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The Performance of Integral and Semi-integral Pre-tensioned Concrete Bridges Under Seismic Loads in Comparison with Conventional Bridges

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ABSTRACT: Bridges are divided into three categories of integral, semi-integral, and conventional (seat type) bridges, based on the connection of deck to abutment. The integral and semi-integral bridges have been widely used recently, while the interactions of soil with abutments and piles are important issue in designing them. However, limited studies have been carried out on the behaviors of integral and semi-integral bridges and, hence, a few specific and suitable designing indices for them can be found. In this study, a 3D finite element model for each type of bridges was developed and analyzed under seismic load. Due to the importance of soil-structure interaction, non-linear springs (links) were employed to simulate the effects of soil behind abutments and soil around piles on the structure. This study determined the effects of seismic loading on the abutment and its backfill soil in the conventional, integral and semi-integral bridge models, and also compared the equivalent exerted force from backfill soil to structure in these three types of bridge models.

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1- Introduction

The use of integral bridges in the United States dates back to 1905, when the state of Colorado used continuous abutments. However, the US Department of Transportation did not accept this type of design until 1930, when Hardy Cross introduced the continuous frame analysis method [1]. In the early of the 1960s, it became clear that the joints and deck supports were the basic problems for maintenance and repair of bridges and the bridges that are constructed in continues and joints-less manner present better behavior than the common bridges in this respect. Hence, the integral bridges remained under longer service life, with minor and occasional maintenance and repairs [2]. During the following decades, the use of integral bridges was progressively extended in USA and several other countries. It was advised by the American Federal Highway Association (FHWA) in 1980 to build integral steel, cast-in-place concrete, and post-tensioned bridges with overall lengths of 90, 150, and 183 m, respectively [3]. The British Highways Agency in 1996 recommended that any bridge up to 60 m length should be constructed using integral system [4]. In Australia, it has been a growing interest of using integral bridges in recent major projects, and Gibbens & McManus (2011) reported that eleven out of thirty bridges have been built as integral system in the Peninsula Link Highway project in Victoria [5]. Integral bridges also have other benefits of fast construction, suitable resistance to catastrophic events, and uniform lateral load distribution [6].

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Integral bridges, which are also named as integral abutment bridges (IABs), joint-less, rigid frame and U-frame bridges, do not have bearings on the abutments and the loads on the structure are tolerated by frame behavior and soil-structure interaction. In these bridges, the connection between superstructure and abutment is fully rigid and this integration results in reduced superstructure displacement and also causes passive backfill soil pressure to increase axial loads of the superstructure [6]. In addition, semi-integral bridges with bearing equipment and short abutment are frequently used instead of conventional bridges. The deck is positioned on an abutment through bearing (usually neoprene) to provide movement in the semi-integral system. The expansion joints in the upper part of the bearing are removed in this system, similar to integral bridges, and hence durability is increased [7].

Although the integral bridges have major benefits, they are not widely used around the world due to the lack of specific and suitable indices for their design-lead and certainty for their thermal and seismic behavior. Therefore, several studies have been performed to characterize the responses of integral bridges under various geotechnical and structural conditions [8-13]. Hambly, in a review article, analyzed different types of integral bridges in UK, USA and Sweden that varied geometrically, in terms of total length, the number of spans and the skew angle, but with the same concept of integrity between the deck and abutments [14]. Erhan and Dicleli conducted a parametric study to determine appropriate structural configurations and geotechnical properties for enhancing the seismic performance of integral bridges [15]. Yen et al. attempted in their paper to present the practical states of joint-less continuous bridges in seismic regions and found suitable connections for integral joint-less bridges in these regions [16]. Qian et al. performed shaking table experiment on soil-micro-pile interaction of semi-integral abutment bridge [17]. Also, Maleki and Mahjoubi presented a new approach about the soil-structure interaction for estimating the seismic soil pressure on retaining walls [18].

As explained above, limited studies have been compared the behaviors of integral and semi-integral bridges. Therefore, the present study is aimed to compare the performance of integral, semi-integral and conventional pre-tensioned concrete bridges under seismic loads, and also to evaluate the effects of backfill soil pressure on the structure of integral and semi-integral bridges under seismic loads.

2- Bridge Modeling Description

A numerical model was made from an actual 5-span pre-tensioned concrete railway bridge with a total length of 265 m. The central and end spans length are 65 m and 35 m, respectively (Figure 1). The bridge's deck is a concrete precast box-girder with 5.6 m width. However, the deck height is variable from 2.5 m in the middle of spans and on the abutments and up to 4.5 m on the piers.

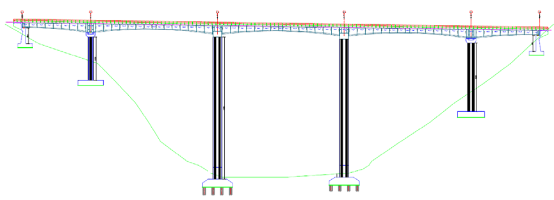

Figure 1. Schematic elevation view of the bridge

3- Finite-Element Model

Three 3D finite element (FE) models for integral abutment bridge, semi-integral abutment bridge, and conventional bridge were developed, with using CSI-BRIDGE V19.0.0 software, to study the effects of seismic load on their superstructure, abutment and its backfill soil. The models were subjected to gravity load in the first step and seismic load in the second step.

The geometry was modeled according to the actual dimensions of the superstructure and substructure of the bridge. Elastic-plastic behavior was considered for concrete and steel materials used in the model. Concrete used on the deck and other components of the bridge had compressive strengths of 32 MPa and 24 MPa, respectively, and Mander's model (1988) was used to define the strength of confined concrete. In addition, the steel used for reinforcements had yield stress of 400 MPa.

The abutment walls were modeled as plate elements (shell). Connection types of deck to abutment were considered as completely rigid for integral bridge model and semi-rigid for semi-integral bridge model, in which abutments were separated from the girders but still connected to the concrete slab. Soil was modeled using two-node non-linear spring elements, in which each spring had one translational (axial) degree of freedom. The soil springs behind the abutment walls were accounted for both passive and active pressures, and the springs at the piles were accounted for passive pressure on each side of the pile.

By studying the behavior of a continuous bridge during several earthquakes in Cap Mendocino, Goel (1997) obtained damping of a continuous bridge structure between 5.6% to 12% in various earthquakes and concluded that the damping of integral bridges could be between 5.6% and 12% [19]. Therefore, the damping of 8% was an acceptable value to be considered for integral and semi-integral models in this study.

4- Soil-Structure Interaction Model

The major problem in analyzing and designing integral abutment bridges is the lack of knowledge in the non-linear behavior of abutment-soil and pile-soil interaction, which affect the magnitude and nature of the stresses and deformations of the soil and structure [11]. Hence, it is important and necessary to study the non-linear interaction of the abutment-soil and pile-soil during seismic events for revealing the essential factors that affect the behavior of these bridges. Consequently, the methods that are most suited to the actual behavior of the structure can be chosen for using in the experimental studies.

The utilization of a series of non-linear springs behind the abutments and around the piles was used in this study as one of the best methods for demonstrating the structure-soil interaction, especially when the structural load is the main item of interest [20-22]. In this type of soil-structure interaction modeling, the lateral load at one point does not affect the lateral load at other points along the depth of the pile [23].

Terzaghi showed that the soil resistance value for each depth can be estimated via performing horizontal loading test on flexible piles. The momentum diagram of the piles length was obtained from the numerical values acquired by the strain gauge, and it was used to determine the horizontal force on the piles. By repeating test for different horizontal forces, a p-y diagram was drawn up, in which "p" was the horizontal pressure input to the pile and "y" was the horizontal deformation [24]. Matlock et al. invented an extremely accurate method that measured the bending moment of a piles and then via two mathematical integrations of the bending moment diagram of the pile provided the first series of p-y curves of piles for the lateral force [25]. This latter approach can be performed by using computer program such as LPILE or COM254, instead of using fairly straightforward analyses. Several non-linear methods have been proposed by different researchers for defining the soil behind the abutment that many of them are very time-consuming to be used in the bridge analysis and sometimes require specific software. Currently, nonlinear diagrams, such as NCHRP and CGS-Canada diagrams, are the most practical methods for analyzing the interaction of backfill soil in the integral bridges.

4- 1- Soil-Abutment Interaction Model

For the soil behind the walls, the effective horizontal stress (σ'_h) and effective vertical normal stress (σ'_v) can be related to each other by using a lateral earth pressure coefficient in the mathematical formula of $K=\sigma'_h/\sigma'_v$, which the value of chosen "K" depends on the level of anticipated displacement. When the same value of K is assumed for all wall depths, it means that a triangular distribution of horizontal earth pressure is presumed, with a resultant reaction force located at H/3 above the base of the wall. The magnitude of resultant reaction force will be F=(1/2)K γ H^2, where γ is the unit weight of the soil. In general, the lateral soil pressure distribution is not triangular.

A literature review research is required to obtain the appropriate non-linear soil response curves behind the abutment wall for applying them to integral abutment bridges. The two most used sets of design curves for estimating the non-linear force-deflection relation behind a rigid retaining wall are NCHRP and CGS-Canada [26]. The NCHRP design curves were used in the current study, the general form of the NCHRP lateral earth pressure K versus deflection design curves was digitized for loose, medium dense, and dense soils and presented in the Figure 2.

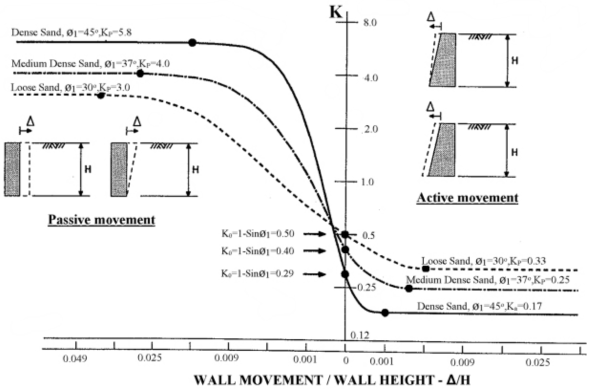

Figure 2. Earth pressure coefficient versus relative wall displacement (NCHRP 1991) [25]

NL-Link elements were used as elastic-plastic for modeling non-linear spring of soil equivalents. For this purpose, the area involved the abutment and the backfill soil was divided into hypothetical panels, and the transferred force from the soil in each of these panels was calculated with the aid of Equation 1, and each of these springs was equivalent to that part.

$$F = \sigma_h' A = K\gamma zwh \tag{1}$$

In the used program (CSI Bridge) and most of the existing commercial programs, the spring-load displacement diagram must necessarily pass from point (0,0). Therefore, based on the study of Easazadeh- Far et al. [27], the whole force displacement diagram of non-linear springs was shifted downward in order that the spring force passed the point (0,0). To compensate this maneuver, a constant force of P_0 was applied to the abutment at the same location and combined with the non-linear force of the spring. The soil behind the abutments was assumed to be the dense type and its properties have been presented in Table 1.

4- 2- Soil-pile interaction model

In most analytical and design methods, the soil around the piles was modeled as a series of springs at the pile's height. So that the p-y pile curve was calculated at the depths of the nodes of the pile elements and multiplied by the pile element length to obtain the curve of force-displacement of non-linear springs, representing the soil around the pile. Then these springs were modeled at the desired depths and connected to the piles nodes. According to the API (1993) recommendation [28], the p-y curve at depth z was obtained from the following Equation 2.

$$P = AP_u \tanh\left[\frac{K_1 z}{AP_u} y\right] \tag{2}$$

Table 1. Properties of soil behind the abutments

modulus of elasticity (N/mm^3)	p-y modulus, (kN/m^3)	friction angle , (deg.)	Unit weight (kN/m^3)
45	64000	45	19

Instead of performing repetitive processes, LPILE software was used in the present study to obtain the p-y pile curve. The total pile height of 35 meters was modeled by 35 perpendicular spring pairs with 1 meter distance between each pair. The piles ended at the bed stone and, hence, the connection of pile to foundation assumed to be fixed in the used model.

The soil around the whole altitude of pile was assumed to be loose type with three different modulus of elasticity (E), including E=10 Mpa for the first 12 m, E=25 Mpa for the next 10 m, and finally E=50 Mpa for the 13 m in the bottom end. The other properties of the used soil in the analysis are as followings:

Dry density of soil: γ = 18.85 kN/m^3

Friction angle: ϕ = 35°

5- Analysis Method

In preliminary analyses, the non-linear modal time history analysis, with very high speed of about 100 times of the non-linear direct integration analysis, showed an acceptable accuracy that were about 2-3% different in most cases from the non-linear direct integration analysis. Therefore, non-linear modal time history analysis was used for structural analysis in the present study. Then, data of the earthquakes shown in Table 2 were entered into each of our 3 considered models and analyzed. In order to compare the results of earthquakes under different conditions, these earthquakes should be applied in the similar conditions to the structure. Hence, according to the section A8 of the FEMA-P695 code [29], we firstly multiplied the normal coefficients of each earthquake to its data, and then the averages of normalized earthquake data were scaled up to the region of actual bridge modeled in this study. Finally, the results of each model were averaged to compare the seismic behaviors of integral, semi-integral and conventional bridges.

6- Verification of the Model

The above mentioned analysis method was initially validated and verified by using the results obtained in the study of Ting & Faraji (2001) for the soil-structure interaction model [30]. With using the same coefficients of thermal expansion α=0.0000117 1/°C for both steel and concrete as to those of Ting & Faraji, the analysis of the model was performed for the loading case of 80°F (45 °C) at situation that no gravity load and internal temperature gradient within the abutment were considered.

Table 2. Data of used earthquakes for analyzing model

PGV	PGA	M	Station	Year	Name	Record
45	0.48	6.7	Canyon Country-WLC	1994	Northridge	1
37	0.38	6.5	El Centro Array #11	1979	Imperial Valley	2
37	0.51	6.9	Nishi-Akashi	1995	Kobe, Japan	3
42	0.42	7.3	Coolwater	1992	Landers	4
35	0.53	6.9	Capitola	1989	Loma Perieta	5
54	0.51	7.4	Abbar	1990	Manjil, Iran	6
19	0.21	6.6	LA-Hollywood Stor	1971	San Fernando	7

In the model of Ting & Faraji the non-linear soil behavior behind the abutment walls and next to the vertical piles were incorporated into a 3D FE bridge model, using commercially available computer package GTSTRUDL [30]. Non-linear analysis capabilities in GTSTRUDL included (1) small-strain geometric; (2) material non-linearity for plane and space truss members; and (3) material non-linearity for support reactions. Details of the bridge construction and finite element model, including the GTSTRUDL input file, were presented in the study of Ting and Faraji (1998) [31]. Figure 3 shows that the analytical modeling used in the present study is sufficiently validated and verified.

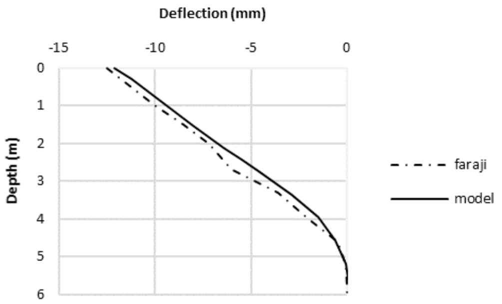

Figure 3. Abutment deflection in the models of Ting & Faraji and also the present study

7- Results and Discussion

Table 3 shows that the conventional bridge has the highest period of longitudinal direction, and those of the integral and semi-integral bridges do not significantly differ from each other. It is also observed that the period of transverse direction is somehow higher in the conventional bridge than the integral and semi-integral bridges, which are nearly equal.

Among the 3 types of bridges presented in the Table 4, conventional bridge has the largest maximum longitudinal and transverse displacements, due to the freedom of its deck on the abutments. The complete rigid connection between deck and abutments in the integral bridge causes its maximum longitudinal displacement to be lower than that of the semi-integral bridge. However, the continuous connections in both integral and semi-integral bridges and also action of abutment as a rigid shear wall result in no difference between their maximum transverse displacements. On the other hand, the more stiffness of connection between deck and abutments causes more deformation along the length of deck in the conventional, semi-integral and integral bridges that result the maximum vertical displacement to be successively increased in them.

Table 3. Periods of bridges

	Conventional bridge	Semi-integral bridge	Integral bridge
Longitudinal direction	2.793	0.895	0.823
Transverse direction	4.234	4.032	4.016

Table 4. Maximum displacement of the structure (mm)

	Longitudinal direction	Transverse direction	Vertical direction
Integral bridge	173.72	649.46	77.73
Semi-integral bridge	261.57	648.75	58.76
Conventional bridge	445.68	710.17	48.11

The above obtained results indicate that integral and semi-integral bridges have lower periods and displacements. These characteristics of the integral and semi-integral bridges are due to their structural frame behavior and soil-structure interaction that reduce transition force to the deck by distributing it to the entire structure and, hence, a more uniform behavior is displayed. Consequently, these types of bridges exhibit better behavior against seismic load than the conventional bridge.

The soil-structure interaction in the integral and semi-integral bridges has not only the above mentioned beneficial effects but also some negative effects on the behaviors of these bridges against seismic load. Regarding these cases, the following results are presented for determining the better performance between the integral and semi-integral bridges. Tables 5 and 6 clearly indicate the importance of integrity in superstructure and abutment connection for preventing abutment rotation. The increase of rotational stiffness and, consequently, the decrease of rotation in the joint of deck to abutment cause the maximum displacement of abutment in the longitudinal direction to be significantly reduced in the integral bridges compared to the semi-integral bridges.

Table 5. Maximum displacement of the abutment with passive soil (mm)

	Left abutment	Right abutment
Integral bridge	18.25	16.71
Semi-integral bridge	77.18	70.45
Conventional bridge	0.32	0.33

Table 6. Maximum displacement of the abutment with active soil (mm)

	Left abutment	Right abutment
Integral bridge	16.85	18.60
Semi-integral bridge	71.50	78.59
Conventional bridge	0.33	0.32

According to the Figures 4 and 5, deflection of the abutments is very small in the conventional bridge, because the deck does not bring direct force into the abutments. However, the existence of deck force in the integral and semi-integral bridges results the deflection of abutments to be increased, especially at the upper heights. Moreover, deflections of abutments show more significant gradual increases with the increases in their heights from 6 m (at the mean height of abutment) in the integral bridge and 8 m (deck sitten on the abutment) in the semi-integral bridge. In addition, the gradual increases of abutments deflection are largely intensified in the higher heights of the semi-integral bridge, because the deck force is concentrated to the top of the abutment and also the rotational stiffness is low in the upper part of abutments, but it is uniform in the integral bridge.

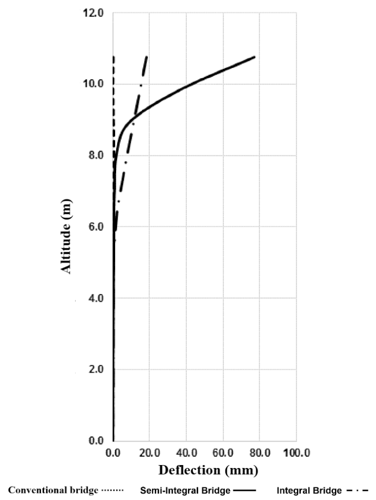

Figure 4. Abutment's deflection with passive soil

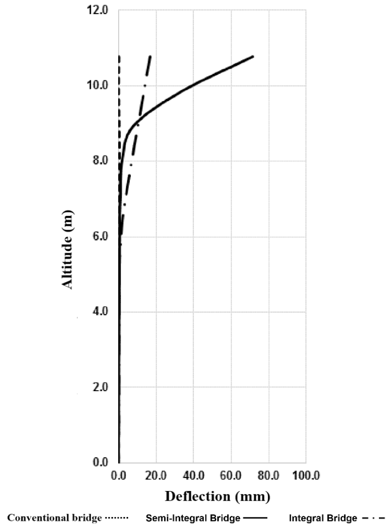

Figure 5. Abutment's deflection with active soil

One of the most important issues in the design of integral and semi-integral bridges is the effect of the interaction between the abutment and its backfill soil that leads to formation of an axial force on the bridge deck. As it has been mentioned above, the maximum displacement of the semi-integral abutment is about 4-times of the integral abutment (Table 5, but the nonlinear behavior of soil in the semi-integral abutment causes the maximum soil pressure-rise in the passive state to be about 2-times more in the semi-integral than the integral bridge (Table 7). In addition, the pattern of pressure-rises for passive soil in the Figure 6 shows that the maximum soil pressure-rise in the bridge with semi-integral abutment occurs at a higher height than that of the integral abutment, which is due to the intensified abutments deflection in the higher heights of the semi-integral bridge. At these high abutment heights and low soil depths in the semi-integral bridge, the decreased soil hardness reduces soil pressure-rise that somehow offsets the effect of large displacement and, therefore, the maximum soil pressure-rise difference between the integral and semi-integral abutment is lessened.

While in the state of active soil, as presented in the Table 8, the maximum pressure-drop produced on the integral and semi-integral bridges are very close to each other. It is well-known that the amounts of maximum pressure-drop in active soil state are much lower than the amounts of maximum pressure-rise in passive soil state. The non-significant difference of maximum pressure-drop between the integral and semi-integral abutments is due to not only the occurrence of maximum pressure-drop at a higher height of abutment in the semi-integral than integral bridge (Figure 7), but also the entrance of nonlinear soil behavior in both types of the bridges.

Table 7. Maximum soil pressure-rise with passive soil (MPa)

	Left abutment	Right abutment
Integral bridge	3.778	3.436
Semi-integral bridge	5.925	5.400
Conventional bridge	0.156	0.161

Table 8. Maximum soil pressure-drop with active soil (MPa)

	Left abutment	Right abutment
Integral bridge	0.454	0.426
Semi-integral bridge	0.324	0.323
Conventional bridge	0.018	0.018

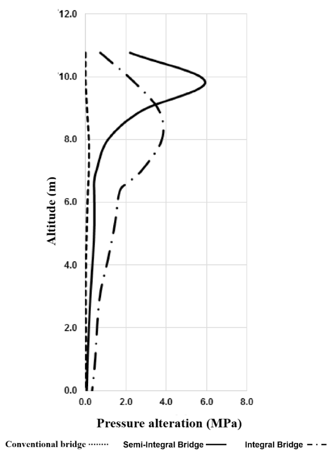

Figure 6. Soil pressure-rise pattern in abutments with passive soil

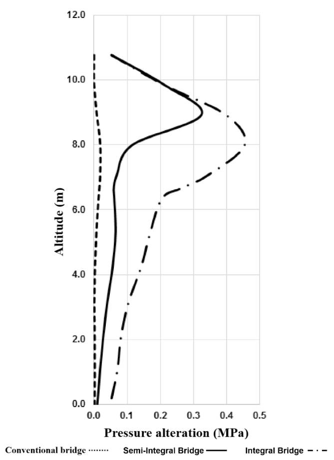

Figure 7. Soil pressure-drop pattern in abutments with active soil

The area under each curve in the Figures 6 and 7 determines the equivalent force applied on the abutment and its values for both types of bridges have been presented in the Table 9 for passive soil and Table 10 for active soil, which both of them are larger in the integral bridges than semi-integral bridges. Since the larger equivalent force applied on the abutment causes the more tolerated force by the deck, hence, the internal force of structure is less in the semi-integral than the integral bridges

Table 9. Equivalent force that is exerted on the abutment with passive soil (kN)

	Left abutment	Right abutment
Integral bridge	101902.8	94536.8
Semi-integral bridge	69633.5	66881.7
Conventional bridge	3330.7	3434.8

Table 10. Equivalent force that is exerted on the abutment with active soil (kN)

	Left abutment	Right abutment
Integral bridge	11797.0	11183.5
Semi-integral bridge	5380.9	5105.3
Conventional bridge	383.7	380.9

It should be mentioned that the alterations of backfill soil pressures by seismic load were considered in all parts of this study, but not the sum of them and the P_0 that was the actual exerted soil pressure. Since the 3 types of bridges had equal P_0, the amounts of alterations in backfill soil pressures by seismic load discriminate these bridges from each other. Therefore, the amounts of alterations in backfill soil pressures were larger in the integral and semi-integral bridges due to abutment displacement, in contrast to the conventional bridges, that makes their weakness point. Consequently, the equivalent force was higher in the integral and semi-integral bridges than the conventional bridges.

The intensified increases of soil pressure alterations reach to a peak at about 8 m (deck sitten on the abutment) for the integral bridge and about 10 m (at the mean height of the upper part of abutment) for the semi-integral bridge, and then are decreased (Figures 6 and 7). Consequently, the equivalent force is exerted at a higher height location for semi-integral than integral bridges (Tables 11 and 12), which also affect the moment of abutment at its bottom in both types of bridges.

Table 11. Equivalent force location with passive soil (m)

	Left abutment	Right abutment
Integral bridge	7.00	6.91
Semi-integral bridge	8.63	8.49
Conventional bridge	6.54	6.54

Table 12. Equivalent force location with active soil (m)

	Left abutment	Right abutment
Integral bridge	6.74	6.80
Semi-integral bridge	7.59	7.76
Conventional bridge	6.53	6.53

8- Conclusions

The obtained results in the present study show that the seismic behaviors of the bridges with integral and semi-integral abutments are nearly identical at the transverse direction, because of the high stiffness of their wall and deck. While, the conventional bridges have longer structural period and displacement at the transverse direction than the other two types of bridges. The integral and semi-integral bridges, because of their structural frame behavior and soil-structure interaction, also reduce the transition force to the deck by distributing it to the entire structure and displaying a more uniform behavior that lead to less displacement. Therefore, both types of bridges have better performance against seismic load than the conventional bridge.

However, the amounts of alterations in backfill soil pressures by seismic load are high in the integral and semi-integral bridges, due to their abutment displacement, that result in largely exerted equivalent force. Finally, the preferable usage of the semi-integral bridge is suggested by this study, because it has less equivalent force applied on the abutment that causes less tolerated force by the deck and, hence, the internal force of structure is lower in the semi-integral bridge than the integral bridge.

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