Comparative analysis of seismic performance of 122-meter long concrete-filled steel tube arched chord truss bridge before and after reinforcement

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ABSTRACT
Finite element analysis of a 122-meter concrete-filled steel tube arched chord truss bridge was performed using ANSYS to obtain the natural vibration characteristics of the bridge, both before and after reinforcement. In addition, the response spectrum and dynamic time history methods were used to analyze and compare its seismic performance. The results show that the transverse stiffness of the bridge’s main truss was relatively low. After the reinforcement, the vertical and the torsional frequencies of the bridge significantly increased by 24% and 32%, respectively. Under the same condition, the axial force at the fixed end of the top chord of the strengthened bridge was reduced by roughly 29%, and the transverse and the vertical displacement at the middle of the top chord span were reduced by roughly 10% and 20%, respectively. Thus, the reinforcement measures significantly improved the vertical stiffness of the bridge. For this bridge, the dynamic time history analysis played a more controlling role in the seismic design. Among the three types of seismic waves, the El Centro wave yielded the largest transverse displacement result and hence, should preferably be used to assess the deflections.

1. Introduction
Concrete-filled steel tube structure is an improvisation based on the traditional steel and concrete structures, whose applications have been growing rapidly (Han, Li, and Bjorhovde 2014). Compared to a hollow steel tube, a concrete-filled steel tube, when used as a chord of the bridge, can fully exhibit the material properties of both concrete and steel. This has the beneficial effect of increasing the chord’s compressive strength and flexural behavior multi-fold. Furthermore, it also improves the bearing capacity of the chord significantly (Han et al. 2015; Hu and Wang 2017; Zanuy 2019). In recent years, both the number and scale of concrete-filled steel tube bridges are increasing significantly, worldwide. Two important considerations in bridge seismic studies are (i) ensuring satisfactory bridge performance during an earth quake and (ii) maintaining the operation of “life-line engineering” (Rajeev, John Peter, and Varkey 2017). Therefore, it is very crucial to study the natural vibration characteristics, as well as the seismic properties of concrete-filled steel structures (Yan, Li, and Chen 2011; Montejo, González-Román, and Kowalsky 2012; Bi, Hao, and Ren 2013; Yuan, Dang, and Aoki 2014; Stephens, Lehman, and Roeder 2018). The concrete-filled steel tube arched chord truss bridges offer advantages, such as flexible component layout, aesthetic modeling, large leapfrogging ability, and excellent mechanical properties, and thus, have been applied in bridge engineering over the past decades, its shape is shown in Figure 1. The mechanical properties of a trussed structure of concrete-filled steel tubes, which is an important component of this type of bridges affects the seismic stability of the whole bridge (Hou and Han 2017; Hou et al. 2017; Xie et al. 2019; Chen et al. 2019). The natural frequency and mode of structure are important indicators that reflect the dynamic performance of the bridge, and they are also the basis for analyzing the seismic performance of the bridge (Fouche, Bruneau, and Chiarito 2017; Xin et al. 2019; Li et al. 2014). Furthermore, the bridge’s response during an earthquake needs to be analyzed by two methods, namely, the response spectrum method and the dynamic time history method. The response spectrum method can evaluate the seismic performance of simple bridges. However, for complex bridges, the results from the response spectrum method need to be compared with those of the dynamic time history method (Ates and Constantinou 2011; Wang, Ma, and Zhu 2017; Huang et al. 2017).

As the concrete-filled steel tube arched chord truss bridges are still relatively less common, studies on the seismic performance of such bridges are rare. Therefore, in this research, we considered a 122-meter long concrete-filled steel tube arched chord truss bridge as the research object. Keeping in view its reinforcement scheme, spatial finite element models for the bridge, before and after reinforcement, were created using the general purpose finite element...
method software ANSYS®. The natural vibration frequencies of the bridge, both before and after the reinforcement, were calculated. The bridge’s seismic responses obtained from the response spectrum and dynamic time history methods were compared and analyzed, and the improvement in its seismic performance post the reinforcement measures was evaluated. The outcome of this research is expected to serve as a reference for the seismic design and reinforcement maintenance of this type of bridges in future.

2. Development of the finite element model

2.1. General description of the bridge

The concrete-filled steel tube arched chord truss bridge, considered in this study, is a simply supported system with a span of 122 m and a deck width of 28.8 m. The spacing between the main trusses is 16.4 m and the maximum height of the trusses is 15.250 m. The upper chord is of concrete-filled steel tube structure. The diameter and the wall thickness of the steel tube are 1.4 m and 24 mm, respectively and the tube is filled with C40 concrete. The lower chord is an open steel box girder with a height of 0.648 m, and top and bottom widths of 0.8 m and 0.56 m, respectively. Three types of hollow steel tubes, \( \Phi 600 \times 8 \text{ mm} \), \( \Phi 600 \times 12 \text{ mm} \) and \( \Phi 600 \times 16 \text{ mm} \), are used for web members and cross braces. The span between the beams is 5 m. The bridge deck is tiled with a 5-cm thick and 29-cm wide preform board between the beams, and the integral bridge deck is formed by “cast-in-place” method.

The main sectional dimensions of the truss are shown in Figure 2.

A transverse beam is set every 1 m along the bridge span, with an open steel box girder. The cross-sectional view of the beam is shown in Figure 3. The two longitudinal beams are 16.4 m apart along the transverse direction of the bridge and open steel box beams are used. Its cross section is shown in Figure 4.

2.2. Overview of the bridge reinforcement scheme

Having been in service for more than 20 years, the bridge has developed a number of defects, which need to be repaired and the bridge is to be reinforced according to a reinforcement scheme. In 2014, a design institute developed the reinforcement and transformation scheme of the bridge, which includes (i) strengthening the ends of the upper chord by adding hangers, diagonal webs, and truss beams; (ii) increasing the length of the...
original bridge beam, web height of the tie beam, and height of the bridge deck, and (iii) adding two small beams in the internodes. The steel bridge deck is to be strengthened by a U-shaped rib orthotropic plate to improve the overall stability of the bridge structure.

Firstly, the ends of the upper chord were polished, and shearing nails were welded on the surface of the upper chord. A total of 12 shearing nails were evenly arranged on each section, and the section spacing along the axis of the upper chord was 30 cm. A schematic of the upper chord end reinforcement with concrete and a reinforced steel casing is shown in Figure 5.

Secondly, the bridge deck was reinforced with U-shaped rib orthotropic plates. The schematic diagram of the integral bridge deck is shown in Figure 6. The height of the transverse beam web was increased to improve the lateral stiffness of the bridge. The cross section of the transverse beam reinforcement is shown in Figure 7. The height of the stringer web was increased on the basis of the original bridge’s longitudinal beam to increase the longitudinal stiffness of the bridge. The cross section of the longitudinal beam is shown in Figure 8. The distance between the transverse beams of the original bridge was 5 m and two additional small beams were added in each of the two transverse beam joints to withstand the force. The section of the additional small beam is shown in Figure 9.

2.3. Finite element models of the bridge

The finite element models of the original bridge, before and after the reinforcement were built using the finite element software ANSYS®. The double-element method was used to simulate the concrete-filled steel tube structure of the upper chord. The steel tube and the concrete elements were included in the model and simulated by common nodes mode. A beam element for the steel tube was established first, and then the concrete beam element was established on the existing nodes (Su and Hu 2003; Wang 2006). The upper chord and the web member were simulated by using beam189 elements; the lower chord, the cross beam, and the cross brace were discretized by using beam188 elements; and the pre-
A stressed steel strand was simulated by using the 3-dimensional rod element, i.e., Link 8 element; the concrete bridge deck was discretized by using Shell63 element. The cross brace and the main truss upper chord realized the degree of freedom coupling between nodes through master-slave node constraints, with the upper chord node being the main node, and the end node of the cross brace being the slave node. According to the actual configuration of the bridge, one side of the left end of the upper chord was a fixed hinge support and the other side was a transverse unidirectional movable support. One side of the right end of the upper chord was a longitudinal, unidirectional movable support and the other side was a bidirectional sliding hinge support. In the model of the bridge before the reinforcement, the bridge had 997 nodes and 1781 elements, which included 1401 beam elements and 380 plate elements, as shown in Figure 10.

The reinforced steel bridge deck was simulated by converting the U rib into the thickness of the bridge deck through the moment of inertia of the cross section. Based on the principle of linear stiffness equivalence, the truss beam strengthened by the diagonal web members was converted into equivalent rectangular section members. The added boom was simulated by a Link8 element, and the initial tension of the boom was considered by way of initial strain. The reinforced bridge model had a total of 1,955 nodes and 4,129 elements, which included 2,837 beam elements and 1,292 plate elements, as shown in Figure 11.
3. Analysis of natural vibration characteristics

To analyze the natural vibration characteristics of the bridge, the Lanczos method in ANSYS modal analysis was used to calculate the natural vibration frequencies of the concrete-filled steel tube arched chord truss bridge before and after the reinforcement. A comparison of the first two natural frequencies of the bridge, before and after the reinforcement, which were predominantly transverse, vertical, and torsional vibrations, is shown in Figure 12.

As can be seen from Figure 12, the natural vibration frequency of the strengthened bridge, especially, the vertical and torsional frequencies, increased to different degrees. The first vibration mode of the bridge before and after the reinforcement was the transverse vibration of the main truss, which indicates that the transverse stiffness of the main truss was low. In addition, the torsional vibration occurred after the transverse and vertical vibrations, indicating that the overall torsional stiffness was greater than the transverse and vertical stiffness.

4. Seismic response spectrum analysis of the bridge before and after reinforcement

Based on the above models, the response spectrum method and the ANSYS software were used to analyze the linear elasticity of the concrete-filled steel tube arched chord truss bridge before and after the reinforcement under an E1 earthquake action.

The geographic area, in which the bridge is located, belongs to seismic category II and is a 7-degree seismic fortification zone. The horizontal acceleration response spectrum based on the “Code for Seismic Design of Highway Bridges” of China, with a corresponding damping ratio of 0.05 is determined by Equation (1):

$$ S = \begin{cases} S_{\text{max}}(5.5T + 0.45), & T < 0.1s \\ S_{\text{max}}, & 0.1s \leq T < T_g \\ S_{\text{max}}(T_g/T), & T \geq T_g \end{cases} $$

where $T_g$ is the characteristic period of the response spectrum and is equal to 0.40 s, as per the characteristic periodic zoning map of the acceleration response spectrum of China’s earthquake ground motion; $T$ is
the natural period of the structure; \( S_{\text{max}} \) is the maximum value of the horizontal design acceleration response spectrum (=1.654m/s²).

The horizontal design response spectrum curve used in the structural seismic analysis is shown in Figure 13.

The complete quadratic combination (CQC) method was used to input the seismic effects in the longitudinal, transverse, and vertical directions of the bridge. From the specification, the following three working conditions were considered:

- Working condition 1: \( \text{EX} + 0.3\text{EY} + 0.3\text{EZ} \);
- Working condition 2: \( 0.3\text{EX} + \text{EY} + 0.3\text{EZ} \);
- Working condition 3: \( 0.3\text{EX} + 0.3\text{EY} + \text{EZ} \).

Here, \( \text{EX}, \text{EY}, \) and \( \text{EZ} \) are the seismic actions along the longitudinal, transverse, and vertical directions of the bridge, respectively.

4.1. Internal force analysis

To analyze the changes in the internal force for typical sections of the bridge under the action of ground motion, before and after the reinforcement, the peaks of internal force at the fixed end of the upper chord of the bridge were extracted and compared, as shown in Table 1, and a comparison chart of the axial force peaks is shown in Figure 14.
As can be seen from Figure 14, under the same working condition, the axial force after the bridge reinforcement was reduced by 27%–30%, compared to that before the reinforcement. As can be seen from Table 1, under various working conditions, the upper chord bore the largest axial force, while the shear force and the bending moment were smaller. Compared to the other two working conditions, the transverse shear force and out-of-plane bending moment of the
bridge under the second working condition increased, as expected, which was owing to the bridge’s sensitivity to transverse ground motion and the relatively weak transverse stiffness of the bridge.

4.2. Displacement analysis

To analyze the displacement change of the bridge under the action of ground motion before and after reinforcement, the peak displacements at the $\frac{1}{4}L$ position of the upper chord of the bridge were extracted and compared, as shown in Table 2, and the comparison diagram of the transverse displacements of the bridge at the $\frac{1}{4}L$ position of the upper chord of the bridge before and after reinforcement is shown in Figure 15.

From Table 2 and Figure 15, it can be seen that under the three working conditions, after the reinforcement, the transverse displacement at $\frac{1}{4}L$ of the upper chord of the strengthened bridge decreased by 7.8%, 7.9%, and 7.9% respectively, when compared to that before the reinforcement. Similarly, the vertical displacement decreased by 18.2%, 20%, and 19.1% respectively, indicating that the reinforcement measures improved the transverse stiffness of the bridge, and significantly increased the vertical stiffness of the bridge. Compared with other working conditions, the lateral displacement response under the second working condition was the most significant, which indicates that the lateral stiffness of the bridge was relatively weak.

5. Dynamic time history analysis of the seismic performance of the bridge before and after reinforcement

The seismic behavior of the concrete-filled steel tubular arched truss girder bridge before and after the reinforcement was also analyzed by using ANSYS. Three types of waves for earthquake simulation, namely, the El Centro wave, Taft wave, and TAR-TARZANA wave were selected from the Pacific earthquake database, and the three seismic waves were calibrated according to the current site conditions. Owing to the limitation of space, only the corrected El Centro wave results are shown in Figure 16. According to the seismic design code for highway bridges, for each seismic wave, three kinds of working conditions similar to those in the response spectrum analysis were considered. Thus, a total of nine working conditions were used.

Table 2. Comparison of displacement peak at $\frac{1}{4}L$ of the bridge upper chord before and after reinforcement.

| Bridge condition | Working condition category | UX (mm) | UY (mm) | UZ (mm) | ROTX (rad) | ROTY (rad) | ROYZ (rad) |
|------------------|---------------------------|---------|---------|---------|------------|------------|-----------|
| Before reinforcement | Working condition 1 | 2.153 | 22.86 | 8.598 | 1.26E-3 | 1.54E-4 | 5.50E-5 |
|                   | Working condition 2 | 1.474 | 76.193 | 7.755 | 4.07E-3 | 1.07E-4 | 1.76E-4 |
|                   | Working condition 3 | 2.069 | 22.867 | 12.519 | 1.29E-3 | 7.20E-5 | 1.76E-4 |
| After reinforcement | Working condition 1 | 1.803 | 21.074 | 7.033 | 1.09E-3 | 1.33E-4 | 6.00E-6 |
|                   | Working condition 2 | 0.949 | 70.186 | 6.203 | 3.54E-3 | 5.20E-5 | 1.50E-4 |
|                   | Working condition 3 | 1.824 | 21.06 | 10.128 | 1.08E-3 | 5.60E-5 | 5.00E-6 |

UX, UY, and UZ indicate the longitudinal, transverse, and vertical displacements of the bridge, respectively; ROTX, ROTY, and ROYZ are the rotation angles around the X-, Y-, and Z- axes, respectively. The following symbols have the same meaning.

Figure 15. Comparison of the transverse displacement peak at $\frac{1}{4}L$ of the upper chord of the bridge before and after reinforcement.
5.1. Internal force analysis

Due to the space limitation, only the calculation results of the Taft wave were extracted for the internal force, and the peak values of the internal force at the fixed end of the upper chord, before and after the bridge reinforcement, were compared and analyzed, as shown in Table 3, and a comparison of the axial forces is shown in Figure 17.

As can be seen from Figure 17, under the same working conditions, the force on the front axle of the bridge after the reinforcement was reduced by 29%–32%. Similarly, from Table 3, it may be observed that the axial force on the upper chord was the largest under each working condition. After the bridge was reinforced, the axial force on both ends of the upper chord was reduced, and the shear force and the bending moment were increased. Under working condition 2, the transverse shear force and the out-of-plane bending moment of the bridge increased compared with the other two working conditions, which shows that it was more sensitive to the transverse ground motion of the bridge and was basically consistent with the internal force law of the above-mentioned response spectrum analysis.

5.2. Displacement analysis

The calculation results of the El Centro wave were extracted, and the displacement peaks at $\frac{1}{2}L$ of the
The upper chord of the bridge was compared and analyzed, as shown in Table 4, and the comparison of the transverse displacement at $\frac{1}{2}L$ of the upper chord of the bridge, before and after reinforcement, is shown in Figure 18.

From Table 4 and Figure 18, it can be seen that the bridge’s transverse displacement was the largest under all the working conditions, indicating that the bridge’s lateral stiffness was still relatively weak. Compared with that before reinforcement, the lateral displacement at the upper chord $\frac{1}{2}L$ of the bridge after reinforcement was reduced by 10%–17%, and the vertical displacement was reduced by 18%–24%. This shows that the vertical stiffness of the bridge was greatly improved. As a result, its seismic performance also improved after post reinforcement. After the

Table 4. Comparison of the displacement peak at $\frac{1}{2}L$ of the bridge upper chord before and after reinforcement.

| Bridge condition   | Working condition category (mm) | UX (mm) | UY (mm) | UZ (mm) |
|--------------------|---------------------------------|---------|---------|---------|
| Before reinforcement| Working condition 1             | 2.5     | 35.4    | 9.7     |
|                    | Working condition 2             | 1.5     | 116.7   | 8.7     |
|                    | Working condition 3             | 1.8     | 39.0    | 11.4    |
| After reinforcement | Working condition 1             | 1.7     | 31.3    | 7.4     |
|                    | Working condition 2             | 1.1     | 104.8   | 7.1     |
|                    | Working condition 3             | 1.3     | 32.4    | 8.9     |

Figure 17. Comparison of the axis force peak at the fixed end of the upper chord of the bridge before and after reinforcement.

Figure 18. Comparison of the lateral displacement peak at $\frac{1}{2}L$ of the bridge upper chord before and after reinforcement.
reinforcement, the lateral displacement response of the bridge was still significant under the second working condition, which indicates that the lateral stiffness of the strengthened bridge was still low.

5.3. Comparative analysis of the results of response spectrum method and the dynamic time history method

According to the seismic responses of the bridge, before and after the reinforcement, as calculated by the response spectrum method and the dynamic time history method, the fixed end, $\frac{1}{4}L$, $\frac{1}{2}L$, $\frac{3}{4}L$ and the movable end at the upper chord were selected as typical sections, along the longitudinal direction to output the internal force and the displacements of the bridge. For the case, in which the seismic wave input was mainly along the longitudinal direction of the bridge (i.e., $EX + 0.3EY + 0.3EZ$), the internal force comparison diagrams for the aforementioned sections at the upper chord are shown in Figures 19 and 20. In these figures, the legend “original bridge fyp” represents the results of the response spectrum method before the bridge reinforcement, and the other legends are self-explanatory.

As can be seen from Figures 19 and 20, the internal force of the bridge obtained by the response spectrum method was smaller than that obtained by the dynamic time history method. The internal force of the upper chord of the bridge was mainly axial force, and the maximum bending moment appeared at the positions of both ends of the upper chord. The axial component of the internal force of the strengthened bridge was reduced post reinforcement.

Under different working conditions, the peak value of the transverse displacement at $\frac{1}{2}L$ of the upper chord of the bridge is shown in Table 5, from which it can be seen that the displacement of the bridge

![Figure 19. Comparison of the axial force before and after the bridge reinforcement under different working conditions.](image1)

![Figure 20. Comparison of the in-plane bending moment of the bridge before and after reinforcement under different working conditions.](image2)
obtained by the response spectrum analysis method was smaller than that calculated by the dynamic time history method overall. Under the action of the El Centro seismic wave, the displacement value of the bridge was the largest.

6. Conclusion

In this study, a 122-meter concrete-filled steel tube arched chord truss bridge, before and after reinforcement, was taken as the research object. The natural vibration characteristics of the bridge were calculated, and the seismic performance of the bridge was analyzed by response spectrum and dynamic time history methods. The following main conclusions could be obtained from this research:

(1) After the reinforcement, the natural frequency of the bridge increased as the fundamental frequency of the bridge increased from 0.719 to 0.773 Hz. Both before and after the reinforcement, the first vibration mode of the bridge was the transverse vibration of the main truss, which indicates that the transverse rigidity of the main truss was relatively low. The torsional vibration occurred after the transverse and the vertical vibrations, indicating that the torsional rigidity was relatively large.

(2) Under the same working condition, the axial force at the fixed end of the upper chord of the bridge post reinforcement decreased by roughly 29% compared to that before the reinforcement. Under various working conditions, the upper chord of this type of bridge bore the largest axial force, and the shear force and the bending moment were smaller. The maximum bending moment of the bridge basically appeared at both ends of the upper chord.

(3) Under different working conditions, the displacement variations before and after the reinforcement were basically the same. The displacement in the transverse direction was greater than that in the longitudinal and vertical directions, and the transverse displacement response was most sensitive when a lateral ground motion was supplied. The transverse and the vertical displacements at the mid-span of the upper chord of the reinforced bridge were reduced by roughly 10% and 20%, respectively compared to that before the reinforcement. Overall, the reinforcement measures have improved the lateral stiffness of the bridge and significantly increased the vertical stiffness of the bridge.

(4) The response of the bridge obtained by the response spectrum method was smaller than that calculated by the dynamic time history method, which shows that the results from the latter played a more controlling role in the seismic design of the bridge; however, their results were fundamentally consistent. Among the three types of seismic waves, under which the bridge was simulated, the displacements from the El Centro wave were the largest. Therefore, the deflection calculation should be carried out using the results of El Centro wave.

Disclosure statement

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Table 5. Comparison of transverse displacement peak at $\frac{1}{2}L$ of the bridge upper chord before and after reinforcement under different working conditions (mm).

| Bridge condition | Main direction of earthquake input | Response spectrum analysis | Dynamic time history analysis |
|------------------|-----------------------------------|----------------------------|----------------------------|
|                  |                                   | El Centro | Taft | Tar-Tarzana |
| Before reinforcement | longitudinal direction           | 22.9      | 35.4 | 25.2 | 19.3 |
|                   | transverse direction                | 76.2      | 116.7 | 81.4 | 62.3 |
|                   | Vertical direction                   | 22.9      | 39.0  | 27.0 | 20.3 |
| After reinforcement  | longitudinal direction           | 21.1      | 31.3  | 23.7 | 19.2 |
|                   | transverse direction                | 70.2      | 104.8 | 75.6 | 62.1 |
|                   | Vertical direction                   | 21.1      | 32.4  | 27.8 | 21.6 |
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