Evaluation of restrainer effects on bridge response subjected to dynamic loads

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Abstract. Highway bridges in seismic prone areas suffer a serious structural issue when subjected to a severe earthquake, which are the longitudinal hinge displacements. The hinge displacements might put the structure integrity in jeopardy when they exceed certain limits. High hinge displacements can cause fatal effects on the bridge structure including; bearing failure, unseating, and pounding of the adjacent superstructure decks. The aim of this study is to implement a numerical model which characterize the standard properties of a highway bridge and put it under a high intensity seismic excitation to examine its response. Then attempting to enhance the structure response using cable restrainers as means of displacement control. A parametric study is to be performed to investigate the effect of the restrainer material properties (Stiffness and Effective damping) on the efficiency of the cable restrainer in performing its intended purpose. The bridge was modelled and analysed using CSIBridge® finite element package. The obtained results have demonstrated a hazardous response of the bridge without the cable restrainer. However, a significant control capability was observed when applying the cable restrainer. Moreover, the parametric study has shown that, the performance of the cable restrainer is closely related to its stiffness as the stiffness value significantly contributes to displacement control and response stabilization. On the other hand, the effect of the effective damping value on the overall response was limited. Optimum cable restrainer have be chosen based on the parametric study and implemented to the model. The response was compared to the initial response without the restrainer. Eventually, the use of the cable restrainer had provided an impressive efficiency on displacement control and response stabilization.

1. Introduction
Highway systems are becoming substantial elements of infrastructure in any developed country. Such systems can accurately represent the evolution and prosperity of the country. Highway bridges are significant components of any highway system to ensure a smooth and safe flow of traffic. Thus, the number and capacity of these bridges continue to grow. Along with this growth, the responsibility to provide high levels of safety for bridge users increase.

This study focuses on earthquake loads and their effects on highway bridge structure. In structural terms, earthquakes can be defined as a dynamic load that can affect the structure in the form of time-dependent force. The horizontal time-dependent force acts through the center of mass of the structure. Thus, the earthquake effect on the structure is dependent on its internal properties such as weight and acceleration (FHWA, 1986).

The structural sensitivity of a bridge can be related to the lack of redundancy. In other words, any failure in any structural component can lead to inevitable progressive failure, which, in turn, develops
to partial or complete collapse (FHWA, 1986). Two requirements, namely, serviceability and seismic resistance, need to be considered in designing a bridge. However, these requirements are conflicting because each one of them imposes demands that are opposite to the other. Serviceability demands usually appear in the longitudinal direction of the superstructure. These demands cater for expansion and contraction, creep, shrinkage, and pre-stressing, which require providing certain deck flexibility and expansion joints. The expansion joint is used to uncouple the response of the bridge decks from each other and from the abutments, as well as to accommodate the in-service and a fraction of the seismic displacements (Stergios A. Mitoulis and Ioannis A. Tegos, 2010). However, the flexibility of the superstructure and the presence of the expansion joint can represent a weakness point in terms of seismic resistance of the structure. Residual displacement usually occurs at the expansion joint even after the seismic loading is over (Huili Wang et al., 2011). The longitudinal displacement of the deck could close or open the existing expansion joint gap, and both cases are undesirable. When the longitudinal displacements are larger than the bearing dimensions, the span is unseated or collapses as a result of bearing failure. However, if the superstructure decks displace toward one another, and if the displacement is larger than the provided expansion joint gap, then the decks collide and pound against each other, which compromises the integrity of the structure. Pounding has a hazardous effect in structural safety terms because it contributes to the magnification of the longitudinal displacements. Pounding contributes by transferring the seismically induced forces to the adjacent decks in the form of impact load, which in turn leads to unseating of the bridge deck (M. AlaSaadeghvaziri, A.R. Yazdani-Motlagh, 2008; ShetataAbdureheem, 2009;Jeong-Hun Won, 2008). Cable restrainers are used to connect the decks of the superstructure across the expansion joints to unify their response and to control longitudinal displacement. Many studies have been conducted to investigate the efficiency of cable restrainers and the parameters that govern their performance. Several parameters such as cross-sectional area, length, and sag length of restrainers are identified to control the performance of the cable restrainer (Saïdi M. et al., 1993). The present study aims to investigate the effect of stiffness and damping of the restrainer on the structural response under seismic excitation. This study analyzes the bridge model under El Centro earthquake to evaluate the effect of an earthquake on the seismic response of the superstructure. The model was adopted from a standard AASHTO LRFD design sheet and was implemented in the analytical software.

Previous experience and observations have shown that bridge structures are vulnerable to earthquake-induced damage or collapse. Numerous design approaches have been proposed and used in different codes and studies to reduce the dynamic and seismic effects on bridge structures. Correct assumptions for these codes and studies should be implemented to ensure design efficiency. For example, the controversial assumptions include the issues of whether to assume the bridge deck to be rigid in its plane or not, and what type of connection should be utilized between the superstructure and substructure. Earlier studies focused on observing the consequences of previous seismic events and on gathering the available data regarding earthquake characteristics such as frequency, amplitude, and the form or direction of the seismic forces affecting the structures, site condition, as well as the recorded subsequent structural failures. The obtained data were implemented in the simulation of various types of analytical and numerical models with different methods and approaches to simulate and observe the seismic behavior of the structures under consideration. Analysis results are then interopera ted, and theories are proposed to explain the reasons for the observed failures and the contributing factors that amplified the seismic effect on the structure. The ultimate objective of all these efforts and considerable research is to present an optimum design in terms of safety, durability, and serviceability of the bridge structure.

Earlier earthquake experience has exposed the vulnerability of bridge structures that are designed without adequately considering seismic loads or previous seismic codes and recommendations. For example, bridges in the central United States are suffering from seismic resistance deficiency because most of them were built during the rapid highway expansion movement in the early 1970s when insufficient attention was given to seismic resistance and before the implementation of any seismic design regulation (M. Dicleli, 2003). Previous experience and observations have shown that bridge
structures are vulnerable to earthquake-induced damage or collapse. This effect can be related to the heavy mass, low inherent damping, and inadequate ductility of the bridge structure (S. Maleki, 2002).

The crucial seismic effect on the bridge structure can be interpreted by mentioning that the natural frequency of bridges, especially those with relatively short piers, lies within the range of the earthquake vibration frequencies. Consequently, the bridge structural components vibrate out of phase and such vibration could lead to severe damage or catastrophic collapse (Shehata E. Abdel Raheem, 2009). Earlier practice and studies have identified the following weak points of bridges: brittle behavior of the rigid superstructure, insufficient resistance and stability of bearing, insufficient lateral resistance of the substructure, inadequacy of transverse reinforcement, and lap splices in pier columns and foundations (M. Dicleli, 2003). As observed from previous earthquake events, the main forms of bridge failure caused by applied seismic loads are pounding between adjacent decks, bearing failure that leads to deck unseating, flexural or shear failure of piers and columns, and rotation of the superstructure (G. Perrone, 2011; Gakuho Watanabe, 2004). The most repeated forms of failure are flexural or shear failure as well as pounding and unseating. Shear and flexure failure is closely related to the rigidity of the superstructure and how it is connected to the piers (S. Maleki, 2002).

Jun Yi Meng (2000) conducted a study on the Foothill Boulevard undercrossing that sustained severe damage during the San Fernando earthquake in 1971. The author stated that previous studies performed to explain the reason for the Foothill bridge failure suggested that such failure could be attributed to the flexural and torsional bridge deck motion, which is due to the superstructure rigidity. Other studies have shown that, if the deck is not rigidly connected to the column, then the seismic response is determined by the in-plane rigid body motions. These studies explained that the observed failure is a result of the rotation-induced lateral forces. The other notorious form of failure is superstructure unseating as a result of large displacements and pounding phenomena. During seismic excitations and because of the varying natural period of the deck segments as well as the spatially varying ground excitations, large displacements occur at the joints of the spans. If the provided seat length at the bent or abutment is insufficient to accommodate these displacements, then the span collapses as a result of the deck unseating (Jeong-Hun Won, 2008). The most delicate components of any bridge structure are the expansion joints. High displacement might occur in these joints because of the amplified seismic forces that might ultimately rupture the continuity of the structure and might jeopardize its overall integrity (Felix D. Ruiz Julian, 2007). When subjected to a design ground motion, a bridge could produce high relative displacements that might reach as many times as the width of the gap available between the adjacent decks. When the displacement exceeds the available clearance, the adjacent decks pound against each other. Moreover, this displacement can have a major function in severe bearing damage, which in turn leads to unseating of the deck and subsequent collapse. The displacement-initiated pounding considerably amplifies the relative displacement. Thus, longer seat width is required to prevent unseating. With the relative response of the different bridge segments, pounding has been identified as the main cause of the collapse. The reason is that pounding causes local damage in the bridge segments and induces transfer of longitudinal forces that changes the entire seismic response of the bridge structure (Shehata E. Abdel Raheem, 2009).

2. Bridge retrofitting techniques

Seismic retrofitting can be defined as the process of modifying existing bridges or bridge components, or adding additional elements to enhance the bridge seismic resistance and to reduce their overall seismic vulnerability to earthquakes. The five most commonly used retrofitting measures are the following:

1. **Seismic isolation:** This technique is based on separating the dynamic response of the substructure and the superstructure by dissipating the seismic energy at the base before it is transmitted in the structure.

2. **Longitudinal or transverse restrainers:** This technique is used to control the longitudinal displacements. As a result, the pounding and unseating of the superstructure are also controlled.
3. **Seat length extension:** Another approach to control the unseating of the superstructure is by providing enough seat length to cater for the longitudinal displacements induced by the seismic action. Seat extenders can be introduced in the form of corbels at the bent cap or at the abutment when the seat length offered by the bridge geometry is insufficient to accommodate the longitudinal displacements.

4. **Column strengthening:** Columns are considered one of the most seismically vulnerable components in a bridge structure. The reason is the low ductility capabilities of the column as a result of the lack of transverse reinforcement and lap splices. This ductility can be enhanced by using steel jacketing, concrete overlays, cable wrapping, and jacket with fiber composite wrap. These techniques can improve the confinement of the column. Thus, seismic performance is enhanced.

5. **Bent cap strengthening:** The main weakness in the bent cap is the lack of shear and flexure resistance capacity, which can be attributed to the insufficient reinforcement. This deficiency results in bent cap failure before the plastic hinge, and consequently, force transfer starts between the bent cap and the column. The bent cap performance can be improved by using post-tensioned rods, external shear reinforcement, and by adding concrete or steel bolster.

These retrofitting measures are used to achieve one or more of the following objectives: reduction of force demand on the vulnerable components of the bridge, reduction of deck displacements, and strengthening the most vital components of the bridge structure (Timothy Wright, 2011).

Bridge retrofitting is a vital measure that dramatically increases the seismic capacity of the bridge. However, this technique has its drawbacks. The proposed conventional seismic retrofitting approaches have been proven imperfect because of their high cost, difficulty of implementation, difficulty of mobilization and traffic control, as well as structural problems. For example, if a bridge component is strengthened, then other components become overstressed and more strengthening is required. Accordingly, the cost dramatically increases over the allocated retrofitting budget (M. Dicleli, 2003).

### 3. Cable and bar restrainers

This retrofitting technique is one of the most commonly used techniques in the world, especially in the United States and Japan. Previous earthquake events in these countries showed the fragility of bridge structures to pounding and unseating of the superstructure. The longitudinal restraining technique utilizes cable or bar restrainers attached to the side of the deck between the adjacent decks or between the deck and abutment to limit or completely eliminate the longitudinal displacements as well as to ultimately avoid the pounding and unseating of the bridge decks.

The most preferable type of restrainer is the cable restrainer. Such a restrainer has lower stiffness and ductility compared with the bar restrainer. Thus, bar restrainers need to be implemented at high lengths to balance their low flexibility. Cable restrainers usually consist of galvanized steel with diameter of 20 mm and length ranging between 3 m and 5 m.

#### 3.1 Advantages of cable restrainers

1. Restrainters are effective to restrain the maximum displacement between the abutment and the adjacent decks.
2. Restrainters can be installed easily.
3. They are economically efficient because this technique is inexpensive.
4. This retrofitting technique requires less time and effort to be implemented.

Several studies have been conducted to examine the seismic response for different configurations of highway bridges. Most of these studies proposed methods and devices to increase the seismic protection of the bridges under study.

Gotam Ghosh et al. (2011) aimed to evaluate the seismic response of a continuous bridge equipped with shape memory alloy (SMA) bearing. Nonlinear analysis was conducted by subjecting the bridge model to five ground motions. The analysis has shown that the proposed method of seismic protection,
which uses conventional elastomeric bearing in combination with stoppers or restrainers, is a practical method with high efficiency. A nonlinear finite element model was implemented by Felix D. Ruiz Julian et al. (2007) to simulate the nonlinearity of expansion joint as well as the nonlinearities of structural elements such as piers, bearing, and cable restrainers. Their study indicates that using cable restrainers can efficiently reduce the displacements at the expansion joints and can ultimately reduce the deck unseating hazard. Furthermore, using extremely flexible LRB bearings has been observed to eliminate the effect of the cable restrainers. An LRB bearing with moderate stiffness is recommended to maintain the effect of the restrainer with higher efficiency.

D. Cardone et al. (2011) proposed the use of multi-performance protection systems based on SAM wires to retrofit multi-span simply supported bridges as well as continuous bridges. Two bridge models were created to represent existing Italian bridges to evaluate the practicality of the proposed devices. These models were subjected to a nonlinear time history analysis. The response of the bridges was checked with and without the use of the proposed retrofitting technique. Analyses for the multi-span simply supported bridge showed that the SMA-based restrainers contributed to the reduction of deck displacement. The maximum deck displacements are compatible with available joint clearance. Minimum force transfer to the piers has also been observed. Furthermore, such a design has contributed to the protection of the abutment and the bearing devices from damage. Analyses have also shown that, in the case of the multi-span continuous bridge, the SMA devices significantly redistributed the seismic force between all the piers of the bridge contrary to the use of fixed bearing. The latter design might cause failure as a result of intense force transfer between the deck and the piers. After demonstrating through their earlier studies that the linear viscous dampers are effective in reducing the displacements at the expansion joints of the bridge, J. M. Kim et al. (2000) proposed the direct use of energy dissipating restrainers after the 1994 Northridge earthquake. Their study proves the efficiency of restrainers in limiting the horizontal displacement at the joint and in reducing the pounding force. However, a slight increase in the ductility demand of a few bridge columns can be observed. Another study was conducted by Shehata E. Abdel Raheem (2009), in which the author initiated a parametric study to investigate the effect of pounding on the overall seismic behavior of the structure, the efficiency of seismic isolation, and the effectiveness of the suggested measures to reduce the pounding effect. Analyses have shown that the efficiency of the base isolation is significantly affected by the pounding prevention measures. The hazard of pounding can be interoperated as pulses of accelerations transmitted to the superstructure, which in turn amplifies the structural response of the bridge. This study has shed light on the effect of the provided gap at the expansion joint on the pounding on the longitudinal direction. The result indicates that a small gap is efficient in limiting the pounding because such a gap has insufficient clearance for the decks to accelerate and to pound against each other. A large gap is enough to accommodate the longitudinal displacements and to completely eliminate the potential pounding of decks. However, if the gap is within the range of the expected displacement, then the gap becomes insufficient to prevent pounding and offers enough clearance for deck acceleration to develop. Unseating protection devices can alter the seismic response of the entire structure by substantially limiting the longitudinal displacements and eventually controlling pounding and unseating. Furthermore, even in the case of unseating, the restrainers can hold the deck and prevent it from falling.

Study by Jeong-Hun Won et al. (2008) showed the effect of using the restrainer as an unseating prevention device. In their study, a simplified mechanical model was prepared to evaluate the effectiveness of the restrainer. The proposed model utilizes the lumped mass system for efficiency and simplicity. Pounding between the adjacent vibrating units was considered by using impact element consisting of spring, damper, and a gap. The seismic response evaluation of the model showed that the restrainers were effective to restrain the maximum displacement between the abutment and the adjacent deck.

The seismic response is highly affected by the rigidity of the superstructure because the assumption of a rigid deck contributed to the simplicity of analysis. However, in case of skewed bridges, the assumption of rigid deck fails to assess the forces transferred to the columns.
If the deck is modeled as rigid element, then high in-plane stresses develop, and a large amount of force transfer occurs between the superstructure and the substructure. Base isolation is suggested to eliminate or to control the transferred forces and stresses by dissipating a part of their energy. Base isolation can be ensured by using elastomeric bearings and slider bearings. However, in addition to its installation difficulties, base isolation can increase the longitudinal displacements beyond the available gap provided in the expansion joint. Furthermore, if designed to cater for the design earthquake, then elastomeric bearings will have a large size and will require large pier dimensions, unless a supporting system of stoppers or restrainers is introduced. A combination of base isolation and a restrainer/stopper system was proven to be successful in improving the seismic performance of the bridge structure.

Restrainers or stoppers can be provided in different types and configurations such as cable restrainers, bar restrainers, and energy dissipating restrainers. Restrainers can be installed to connect directly between two adjacent decks with or without a shear key and can also be installed in a deck-to-abutment or deck-to-peer configuration.

4. Definition of retrofitting techniques
In this study, the proposed measure to overcome the longitudinal displacements is cable and bar restraining systems. The model is analyzed in three phases. In the first phase, the model is analyzed without using any retrofitting technique. In the second phase, a cable restrainer is modeled as a link element on the sides of the superstructure across the expansion joint to connect each of the two adjacent decks. A parametric study is also performed to investigate the effect of stiffness and effective damping of the restrainer on the overall efficiency of the restrainer. In the third phase, optimum values of the stiffness and effective damping of restrainer are selected and implemented on the model to evaluate the response of the optimized structure.

In the second phase, one restrainer is provided at each side of the expansion joints connecting deck to deck or deck to abutment. In other words, eight restrainers are installed on the entire bridge structure. The restrainer is modeled as a tension-only spring.

Thermal requirement is considered by providing a slack length of 50mm. When the displacements exceed the provided slack length, the restrainer is engaged to resist or to control the displacements through its stiffness.

5. Restrainer stiffness
According to AASHTO LRFD (1996) (Bridge Design Aid, 14-1), the governing assumptions and the equation of the stiffness of the cable restrainer are as follows:
1. The effect of friction is negligible.
2. The skew is less than 30 degrees.
3. The hinge is modeled to be linear.

The stiffness value required to control unrestrained relative hinge displacement of $D_{eq0}$ can be calculated by

$$D_{eq0} = \sqrt{(D_{1a}^2 + D_{2a}^2 - 2\rho_{12}D_{1a}D_{2a})}.$$  \hspace{1cm} (Equation 1)

where

$D_{eq0}$ = unrestrained relative hinge displacement;
$D_1$ = displacement demand of the less flexible frame;
$D_2$ = displacement demand of the high flexible frame.

$K_r$ is the restrainer stiffness and can be obtained using the following equation:

$$K_r = R F K_{mod},$$  \hspace{1cm} (Equation 2)

where

$K_r$ = restrainer stiffness.

$R$ can be obtained from the following equation:
\[ R = r (1 - 1.66L + 0.67/L), \]  
(Equation 3)

where

- \( r \) = adjustment factor for \( R \);
- \( L \) = relative hinge displacement limit.

\( F \) is determined from

\[ F = f \cdot F_{\text{eff}}, \]  
(Equation 4)

where

- \( f \) = effective stiffness adjustment factor;
- \( F_{\text{eff}} \) = effective stiffness factor.

\[ K_{\text{mod}} = \frac{K_1 K_2}{(K_1 + K_2)}, \]  
(Equation 5)

where

- \( K_1 \) = stiffness of the less flexible frame;
- \( K_2 \) = stiffness of the more flexible frame.

According to AASHTO LRFD (1996) specifications, three types of analysis for the seismic load on bridges are recommended. Single mode and uniform mode are recommended to be used for regular bridges. Meanwhile, multi-mode method is proposed as response spectrum analysis for irregular bridges.

AASHTO also specified the maximum allowable displacement that needs to be controlled by the restrainer system in the expansion joint by the formula

\[ D_R = D_Y + D_S, \]  
(Equation 6)

where

- \( D_R \) = available hinge seat and minimum bearing length and initial gap;
- \( D_Y \) = restrainer elongation at yield;
- \( D_S \) = restrainer slack.

AASHTO also recommended the initial hinge displacement that can be obtained by

\[ D_{eq} = (D_{1o}^2 + D_{2o}^2 - 2D_{1o}D_{2o}D_{eq})^{1/2}, \]  
(Equation 7)

where

- \( D_{eq} \) = initial hinge displacement;
- \( D_{1o}, D_{2o} \) = individual frame displacements.

According to the AASHTO code, restrainers are recommended if \( D_R \geq D_{eq} \). Moreover, restrainers are unnecessary to limit hinge displacement, but a minimum number of restrainers should be provided.

6. Numerical model characteristics

The bridge was selected from a standard (AASHTO LRFD, 1996) design sheet. In this section, a detailed illustration of the model characteristics is provided. The geometry, structural properties, materials properties, support condition, and connection between the superstructure and substructure are discussed in detail.

6.1 Geometry

The numerical model represents an existing simply supported three-span bridge. Both end spans are 30m, whereas the middle span is 36m long with a total width of superstructure of 14.2m and curb-to-curb width of 13.4m. As shown in Figure (1), the superstructure of the bridge is a pre-stressed concrete four-bay box girder. The superstructure is supported on three columns at each joint.
Figure 1. Bridge model.

The superstructure was modeled as a box girder. The girder has four bays with web width of 105mm and a flange thickness of 205mm. The total height of the box girder from top to bottom is 1800mm. A detailed dimension description is shown in Figure (1). Five pre-stressed 7-wire grade 270 strands with wire diameter of 12.70mm and $f_y = 1860$ MPa are implemented in each span. The low relaxation pre-stressed strands with modulus of elasticity $= 197$ GPa are installed in the central and external webs, as shown in Figure (4.3), following ASHTTO LRFD (1994), Bridge Specifications, Section 5, Appendix A5.3.

Two expansion joints were created in the model as gap elements; one was located at the end of the first span, and the other was at the end of the middle span. The expansion joint was 10 cm wide. Two bearing configurations were introduced in the bridge model; one type was a standard rubber bearing with low stiffness, and the other was a roller bearing. These bearing configurations were selected to provide a certain amount of energy dissipation capability to compensate for the relatively rigid shell superstructure and to minimize the stress transferred between the superstructure and the substructure, and vice versa. The two abutments in the bridge model were modeled as a system of springs, in which the bearing spring was connected to the foundation springs to simulate the function of the abutment. Two bent caps were introduced in the model; one bent cap was located at the end of the first span, and the other bent cap was located at the end of the middle span. The bent cap was modeled as a frame element using the section designer feature in CSIBridge software. Each of the two bent caps was resting on three concrete columns. The columns provided at each span end had a hexagonal cross section. The total height of the column is 6.1m with a cross-sectional diameter of 1.25m, which provides a seat length of 0.575m, as shown in Figure (2).
The foundation of the structure was modeled as a spring system connected to the bottom of the columns and abutments. To simulate the soil structure interaction, the foundation springs are partially fixed, and stiffness values are assigned in each direction. The stiffness of the springs and the corresponding directions are illustrated in Table (1).

**Table 1. Foundation spring stiffness (KN/mm) (AASHTO).**

| DIRECTION | U    | R        |
|-----------|------|----------|
| X         | 3294 | 1.16e+11 |
| Y         | 2946 | 4.71e+10 |
| Z         | 3152 | 13.30e10 |

6.2 Material properties

The bridge superstructure was modeled with normal strength concrete with $f'_c$ = 35 MPa and tensile strength of 3 MPa with modulus of elasticity $E$=28000 MPa. Grade 50 concrete was used in the columns because of the high sensitivity of this structural member. The column concrete has $f'_c$ = 50 MPa, a tensile strength of 5 MPa, and a modulus of elasticity $E$ = 36000 MPa. The concrete used for both grades is a conventional type of concrete with an average density of 2400 Kg/m$^3$, and Poisson’s ratio is set equal to 0.3 with reinforcement of $f_y$ = 414 MPa.
6.3 Support condition
The superstructure was supported by two abutments and two bents. The connection of the superstructure to the substructure was achieved by using rubber bearings and roller bearings depending on the location. A rubber bearing was used in the first abutment and a roller bearing was used in the final abutment. The support condition was identical on both of the bent caps. A roller bearing was installed on the left side of the bent cap, whereas a rubber bearing was used on the right side. In other words, each span was supported on a roller bearing from the left and a rubber bearing that represented partial fixity on the right.

6.4 Loading
The bridge model was loaded with the following two categories of loading:

6.4.1 Static loads
1- **Dead load:** Only the self-weight of the structure was applied without the pavement load and other loads because they are unnecessary for the purpose of this study.
2- **Pre-stressing load:** This load includes the loads generated from the effect of the pre-stressing of the box girder.

6.4.2 Dynamic loads
1- **Moving load:** This load is generated by the passage of different types and sizes of vehicles on the bridge superstructure at varying speeds. To extend the effect of the simulated loading, two moving load cases were defined; the first case is HSn-44 truck load and the second is HSn-44L lane load.
2- **Time history load:** This load simulates an intense earthquake, which is the El Centro earthquake in the US (1940). This earthquake was a 3D ground excitation that had three components; the first component was directed from north to south, the second was from east to west, and the third was in the vertical direction. In this study, the longitudinal displacements of the superstructure are considered under each excitation component and under the combined effect of the three components.

7. Results
In this section, the results from the analysis of the bridge model under three separate components of the seismic wave are discussed. These components are the north–south, east–west, and the vertical component of the ground excitation, as well as under the combined effect of the three components (3D excitation).

To fulfill the objective of this study, the result required for consideration is the longitudinal displacement (along the layout line); six nodes are chosen as the reference to exhibit the longitudinal displacements. These points are selected in a pattern in which each span is represented by a corner node at each end, as shown in Table (2) and Figure (2).

![Figure 4. Reference node location.](image-url)
Table 2. Node notation.

| SPAN   | SPAN 1 | SPAN 2 | SPAN 3 |
|--------|--------|--------|--------|
| NOTATION | A    | B    | C    | D    | E    | F    |
| NODE ID. | 131  | 6104 | 6100 | 2426 | 3632 | 3460 |

7.1 Bridge response without restrainer

This study was conducted in two phases; the first phase previewed the results of analyzing the bridge model subjected to the time history seismic load in addition to the other static load cases. This analysis was performed without installing the cable restrainer. The result was used as a benchmark to compare the bridge response with and without the restrainer and to check the efficiency of using the cable restrainer.

After running the first phase of analysis, the result was displayed, the maximum displacements in the X-direction (along the layout line) for the six reference nodes were obtained, and the following benchmark table was established.

Table 3: Nodal displacement of the structure without restrainer.

| STRUCTURE WITHOUT RESTRAINER |
|-------------------------------|
| DIRECTION | PEAK DISPLACEMENT (mm) |
| SPAN 1 | SPAN 2 | SPAN 3 | A | B | C | D | E | F |
| Acc. In X Direction | 73.98 | 74.08 | 61.47 | 59.94 | 46.3 | 46.38 |
| 3D Acceleration | 97.02 | 96.96 | 84.83 | 84.61 | 73.43 | 73.31 |

The displacements presented in Table 3 are the peak displacements. As shown in this table, the largest amounts of displacements occur when the bridge under study is subjected to ground shaking in the longitudinal direction and in the 3D ground shake. Evidently, the maximum displacements occur in the first span, which reaches up to 7.4cm for the longitudinal excitation and 9.7cm for the 3D excitation. Relatively lower displacements are also found in the two other spans. This finding can be related to the different geometry of the spans and the difference in the support condition at the different bents and abutments. The aforementioned value of displacement is extremely critical to the integrity of the bridge because such value magnifies the hazard of bearing failure. Bearing failure can occur when the superstructure displacement exceeds the provided bearing parameter dimensions. Moreover, this failure can occur when the superstructure displaces in a relatively large displacement, away from the centroid of the bearing. This phenomenon jeopardizes the stability of the bearing, which in turn jeopardizes the superstructure.
The difference in displacement values between the adjacent spans (Table 3) can raise another concern, that is, pounding between the decks. Pounding occurs when two adjacent decks displace in opposite directions and close the available gap in the expansion joint. If the displacement values exceed the available gap width, then the decks would pound against each other. The criticality arises from the fact that pounding would cause undesirable force transfer between the decks and would magnify the longitudinal displacements. Such a phenomenon ultimately leads to bearing failure and unseating.

7.2 Stiffness comparison of bridge response with cable restrainers

In most previous parametric studies, the same parameters were investigated such as the length, area, and slag length of the restrainer. In the current study, a parametric investigation is performed to check the effect of stiffness and the effective damping properties in the material of the cable restrainer. Six stiffness values of the restrainer were considered to extend the range of the examined stiffness from low to medium to high stiffness. These stiffness values are 50, 75, 100, 150, 200 and 300 KN/mm (ShehataAbdulreheem, 2009).

Moreover, the effective damping is set to zero in this section. The analysis was run after assigning each of aforementioned stiffness values to the cable restrainer. The peak displacement at each of the six nodes was denoted and compared with the corresponding benchmark displacement value. Table (4) shows the displacement values of the superstructure with the various stiffness values of restrainers.

| DIRECTION          | PEAK DISPLACEMENT (mm) |
|--------------------|------------------------|
|                    | span 1 | span 2 | span 3 |
|                    | A   | B   | C   | D   | E   | F   |
| Acc. In X Direction| K=50 | 67.26 | 67.45 | 58.96 | 60.15 | 53.10 | 53.30 |
|                    | K=75 | 65.15 | 65.35 | 57.23 | 59.81 | 54.17 | 54.40 |
|                    | K=100| 63.66 | 63.88 | 56.00 | 59.73 | 54.64 | 54.88 |
|                    | K=150| 61.82 | 62.05 | 55.48 | 58.63 | 54.88 | 55.14 |
|                    | K=200| 60.61 | 60.84 | 55.32 | 58.13 | 54.85 | 55.10 |
|                    | K=300| 59.95 | 60.12 | 55.29 | 57.61 | 54.79 | 55.09 |
| 3D Acceleration    | K=50 | 89.04 | 89.10 | 84.20 | 83.92 | 78.15 | 78.24 |
|                    | K=75 | 86.87 | 86.95 | 84.95 | 83.03 | 78.42 | 78.52 |
|                    | K=100| 88.48 | 88.37 | 80.17 | 84.17 | 79.10 | 78.91 |
|                    | K=150| 89.99 | 89.86 | 81.40 | 86.66 | 82.68 | 82.51 |
|                    | K=200| 90.43 | 90.32 | 81.78 | 87.96 | 84.79 | 84.66 |
|                    | K=300| 90.85 | 90.75 | 82.14 | 89.18 | 86.62 | 86.49 |

The effect of the cable restrainer is clearly presented in Table 4. The effect is only limited to the longitudinal and 3D excitation. This finding is compatible with the fact that the cable restrainer is installed only in the longitudinal direction.

The result also shows that the cable restrainer has impressive efficiency in limiting the longitudinal displacements at the supports that have relatively high displacements. On the contrary, at the supports with low displacements, the cable restrainer shows an opposite effect because the displacement increased rather than decreased. This finding is not an anomaly because it is compatible with the theoretical objective of using a cable restrainer.

Theoretically, a restrainer connects the separated superstructure segments and unifies its response as well as distributes the seismically induced displacements almost equally between the different
spans. Such effects can be confirmed by the displacement values obtained in Table (4). The high displacements that are concentrated on the left side of the bridge were reduced and were transferred to the other segments with lower displacements. In other words, the displacements of each of the six nodes were brought one step closer to the average displacement of all of the six nodes.

The preceding interpretation applies to the results obtained using other stiffness values, but the reduction percentage is expected to vary depending on the stiffness of the cable restrainer. For example, using a stiffness of 50 KN/mm resulted in a displacement reduction of -9.08% to -8.95% at the first span under seismic wave in X-direction, whereas less reduction occurs at the same span under the 3D excitation. The middle span values reduce at the left support and increase at the right support with -4.08% to +3.49%, respectively. The inflection point of the displacement is expected to occur close to the center of the middle span. A different case is observed under the 3D load as reduction of displacement occurs at the left and right supports of the middle span with -7.81% and -5.81%, respectively. At the third span, the effect is nearly similar in general because the displacements increase. The displacements can reach up to +12.80% and +12.98% in the case of ground excitation in the X-direction and can increase to +6.03% and +6.30% in the case of a 3D excitation. The efficiency of the restrainer in equalizing the response of the entire structure is closely related to the stiffness of the cable restrainer material.

For example, if node (A) is used as a reference to the first span, the initial longitudinal displacement is 73.98mm when the bridge is under acceleration in the X-direction. When stiffness values of 50, 75, 100, 150, 200, and 300 KN/mm are implemented, the restrainer provides a reduction percentage of -9.08%, -11.93%, -13.95%, -16.43%, -18.07%, and -18.96%, respectively, in the initial longitudinal displacement.

However, the longitudinal displacements induced by the 3D excitation are higher in magnitude because of the combined effect of the seismic wave components. A longitudinal displacement of 97.02 mm occurs at node A under 3D excitation without a restrainer. With the use of the aforementioned stiffness values, the corresponding reduction values are -8.22%, -10.46%, -8.80%, -7.24%, -6.79%, and -6.33%.

The cable restrainer is more effective in reducing the longitudinal displacements generated by the seismic wave in the X-direction, compared with reducing the 3D seismic waves. This result can be related to the fact that the full tensile capability of the cable restrainer in the first case is located exactly within the line of effect of the seismic wave; thus, full tensile resistance is provided. On the contrary, only a certain component of the tensile resistance of the cable restrainer in the second case is located within the integrated line of effect for the combined seismic wave components.

In the case of 3D excitation, the displacement reduces when the restrainer is implemented. However, as the restrainer stiffness increases, the longitudinal displacements also start to increase slightly. This finding can be justified by the fact that a high stiffness restrainer has lower energy dissipation capability. The restrainer then starts to act as a connection to transfer the undesirable loads between the adjacent decks. This effect is clearer in the result of the bridge under the 3D wave because the effect of this combined seismic load is more intense than that of the other individual directions.

The same effect can be observed in the other node displacements. However, the displacement alteration becomes smaller in the middle span because the longitudinal displacements in the span are not far in magnitude from the desired average displacement. For example, the initial longitudinal displacement at node C is 61.47 mm and is then reduced as a result of the effect of the restrainer by -4.08%, -6.89%, -8.90%, -9.74%, -9.74%, and -9.07%.

After the inflection point closes to the center of the second span, the displacement trend becomes similar to the first span. However, the displacement has increased rather than decreased as a result of the effect of the restrainer that ensures distributing the displacements as close as possible along the three spans that form the superstructure.
The initial and average displacements with and without a restrainer are presented in Figure 5. In this figure, the blue line represents the initial displacement, the red line represents the average displacement, and the purple line represents the displacement with restrainer stiffness of 300 KN/mm. The variations of displacements at nodes A, C, and E associated with different stiffness values of restrainer are shown in Figure 6.

Figure 6 provides a clear image of the trend of displacement at each of the three spans through its representative point. The longitudinal displacements are decreased before the inflection point at the middle of the second span with the increase of the stiffness. However, an increase of displacement is observed after the inflection point. Accordingly, the displacement is brought closer to the average value of displacement.

At low to medium stiffness values of 50KN/mm to 150 KN/mm, the graph drops dramatically and the displacement reduction is relatively high. However, after the stiffness increases (above medium stiffness), the trend of the graph stabilizes with substantially less reduction percentage. As mentioned, this result can be related to the limited energy dissipation capability and high potential of force transfer associated with the stiffness increase.

To present the time history response of the bridge structure, a time history versus displacement graph is needed. For simplicity and clarity, three stiffness values were considered to represent an acceptable range of stiffness. A value of 75 KN/mm is chosen to represent low stiffness, whereas 150 KN/mm and 300 KN/mm are chosen to represent medium and high stiffness, respectively. The graphs were generated for each of the six nodes under the X-direction and 3D excitation.
The same response information presented earlier can be verified by the time history-displacement graphs for both cases of X-direction excitation and 3D excitation. The relationship between the displacement and the stiffness can be observed through the peak value. This relationship can also be found throughout the entire case of the time history loading, starting from 0 s until 53.5 s, in which the ground excitation was concluded at an interval of 0.02 s. This result confirms that the effect observed by the peak displacements is valid throughout the time history loading.

7.3 Expansion joint gap

One of the major concerns of this study is to evaluate the effect of the earthquake on the displacement of the superstructure, particularly at the expansion joint. This effect can be related to the sensitivity of this structural member to the developed displacements. These displacements can place this structural member in a critical position in which it is prone to several hazards such as unseating, bearing failure, and pounding between the adjacent decks.

Tables 5 and 6 show the displacements of the two corner nodes at the left and right sides of the expansion joint. The difference between them represents the residual gap at the expansion joint after the displacement occurs. The residual gaps are plotted in Figures 7 and 8 to obtain a graphic display for the expansion joint behavior and to compare the effect under different stiffness values of the restrainer at each of the two expansion joints in the superstructure. Tables 5 and 6 and Figures 8 and 9 indicate that the cable restrainer has two apparent effects on the expansion joint behavior; the first effect can be noticed by stabilizing the out-of-phase displacements at the expansion joints and bringing the gap to a predictable and less deferential mode of displacement. This effect can be observed particularly in the graphs where a partially random displacement control occurs in the case of low stiffness values. Within the range of low stiffness, the gap displacement varies significantly from gap to gap and from different stiffness value to another. However, as the stiffness value increases, the residual gap width values become stable and take a more predictable path.

**Table 5.** Gap 1 displacements with different stiffness values.

| Peak Displacement in GAP1 (mm) | in X-direction | in 3-dimensions |
|-------------------------------|----------------|-----------------|
|                               | Average Disp. | Righ Disp.      |
|                               | Left Disp.    | Average Disp.   |
|                               | Righ Disp.    |                 |
| K                             | Left Disp.    | Average Disp.   |
| 0                             | 74.08         | 67.76           |
| 50                            | 67.45         | 63.21           |
| 75                            | 65.35         | 61.29           |
| 100                           | 63.88         | 59.94           |
| 150                           | 62.05         | 58.77           |
| 200                           | 60.84         | 57.66           |
| 300                           | 60.12         | 58.01           |
Figure 7. Gap 1 displacement with acceleration in X-direction with different stiffness values.

Figure 8. Gap 1 displacement with acceleration in three dimensions with different stiffness values.

Table 6. Gap 2 displacements with different stiffness values.

| K     | Peak Displacement in GAP2 (mm) | in 3-dimensions |
|-------|--------------------------------|-----------------|
|       | Left Disp. | Average Disp. | Right Disp. | Left Disp. | Average Disp. | Right Disp. |
| 0     | 59.94      | 53.12         | 46.3        | 84.61      | 79.02         | 73.43       |
| 50    | 60.15      | 56.625        | 53.10       | 83.92      | 78.925        | 78.15       |
| 75    | 59.81      | 56.99         | 54.17       | 83.03      | 80.725        | 78.42       |
| 100   | 59.73      | 57.185        | 54.64       | 84.17      | 81.635        | 79.10       |
| 150   | 58.13      | 56.505        | 54.88       | 86.66      | 84.67         | 82.68       |
| 200   | 58.63      | 56.74         | 54.85       | 87.96      | 86.375        | 84.79       |
| 300   | 57.61      | 56.2          | 54.79       | 82.14      | 85.66         | 89.18       |
The second form of effect provided by the cable restrainer is high efficiency in equalizing the expansion joint displacements. This effect can be observed by reducing the joint displacements, which have higher magnitude than the average displacements, and by increasing the joint displacements that are lower than the average displacements.

For example, a displacement reduction can be observed in the first gap, where as a mixed effect of reduction and increase is apparent in the second gap. This mixed effect can be related to the facts that the joint displacement value at the left of the expansion joint is higher than the average displacement and that this displacement is decreased to be close to the average. Meanwhile, the displacement on the right side of the expansion joint is lower than the average and needs to be increased to bring it closer to the average displacement (Figures 6 and 7).

The desired effect of maintaining the longitudinal displacement near the average value comes from the need to avoid a displacement anomaly that can subsequently lead to out-of-phase response. Accordingly, pounding or unseating occur in the worst-case scenarios. As observed in this section, the relation between the longitudinal displacement and the stiffness value varies with the different stiffness categories (low, medium, and high). At the range of low and medium stiffness, the displacement adjustment (reduction or increase) can be significant with a reduction percentage that reaches up to 18% and an increase of up to 15%. However, the effect of the restrainer within this range of displacements is highly unpredictable.

At the range of high displacements, the cable restrainer maintained the adjustment efficiency in the low and medium stiffness ranges, but this efficiency is insignificant. However, in the high range of stiffness, the cable restrainer gains a high displacement stabilization capability without sacrificing its displacement adjustment (decrease or increase) capability. Therefore, a cable restrainer with a high stiffness (from 200 KN/mm to 300 KN/mm) could be the optimum choice for the cable restrainer stiffness to achieve optimum adjustment and stabilization capabilities.

### 7.4 Effective damping comparison

The effective damping of the cable restrainer is represented by the material property of the cable rather than by providing a damping element. The cable material properties are defined to provide effective damping values of 0.025, 0.050, 0.075, and 0.100. The bridge was analyzed with these effective damping values and with a constant stiffness of 300 KN/mm. The obtained results are illustrated in Figure 9.

![Figure 9. Variation of displacement with effective damping (nodes A, C, and E).](image_url)

As shown in Figure 9, the effective damping property for the cable restrainer material has a slight effect on the longitudinal displacements. This effect is generally concentrated at the range of the low effective damping. However, as the effective damping increases, the changes in the displacement...
values decrease until they nearly stabilize. Therefore, if a certain value of effective damping is provided, then keeping the displacement within the low range is recommended because no significant effect is gained by increasing it.

7.5 Selecting the optimum restrainer properties

The parametric study that has been conducted and presented in the previous section shows that the restrainer stiffness significantly affects its efficiency in controlling the longitudinal displacements, especially at the expansion joint. The effective damping properties also have only a slight effect on the response of the bridge.

Accordingly, the optimum properties of the cable restrainer were selected. A high value of stiffness is recommended to achieve a high efficiency in displacement control (which was earlier referred to as displacement adjustment) as well as in response stabilization. A stiffness value of 300 KN/mm is chosen to achieve maximum efficiency. For effective damping, a low value of 0.025 is chosen to achieve the same purpose.

These values were combined in one cable restrainer. The analysis was performed to evaluate the seismic response for the bridge under study when equipped with the optimized restrainer under X-direction and 3D seismic excitation. The result of the analysis is displayed in the form of tables and time history graphs to be compared with the seismic response of the bridge without a cable restrainer. Figure 10 shows the time history displacement of a sample node (node A) under seismic excitation in the X-direction without the use of a cable restrainer. The optimum restrainer with stiffness of 300 KN/mm and effective damping of 0.025 is implemented to the bridge, and seismic response of different nodes under seismic excitation is evaluated.

![Figure 10. Overall time history displacement (without restrainer).](image-url)
Table 7. Nodal displacements of the structure with restrainer (effective damping = 0.025).

| DIRECTION | DISPLACEMENT (mm) |
|-----------|-------------------|
|           | SPAN 1  | SPAN 2  | SPAN 3  |
| A         | B       | C       | D       | E       | F       |
| Acc. In x Direction | 59.95   | 60.12   | 56.85   | 57.61   | 54.68   | 54.93   |
| 3D Acceleration          | 90.84   | 90.75   | 82.14   | 83.18   | 86.62   | 85.80   |
| X CHANGE (%)            | -18.96  | -18.84  | -7.96   | -3.88   | 15.32   | 15.56   |
| T CHANGE (%)            | -6.37   | -6.40   | -3.17   | -1.69   | 15.22   | 14.55   |

As shown in Table 7, the maximum reduction is 18% at node A when the bridge is subjected to the seismic wave in the X-direction. Peak displacements are increased by approximately 15% by using the optimum restrainer in the bridge. The smallest effect of the restrainer is observed when the earthquake acceleration is applied in the Y- and Z-directions that are perpendicular to the acting direction of the restrainer. The time history displacement of the seismic response of the selected nodes during the seismic loading with and without the optimum restrainer is shown in Figures 11 and 12.

Figure 11. Time history displacement of node A (X-direction).
Table 7 and Figures 11 and 12 show that the restrainer with the chosen optimum properties provides maximum efficiency in controlling the displacements with high adjustment percentages of -18.96%, -7.96%, and +15.56% for nodes A, C, and E, respectively. As shown in Figure 10, the response of the structure fluctuates and nearly follows a random pattern. This behavior is extremely hazardous because it causes the superstructure segment to vibrate out of phase, and this type of vibration causes the decks to pound against one another. The force transfer and the displacement magnification resulting from pounding causes the unseating of the span decks. Figures 11 and 12 show that the random mode of vibration is stabilized and replaced with a more uniform mode, with a stable gradient from maximum to zero when the cable restrainer is used.

8. Conclusion
In this study, considerable literature has been reviewed to gain in-depth understanding of the response of similar structures under the effect of seismic excitations, and to shed light on the methods and results obtained from earlier studies by other researchers. Previous studies have set a solid foundation by setting the bases on which the present study is conducted.

Reviewing the available literature has shown the importance and the consequences of several assumptions such as the rigidity of the deck and its effect on the overall response of the structure. The conducted seismic retrofitting approaches, their benefits and drawbacks, and their effect on the bridge response under the seismic excitation are considered.

The efficiency of using the cable restrainer alone or combined with the base isolation measure has also been discussed. The capabilities of the cable restrainer to mitigate the longitudinal displacements, pounding, and unseating of the bridge decks; to control the residual rotation in the case of skewed bridges; and to hold an unseated span and prevent it from falling are discussed in detail. The results showed that high longitudinal displacements occurred along the layout line although the bridge was modeled with high safety factors. These displacements are large enough to cause bearing failure and subsequent unseating.

The parametric study conducted on the stiffness and effective damping properties of the cable restrainer has demonstrated interesting results. The cable restrainer acquires its displacement control capability at low to medium stiffness values, but these values are variable and unpredictable within the range of stiffness values.

However, as the stiffness value increases toward the high range, the cable restrainer maintains the displacement control capability attained at low stiffness values but with substantially more stable and reliable performance.

The parametric study has also shown that the effect of the effective damping, defined in the cable restrainer material properties, provided limited significance in enhancing the seismic response of the
bridge structure. Within this limited range, a low value of effective damping achieves a certain capability of displacement control. However, no significant increase in this capability was associated with increasing the value of the effective damping in the cable restrainer.

Optimum values of stiffness and effective damping were chosen from the range of values considered in the parametric study and were implemented in modeling an optimum restrainer property. The efficiency of this restrainer was tested by comparing the structural seismic response with and without the optimum restrainer. The result showed that a significant percentage of displacement control and stabilization was achieved. Through this test, the efficiency of the cable restrainer retrofitting technique was proven to be effective and viable because of its ease of implementation, reliability, and low cost.

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