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Seismic Analysis of Continuous Steel Concrete Composite Bridge

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Abstract:

The bridge analyzed with ANSYS software which is based on the Finite Element Method. Finite Element method is one of the widely used methods nowadays to study the dynamic responses of the steel concrete bridges. Static analysis is performed to find the response of the composite bridge under the gravity load. The modal analysis was based on the Block Lanczos method is performed to get the mode shapes and natural frequencies of continuous steel concrete composite bridge. The results show that the bridge has complex modes due to the coupling effects of the free vibrations and it is more rigid in the vertical direction than in transverse and longitudinal directions. Time history analysis is performed by input of the EI Centro earthquake acceleration waves to the bridge system. The time period considered for 10 seconds with a time step of 0.0001seconds. The dynamic responses of the bridge, including the displacement, accelerations, and the stresses of the mid-span are calculated.

Keywords: Dynamic Behavior, Seismic Analysis, Steel Concrete Composite Bridge, Time History Analysis.

1. INTRODUCTION

Composite bridges are that type of bridges combines of more than one material such as concrete, steel, timber or masonry in any combination. In recent days, the common usage of composite construction is meaning either steel - concrete construction or in-situ concrete or precast concrete bridges. Steel concrete composite bridges became commonly used all over the world, because of their aesthetic appearance and economically.

Composite steel plate girder bridges have many advantages over other types of the composite steel bridges. Their capability of covering long spans without requiring of falsework make them more desirable in metropolitan areas. In addition, there is a significant difference in the weight of steel plate girder bridges and concrete box girder bridges have an important effect on the seismic design. The inertia forces generated by an earthquake in steel plate are significantly less than the forces generated by concrete bridges. These differents in the inertia forces provide an economy for the substructure design [1].

The performance of steel bridges in the earthquakes can be considered generally good, and the indication is often made that they should be used in seismically active regions more frequently. It seems that reason is based on the fact that few steel bridges have collapsed in earthquakes in the United States, in comparison with the performance of concrete bridges [2].

In the western North America during the last decade, there are very few steel bridges with a steel superstructure and steel substructure, and even fewer of this kind of bridges have been exposed to strong ground motions. However for another type of steel bridges with concrete substructures (piers), (abutments) there is significantly increasing in the population but still less than the concrete bridges. Even so, seismic performance data for these bridges is difficult to find, and especially for these bridges subjected to strong shaking (experience from recent North American earthquakes is generally for bridges in areas of low-to-moderate shaking). This data showed that steel bridge superstructures are easy to damage, even during low-to-moderate shaking, and appeared more fragile than concrete superstructures in this concern if not designed properly. The common damage such unseated girders and failures in connections, cross-frames, bearings, and expansion joints. In a few cases (during the Kobe earthquake) major carrying members have failed, triggered in some instances by the failure of components in the superstructure like the bearing for example [3], [4].

2. LITERATURE REVIEW

Basically, earthquakes do not kill people, but bad structures do. This sentence is quite familiar to the designers in structural engineering. Usually, earthquakes only cause death by the damage they induce in structures such as buildings, bridges and other works of man. In other words, damage caused by earthquakes most commonly relates to man-made structures.

2.1. Performance of Steel Bridges during Recent Earthquakes

Concrete structures have generally suffered significant damage in past earthquakes compared to the minor or moderate damage suffered by Steel plate girder bridges. However, these earthquakes have exposed components in the superstructure and substructure, which should be design and detail carefully to resist seismic demand.

These earthquakes caused some remarkable damages in two steel plate girder bridges as follow:

2.1.1. Behavior of Steel Bridges during the Petrolia Earthquakes

Strong earthquakes (7.0, 6.4) occurred near Petrolia, California at on April 25, 1992. Many aftershocks followed including two earthquakes over magnitude 6.5 occurred during the early morning hours of April 26, 1992 (6.8, 6.0, 6.6, and 6.0). The earthquakes caused damage in the towns of Ferndale, Petrolia. Rio Dell, Honeydew, Fortuna and Scotia with the surrounding areas [5], [6].

- This bridges suffered major damages including buckling and fracture in the end cross frames and their connections and also at the hinge locations.
- Spalling of concrete also found at the connection of top flange of the steel girders and the reinforced concrete deck at the end span.
- Because of the large impacts on the service load capacity of the bridges, large deformations occurred during the passage of trucks.
- The Petrolia Earthquakes highlighted the importance of shear connectors in transferring the lateral forces that are generated by the mass of the bridge’s superstructure.

2.1.2. Behavior of Steel Bridges during the Northridge Earthquake

The Northridge earthquake on January 17, 1994, with a magnitude of 6.7, caused failure of a number of transportation structures that were not designed according to modern seismic codes. Five reinforced concrete bridges in the greater Los Angeles area failed and had to be replaced [7]. The damage to the

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steel structures was minor, with the exception of four that sustained some structural damage to their reinforced concrete substructures, steel diaphragms, and connections of the reinforced concrete substructures to steel superstructures. None of the steel bridges had significant damage to their primary load carrying members. The steel bridges remained functional after the earthquake [8]. This review will focus on the general seismic behavior of five steel bridges during Northridge earthquake.

- The performance of the composite steel girders and concrete slab superstructures were very good.
- In some bridges, there were no signs of transverse movement while in others bridges there was permanent transverse movement in the supports remaining after the earthquake.
- Due to the longitudinal movement of some bridges, the elastomeric pads supporting the bridge girders showed signs of elastic deformation.
- In the most composite bridges, there was no sign of visible earthquake damage in the reinforced concrete substructure above the ground.

2.1.3. Behavior of Steel Superstructure during the Hyogoken-Nanbu 'Kobe' Earthquake

In January 1995 near Kobe, Japan Hyogoken-Nanbu earthquake occurred many bridges in that area were suffered extensive damage, many major roads and rail liens near from Kobe to Osaka due to damaged or collapsed bridges as a result of that earthquake. In the area of severe shaking, the concentration of steel bridges was considerably larger than for any previous earthquake in recorded history. Damage was suffered by superstructure components, seismic restrainers, bearings, many steel piers, and some spectacular collapses resulted from this damage [3] [9] [10].

- For the superstructure the lateral displacements observed for bridge spans that were unseated from their bearings was often impressively large, sometimes producing localized severe lateral bending of the steel girders and even rupture of the end cross frames. Tensile fracture of the bolts connecting end cross frames to the main girders, and fracture through the cross frame extension haunch near the tip of the haunch.
- The bearings suffered a considerable amount of damage during this earthquake. Frequently they were the second structural element to fail following major substructure damage, but many also failed even though the substructure remained intact.
- Many seismic restrainers worked effectively during this earthquake, and prevented simply supported spans from being unseated, numerous restrainers showed signs of plastic yielding or buckling. Others were strained to their limit, often due to excessive sub-structure displacements, and failed.

2.2. Review of Bridges Dynamics

In 1969, Smith [11] presented a study on the dynamic behavior of highway bridge structures as part of the Ontario Joint Highway Research Program. This report was concerned with current knowledge on the dynamic amplification factor, fatigue in bridges and human sensitivity to vibrations. And he also stated the significant parameters that influencing the dynamic amplification factors such as span length, vehicle speed, axles spacing and fundamental frequency.

Creed 1987 [12] in his paper showed that if damage occurs which is sufficient to cause a change in overall stiffness in the structure, the natural frequencies and mode shapes will change. Thus serious damage to a structure can be detected and in some structures approximately located. Finite element model also was used as a theoretical study to make a comparison between field data and theoretical data was obtained by modal 'tuning' or modal updating. The result from the modal updating shows a good agreement for all spans to within 3.5%.

Brownjohn et al. (2003)[13] presented the experimental program clearly demonstrated the viability of noninvasive full-scale dynamic testing allied with FE modal updating for assessing modal and structural parameters of a highway bridge. The application of the procedure to a relatively simple bridge showed clearly the potential for such a procedure to assist bridge managers in assessing their structures by providing validated structural models that may be used for load capacity assessment. For larger bridges, it may be more appropriate and simpler to use ambient or output only testing, but the same identification and updating processes could be applied.

Ren et al. (2004) [14] presented the investigation of historic bridges, John A. Roebling suspension bridge built in the year 1867. The primary objectives are to access the bridge's load carrying capacity and compare this capacity with current standards of safety. Dynamic based evaluation is used, which requires combining FE bridge analysis and field testing. Stiffness parameter variations have been shown to cause some reordering in the sequencing of the natural modes of vibration. FE modal updating is carried out in the comparison paper by adjusting these design parameters so that the live loaded analytical frequencies and mode shapes match the ambient field test frequencies and mode shapes [15].

Zhao and DeWolf (2002) [16] presented the use of dynamic properties that has advantages over static properties in the structural evaluation of bridges since components of the dynamic properties are only marginally influenced by variations in the loading. When dynamic properties are used, field studies have shown that it is not always sufficient to use only natural frequencies and the modal displacements. Some research for structural evaluation of bridges indicates that techniques based on the use of derivatives of the natural frequencies and the modal displacements may be more effectively used to generate effective diagnostic parameters for structural identification. This paper presents the results of applying one of these methods, the modal flexibility approach. The results show that the modified modal flexibility method provides a clear indication that there have been changes in the bridge's structural behavior.

3. FINITE ELEMENT METHOD & MODELING

Finite element analysis is an efficient method can use to determining the static and dynamic performance of structures for some reasons like it can be applied to structures of all forms and complexity, and for some structures or parts, saving in design time, expense of field and laboratory testing of civil engineering structures and increase the safety of the structure.

Three dimensions finite element model of a typical two span composite bridge by done by using ANSYS. ANSYS is in the lead of the commercial finite element software packages in the world and can be applied to a large number of applications in civil engineering. Finite element analysis is available for several engineering disciplines linear, nonlinear, static and dynamic analysis [17].However, the civil engineers need to be trained with the dynamic analysis and create accurate finite element models to be able to develop a reliable bridge analysis.

The Steel concrete composite bridge model first analyzed for the dead loads by static analysis to get the deformed configuration. Modal analysis is done after static analysis with consideration of the effects of deformed structural equilibrium. Deformation under the self-weight of the structure should be small. A time history analysis is performed through the dynamic analysis of the steel concrete composite bridge to know the dynamic response of a steel concrete composite bridge under the earthquake.

3.1. Bridge Modeling Description

The finite element models developed in this research include specifically detailed bridge components. Generally, these components include facets of the plate girders, the cross frames and the concrete deck, each of which will be addressed in the following subsections.

3.1.1. Plate Girders

The model of the plate girders done by creating six key points for every part of the top flange and bottom flange that the cross section thickness varies in it, then areas created by linked the key points together with lines .the web created by four key points with the length of the tow spans of the bridge. To establish the entire girders framework, the areas are generated to the desired locations of the girders. The section properties are then adjusted by applying real constant sets appropriately within ANSYS, i.e.

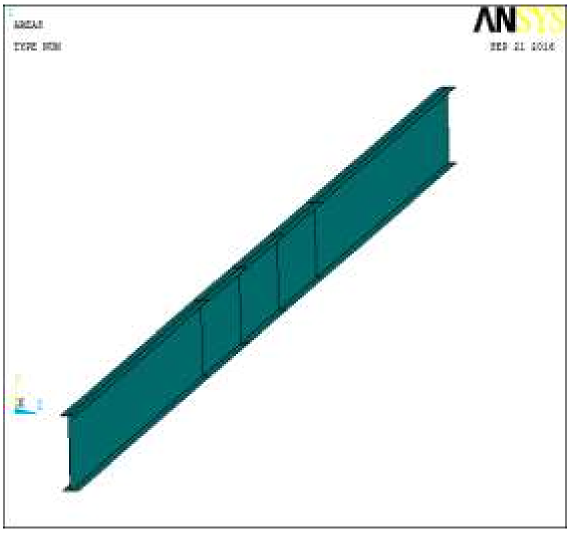

Figure 1. Single plate girder model

3.1.2. Cross Frames

K-type cross frames are used in the bridge of the study. According to the AASHTO LRFD Bridge Design Specifications (2004) [18], the cross frames must: transfer lateral loads from the bottom of the girders to the deck, support bottom flange in negative moment regions, stabilize the top flange before the deck has cured, and distribute the all vertical dead and live loads applied to the bridge. Each cross frame is modeled by creating areas between the girders at the intersection of the web and flange. In the finite element models, each cross frame member is modeled as area element. The cross frame member section properties were acquired from the AISC Manual of Steel Construction with real constant and applied directly into ANSYS [19], [20].

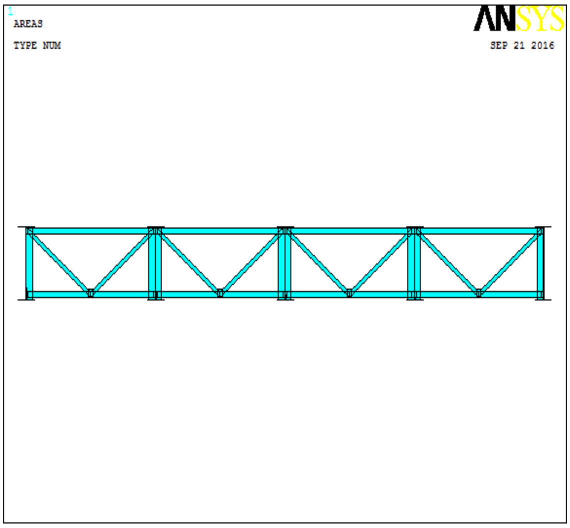

Figure 2. Finite element model of the girders with cross frames

3.1.3. The Concrete Deck

The concrete deck slab was also included in the ANSYS models. The slab was modeled using four node SOILID45 elements. The deck slab modeled by offset the areas of the top flange to the desired location to create the deck slab thickness for every girder .Figure 3 illustrates the technique used to represent the concrete deck slab.

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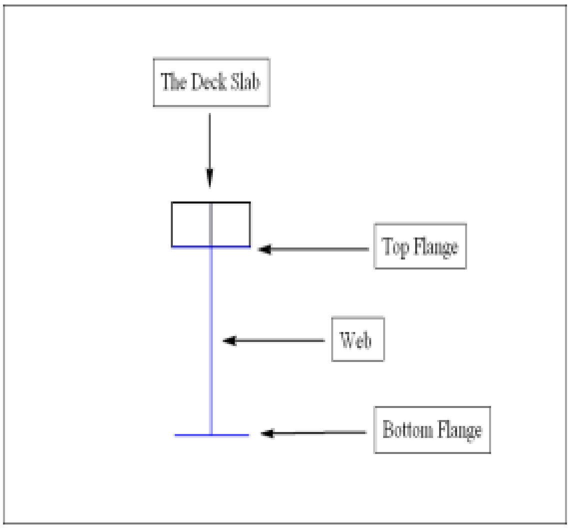

Figure 3. Modeling technique used for concrete deck slab

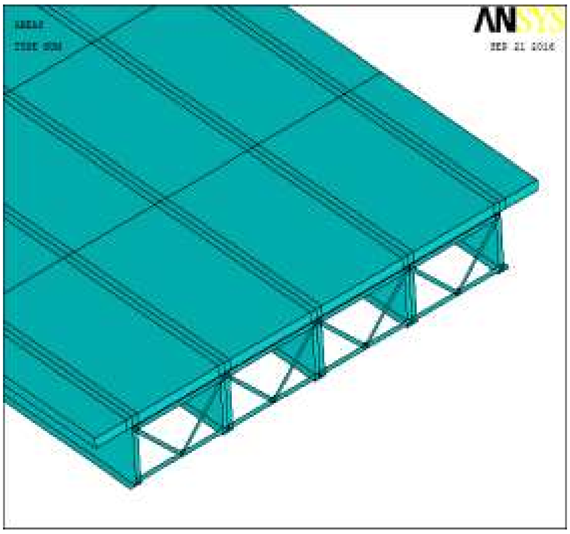

Figure 4. Finite element model of the deck slab

3.1.4. The bearings with abutments and pier

The bearings created with the same technique used for creating the deck slab but the difference was the offset was in the bottom flanges of the girders while the abutments and pier created by creating the key points to create the areas and volumes.

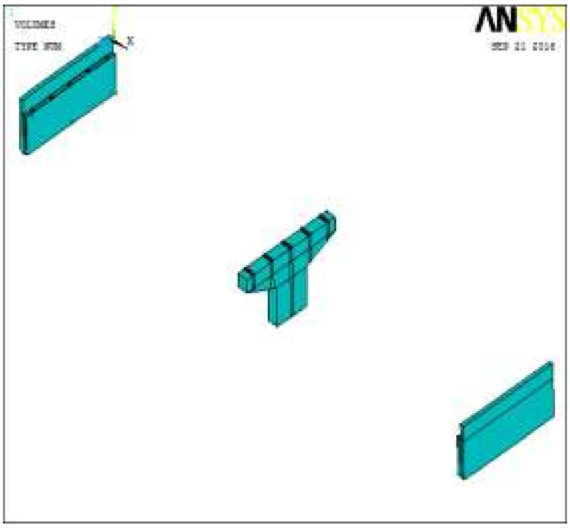

Figure 5. Finite element model of bearings with abutments and pier

3.2. Finite Element Model Input Data for Materials

To simulate the materials behavior in ANSYS different material properties should be input. Table 1 shows the materials properties data used for modeling of the bridge.

Table 1. Materials properties

Material	Elastic Modulus N/mm2	Density ton/mm3	Poisson's ratio
Concrete	2.4×10^{-9}	35000	0.2
Steel	85×10^{-9}	180000	0.3
Rubber	2.5×10^{-9}	60000	0.4999

4. ANALYSIS & RESULTS

4.1. Static Analysis

Results of the static analysis under gravity load are presented in Figures 6 to 10. Figure 6 shows the deflected shape pattern of the steel concrete composite bridge as a result of the self-weight loads under the gravity applied to the model, and Figure 7 shows contours of distribution of the vertical displacement throughout the bridge.

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The maximum vertical displacement is attained at the mid-span region of the spans. The displacement at the midpoint of the spans is evaluated as 23.25 mm while the allowable deflection of the bridge is 46 mm. Figure 8 shows the contours of distribution of the shear stresses in the XY-directions τxy throughout the deck. The shear stresses are distributed in an anti-symmetrical pattern with respect to the centerline of the deck, the results showed that the maximum negative shear stress is 50.9 MPa and the maximum positive shear stress is 26.5 MPa.

The maximum stresses appeared at twelve nodes in the three dimensions, six in the deck slab and six in the girder. On the six nodes in the concrete slab are the maximum stresses as listed in table 2 and in the figure 9 the positions of the nodes.

The six nodes in the girder are also the maximum values of the stress in the three dimensions. The results for the steel girders are listed in Table 3, and the nodes positions showed in figure 10.

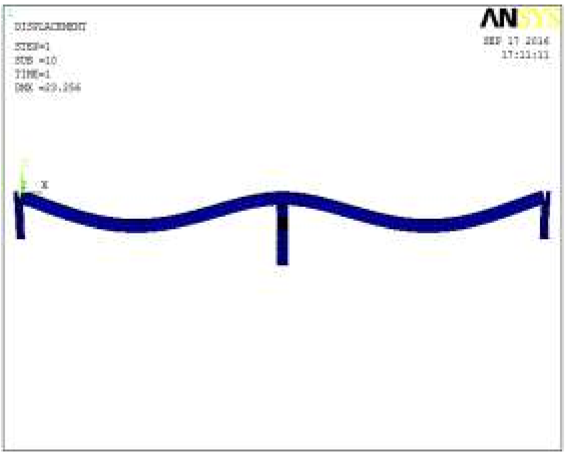

Figure 6. Deflected shape pattern of the composite bridge under static analysis

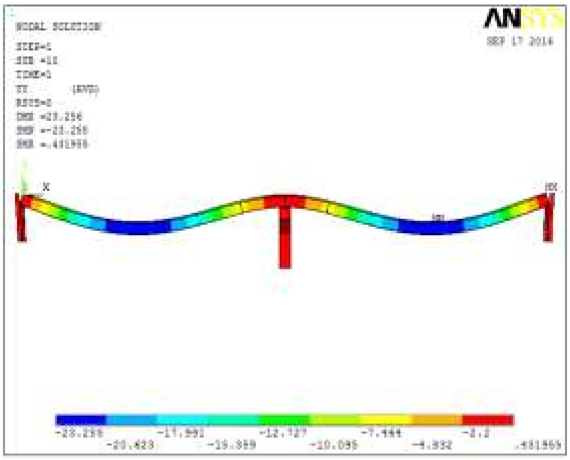

Figure 7. Distribution of vertical displacement (mm) throughout the bridge static analysis

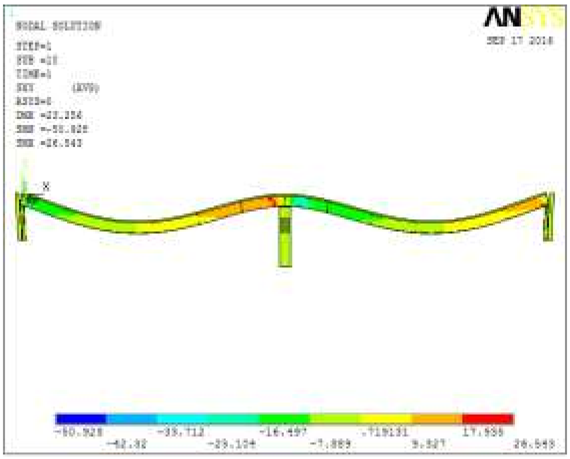

Figure 8. Distribution of shear stress (MPa) throughout the bridge static analysis

Table 2. Maximum stresses on the deck slab (MPa)

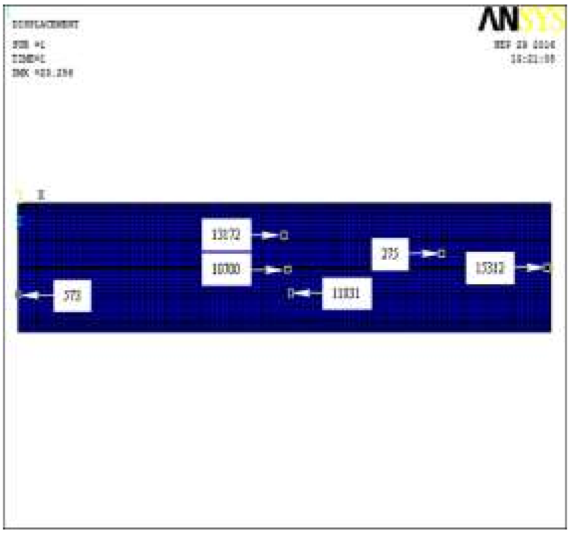

Figure 9. Nodes positions on the deck slab

Table 3. Maximum stresses on the steel girders

Node No.	σx	σy	σz
4552	-53.05	$-0.95555x10^{-6}$	-25.485
2462	-44.739	-54.41	-46.528
5281	-32.683	66.068	-66.544
3421	52.261	$0.12547x10^{-6}$	0.033465
2563	-37.736	130.61	-50.287
2499	21.740	65.912	74.909

(Mpa)

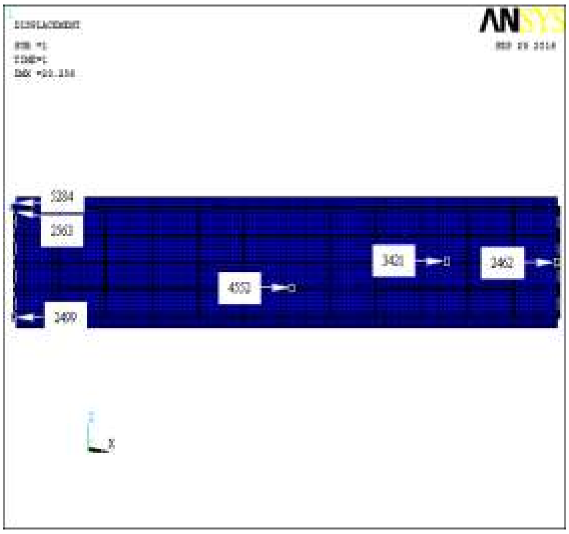

Figure 10. Nodes positions on the steel girders

Node No.	σx	σy	σz
15312	-3.1317	$-0.14044x10^{-3}$	-0.49934
275	-0.36185	-0.69721	-0.42443
13172	0.43324	$-0.62496x10^{-2}$	-0.53242
10700	3.9279	0.012547	0.70126
573	0.20246	1.2023	0.60443
11831	0.38592	0.024273	0.73833

4.2. Model analysis of the bridge

In this study, a suitable model for modal analysis was developed and then used to determine the natural frequencies and mode shapes of the selected bridge. Modal analysis was used as an analysis method for the linear elastic dynamic problem. The natural vibration properties of the bridge are analyzed by the general structural analysis software ANSYS. There are 10 orders of natural frequencies, and mode shapes for the bridge are obtained and used in the calculation of the bridge responses. The descriptions for the first 10 modes are given in In Table 4; the first mode of the two-span continuous steel concrete composite bridge is a bending with a double curvature with a frequency of 2.7441 Hz. The second mode is a torsional mode, with girders in phase and a frequency of 3.1762 Hz. Note that the natural frequencies of the bridge are high, with its tenth frequency only at 5.3288 Hz in maximum. This result indicates that the two-span continuous steel concrete composite bridge is rather rigid. The first five natural frequencies and mode shapes for the Bridge are shown in Figures 11 to 15 below.

Table 4. Natural Frequencies for Bridge

Mode No.	Frequency Hz	Mode description

1	2.7441	Bending with double curvature
2	3.1762	Torsional
3	3.7326	Bending with double curvature
4	3.9721	Torsional
5	4.7558	Lateral
6	5.1548	Lateral
7	5.2785	Coupled mode
8	5.3252	Torsional
9	5.3273	Bending partially with double curvature
10	5.3288	Coupled mode

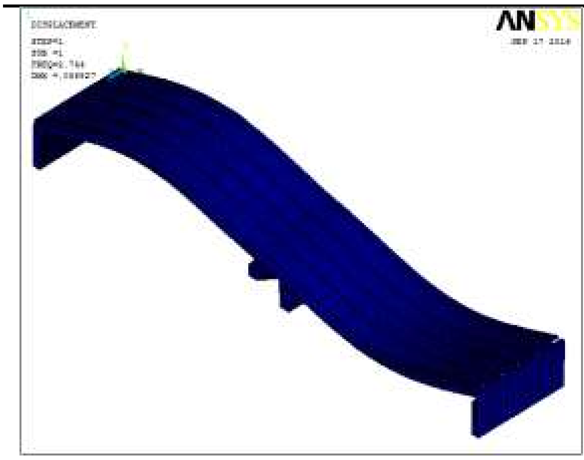

Figure 21. 1st Mode (isometric view)

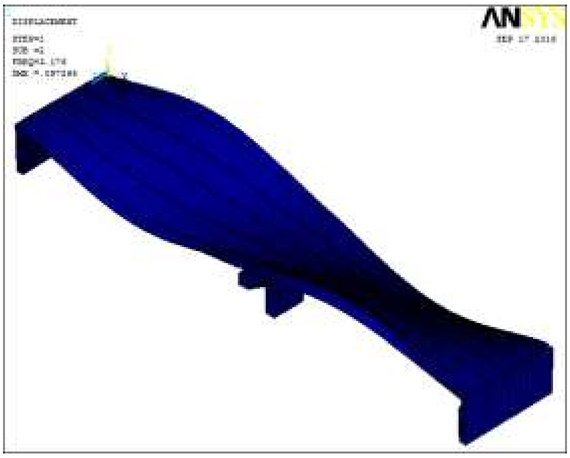

Figure 32. 2nd Mode (isometric view)

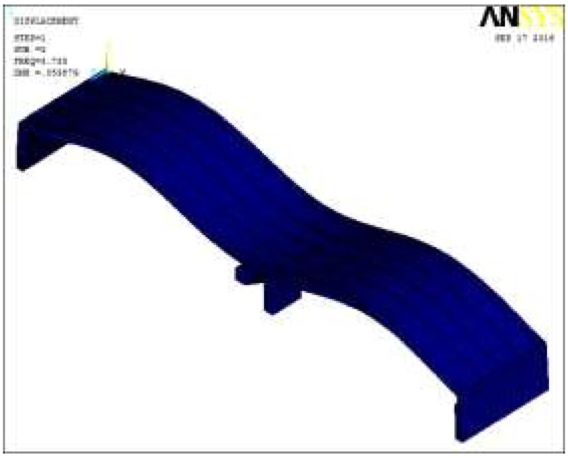

Figure 43. 3rd Mode (isometric view)

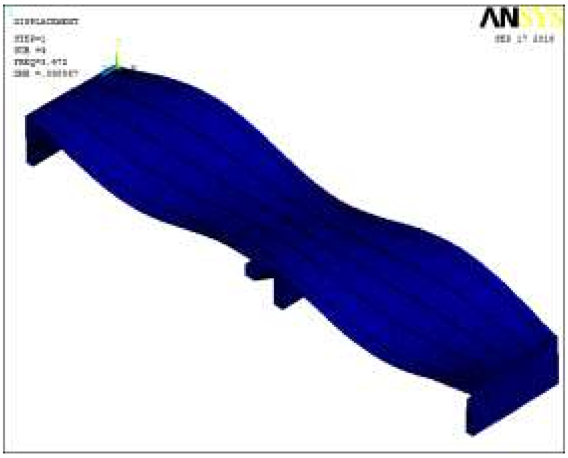

Figure 54. 4th Mode (isometric view)

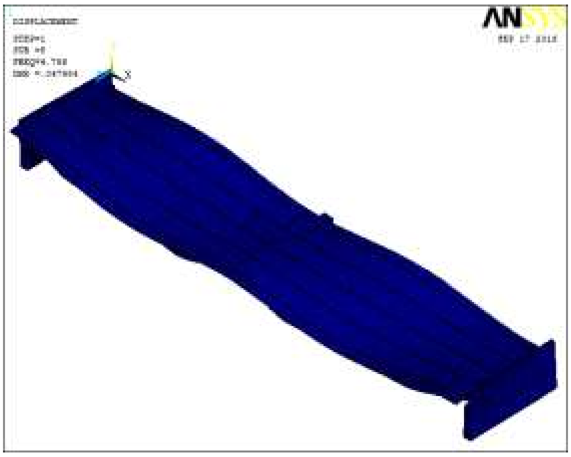

Figure 65. 5th Mode (isometric view)

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4.3. Seismic Analysis of Bridge

4.3.1. Earthquake excitation to the bridge

In US, California during May 18, 1940, EL CENTRO earthquake occurred the maximum peak accelerations in x, y and z directions are, 0.349g, 0.210g and 0.214g, where g is the gravitational acceleration. The acceleration records are presented in figures 16, 17 and 18.

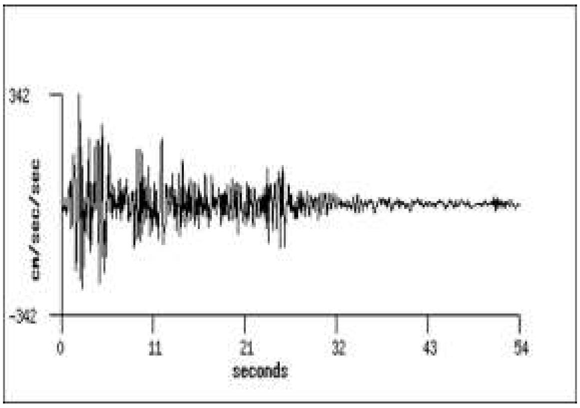

Figure 76. Horizontal acceleration (X direction)

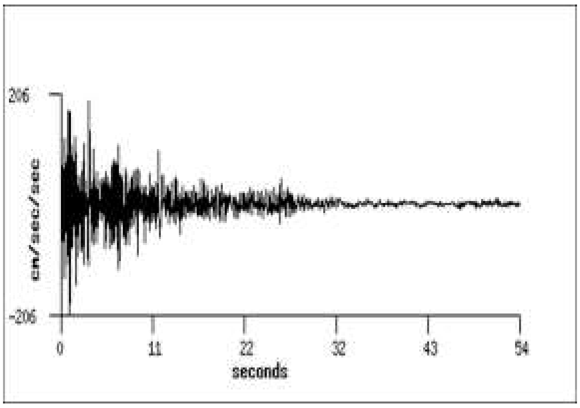

Figure 87. Vertical acceleration (Y direction)

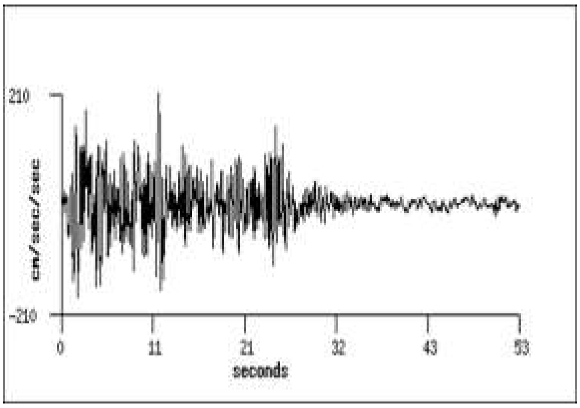

Figure 98. Lateral acceleration

4.3.2. Earthquake responses of the bridge

The dynamic responses of the bridge under the EL CENTRO earthquake accelerations were analyzed. The time period considered for 10 seconds with a time step of 0.0001seconds. Figures 19, 20 and 21 showed the time histories of the displacement responses at the mid-span node of the deck slab. Figures 22, 23 and 24 showed the time histories of the acceleration responses at the mid-span node of the deck slab.

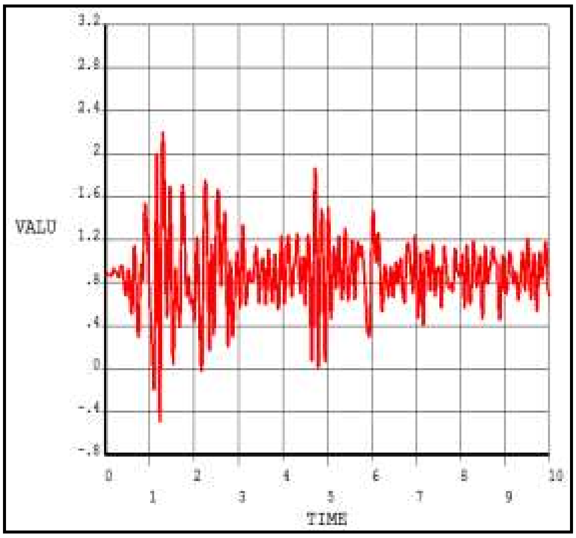

Figure 109. Longitudinal Displacement at the Mid-span (mm)

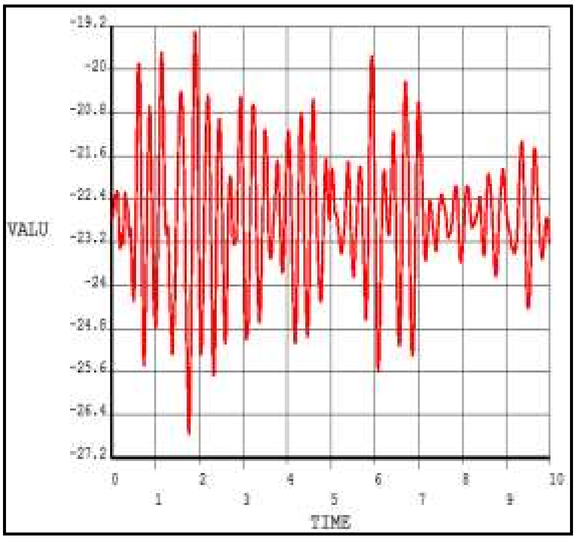

Figure 20. Vertical Displacement at the Mid-span (mm)

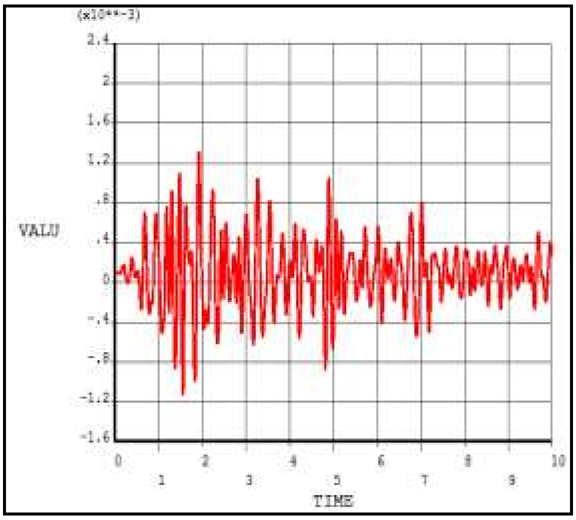

Figure 211. Transverse Displacement at the Mid-span (mm)

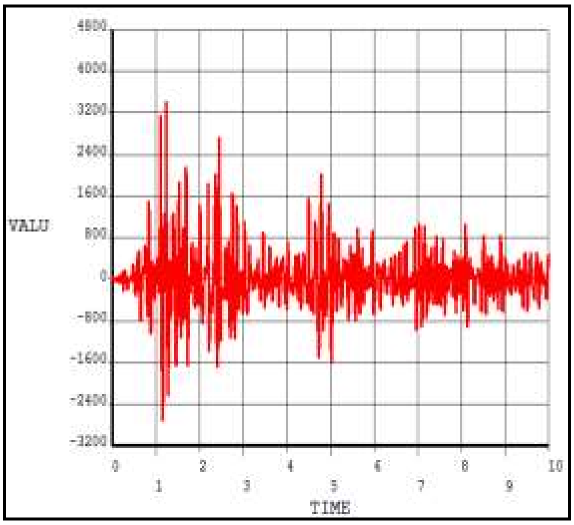

Figure 22. Longitudinal Acceleration at the Mid-span (mm/s^2)

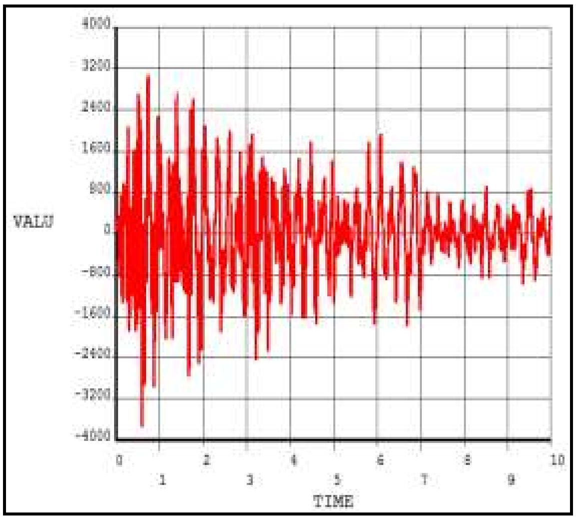

Figure 23. Vertical Acceleration at the Mid-span (mm/s^2)

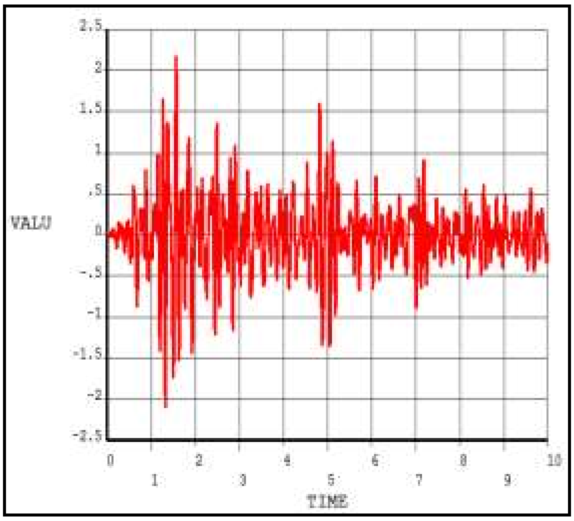

Figure 24. Transverse Acceleration at the Mid-span (mm/s^2)

The maximum displacements and accelerations of the continuous steel concrete composite bridge are listed in Table 5.

Table 5. Maximum responses of the bridge members

Position	Units	Values
Longitudinal displacement	mm	2.3
Vertical displacement	mm	-26.7
Transverse displacement	mm	$2.4x10^{-3}$

Longitudinal acceleration	mm/s^2	3950
Vertical acceleration	mm/s^2	-3800
Transverse acceleration	mm/s^2	2.5

The results in Table 5 indicate that the Vertical maximum displacement is -26.7mm, which occurs at the deck slab node of the mid-span. The vertical maximum displacement is Longitudinal 2.3 mm, which occurs at the mid-span. The Longitudinal maximum acceleration is 3950 mm/s^2, which occurs at the girder mid-span. The vertical maximum acceleration is -3800 mm/s^2, which occurred also at the girder mid-span.

5. CONCLUSION

The study limited to a determination of the seismic behavior of the Steel concrete composite bridge through using structural dynamic properties such as natural frequencies and mode shapes and the dynamic analysis is performed. The modeling and analyzes were done by using ANSYS as finite element software in this study. SOLID45 and SHELL63 selected as elements to modeling the bridge. The boundary conditions chose to be fixed at the bottom of the abutments and the pier. For the bridge model, there is no external loading considered. Just the mass of the bridge model is considered. The selected bridge is modeled with three dimensional within the linear elastic analysis.

The modal analysis with Block Lanczos mode extraction method is selected as a method of the analysis of dynamic response. From this type of analysis, the result of natural frequencies and mode shapes are obtained. For the selected bridge mode shapes have been successfully developed based on its natural frequencies. These mode shapes represent the vibrational characteristics of the bridge decks in bending and torsion. The first mode of the two-span continuous steel concrete composite bridge is a bending with a double curvature with a frequency of 2.7441 Hz. The second mode is a torsional mode, with girders in phase and a frequency of 3.1762 Hz. Note that the natural frequencies of the bridge are high, with its tenth frequency only at 5.3288 Hz in maximum. This result indicates that the two-span continuous steel concrete composite bridge is rather rigid.

The maximum displacements of the bridge under the EL CENTRO earthquake record with design scales are 26.7mm in the vertical direction at the deck slab node of the mid-span, and 2.3 mm in the longitudinal direction at the deck slab node of the mid-span, respectively.

The maximum accelerations of the bridge under the EL CENTRO earthquake record with design scales are 3950 mm/s^2 in longitudinal at the steel girder node of the mid-span, and -3800 mm/s^2 in vertical at the steel girder node of the mid-span, respectively.

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