Dynamic Behaviour of a Cable - Stayed Bridge under Earthquake and Traffic Loads

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Abstract

This paper focuses on the effects of earthquake and traffic loadings on the dynamic behaviour of cable-stayed bridges. Tatara Bridge, which is the second longest cable-stayed bridge in the world, is considered as a numerical example in this study. For the dynamic analysis of the 3D finite element model of the Tatara Bridge, multi-support earthquake ground motion and traffic loading applications are considered. For the multi-support excitation analysis, two close recordings of Chi Chi-Taiwan earthquake ground motions recorded on medium soil condition are used. Traffic loading is taken into account by considering the traffic loading as moving traffic load. A combined excitation case considering the traffic and earthquake loadings together are also applied to the bridge model. The results show that the structural responses obtained for the multi-support earthquake excitation case are usually larger than the responses determined for the traffic loading case. Depending on the results, it is concluded that the combined effect of the earthquake and traffic loadings should be considered for the realistic design of cable-stayed bridges.

Keywords: Cable - Stayed Bridges, Multi Support Excitation, Traffic Loading.

1 Introduction

Due to the increasing traffic demand and construction of cable-stayed bridges in earthquake zones with soft soil conditions, detailed dynamic analyses of these bridges under earthquake and traffic loads has become very important for realistic design. For the dynamic analysis of these structures, it is usually assumed that the bridge supports are subjected to uniform ground motion. Considering the increment in the center span of cable-stayed bridges in recent years, non-uniform nature of the ground motion should be considered in the analyses and applied to these bridge systems as multi-support excitations. It is also obvious that cable-stayed bridges will be subjected to significant traffic loads, especially in heavily populated regions like Marmara region.

No study has been encountered in the literature examining the relative effects of multi-support and traffic loadings in determining the dynamic responses of cable-stayed bridges. Several past studies concerning bridge structures focused on the effects of multi-support excitation and traffic loading, separately. For example, cable-stayed bridge models were investigated for the multi-support excitation without considering the traffic loadings (Gurevski et al., 1987, Nazmy and Abdel-Ghaffar, 1992, Soyluk and Dumanoglu, 2000, Allam and Dutta, 1999, Lin et al., 2004, Ferreira and Negrao, 2006, Soyluk and Sıvacık, 2012). All these studies underline the importance of the multi-support ground motion effect. Cable-stayed bridges were even investigated for traffic loadings without considering the multi-support ground motion excitation (Khalifa, 1993, Zaman et al., 1996, Yang and Folder, 1998, Bruno et al., 2008, Cengiz et al., 2011). The importance of traffic loading on cable-stayed bridges was emphasized in these studies.

It seems that the relative importance of the multi-support ground motion and traffic loading effects on cable-stayed bridges has not been carried out so far. Therefore, this study focuses on defining and comparing the dynamic behaviour of cable-stayed bridges under earthquake and traffic loads. For this purpose, two close recordings of Chi Chi-Taiwan earthquake ground motions recorded on medium soil condition are used in this
study to define the multi-support excitation case. On the other hand, traffic loading is taken into account by considering the traffic loading as moving traffic load. A combined excitation case considering the traffic and earthquake loadings together are also applied to the bridge model.

2 Theoretical Formulations

2.1 Multi-Support Excitation

Total structural displacement \( v \) that results from the ground motion excitation can be expressed as a sum of two components. One of these components is the result of differential ground motions and the other one is the result of the inertia forces. Since the inertia forces are the product of acceleration and mass, the corresponding displacement component is defined as dynamic and expressed by \( v_d \). If differential ground motions are considered, various displacements will be observed at the support degrees of freedom of the structural system. Since the effect of the mass will be disregarded during the calculation of these displacements, these displacements are called as pseudo-static displacements and expressed by \( v_s \). Therefore; the total displacement will be defined as

\[
v = v_s + v_d
\]

(1)

where \( \{v\}, \{v_s\}, \{v_d\} \) are the total, pseudo-static and dynamic displacement vectors, respectively. The dynamic displacement vector is defined as

\[
\{v_d\} = \sum_i \{\phi_i\} Y_i(t)
\]

(2)

where \( \{\phi_i\} \) is the \( i \)th modal vector and \( Y_i(t) \) is the \( i \)th modal amplitude with respect to time. The pseudo-static displacement vector is defined as

\[
v_s = r_1 v_{ig} (r_1, t) + r_2 v_{ig} (r_2, t) + ...
\]

(3)

where \( \{r_i\} \) is the shape vector of ground displacement, \( \{v_{ig}\} \) is the displacement vector of ground acceleration, \( t \) is time and \( r_{ij} \) is the time of arrival of ground acceleration to the point of support \( i \) or to the area beginning from certain reference point (Soyluk and Dumanoğlu, 2000).

2.2 Vehicle - Bridge Interaction

The elastic and the inertia effects of vehicles may be omitted to define simpler models in cases where the vehicle mass is less than the bridge mass. This kind of vehicle model is defined as moving load model (Figure 1). If the bridge–vehicle interaction is neglected, the moving load model is used only in the calculation of the reaction forces of the main structural system. In this model, although the contact forces are neglected, the basic dynamic characteristics of the bridge under moving loads can be reflected quite accurately (Yang et al., 2004).

![Figure 1. Moving load model (Yang et al., 2004).](image)
In cases where the inertia of the vehicle can not be neglected, the moving mass model shown in Figure 2 should be used instead of the moving load model.

![Figure 2. Moving mass model (Yang et al., 2004).](image)

In the moving load model, bouncing of the moving mass relative to the bridge can not be taken into account. If there is pavement roughness or vehicles are moving with rather high speeds along the bridge, then it is expected that the bouncing effect will be important. The corresponding vehicle model in this case will be a moving mass supported by a spring-dashpot unit, the so called sprung mass model (Figure 3) (Yang et al., 2004).

![Figure 3. Sprung mass model (Yang et al., 2004).](image)

Because the ratio of the considered vehicle mass to the bridge mass is low and only the dynamic behavior of the bridge is investigated, the moving load model is used in this study as a vehicle model.

3 Numerical Studies

Tatara cable-stayed bridge is preferred as a numeric example in this study to determine the dynamic behaviour of cable-stayed bridges under earthquake and traffic loads. The Tatara Bridge ranked the longest cable–stayed bridge in the world with a main span of 890 m at the date of construction. 3D finite element model of the bridge is prepared with SAP 2000 Advanced V4.1.0 Software. Since the main span of the bridge is quite long, different ground motion recordings are applied to the support points of the bridge through the multi-support excitation. For traffic loading, moving traffic loads are used to determine the maximum responses of the bridge system. As vehicle types, HS20-44 heavy truck and H20-44 medium truck defined in AASHTO are used.

In this study, static traffic loading case that corresponds to medium traffic loading on the bridge deck is considered and combined with the multi support excitation case to define a combined excitation case. If the design criteria of the Tatara Bridge and AASHTO are examined, it can be observed that half of the traffic load is taken into account when earthquake and traffic loads are taken into consideration together. As a result of this observation, only H20-44 truck is used as the traffic load when earthquake and traffic loads are considered together. Due to non-linear behaviour of cables, non-linear time history analysis of the bridge is considered. This non-linear analysis has been started from the dead load deformed shape.

3.1 General Characteristics of Tatara Cable-Stayed Bridge

Tatara Bridge ranked the longest cable–stayed bridge in the world at the date of construction (1999) is connecting Ikuchijima and Ohmishima Islands in Japan. Total length of the bridge is 1480 m having a main span of 890 m, and side spans of 270 m and 320 m. While the main span deck is made from steel, the side span decks are made from steel and prestressed concrete. General layout of the bridge is given in Figure 4. As the deck section is mostly steel along the bridge length (1312 m), prestressed concrete is used at each end of side span sections to prevent negative reaction. The support conditions are summarized in Table 1 (Yabuna et al., 2001).
### Table 1 Supporting conditions of the bridge (Yabuna et al., 2001).

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<td>Vertical direction</td>
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<tr>
<td>Horizontal to the bridge direction</td>
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<td>Longitudinal to the bridge direction</td>
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<td>20000kN/m</td>
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(S: fix H: move)

#### Figure 4. General Layout of Tatara Bridge (Yabuna et al., 2001).

### 3.2 Definition of Earthquake Ground Motion

1999 Chi Chi-Taiwan earthquake records are used in this study as earthquake ground motions that will be applied to Tatara Bridge. The selected earthquake motions are recorded on medium soil condition. To consider the multi-support excitation analysis, two different close recordings of Chi Chi earthquake ground accelerations are used. These recordings are obtained from the University of California, Berkeley, PEER (Pacific Earthquake Engineering Research) Strong Motion Database.

While the peak ground acceleration of Chi-1 record is 0.165 g in the vertical direction, it is 0.416 g in the north direction, and 0.303 g in the east direction. The corresponding peak ground displacements are 173.9 mm, 353.7 mm and 314.7 mm in the vertical, north and east directions, respectively. Peak ground acceleration data of Chi-2 record is 0.281 g in the vertical direction, 0.44 g in the north direction, and 0.353 g in the east direction. The corresponding peak ground displacements are 197.3 mm, 387.5 mm and 452.8 mm in the vertical, north and east directions, respectively. Displacement–time history plot of Chi-1 record in the east direction is given in Figure 5.

#### Figure 5. Chi-1 displacement-time history record in the east direction.
For multi-support excitation case, Chi-1 and Chi-2 earthquake records are applied to the bridge supports. While Chi-1 displacement-time history records are applied to the Ikuchijima Island side supports, Chi-2 displacement-time history records are applied to the Ohmishima Island side supports as three directional multi-support ground motions.

### 3.3 Traffic Loading

While examining the dynamic behaviour of Tatara Bridge under traffic loading, moving load model is considered by using the heavy truck HS20-44 defined in AASHTO. In this modeling, the vehicles enter into the bridge in predefined durations and travel along the selected traffic lanes until completing their predefined total durations.

Tatara Bridge has four lanes having two lanes in each direction. In the application of moving traffic load pattern, two lanes in each opposite direction are loaded mutually and when the loading duration is over, all four lanes of the bridge will be full of vehicles. Since various loading options in different intervals are possible, many critical situations are also analysed. The length of Tatara Bridge is 1480 m and the design speed is 80 km/h (22 m/s) (Yabuna et al., 2001). The duration of the analyses will be the time that begins when the vehicle enters into the bridge with a speed of 80 km/h and ends when it leaves the bridge and this duration is calculated as 68 s. This duration is calculated by dividing the length of the bridge (1480 m) to the design speed of 22 m/s. Traffic loading application is given in Figure 6.

![Figure 6. Traffic load application.](image)

The distance between the vehicles is an important factor while determining the magnitude of the load that will be applied to the bridge. This distance varies with reference to the intended purpose of the bridge and the density of the traffic in long-span bridges. For instance, the distance between vehicles is determined as 25 m in Fatih Sultan Mehmet Bridge for the purpose of providing moving load combination (Dost and Dedeoğlu, 2008). The length of a truck is also considered as 9 m for the most unfavourable situation. When the traffic and earthquake loads are considered together, then uniformly distributed static traffic load application by mid-sized vehicles is considered as traffic loading. While calculating uniformly distributed static traffic load, the distance between vehicles is considered as 25 m, and the length of a truck is considered as 8 m. Since the weight of H20-44 truck is 180 kN, the total distributed weight in each lane in which the trucks are lined up will be 6 kN/m. As a result of this application, the distance between two vehicles will be 33 m if they enter into the bridge with a speed of 22 m/s and with 1.5 s intervals. While defining uniformly distributed traffic loading, non-linear static loading is preferred. The size and load capacity of HS 20-44 and H 20-44 type trucks are given in Figure 7.

![Figure 7. Sizes and load specifications of HS 20-44 and H 20-44 trucks (AASHTO).](image)
3.4 Finite Element Model of the Bridge

3D finite element model of the Tatara Bridge is prepared with SAP 2000 Advanced 14.1.0 software (Figure 8). 381 nodal points, 214 frame elements, 168 cable elements, and 170 rigid-link elements are used in this model. While the deck of the bridge and towers are defined as frame elements, the cables are defined as special type of cable elements that exist in SAP 2000 Software. Material properties of the deck and towers are given in Table 2. The nonlinearity of the cables is included with an equivalent modulus of elasticity. The nonlinearity of the cables originates with an increase in the loading followed by a decrease in the cable sag. To overcome this nonlinear effect the following equivalent modulus of elasticity model proposed by Ernst is employed.

\[ E_i = \frac{E}{1 + (\gamma^2 L^2 E / 12\sigma^3)} \]

where, E is the modulus of elasticity of the straight cable, L is the horizontal length of the cable, \( \gamma \) is specific weight of the cable and \( \sigma \) is the tensile stress in the cable (Troitsky, 1988).

![Figure 8. Three dimensional finite element model of Tatara Bridge.](image)

| Table 2 Material properties of the deck and towers (Yabuna et al., 2001). |
|---------------------------|-----------------|-----------------|
| **Main Steel Members**    | **Main tower**  | **SS400, SM490Y, SM570** |
| **PC girder member standard** | **Main girder** | **SS400, SM490Y** |
| **design strength**       | **24 N/mm²**    |
| **Cable strand allowable stress** | **640 N/mm²** |

Link elements are used at the points where deck frame elements and tower frame elements intersect. These link elements are vertically and horizontally constant and have a stiffness of 20000 kN/m in the longitudinal direction (Endo et al., 1991). While defining multi-support excitation in SAP 2000 software, displacement–time recordings are used as time history functions. Unit displacements are applied to each support of the bridge in the longitudinal, horizontal and vertical directions. Then, these displacements are defined as loading conditions. Multi-support excitation analyses are started from the dead load induced deformed shape of the bridge. “Multi Step Bridge Live Load Pattern Generation” module of SAP 2000 Software is used while applying the moving traffic loads to the bridge model. The standards of the vehicles, the lanes on which these vehicles are moving, starting and leaving times of the vehicles on the bridge deck, directions of the vehicles and the speeds of these vehicles are defined in this module.
4 Analysis Results

4.1 Displacements

When the vertical displacements of the deck due to the multi-support excitation and traffic loading cases are compared, it can be observed that the displacement values determined for the combined loading case are larger than the displacement values obtained from the separate analyses of earthquake or traffic loadings (Figure 9). Although the displacement values determined from the traffic and multi support excitation cases are similar to each other, it is observed that the displacement values due to the traffic loading is especially larger at the center of bridge deck. If the center point vertical displacement values are compared, it can be noticed that the traffic loading case causes 5% larger displacements than those of the multi support excitation case. When the combined effect of traffic and earthquake loadings are considered, this ratio increases to 15%.

![Figure 9. Vertical displacements of the deck due to the traffic and earthquake loadings.](image)

When the longitudinal tower displacements due to the multi-support excitation and traffic loadings are compared, it can be observed that the displacement values determined for the combined loading case are larger than the displacement values obtained from the separate effects of earthquake or traffic loading cases (Figure 10). Opposite to the variation obtained for the deck, the displacements due to the multi-support excitation case are much larger than those of the traffic induced displacement values. Hence, the multi-support excitation case induces 108% larger displacement value than those of the traffic loading case at the top of the tower. When the traffic and earthquake loadings are considered together, this displacement value reaches to a ratio of 122%.
Figure 10. Longitudinal displacements of the tower due to the traffic and earthquake loadings.

4.2 Bending Moments

The bending moment diagram of the bridge deck obtained for the traffic and earthquake loadings can be observed in Figure 11. While the largest support moments are observed at the point where the deck is supported with towers, the largest span moments are observed at the center of the bridge span and at the edges where prestressed concrete fabrications are used. When the deck bending moments due to the multi-support excitation and traffic loads are compared, it can be observed that the earthquake induced moments are obviously larger than the moment values obtained from the traffic loading case. As a matter of fact, the moment values observed at the middle of deck span, at the side span and at the point where the deck is supported with towers for multi-support earthquake excitation case are 244%, 206%, and 53% larger than the moment values obtained from the traffic loading case, respectively. When the traffic and earthquake loadings are considered together, these ratios increase to 253%, 212%, and 68%, respectively.

Figure 11. Deck bending moments due to the traffic and earthquake loadings.
The tower bending moments obtained for the traffic and earthquake loading cases is shown in Figure 12. As can be observed from this figure, the largest moments are observed at the base of the tower. If the bending moments due to the multi-support excitation and traffic loading cases are compared, it can be observed that the earthquake induced moment values are much larger than the moment values determined from the traffic loading case over the height of the tower. The maximum moment values determined at the middle and at the base of the tower from the multi-support excitation case are 237% and 660% larger than the traffic induced loading case, respectively. When the traffic and earthquake loadings are considered together, these ratios increase to 262% and 690%, respectively.

**Figure 12.** Tower bending moments due to the traffic and earthquake loadings.

### 5 Conclusions

The purpose of this study is to define and compare the dynamic behaviour of a cable-stayed bridge under multi-support earthquake and traffic loadings. Since it is very important for the design of cable-stayed bridges, the behaviour of a cable-stayed bridge is examined for multi-support earthquake and traffic loadings separately and then examined for the combined effect of earthquake and traffic loadings. As a numerical example, Tatara Bridge located in Japan is chosen as the second longest cable-stayed bridge in the world. To define the dynamic behaviour of the considered cable-stayed bridge under multi-support earthquake loading, two different ground motions are applied to the support points and non-linear time history analyses are performed. Considering the distance between the support points of the bridge, close earthquake recordings are selected to apply to the bridge. As a result of the conducted analyses, the following conclusion can be drawn from this study:

It is observed that the nodal displacements and member forces obtained for the combined effect of the multi-support earthquake excitation case, corresponding to the Taiwan Chi Chi earthquake motions recorded on medium soil condition, and traffic loading case are larger than the responses determined for the multi-support earthquake and traffic loading cases, separately. Furthermore, it is also observed that the structural responses obtained for the multi-support earthquake excitation case are usually larger than the responses determined for the traffic loading case. These results imply that the combined effect of the earthquake and traffic loadings should be considered for the realistic design of cable-stayed bridges.
References