	9.14	8.5
	Tower (wind-ward)	0.5	1.2	172^2	1.60	1.31	0.46	17
	Tower (lee-ward)	0.5	1.2	172^2	0.8	1.31	0.46	8.6

B. Allowable Limits

Deflection is the vertical displacement of a member subjected to loading. Gross sectional area of a member shall be used to calculate allowable deflection. The deflection of main girders, floor beams and stringers of a cable-stayed bridge due to live load should be less than the allowable deflection. So, allowable deflection for cable-stayed bridge is L/400. L is main span length of proposed bridge.

Allowable combined stress according to AASHTO is used for checking by (3).

$$\left(\frac{f_b}{F_b}\right)^2 + \left(\frac{f_v}{F_v}\right)^2 \le 1.2 \qquad (3)$$

In which,

F_b = 0.55 F_y = Allowable bending stress

f_b = M / Z = Maximum bending stress

M = Maximum moment

Z = Section modulus

F_v = 0.33 F_y = Allowable shear stress

f_v = V / A_w= Maximum shear stress

V = Maximum shear

A_w = Area of web

V. Analytical Results of Proposed Bridge

A. Analysis Results due to Own Weight

The displacements of girder along the bridge length are shown in their respective figures. Fig 4 is girder displacement about X-axis and Fig 5 is also girder displacement about Y-axis. Fig 6 is the illustration of girder displacement about Z-axis.

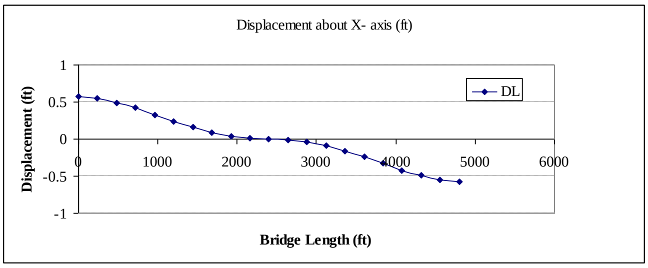

Fig. 5. Girder displacement about X-axis.

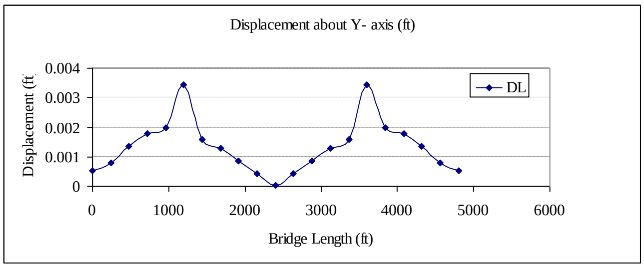

Fig. 6. Girder displacement about Y-axis.

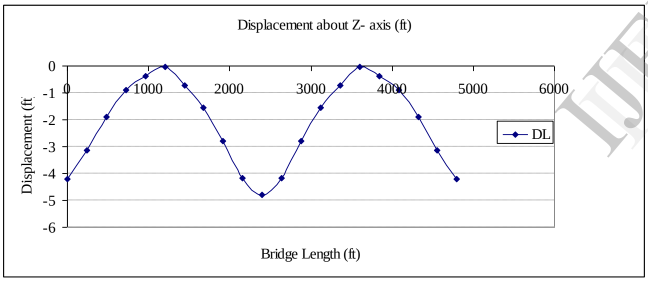

Fig. 7. Girder displacement about Z-axis.

Under own weight condition, Fig. 7 shows axial force along the bridge length, Fig. 8 shows vertical shear along the bridge length, Fig. 9 shows horizontal shear along the bridge length, Fig. 10 shows torsion along the bridge length, Fig. 11 shows vertical moment along the bridge length and Fig. 12 shows horizontal moment along the bridge length due to own weight.

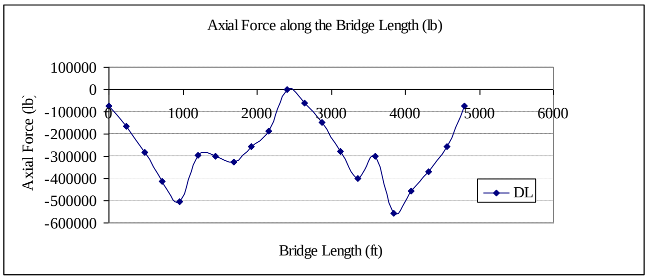

Fig. 8. Axial force along the bridge length.

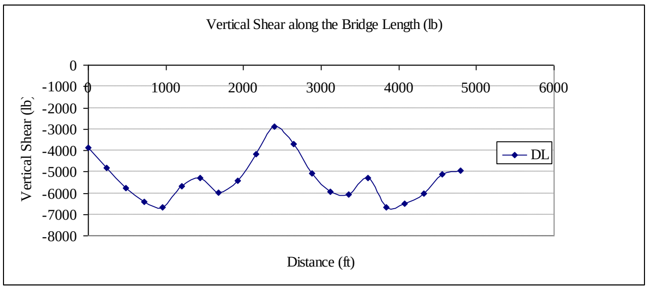

Fig. 9. Vertical shear along the bridge length.

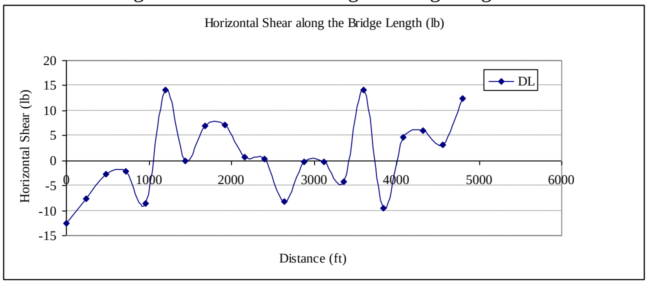

Fig. 10. Horizontal shear along the bridge length.

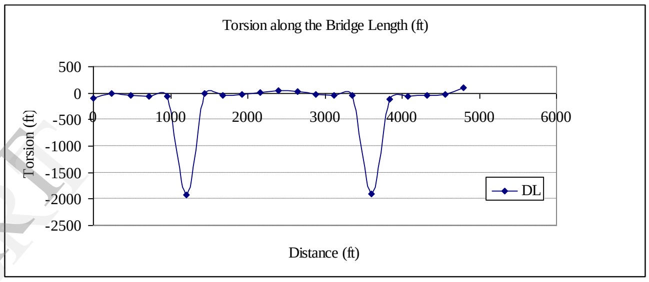

Fig. 11. Torsion along the bridge length.

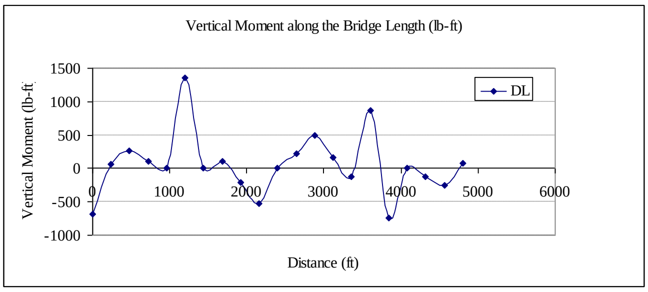

Fig. 12. Vertical moment along the bridge length.

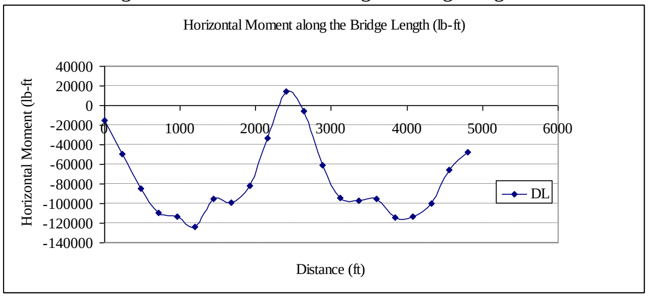

Fig. 13. Horizontal moment along the bridge length.

B. Analysis Results due to Moving Load

In moving load condition, Fig. 13 shows axial force along the bridge length, Fig. 14 shows vertical shear along the bridge length, Fig. 15 shows horizontal shear along the bridge length, Fig. 16 shows torsion along the bridge length, Fig. 17 shows vertical moment along the bridge length and Fig. 18 shows horizontal moment along the bridge length under moving load.

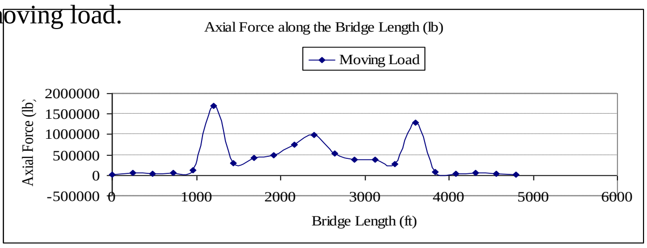

Fig. 14. Axial force along the bridge length.

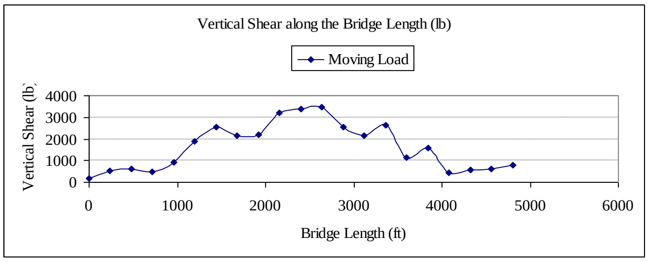

Fig. 15. Vertical shear along the bridge length.

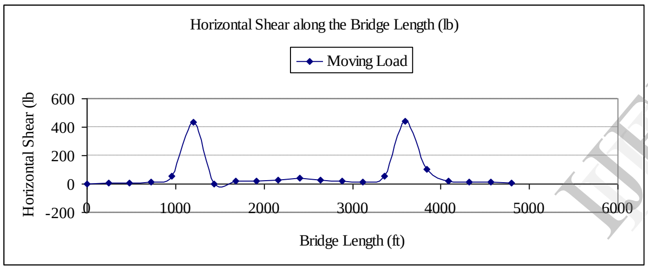

Fig. 16. Horizontal shear along the bridge length.

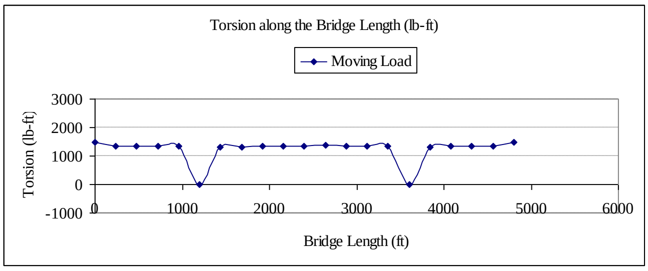

Fig. 17. Torsion along the bridge length.

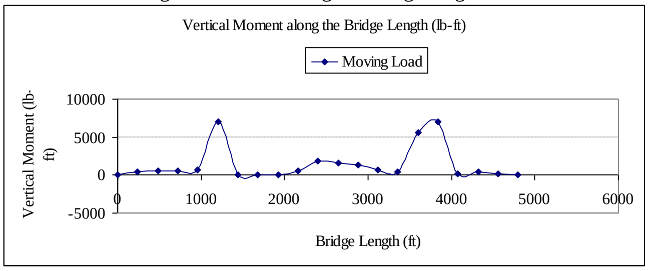

Fig. 18. Vertical moment along the bridge length.

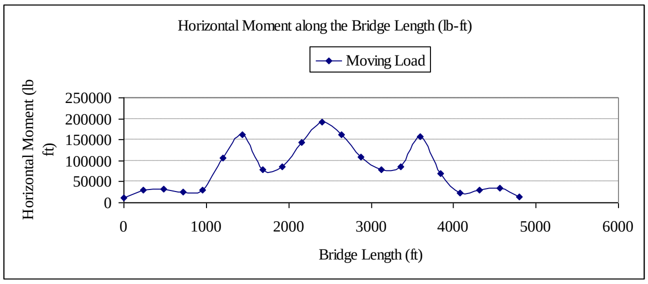

Fig. 19. Horizontal moment along the bridge length.

C. Analysis Results due to Wind Load

In wind load condition, wind loads are calculated with three basic wind speeds such as 110 mph, 130 mph and 155 mph according to tropical cyclone categories. The following figures: Fig 19 shows axial force along the bridge length, Fig. 20 shows vertical shear along the bridge length, Fig. 21 shows horizontal shear along the bridge length, Fig. 22 shows torsion along the bridge length, Fig. 23 shows vertical moment along the bridge length and Fig. 24 shows horizontal moment along the bridge length under wind load.

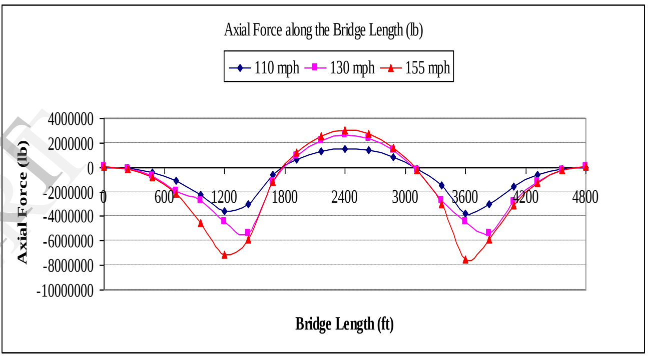

Fig. 20. Axial force along the bridge length.

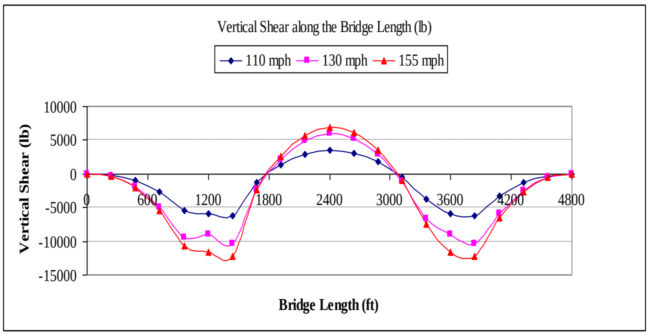

Fig. 21. Vertical ahear along the bridge length.

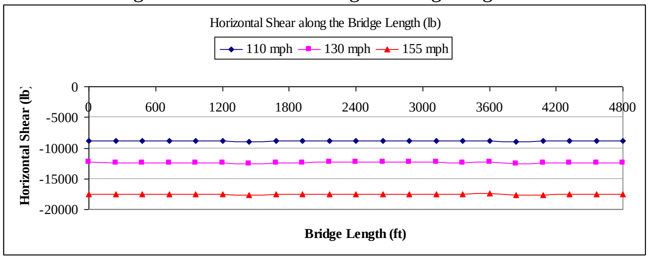

Fig. 22. Horizontal ahear along the bridge length.

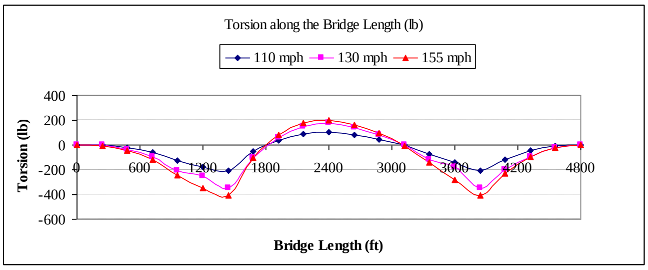

Fig. 23. Torsion along the bridge length.

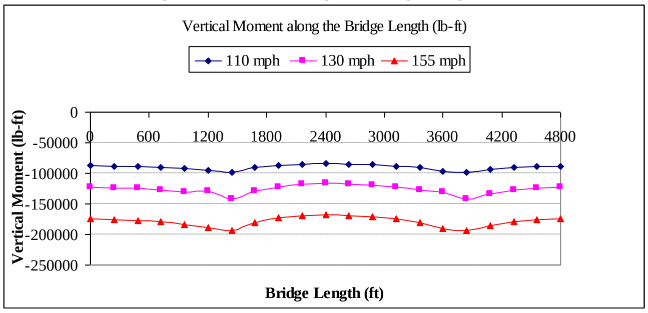

Fig. 24. Vertical moment along the bridge length.

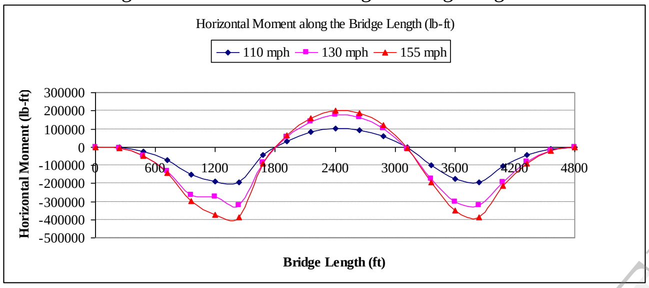

Fig. 25. Horizontal moment along the bridge length.

VI. CHECKING FOR PROPOSED BRIDGE

A. *Deflection Check*

Allowable deflection for cable-stayed bridge according to JRA, δ_{all} = L/400

Main span length, L= 4800 ft = 731.52 m

δ_{all}= 731.52/400= 1.83 m=6 ft

Maximum deflection for A-shaped tower model due to moving load=5.50 ft < 6 ft

Maximum deflection due to moving load is 5.50 ft for A-tower model and it is within acceptable limit 6 ft. So, it can be concluded that the results are satisfactory conditions.

B. *Allowable Stress Check*

In checking of combined stress, maximum values of bending and shear are used.

TABLE V. CHECKING FOR COMBINED STRESS

Member Section	$\left(\frac{f_b}{F_b}\right)^2+\left(\frac{f_v}{F_v}\right)^2$	**Factor**
W 21 × 147: Left Side Span	0.33	1.2
W 21 × 147: Right Side Span	0.88	1.2
W 21 × 147: Mid Span	0.92	1.2
W 18× 158: Tower (Upper)	0.43	1.2
W 18× 158: Tower (Lower)	0.19	1.2

According to checking for stresses, allowable stresses are greater than the maximum values of stresses. So, these sections are satisfied sections.

VII. CONCLUSIONS

Cable-stayed bridges are widely constructed all over the world. This type of bridge is very competitive economical for medium and long span. In comparison with other types of bridge, cable-stayed bridges are particularly pleasing to the visual senses. Moreover, this type of bridge fills the gap of efficient span range between conventional girder bridges and the very long span bridges. In this study, the 2400 ft main span cable-stayed bridge is analyzed by using SAP 2000 software. Therefore, this study would give some knowledge of analysis and design of three-span cable-stayed bridge with A-shaped tower and for the way how to use SAP 2000 software. According to analysis results due to own weight, displacements and rotations are symmetric. The maximum vertical displacement due to moving load is (5.50 ft) in A -tower model and it is satisfactory condition. For A- tower model, maximum value of axial force (1691.14 kips) and maximum value of vertical shear (3.48 kips) occurred in moving load condition. And then, in A- tower model, maximum value of axial force is (3053.69 kips) whereas maximum vertical shear is (6.84 kips) under wind load condition. According to the analysis results, the maximum case in wind load cases is under wind speed 155 mph. By checking stresses for combined for members, the actual stresses are within acceptable limits.

ACKNOWLEDGMENT

This study is the achievement of invaluable help from teachers and many other persons. The author would like to express her deep gratitude to Dr. Kyaw Moe Aung, Associate Professor and Head of Civil Department, Mandalay Technology University, for his invaluable supports and managements. The author would like to special thank to her supervisor and co-supervisor, Dr. San Yu Khaing and Dr. Kyaw Lin Htat, Associate Professors of Civil Department, Mandalay Technological University, for their invaluable guidance, kind consideration and knowledge.

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