COMPARATIVE STUDY ON BEHAVIOUR OF CABLE-STAYED BRIDGE WITH NORMAL SUPPORT AND SPRING CONTROLLED SYSTEM

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ABSTRACT: The main objective of this research is to study the damped spring controlled system to reduce the earthquake-induced forces in the structure of cable-stayed bridges by applying STAAD-Pro structural analysis. The total length of the proposed cable-stayed bridge is 450 ft from abutment to abutment. It has symmetrical span arrangement with 250 ft (main span length) and 100 ft (each of side span length) respectively. It consists of dual H-type tower and the warren truss type steel girder. The girder has 10ft each panel bays and 5ft in height. A reinforced concrete, H-type tower, rises 50 ft above the truss girder base. Its apex is 15 ft high in pylon that supports a dual-plane cable system. Two pairs of cables on each side are arranged as a fan type cable system. Cables are installed from the outermost cable connections; from the ends of the bridge to the tower with 50 ft spacing to distribute axial load throughout the deck. HS 20-44 AASHTO loading will be applied. Effects of impact loading, wind loading, temperature forces and horizontal seismic loading will also be considered.

Firstly, the static analysis of cable-stayed bridge is accomplished without considering the effect of earthquake forces. After that the spring supports are installed between tower and girder for effective flexible responses of the cable-stayed bridge due to the horizontal earthquake forces only. The intelligent control system is used to guarantee the relative movements between the connected structural parts. The bridge is analyzed and designed by using the STAAD-Pro engineering software in this study. The materials and loads are specified according to the AASHTO specifications. This study provides the proof of seismic protection and the comparative results of the controlled and uncontrolled structural system such as tower deformations, truss girder axial forces, cable axial forces, truss girder displacement and support reactions.

Keywords: cable, earthquake force, spring stiffness, displacement, axial forces

1. INTRODUCTION

Bridges are lifeline structures. In Myanmar, bridges play an important role because of natural conditions that the topography is steep and many rivers flow. Among several types of bridge, cable-stayed bridges are the most popular bridge type for long-span bridges. This can be attributed to several advantages, predominantly being associated with the relaxed foundation requirements. This leads to economical benefits which can favor cable-stayed bridges in free spans of up to 1000 m.

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. Because the only part of the structure that extends above the road is the towers and cables, cable-stayed bridges have a simple and elegant look. Cable-stayed bridges offer outstanding architectural appearances due to its small diameter cables, minimum overhead structure, and wide choice of design methods.

2. **COMPONENTS OF CABLE-STAYED BRIDGE**

The cable-stayed bridge is the structure in which the stiffening girder is supported by straight inclined cables which are anchored at the towers. Its structural type is regarded as one of the continuous girder types. The structural components of cable-stayed bridge: the cables, the pylon or the tower and the girder.

It is apparent that the close supporting points enable the deck to be very slim. Even though it has to support considerable vertical loads, it is loaded mainly in compression with the largest prestress being at the intersection with the towers. This is due to the horizontal force which is applied by each of the cables. This characteristic also distinguishes the cable-stayed bridge from the suspension bridge because necessary provisions for anchoring cables are much more relaxed.

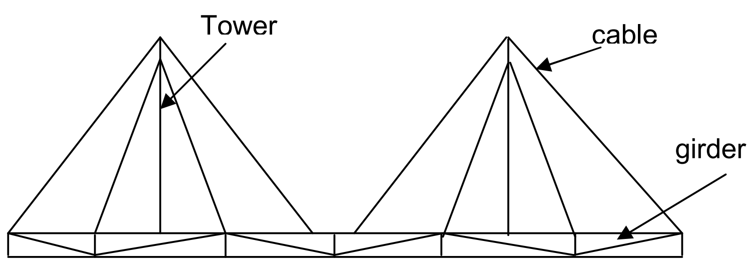


Figure.1. Structural Components of Cable-Stayed Bridge

3. **TYPES OF LOADING**

In this study, the loadings applied to the structural model for the proposed bridge by using STAAD-Pro engineering software are followings:

(1) Dead load
(2) Live load
(3) Impact or Dynamic effect of the Live load
(4) Wind load

(5) Thermal forces and

(6) Seismic loads due to lateral direction.

4. ANALYSIS METHODS

Generally speaking, three methods are available for analysis and design of structures for seismic loads, namely, response spectrum modal analysis, the time history method, and the equivalent static force method. Of these, the first two are called dynamic analysis methods. Firstly, the bridge structure is designed by static analysis method. And then the dynamic analysis method is applied to the structure with controlled spring support condition and uncontrolled system condition.

Response-spectrum analysis shall be performed using a modal damping value for the fundamental mode in the direction of interest not greater than the effective damping of the isolation system or 30 percent of critical, whichever is less. Modal damping values for higher modes shall be selected consistent with those appropriate for response spectrum analysis of the structure above the isolation system with a fixed base. For dynamic analysis, the isolated bridge can be modeled as a continuous beam for simplicity. Since this model is flexible in the vertical direction, it cannot be considered as a rigid block in that direction.

The basic factors including the spring stiffness involved in engineering design are taken into consideration for the three-span bridge model in this study. The span length is based on the continuous beam model. Once the span length is decided, the size of the cross section can be calculated by applying traffic load as a live load plus the dead load of the bridge model. Note that the deflection under normal bridge loading must be controlled and can be the determinant for the bridge stiffness. From the point of seismic isolation, the bearings are expected to be as flexible as possible. The stiffness of the springs can be calculated based on the reaction at the bearing and the static settlement limit.

5. THE USE OF DAMPED SPRING SUBJECTED TO EARTHQUAKE FORCES

In the proposed configuration of the cable-stayed bridge, the deck and girders can be considered to be floating on helical spring bearings. Helical springs, which have both vertical and shear stiffness, are designed to support vertical loads, including the self-weight of the bridge, providing the mechanism to accommodate movement in all directions. To protect the bridge deck and abutment from damage by an earthquake in the horizontal direction, helical springs are also installed between the deck and towers.

6. DESIGN CONFIGURATIONS OF THE PROPOSED BRIDGE

In this study, the total length of the proposed cable-stayed bridge is 450 ft from abutment to abutment. It has symmetrical span arrangement with 250ft (main span length) and 100ft (each of side span length) respectively. Dual H-type reinforced concrete tower and dual-plane cable system are selected. The girder, warren truss type steel girder, is used. The total width of bridge surface is 30ft, the carriage way width is 24ft and the two sidewalks are 3ft each as shown in Fig. 2.

The deck is supported by steel stay cables at main pylons. These are anchored directly above the web of the main girders. The slab thickness is 9 inches above the girders.

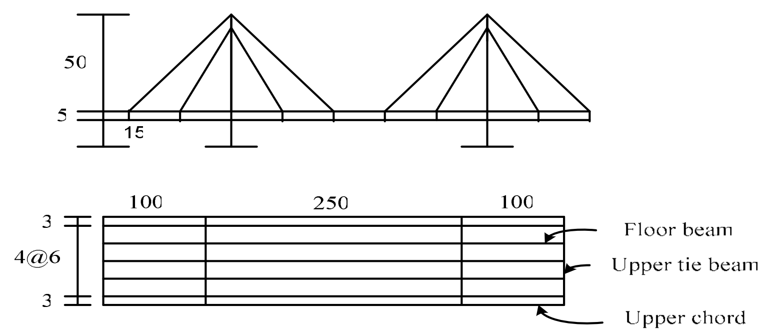
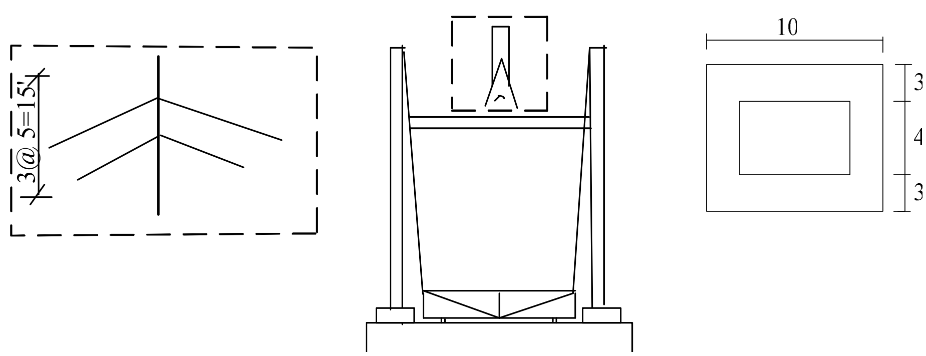


Figure 2. Elevation and Plan View of Proposed Bridge

The box section H-type tower, which rises 65ft above the supporting pier cap and its apex is 15ft high as shown in Fig. 3.



(a) Tower Apex (b) Cable Anchorage of Tower (c) Tower Section

Figure 3. Tower with Dual- Plane Cable System

The total traffic load is acting on the deck of the girder, and both the dead load and the wind area in most cases are larger for the girder than for the cable system. In this study, the stiffening truss girder is chosen.

According to the standard configuration, the girder has 10ft each panel bays and 5ft in height. It is 32ft wide. The warren truss type steel girder is adopted for the proposed bridge as shown in Fig. 4.

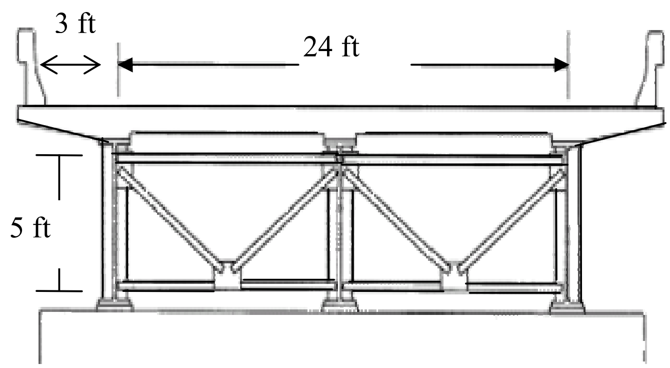


Figure 4. The Proposed Warren Truss Type Steel Girder

The two planes of stay cables are arranged in the fan pattern. Cables are installed from the outermost cable connections; from the ends of the bridge to the tower with 50ft spacing to distribute axial load throughout the deck.

Each strand consists of seven twisted wires, with an external diameter of 0.5 inch or 0.7 inch as shown in Fig. 5. 37, 61 and 91 number of strands are generally used.

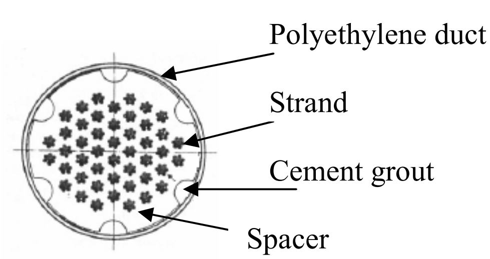


Figure 5. Grouting Type Polyethylene Strand

The type of deck-tower connection of cable-stayed bridge is very important to protect the bridge under seismic excitations both under construction and after construction.

6.1. **Design Condition**

Type	_ Three-Span Cable-Stayed bridge
Span Length	– main span 250 ft
	side span 100 ft
Carriageway width	– 24 ft
Sidewalk width	_ 3 ft
Tower Height	_ 50 ft
Tower Type	_ H-type
Girder Type	_ Warren Truss type
Girder Height	_ 5 ft
Live load	_ HS 20-44, 20tons
Slab	_ Asphalt concrete 9 inches thick

6.2. **Modeling of the Proposed Cable-Stayed Bridge**

In this study, the type of cable-stayed bridge is modeled as three-span continuous bridge which has a main span of 250 ft and side spans of 100 ft respectively. It is modeled as space frame model. Its roadway width is 24 ft and sidewalks width is 3 ft respectively. In this model, the total number of 752 nodes, 1403 beam elements and 108 plate numbers are assigned respectively. The steel girder and pylon are modeled as space frame and the slabs are as plates. The cable is modeled as space solid. The cable system of the bridge was of the efficient fan-shaped type, which is in good harmony with the H-type pylon. The tower height is 50 ft above the bottom of the girder. The anchor portion of the pylon is 15 ft and it has 15 ft above from pile cap. The cross section of the stiffening girder is the warren truss type girder that has the length of 450 ft is divided into segments. Each panel bay has 10 ft long. It is 32 ft wide and 5 ft high. The 100 ft span has two pairs of cables with 50 ft spacing each. The support of the pylon base is assumed as fixed base on the pie cap. On one side of the abutment, truss girder is installed with pinned type support and on the other is enforced but support type. And the damped spring support type is connected girder and pylon in horizontal and vertical directions. The outermost cable connections are edges of the bridge to distribute axial load throughout the deck.

6.2.1. Boundary conditions of the proposed model

(a) ***Material properties***

Material assumptions are the following Table 1.

(b) ***Properties of stayed cables***

For the proposed bridge, parallel wire strands are used, each strand consists of seven twisted wires with an external diameter of 0.5 in. The properties of 61 Ø 0.5 inch of 7 twisted wires are described in Table 2.

Table 1. Properties of Structural Steel

Structural Steel (Grade 60, ASTM A 572)	Minimum tensile strength Minimum yield strength	fu = 75.0 ksi fy = 60.0 ksi
	Modulus of elasticity	E = 29,000.0 ksi
	Shear modulus	G = 11200.0 ksi
	Poisson's ratio	v = 0.3
	Coefficient of linear expansion	$\alpha = 12.0\text{x}10^{-6}$ (°C)
	Unit weight	w = 490.0 (lb/ft^3)

Table 2. Strand Capacities

Nominal cross-section of steel (inch^2)	12.00
0.2% proof stress, $\sigma_{0.2}$ (ksi)	242.22
Ultimate tensile strength, β_z (ksi)	270.00
Ultimate load (k)	3570.00
Modulus of elasticity, E (ksi)	29,000.00
Poisson's ratio, v	0.30.

7. **MAXIMUM OUTPUT RESULTS**

From the analysis, the maximum node displacement, beam end forces and axial forces of the structure with controlled spring and uncontrolled system due to horizontal earthquake forces (EQx and EQz) are shown in tables and figures as below.

Table 3. Maximum Node Displacement with Control System

			Horizontal	Vertical	Horizontal
	Node	L/C	X in	Y in	Z in
Max X	454	247 COMBINATION LOAD CASE 247	**0.844**	1.38	0.012
Min Y	59	276 LOAD CASE 278	-0.009	**-5.152**	0.011
Max Z	207	240 EQZ	0.036	0.001	**0.871**

Table 4. Maximum Node Displacement without Control System

			Horizontal	Vertical	Horizontal
	Node	L/C	X in	Y in	Z in
Max X	300	247 COMBINATION LOAD CASE 247	**0.775**	0.434	0.004
Min Y	59	276 LOAD CASE 278	-0.025	**-4.84**	0.007
Max Z	207	240 EQZ	0.112	0.001	**0.696**

Table 5. Maximum Beam End Forces with Control System

	Beam	L/C	Node	Fx kip	Fy kip	Fz kip
Max Fx	428	398 LOAD CASE 399	132	4269.66	-49.23	24.07
Min Fx	1508	400 LOAD CASE 401	109	-2602.35	25.85	-17.97
Max Fy	408	248 COMBINATION LOAD CASE 248	220	32.35	662.38	32.83
Min Fy	429	296 LOAD CASE 298	201	3268.71	-750.94	-77.68
Max Fz	1511	240 EQZ	132	668.96	583.72	744.95

Continued;

Min Fz	1511	248 COMBINATION LOAD CASE 248	755	-350.37	-700.99	-762.20
Max Mx	424	398 LOAD CASE 399	3	1437.50	23.38	-88.54
Max My	427	240 EQZ	109	29.43	30.38	596.01
Max Mz	429	247 COMBINATION LOAD CASE 247	201	435.7	483.67	-12.09

Table 6. Maximum Beam End Forces without Control System

	Beam	L/C	Node	Fx kip	Fy kip	Fz kip
Max Fx	428	398 LOAD CASE 399	132	4270.82	-4.59	21.95
Min Fx	1508	398 LOAD CASE 399	109	-2596.94	1.42	-7.13
Max Fy	408	248 COMBINATION LOAD CASE 248	220	22.52	728.40	40.92
Min Fy	1511	247 COMBINATION LOAD CASE 247	755	0	-870.61	-2.44
Max Fz	1512	248 COMBINATION LOAD CASE 248	191	0	534.23	2627.97
Min Fz	1512	248 COMBINATION LOAD CASE 248	756	0	-178.41	-2620.43
Max Mx	424	398 LOAD CASE 399	3	1447.80	14.13	-87.62
Max My	1512	240 EQZ	191	0	356.32	2624.20
Max Mz	1513	296 LOAD CASE 298	757	188.98	-328.91	-36.15

Table 7. Axial, Shear and Torsion Forces of Tower with Control System

Beam	L/C	Node	Axial Force kip	Shear-Y kip	Shear-Z kip	Torsion kip-in
424	240	3	29.11	26.71	506.59	0
425	240	9	662.09	24.15	559.92	4959.56
427	240	109	29.43	30.38	596.01	0
430	240	115	663.89	21.82	494.96	4741.17

Table 8. Axial, Shear and Torsion Forces of Tower without Control System

Beam	L/C	Node	Axial Force kip	Shear-Y kip	Shear-Z kip	Torsion kip-in
424	240	3	52.18	68.36	519.87	0
425	240	9	739.02	48.67	545.07	7103.44
427	240	109	50.40	72.12	590.21	0
430	240	115	739.73	46.32	499.81	6853.51

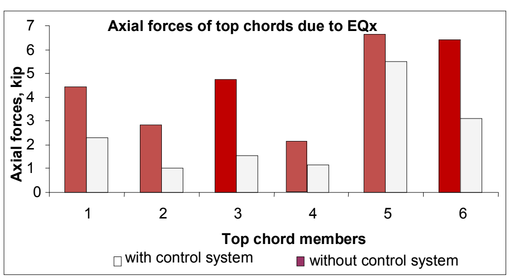


Figure 6. Axial Forces of Top Chord Members due to EQx

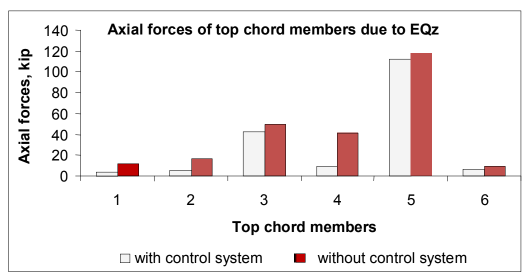


Figure 7. Axial Forces of Top Chord Members due to EQz

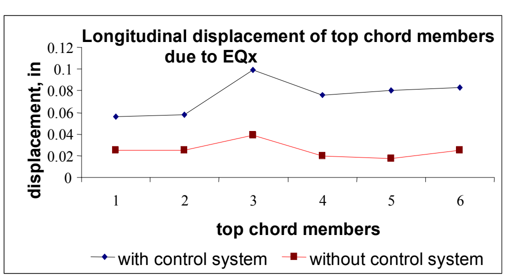


Figure 8. Longitudinal Displacement of Top Chord Members due to EQx

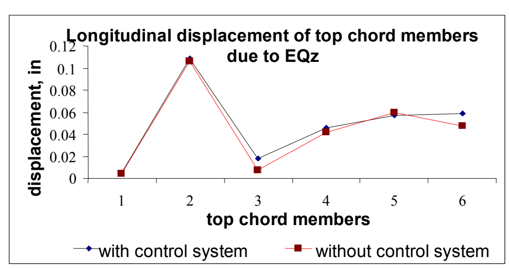


Figure 9. Longitudinal Displacement of Top Chord Members due to EQz

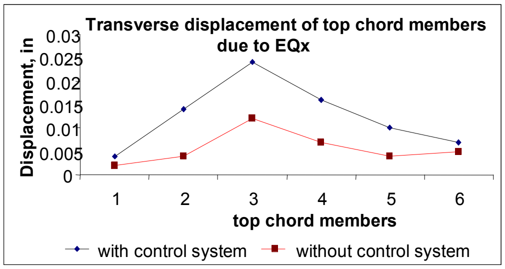


Figure 10. Transverse Displacement of Top Chord Members due to EQx

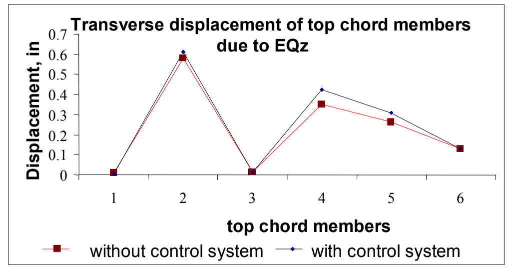


Figure 11. Transverse Displacement of Top Chord Members due to EQz

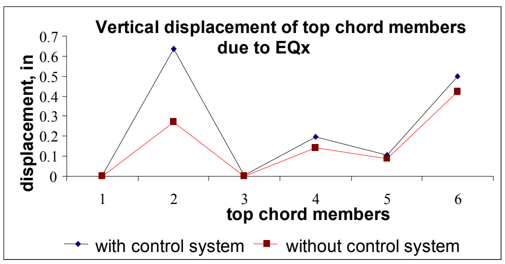


Figure 12. Vertical Displacement of Top Chord Members due to EQx

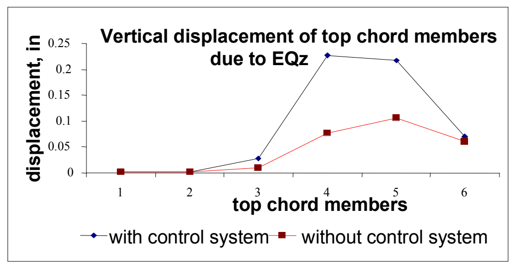


Figure 13. Vertical Displacement of Top Chord Members due to EQz

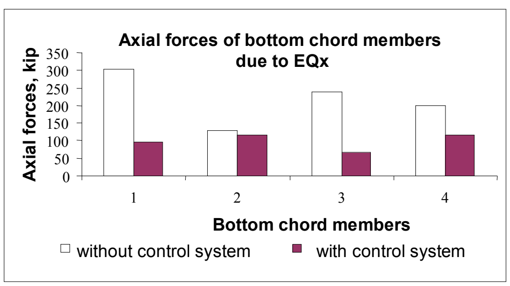


Figure 14. Axial Forces of Bottom Chord Members due to EQx

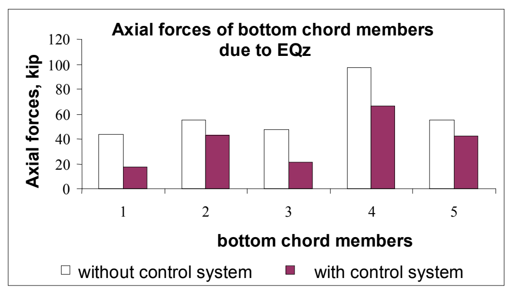


Figure 15. Axial Forces of Bottom Chord Members due to EQz

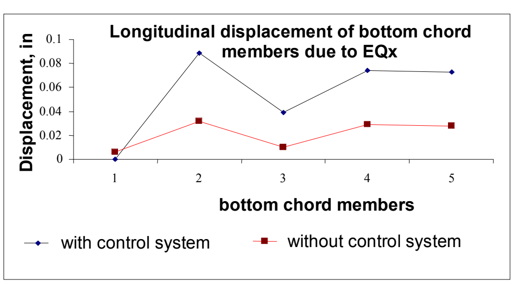


Figure 16. Longitudinal Displacement of Bottom Chord Members due to EQx

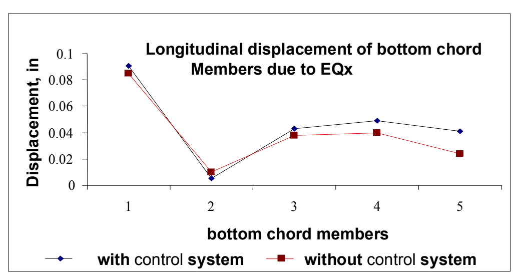


Figure 17. Longitudinal Displacement of Bottom Chord Members due to EQz

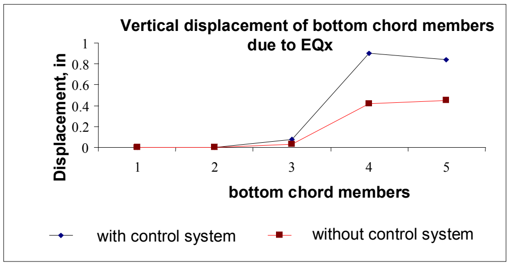


Figure 18. Vertical Displacement of Bottom Chord Members due to EQx

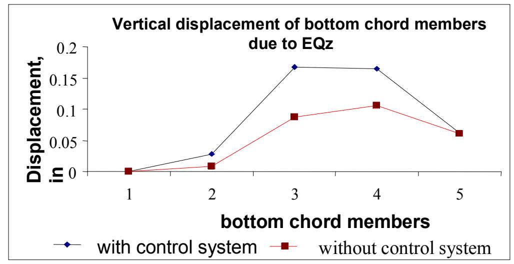


Figure 19. Vertical Displacement of Bottom Chord Members due to EQz

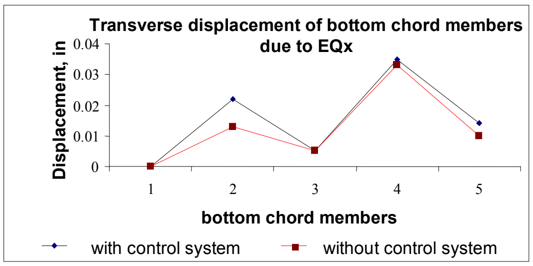


Figure 20. Transverse Displacement of Bottom Chord Members due to EQx

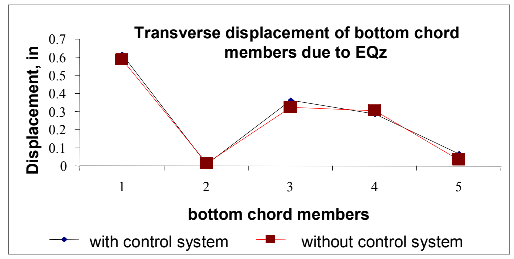


Figure 21.Transverse Displacement of Bottom Chord Members due to EQz

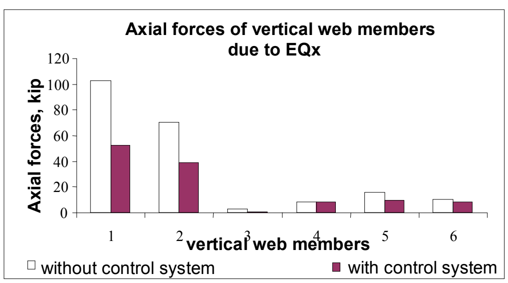


Figure 22. Axial Forces of Vertical Web Members due to EQx

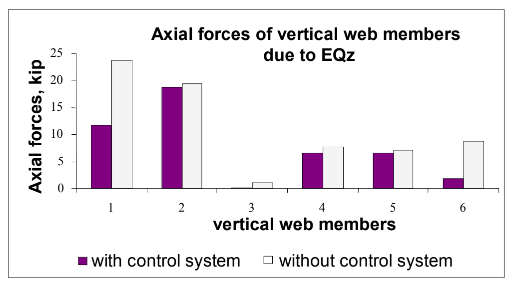


Figure 23. Axial Forces of Vertical Web Members due to EQz

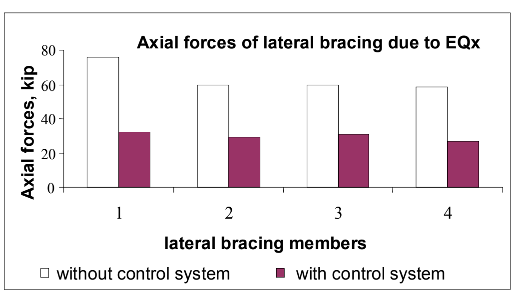


Figure 24. Axial Forces of Lateral Bracing due to EQx

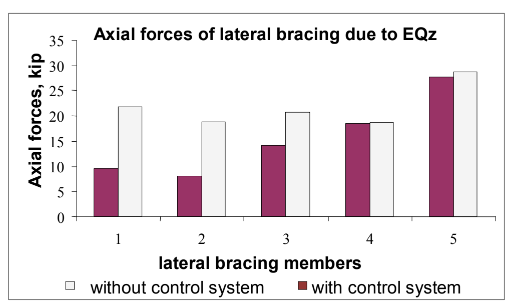


Figure 25. Axial Forces of Lateral Bracing due to EQz

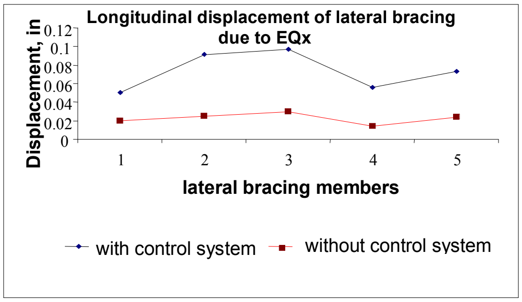


Figure 26. Longitudinal Displacement of of Lateral Bracing due to EQx

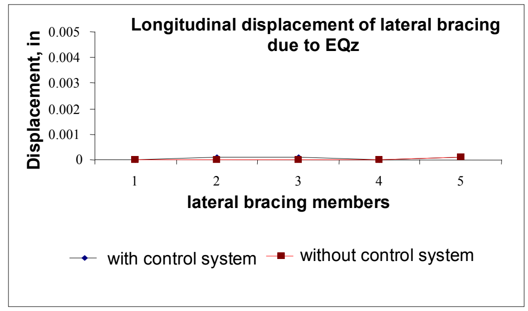


Figure 27. Longitudinal Displacement of Lateral Bracing due to EQz

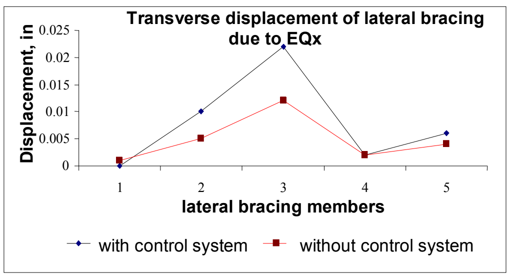


Figure 28.Transverse Displacement of Lateral Bracing due to EQx

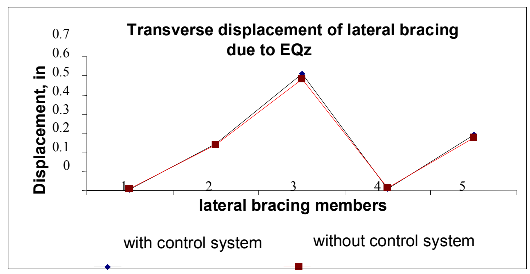


Figure 29.Transverse Displacement of Lateral Bracing due to EQz

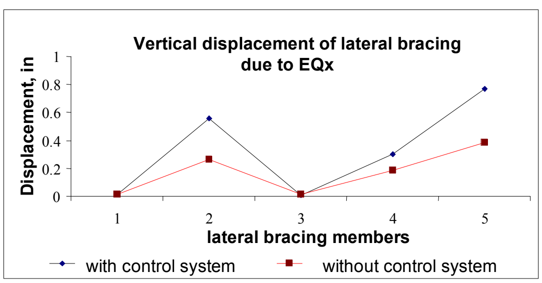


Figure 30.Vertical Displacement of Lateral Bracing due to EQx

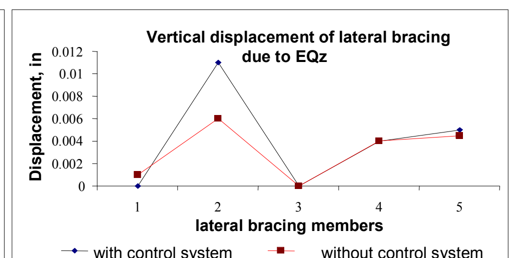
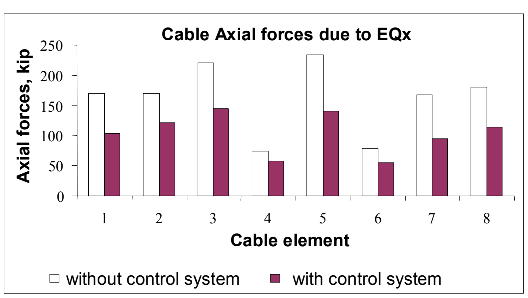


Figure 31.Vertical Displacement of Lateral Bracing due to EQz



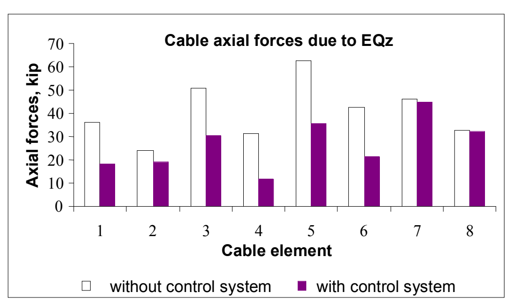


Figure 32.Cable Axial Forces due to EQx *Figure 33.Cable Axial Forces due to EQz*

8. **DISCUSSIONS AND CONCLUSION**

The objective of this research is to study the behavior of cable-stayed bridge with normal support and spring controlled system. As a first step, static analyses are conducted on the bridge subjected to relative displacements between the girder and pylon to study the impact of tectonic movements on the structure. The most responses especially vertical deck displacements of the damped spring connected between deck and tower that are better than those of the hinged connection as shown in previous chapter. The helical spring possesses stiffness in all directions. Its stiffness can be customized according to design requirements. Compared to a non-isolated bridge, a spring-supported bridge is relatively flexible in the vertical direction of the structure. And the maximum forces of the members without damped springs are greater than installing with the damped springs. In this study:

1. Maximum deformation was occurred at mid-span under seismic loadings.
2. In cable-stayed bridge, height of the tower influenced the cable forces and deformations of main girders as the loads were finally transferred to the tower.
3. The deformation of the structure was found to be considerably reduced by the spring control system.
4. Maximum cable axial forces, girder and tower forces for spring controlled system were reduced effectively under horizontal earthquake loadings.
5. Good results can be obtained by balancing the reduction in forces along the bridge with control system as compared to the uncontrolled system.

9. **REFERENCES**

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