Evaluation on influences of inertial mass on seismic responses and structure-soil interactions of pile-soil-piers

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Abstract: This research is to assess the influences of the inertial mass from the girder on the dynamic characteristic, dynamic response, and structure-soil interaction of a pile-soil-pier subsystem in a scale-model of a cable-stayed bridge. Therefore, both connection configurations between the pile-soil-pier and girder, including the sliding and fixed connections, were designed to present various inertial mass from the superstructure delivered to the pile-soil-pier. The pile-soil-pier supported by a 3×3 pile-group in mixed soil placed in a shear box was tested using shaking tables. The dynamic characteristics, seismic responses, inertial interactions, and pile group effects of the pile-soil-pier between the sliding and fixed connections were analyzed under three input motions with different shaking amplitudes. These results showed that more inertial mass from the girder significantly increased the reinforcement strain and bending moment at the column bottom and pile top, displacement at the column top, inertial interaction effects, and pile group effects of the pile-soil-pier due to the sliding connection changing to the fixed connection. The inertial mass increment from the girder noticeably decreased the peak accelerations of the column of the pile-soil-pier when subjected to three input motions with different amplitudes. However, the inertial mass insignificantly affected the accelerations of the pile and free-soil. Therefore, the corresponding kinematic interaction effects were almost unaffected by the inertial mass. Additionally, the evident pile group effects were observed in the sliding and
fixed connections between the pile-soil-pier and girder. The numerical model could approximately reproduce the
macroscopic seismic responses of the pile-soil-piers with sliding and fixed connections and capture the typical
response variations induced by the connection configuration change.

**Key Words:** Pile-soil-pier; inertial mass; seismic responses; inertial effects; pile group effects; shaking table tests

1. **Introduction**

Pile foundations are often employed in bridge structures located in soil deposits. The bridge structure with the pile
foundation always interacts with the soil a certain degree subjected to earthquake excitations, compelling soil
deformations that lead to the movements of the pile foundation and bridge structure, in turn, termed as structure-
soil interaction (SSI) (Clough and Penzien 2003). The SSI effects play crucial roles in the seismic response of bridge
structures according to the post-earthquake observations (Ganev et al. 1998), field tests (Ashford et al. 2006, Ko
and Chen 2010), test studies (González et al. 2009), and numerical investigations (Vlassis and Spyroskos 2001,
Zhang et al. 2008). The interaction effects are influenced by the soil motions, which produces the kinematic
interaction, and the mass from the superstructure, which generates the inertial interaction. Therefore, extensive tests
and simulations focused on the SSI effects and seismic behavior of the pile-supported piers or piles in the past
decades (Makris et al. 1997, Yao et al. 2004, Tokimatsu et al. 2005, Cubrinovski et al. 2006, Chau et al. 2009,
Motamed and Towhata 2010, Gao et al. 2011, Haeri et al. 2012, Goit and Saitoh 2014, Durante et al. 2015, Wang et
al. 2015, Durante et al. 2016, Su et al. 2016, Durante et al. 2017). However, there are few experimental studies on
the dynamic response and SSI effects of bridges by considering the influences of an actual superstructure. In other
words, the superstructure was generally considered as either a simplified structure or even an oscillation mass in the
previous studies. Therefore, experimental studies on evaluating the influences of the inertial mass from an actual
superstructure on the dynamic response and SSI effect in bridges need to be performed to understand the knowledge
In the past, many tests have studied the SSI effects and seismic behavior of pile-supported piers with a concentrated oscillation mass or even neglecting an oscillation mass in different soils (Makris et al. 1997, Yao et al. 2004, Rollins et al. 2005, Tokimatsu et al. 2005, Cubrinovski et al. 2006, Dungca et al. 2006, Chau et al. 2009, Motamed and Towhata 2010, Gao et al. 2011, Haeri et al. 2012, Chang and Hutchinson 2013, Motamed et al. 2013, Goit and Saitoh 2014, Durante et al. 2015, Wang et al. 2015, Durante et al. 2016, Su et al. 2016, Durante et al. 2017, Liu et al. 2017, Wang et al. 2019). The research work validated experimentally the Winkler foundation model using the displacement transfer function and strain spectrum of a SDOF superstructure supported by a single pile (Makris et al. 1997). Yao et al. (2004) discussed experimentally the interactive behavior of a two-story structure supported by a pile group in liquefiable soils. The effects of inertial and kinematic forces on the pile-soil-superstructure placed in liquefiable or dry sand were studied in the published paper (Tokimatsu et al. 2005). Experimental results indicated that the displacement was beyond phase with the inertial force, stopping the stress of the pile from increasing as the period of the superstructure is more than that of the ground. Chau et al. (2009) studied experimentally a pile-soil-structure model embedded in river sand. Test results revealed that a pounding between the pile and soil occurred, and the peak acceleration of the pile cap was 3.0 times greater than that of the structural response. Motamed and Towhata (2010) conducted shaking table tests to investigate the seismic response of a pile group behind a quay wall subjected to soil displacement. The pile-head constraint condition had marked impacts on the bending moment of the pile. Gao et al. (2011) explored the interactive behavior of soil-pile foundation in liquefiable soil under the El Centro wave with different shaking frequencies and amplitudes. Results revealed that the shaking frequencies had insignificant influences on the bending moment and acceleration of the pile. Goit and Saitoh (2014) carried out shaking table tests on an inclined pile-group model to assess the soil nonlinearity influences on the seismic behavior.
Results indicated the profound impacts of local nonlinearity on the impedance functions. Wang et al. (2015) investigated experimentally scoured bridge models with pile foundations to estimate SSI and seismic performance. Test results showed the bending moment increment of piles whereas the bending moment reduction of piers as the scoured depth increased. Durante et al. (2015 and 2016) studied experimentally a model comprised of a concentrated mass and a single pile or a pile-group embedded in soil deposits. The bending moment of piles was not affected by the oscillator mass, but it was dependent upon the dominant frequencies of the structure-soil system and the frequency contents of the input motion. Motamed et al. (2013) investigated the behavior of a pile-group behind a gravity-type quay wall under large displacement induced by liquefaction. Experiment results indicated that the bending strain of the pile matched well with the failure mode of the pile, and the rear-row piles suffered from larger bending moments. Liu et al. (2017) presented test results of a 2×2 pile-group behind a quay wall that evident pile group effects were observed by comparing the bending moment between piles. Wang et al. (2019) investigated the inertial and kinematic effects on the curvature of the pile using shaking table tests on pile-soil-pier models in liquefiable and dry sand. Results indicated the near pile in liquefiable sand behaving more significant dilation tendency compared to the far-field. Previous studies emphasize the SSI significance in seismic behavior. However, rare test studies have checked the SSI and its effects on the dynamic responses of bridges considering the influences of an actual superstructure.

The aims of this study are to evaluate the influences of the inertial mass from the girder on the dynamic properties, seismic responses, and SSI effects of the pile-soil-pier in a 1/70-scaled cable-stayed bridge model. Both connection configurations between the pile-soil-pier and girder—one is named as the sliding connection, and the other is designated as the fixed connection—were designed in the bridge model with both structural systems, which could present the various inertial mass from the girder transferred to the pile-soil-pier. The pile-soil-pier consisted of both
columns and a cap supported by a 3×3 pile group in mixed soil in the shear box. The dynamic characteristics, seismic responses (displacements, accelerations, strains, and bending moment), inertial interaction effects, and pile group effects of the pile-soil-pier between the sliding and fixed connections were discussed for the test cases under three typical input motions with different amplitudes. A numerical model was established to replicate the seismic responses of the pile-soil-piers with sliding and fixed connections.

2. Experimental setting

2.1 Pile-soil-pier in cable-stayed bridge model

A scaled cable-stayed bridge model with 20 m central-span was designed following the similarity rules. Three essential similitude ratios, including the geometric dimension, Young’s modulus, and acceleration, were selected for the bridge model. The three similitude ratios were 0.01429, 0.3, and 2.0, respectively, according to the payload of the shaking table and project budgets. Other similitude ratios, such as the structural frequency (or period) and material density, were derived from the three essential similitude ratios based on the similarity formulas (Harris and Sabnis 1999). For example, the similitude ratios of the structural frequency (or period) and material density were 11.8322 (or 0.0845) and 10.5, respectively.

As illustrated in Figure 1, the bridge model was comprised of the pylon, pier, girder, and pile-group foundation in the mixture soil placed in the shear boxes. The height and central span of the bridge model were 6.386 m and 20.0 m, respectively. The piers consisted of the double-column pier (Piers 1 and 8) including corresponding pile groups (Pile-groups 1 and 8), double-column piers with shear beams (Piers 2 and 7) including pile groups (Pile-groups 2 and 7), and single-column piers (Piers 3 and 6) including pile groups (Pile-groups 3 and 6). All pile groups were in the mixed soil placed in the shear boxes. This paper aims to assess the influences of the inertial mass from the girder on the pile-soil-pier. The boundary and connection conditions of Pier 2 were similar to those of Pier 3 because they
were placed in an identical shear box and connected to the same girder by the fixed connections. Therefore, Pier 2, including a cap, Pile-group 2, and the mixture soil placed in Shear-box 1, was typically taken as an example to clarify the influences of the inertial mass from the girder on the pile-soil-pier. Herein, only the detailed information on the pile-soil-pier was presented. Other information on the bridge model could refer to the published paper (Xie and Sun 2019).

The scaled height was 860 mm for the column supported by the pile group (Pile-group 2). The size dimension of the box section of the column was $75 \times 122$ mm (see Figure 2). The micro-concrete and iron wires were employed to build the column with a reinforcement ratio of 1.7%. Figure 2 (b)-(c) plots the arrangement of the reinforcement bars of the column and cross-section, respectively. Eight supplemental mass boxes were installed to each side face of the column to satisfy the similitude rules. The height of the scaled pile was 1000 mm for Pile-group 2 comprised of 3×3 piles with 105 mm-diameter (D). The C40 concrete and ribbed bar were utilized to construct Pile-group 2 with a reinforcement ratio of 3.26%. The size dimension and reinforcement bars arrangement of the pile are depicted in Figure 2 (d)-(e). The spacing between both adjacent piles was 370 mm and 300 mm in the transverse and longitudinal directions (Figure 2 (f)), respectively, which is within the range of 3–4 D in practice (Fleming et al. 2009). It is worth noting that the scaled model did not consider the additional mass to the pile-group because of the

![Figure 1. Elevation view of cable-stayed bridge model (Units: m)](image-url)
model construction. However, there are insignificant influences on the dynamic response of the pile-soil-piers between the sliding and fixed connections because the pile groups were identical. The length, wideness, and height of the cap were $765 \times 765 \times 140$ mm, respectively. Shear-box 1 was comprised of the sliding frame that was manufactured using 8 mm-thickness/100 mm-height steel tubes. Its outside dimensions were $6700 \times 1700 \times 1106$ mm (see Figure 1). The sand and sawdust were used to produce the mixed soil, the mass ratios of which were 3:1 for the mixed soil. The material parameters could refer to in the published paper (Sun and Xie 2019).

Additionally, five I-type shear beams were designed and installed between both columns of the pile-soil-pier (Pier 2) according to the prototype of the shear beam (Sun et al. 2013, Xie et al. 2014). The length of the shear beam was 70 mm. The height and wideness of the cross-section were 20 mm and 40 mm, respectively. The flange and web thickness of the cross-section were only 2.0 mm, as shown in Figure 2 (g)-(h). Both column tops were hinged connections by a lateral steel beam (see Figure 2).

**2.2 Connection configurations**

The floating and supporting pier structural systems have been presented to assess the seismic performance of the
cable-stayed bridge model in the past (Sun and Xie 2019). For the floating structural system, the sliding bearings were mounted to the column top of the piers, such as the pile-soil-pier (Pier 2), supporting the girder. The corresponding connection configuration between the pile-soil-pier and girder was named as the sliding connection, as shown in Figure 3 (a) and (c). However, the fixed connection configuration between the pile-soil-pier and girder was employed using the welded steel bars for the supporting pier structural system (Figure 3 (b) and (d)). Pile-group 2 supporting the pile-soil-pier were secured to the bottom of Shear-box 1 bolted to Table A (Figure 3 (e) and (f)). The connection configurations between the pile-soil-pier and girder changed from the sliding connection to the fixed connection when the floating structural system varied to the supporting structural system. The corresponding inertial mass from the girder transferred to the pile-soil-pier increased. Therefore, both connection configurations could be taken to investigate the impacts of the inertial mass from the girder on the seismic responses, dynamic characteristics, and SSI effects.

![Schematic of sliding connections between the pile-soil-pier and girder in the floating structural system](image1)

(a) Schematic of sliding connections between the pile-soil-pier and girder in the floating structural system

![Schematic of fixed connections between the pile-soil-pier and girder in the supporting pier structural system](image2)

(b) Schematic of fixed connections between the pile-soil-pier and girder in the supporting pier structural system

![Figure 3. Connection configurations between the pile-soil-pier and girder](image3)
2.3 Instrumentations

Three types of sensors – the displacement transducer, accelerometer, and strain gauge – were employed to document the dynamic response of the test model (Xie and Sun 2019). However, only a part of the sensors attached to the pile-soil-pier and free-soil in Shear-box 1 were introduced. As illustrated in Figure 4, longitudinal displacements at the top and bottom of the column of the pile-soil-pier were measured using two displacement transducers. The longitudinal acceleration of the column and pile of the pile-soil-pier was recorded using six accelerometers. Three accelerometers were used to track the longitudinal acceleration for the free-soil. Reinforcement strains at the bottom of the column and the top of the corner, side, and center piles were detected using strain gauges. Using the National Instruments data acquisition gathered the displacements and accelerations. However, the reinforcement strain was collected using other data acquisition because the channels of the National Instruments data acquisition are not enough.

Figure 4. Instrumentations of pile-soil-pier in Shear-box 1
2.4 Input motions

A synthetic wave was obtained according to the seismic design guidelines of bridges in China (Xie 2013). Its frequency contents could present the site characteristics where the preliminary design cable-stayed bridge is assumed to be in coastal regions of China. Other two typical waves recorded from the 1994 El Centro and 1985 Mexico City earthquakes were taken as input motions to further assess the influences of the inertial mass from the girder on the pile-soil-piers because the seismic responses of the bridge model are easily influenced by the various frequency contents of input motions [39]. Also, the El Centro wave is a representative earthquake record frequently employed in shaking table tests. The typical Mexico City wave includes the long period contents, and it may induce a considerable response of the bridge model with a long period and a larger inertial mass from the girder transferred to the pile-soil-pier. It is worth noting that the duration of three input motions was scaled to 0.0845 times the prototype one due to the scaled effects of the pile-soil-pier in the cable-stayed bridge model.

Figure 5 illustrated the acceleration time histories with 0.10 g and corresponding acceleration frequency spectra, where 'g' is the gravitational acceleration. The dominant frequency contents of the synthetic wave, El Centro wave, and Mexico City wave were 8-20 Hz, 20-50 Hz, and 4-7 Hz, respectively. Figure 6 also plots the displacement and...
acceleration response spectra under 3% damping proposed by the guidelines for seismic design of highway bridge in China (MCPRC, 2008).

![Displacement and acceleration spectra of three input motions](image)

**Figure 6.** Displacement and acceleration spectra of three input motions

### 2.5 Test cases

The aim of this research is to estimate the influences of the inertial mass from the superstructure on the dynamic properties, seismic responses, and SSI effects of the pile-soil-piers. Firstly, the white-noise excitations were applied to the test model to capture the dynamic characteristic changes of the pile-soil-piers due to the inertial mass change caused by the connection configurations between the pile-soil-pier and girder. Then, the synthetic wave, El Centro wave, and Mexico City wave with 0.10, 0.15, and 0.20 g were longitudinally applied to the bridge model. Note that higher shaking amplitudes were not considered as possible to reduce the impacts of the constraint condition from the girder on the pile-soil-piers. Table 1 lists all test cases of the pile-soil-piers under three input motions in the longitudinal direction. 'S' and 'F' represent the sliding and fixed connection configurations between the pile-soil-piers and girder in the first column, respectively. However, the records were not imposed on the bridge model in the transverse direction because there were less influences of the inertial mass from the girder on the pile-soil-pier in the transverse direction.
Table 1. Test cases of the pile-soil-piers under longitudinal three input motions

| Test cases  | Excitation modes       | Input motions     | Peak acceleration (g) |
|-------------|------------------------|-------------------|-----------------------|
| S1 (F1)     | Uniform excitations    | White noise       | 0.08                  |
| S2 (F2), S3 (F3), S4 (F4) | Uniform excitations    | Synthetic wave   | 0.10, 0.15, 0.20      |
| S5 (F5)     | Uniform excitations    | White noise       | 0.08                  |
| S6 (F6), S7 (F7), S8 (F8) | Uniform excitations    | El Centro wave   | 0.10, 0.15, 0.20      |
| S9 (F9)     | Uniform excitations    | White noise       | 0.08                  |
| S10 (F10), S11 (F11), S12 (F12) | Uniform excitations    | Mexico City wave | 0.10, 0.15, 0.20      |
| S13 (F13)   | Uniform excitations    | White noise       | 0.08                  |

3. **Test results**

3.1 **Frequencies and damping of pile-soil-pier**

The dominant frequencies between the pile-soil-piers with sliding and fixed connections in the bridge model were compared (Figure 7), which were obtained through the peak picking procedure based on the acceleration cross-power spectra at the top and bottom of the column under white-noise excitations. The first and higher dominant frequencies of the pile-soil-pier with sliding connections were approximately 7 Hz and 14 Hz, which were far more than that (about 3.6 Hz) of the pile-soil-pier with fixed connections. This phenomenon could be an explanation that the inertial mass of the girder transferred to the pile-soil-pier increased when the sliding connection between the pile-soil-pier and girder changed to the fixed connection, decreasing the predominant frequencies of the pile-soil-pier. Accordingly, the inertial mass from the girder significantly elongated the dominant periods of the pile-soil-pier (see Table 2). Table 2 also lists the predominant frequencies identified from the stochastic subspace identification procedure (Van Overschee and De Moor 2012) based on the white noise excitations.
Figure 7. Dominant frequencies of pile-soil-piers with sliding and fixed connections

Table 2. Dominant frequencies and damping ratios of pile-soil-piers

| Connection configurations       | Dominant frequencies (Hz) | Dominant periods (s) | Damping ratios (%) |
|--------------------------------|---------------------------|----------------------|--------------------|
| Pile-soil-pier with sliding     | 7.04 (7.024)              | 0.142 (0.142)        | 6.94 (8.50)        |
| connections                     | 14.16 (14.00)             | 0.071 (0.071)        | 2.79 (2.88)        |
| Pile-soil-pier with fixed       | 3.62 (3.625)              | 0.276 (0.276)        | 4.32 (3.59)        |
| connections                     |                           |                      |                    |

Notes: Data in brackets was determined from the stochastic subspace identification method.

Additionally, the damping ratios of the pile-soil-pier were identified from the simple half-power bandwidth and the advanced stochastic subspace identification procedures (Van Overschee and De Moor 2012) based on the white noise excitations. Table 2 lists the damping ratios of the pile-soil-pier with sliding and fixed connections. The pile-soil-pier with sliding connections showed higher damping ratios compared to the pile-soil-pier with fixed connections. For instance, the damping ratio of 6.94% corresponding to the first predominant frequency of the pile-soil-pier with sliding connections decreased by 37.7% to 4.32% for the pile-soil-pier with fixed connections. This variation is because the inertial mass from the girder transferred to the pile-soil-pier increased. Therefore, the inertial mass significantly reduced the damping ratios of the first dominant frequency of the pile-soil-pier, and the damping was highly dependent on the boundary condition or inertial mass for the pile-soil-pier. Also, the identified damping ratios could be considered as references for the numerical simulation of pile-soil-structure in future work. However, extensive experimental investigations are necessary to quantify the variation tendency of the damping ratios.
Figure 8. Frequency characteristics of free-soil in Shear-box 1 for pile-soil-piers with sliding and fixed connections

Figure 8 compares the influences of the different connections on the frequency characteristics of the free-soil. The frequency contents were obtained from the power spectra density function based on the acceleration at the soil surface in Shear-box 1. Compared to the frequency contents of the free-soil for the sliding connection, the free-soil behaved new primary frequency content \( f = 3.523 \) Hz that was almost the same as that of the column (see Figure 7 (a)) for the pile-soil-pier with fixed connections. Therefore, the boundary connections from the girder significantly affected the frequency characteristics of the free-soil. The structure-soil interaction for the pile-soil-pier with fixed connections became more significant than the pile-soil-pier with sliding connections due to the inertial mass increasing. However, this paper did not show the dynamic properties of the bridge model for brevity. It has been reported by the authors (Xie et al. 2020).

3.2 Structural displacement and acceleration demand

Figures 9-11 show the comparisons of the peak displacement at the top and bottom of the column between the pile-soil-piers with sliding and fixed connections under three input motions with different amplitudes. The peak displacements at the column top of the pile-soil-pier with fixed connections under three input motions were generally more than the pile-soil-pier with sliding connections. For example, the peak displacements at the column
top of the pile-soil-pier with fixed connections were 49.2%, 54.9%, and 58.1% more than the pile-soil-pier with sliding connections in the cases with 0.10, 0.15, and 0.20 g El Centro waves, respectively. Compared to the peak displacements at the column top of the pile-soil-pier with sliding connections, the pile-soil-pier with the fixed connection grew by 278.9%, 167.0%, and 170.0% for the cases with 0.10, 0.15, and 0.20 g Mexico City waves, respectively. It is since that the inertial mass transferred to the pile-soil-piers increased, which elongated the dominant period of the pile-soil-piers (see Figure 7 (a)). Therefore, the corresponding displacement response spectra were more significant, resulting in the displacement increment. For instance, the displacement response spectrum value of 0.357 at the dominant period (0.276 s) of the pile-soil-pier with fixed connections was far more than 0.015 at the dominant period (0.071 s) of the pile-soil-pier with sliding connections for the Mexico City wave (see Figure 6 (a)).

![Figure 9. Peak displacements of the column of the pile-soil-pier with sliding and fixed connections under the synthetic wave](image)

However, there is a decrease concerning the peak displacement at the column top of the pile-soil-pier with fixed connections compared to the pile-soil-pier with sliding connections under the 0.20 g synthetic wave. It is because that the dominant period of the free-soil of the pile-soil-pier with sliding connections under the 0.20 g synthetic wave was slightly more than that under the 0.10 g artificial wave and was higher than that of the pile-soil-pier with
fixed connections under the 0.20 g synthetic wave, as shown in Figure 12. Consequently, the corresponding displacement spectra value became larger (see Figure 6 (a)), resulting in the displacement increment at the column top of the pile-soil-pier with sliding connections.

Figure 10. Peak displacements of the column of the pile-soil-pier with sliding and fixed connections under the El Centro wave

Figure 11. Peak displacements of the column of pile-soil-pier with sliding and fixed connections under the Mexico City wave
As shown in Figure 9 (b) and Figure 10 (b), the peak displacements at the column bottom of the pile-soil-pier with fixed connections were slightly less than the pile-soil-pier with sliding connections for the cases with the synthetic wave and El Centro wave. For example, compared with the peak displacements at the bottom of the column of the pile-soil-pier with sliding connections, the pile-soil-pier with fixed connections decreased by 15.9%, 5.3%, and 9.5% for the cases with 0.10, 0.15, and 0.20 g synthetic wave, respectively. The differences are due to the facts that the dominant frequencies of the pile-soil-pier with sliding connections were nearer to the primary frequency contents (8-20 Hz) of the synthetic wave (see Figure 5 (b) and Figure 7 (b)), which was more easily excited by the artificial wave. Therefore, it caused more displacement at the bottom of the column for the pile-soil-pier with sliding connections than the pile-soil-pier with fixed connections. Conversely, the displacement spectrum value of the synthetic wave of 0.124 at the dominant period (0.276 s) of the pile-soil-pier with fixed connections was more than 0.066 at the dominant period (0.142 s) of the pile-soil-pier with sliding connections (see Figure 6 (a)). As a result, the abovementioned factors cooperatively resulted in larger displacement at the column bottom of the pile-soil-pier with sliding connections.

Additionally, the peak displacements at the column bottom of the pile-soil-pier with fixed connections were more
than the pile-soil-pier with sliding connections under the Mexico City wave (Figure 11 (b)). For example, the maximum displacements of the column bottom of the pile-soil-pier with fixed connections were 95.2%, 98.9%, and 62.1% more than the pile-soil-pier with sliding connections under the 0.10 g, 0.15 g, and 0.20 g Mexico City wave, respectively. The variation is because that the inertial mass from the girder significantly elongated the predominant periods of the pile-soil-piers (Figure 7 (b)). Accordingly, the displacement spectrum value of 0.357 at the dominant period (0.276 s) of the pile-soil-pier with fixed connections was far more than 0.079 at the dominant period (0.142 s) of the pile-soil-pier with sliding connections for the Mexico City wave (Figure 6 (a)). Therefore, the variations of the peak displacement of the column induced by the inertial mass are dependent upon the primary frequency contents of the input motions and the dominant periods of the pile-soil-piers.

Figures 13-15 compare the peak accelerations along the column and pile between the pile-soil-piers with sliding and fixed connections under three input motions with different amplitudes, together with the acceleration of Table A. Note that the maximum accelerations between both cases with identical target amplitudes were normalized to eliminate the influences of the variations of the outputting accelerations between both runs on the acceleration responses. In other words, the recorded acceleration of the column divided by the target acceleration, such as 0.10 g, of Table A is the shown peak acceleration. The peak accelerations of the column of the pile-soil-pier with fixed connections were significantly less as compared to the pile-soil-pier with sliding connections under three input motions. For instance, the maximum accelerations of the column top of the pile-soil-pier with fixed connections were 72.4%, 73.8%, and 75.3% smaller than the pile-soil-pier with sliding connections in the cases with 0.10 g, 0.15 g, and 0.20 g synthetic waves, respectively. Compared with the accelerations at the column top of the pile-soil-pier with sliding connections under the 0.10 g, 0.15 g, and 0.20 g Mexico City wave, the pile-soil-pier with the fixed connection reduced by 48.7%, 43.8%, and 42.8%, respectively. It is since that the inertial mass transferred to the
pile-soil-piers increased, and its dominant period elongated (see Figure 7 (a)). Therefore, the corresponding acceleration response spectra became smaller, and the corresponding acceleration responses decreased. For example, the acceleration spectrum value of 0.077 at the dominant period (0.276 s) of the pile-soil-pier with fixed connections was far less than 0.320 at the dominant period (0.071 s) of the pile-soil-pier with sliding connections for the synthetic wave (see Figure 6 (b)). Additionally, there are the same variation trends on the peak accelerations at the column middle caused by the inertial mass. However, the inertial mass increment had insignificant influences on the peak accelerations of the pile.

Figure 13. Comparisons of peak accelerations of the column and pile between pile-soil-piers with sliding and fixed connections under the synthetic wave

Figure 14. Comparisons of peak accelerations of the column and pile between pile-soil-piers with sliding and fixed connections under the El Centro wave
3.3 Structural strain and bending moment demand

Figures 16-18 illustrate the comparisons of the maximum strain in the reinforcement bar at the column bottom and pile top between the pile-soil-piers with sliding and fixed connections under three input motions with various amplitudes. Figure 19 only plots the strain histories between the pile-soil-piers with sliding and fixed connections under three input motions with 0.20 g. Compared to the pile-soil-pier with sliding connections, the maximum strain of the column bottom and pile top of the pile-soil-pier with fixed connections significantly increased due to the inertial mass increment from the girder. For example, the maximum strain of the column bottom of the pile-soil-pier with fixed connections was 2.8-3.4 times that of the pile-soil-pier with sliding connections under 0.10 g, 0.15 g, and 0.20 g El Centro waves. The maximum strain of the column bottom of the pile-soil-pier with fixed connections was 5.6-7.1 times than the pile-soil-pier with sliding connections under Mexico City wave with 0.10 g, 0.15 g, and 0.20 g. Under the El Centro wave with 0.10 g, 0.15 g, and 0.20 g, the peak strains at the pile top of the pile-soil-pier with fixed connections were more 36.8%, 56.4%, and 54.2%, respectively, than the pile-soil-pier with sliding connections. The peak strains at the pile top of the pile-soil-pier with fixed connections grew by 83.3%, 46.3%, and 16.8%, respectively, compared to the pile-soil-pier with sliding connections under Mexico City wave with 0.10 g, 0.15 g, and 0.20 g. It is because that the pile-soil-pier shared more inertial mass from the girder, significantly increasing the inertial force, such as the axial force and bending moment,
of the pile-soil-piers. Therefore, more inertial mass from the girder is unfavorable effects on the seismic behavior of the pile-soil-piers at the microscopic level. It is worth noting that the maximum strains in the reinforcement bar of the pile-soil-piers are still below the yield strain (2860 microstrain) for all cases.

Figure 16. Comparisons of reinforcement strain between pile-soil-piers with sliding and fixed connections under the synthetic wave

Figure 17. Comparisons of reinforcement strain between pile-soil-piers with sliding and fixed connections under the El Centro wave
Figure 18. Comparisons of reinforcement strain between pile-soil-piers with sliding and fixed connections under the
Mexico City wave

Figure 19. Reinforcement strain histories of pile-soil-piers with sliding and fixed connections under three input motions
with 0.20 g

Figures 20-22 show the peak bending moment at the column bottom and pile top of the pile-soil-piers with sliding and
fixed connections under three input motions with various amplitudes. Figure 23 indicatively displays the bending moment
histories of the pile-soil-piers with sliding and fixed connections under three input motions with 0.2 g. The bending
moment was determined from the recorded strain in the reinforcement bars of the column and pile, as well as the moment-
curvature-strain relationship deduced from the section analyses [38]. Compared to the pile-soil-pier with sliding
connections, the peak bending moment at the column bottom and pile top of the pile-soil-pier with fixed connections
significantly enlarged because of the inertial mass increment from the girder. For example, the peak bending moment at the column bottom of the pile-soil-pier with fixed connections was 2.8-3.3 and 5.3-6.4 times that of the pile-soil-pier with sliding connections for the El Centro and Mexico City waves, respectively. The peak bending moment of the pile top of the pile-soil-pier with fixed connections were more 23.1%, 58.1%, and 59.4%, respectively, than the pile-soil-pier with sliding connections under the El Centro wave with 0.10 g, 0.15 g, and 0.20 g. The corresponding peak bending moment of the pile top increased by 39.2%, 21.8%, and 5.2%, respectively, compared with the pile-soil-pier with sliding connections under the Mexico City wave with 0.10 g, 0.15 g, and 0.20 g. It is because that more inertial mass from the girder significantly increased the inertial force of the pile-soil-piers. Consequently, more inertial mass from the girder adversely affects the macroscopic seismic behavior of the pile-soil-piers.

Figure 20. Comparisons of bending moment demands between pile-soil-piers with sliding and fixed connections under the synthetic wave
Figure 21. Comparisons of bending moment demands between pile-soil-piers with sliding and fixed connections under the El Centro wave.

Figure 22. Comparisons of bending moment demands between pile-soil-piers with sliding and fixed connections under the Mexico City wave.

Figure 23. Comparisons of bending moment demands between pile-soil-piers with sliding and fixed connections under input motions with 0.20 g.

3.4 Kinematic and inertial effects

The peak accelerations of the free-soil and Table A recorded in the shaking tests with both connection configurations between the pile-soil-piers and girder are depicted in Figure 24 for three input motions excitations with various shaking...
amplitudes. Note that the peak accelerations between both connection configurations with identical shaking amplitudes were normalized to eliminate the impacts of the outputting acceleration variations between both cases on the acceleration response. In other words, the recorded acceleration of the free-soil divided by the target acceleration, such as 0.10 g, of Table A is the shown peak acceleration. Additionally, the accelerometer positions instrumented in the free-soil (Figure 24 (a)) were far enough from piles so that the peak accelerations of the free-soil could be unaffected by them. The peak accelerations of the free-soil almost did not change for the different connection configurations. Thus, it should generate the same kinematic interaction effects on the pile. This phenomenon was the same as the observation in the published paper (Durante et al. 2016). Consequently, the observed variations in the seismic responses of the pile should be associated only with the inertial effect caused by the inertial mass of the girder because the connection configurations between the pile-soil-pier and girder changed.
As seen in Figures 16 (b)-18 (b) again, the fixed connection configuration between the pile-soil-pier and girder significantly amplified the inertial effects on the piles when compared to the sliding connection configuration. It is since that the inertial mass transferred to the pile-soil-piers noticeably increased when the connection configurations between the pile-soil-pier and girder changed from the sliding connection to the fixed connection. Compared to the sliding connection configuration, the inertial effects on the piles for the fixed connection configuration increased by 61.5%, 66.7%, and 34.6%, respectively, for the cases with the synthetic wave with 0.10, 0.15, and 0.20 g. The inertial effects on the piles induced by the fixed connection configuration separately grew by 36.8%, 56.4%, and 54.2% compared to the sliding connection configuration under the El Centro wave with 0.10, 0.15, and 0.20 g. In the cases with the Mexico City wave with 0.10, 0.15, and 0.20 g, the inertial effects on the piles for the fixed connection configuration enhanced 83.3%, 46.3%, and 16.8%, respectively, compared to the sliding connection configuration. Consequently, the more inertial mass of the girder produced more inertial effects on the piles of the pile-soil-piers. Similarly, the bending moment increment at the pile top provided evidence (Figure 20 (b)-Figure 22 (b)) to the observation results that the fixed connection configuration between the pile-soil-pier and girder significantly magnified the inertial effects on the piles.

3.5 Pile group effects

The pile group effects of the pile-soil-piers were estimated by comparing the peak reinforcement strains at the top between the corner, side, and center piles under three input motions with different shaking amplitudes, as shown in Figure 25-27. The reinforcement strains of the side pile were the largest, following by the center pile. The corner pile showed the smallest strains. For instance, the most reinforcement strains of the side pile were 65.6, 42.0, and 57.0 με, respectively, for the pile-soil-pier with sliding connections under the synthetic wave, El Centro wave, and Mexico City wave with 0.15
g. The lightest reinforcement strains of the corner pile were 22.4, 14.9, and 19.5 μɛ, respectively, for the pile-soil-pier with sliding connections under the synthetic wave, El Centro wave, and Mexico City wave with 0.15 g. Therefore, there were noticeable variations in the reinforced strain between the corner, side, and center piles, regardless of which in the sliding connection or fixed connections between the pile-soil-piers and girder. The reason is due to the rotation of the cap under earthquake excitations. It indicated the significant pile group effects on the pile group, which was the same as the observation in this published paper (Liu et al. 2017).

Figure 25. Pile group effects of pile-soil-piers with sliding and fixed connections under the synthetic wave

Figure 26. Pile group effects of pile-soil-piers with sliding and fixed connections under the El Centro wave

Additionally, there are more significant differences in the peak strains between the corner, side, and center piles of the pile-soil-piers with the fixed connection configuration than the sliding connection configuration (see Figures 25-27). For instance, the reinforcement strains of the side pile were more approximately 50% than the center pile.
for the sliding connection between the pile-soil-pier and girder under the synthetic wave. The reinforcement strains
of the side pile of the pile-soil-pier with fixed connections increased by about 80% compared to the center pile.
Consequently, the tested 3×3 pile group in the fixed connection configuration showed more significant pile group
effects. It is due to more inertial mass from the girder transferred to the pile-soil-pier. For other cases with the El
Centro and Mexico City waves with different amplitudes, we could observe similar trends of the peak strain
increments in the fixed connection compared to the sliding connection, demonstrating that the pile group effects in
the dynamic responses of piles are independent of the frequency contents of input motions. Therefore, the fixed
connection configuration significantly amplified the pile group effects due to the inertial mass increment from the
girder when the connection configurations between the pile-soil-pier and girder changed from the sliding connection
to the fixed connection.

Figure 27. Pile group effects of pile-soil-piers with sliding and fixed connections under the Mexico City wave

4. Simulation results

4.1 Overview of numerical model

A numerical model of the bridge model, including the pile-soil-piers with sliding and fixed connections, was built
to reproduce the shaking table results, as shown in Figure 28 (a). The tower, girder, and cable were simulated by the
fiber, elastic, and truss elements, respectively (Taucer et al. 1991). The detailed information was introduced by this
literature (Mazzoni et al. 2006, Xie and Sun 2019). Herein, the following paragraphs mainly introduced the
numerical model of the pile-soil-pier.

For the pile-soil-pier, the column and pile were modeled using the fiber element. The lateral beam was resembled
using one elastic element, the rotation degrees of which were released to behave the hinge connection. The shear
beam was simulated by one fiber element considering shear deformation. It connected to the columns by rigid links,
as shown in Figure 28 (b). The nonlinear zero-length p-y spring was used to model the pile-soil interactions, which
could represent the plastic, gap, and closure components of the pile-soil interactions and the radiation damping
effects of the mixed soil (Boulanger et al. 1999). The spring parameters were determined from the procedure (Xie
and Sun 2019). For example, the ultimate strength of the p-y spring at the pile head (0.95 m-height) and the pile
base (0.05 m-height) were 140 N and 14762 N, respectively. The deformation corresponding to 0.5 times of the
ultimate strength were 4.844e-4 m and 5.126e-2 m, respectively. The rigid link was adopted to model the cap, which
connected the column bottom to the pile top (Figure 28 (b)-(c)). The zero-length element was used to resemble the
sliding and fixed connections between the pile-soil-pier and the girder. For the sliding connections between the pile-
soil-pier and girder, the updated longitudinal, transverse, and vertical stiffnesses of the zero-length element were 90
kN/m, 100000 kN/m, and 650 kN/m, respectively. The corresponding parameters were 100000 kN/m, 100000 kN/m,
and 980 kN/m for the fixed connection between the pile-soil-pier and girder. The base of the pile group was fixedly
connected to the shear box to resemble the fixed scenario.

The section of the column and pile was discretized into hundreds of fibers based on the concrete and reinforcement
bar (Figure 28 (d)). Concrete04 in OpenSees was adopted to represent the mechanical behavior of the unconfined
and confined concrete (Figure 28 (e)). The maximum compressive strength, \( f_c \), and corresponding strain, \( \epsilon_c \), of
the unconfined concrete of the column were 19.02 MPa and 0.002, respectively. The maximum compressive strength,
$f_{cc}$, peak strain at maximum strength, $\varepsilon_{cc}$, ultimate strain at the crushing strength, $\varepsilon_{cu}$, of the confined concrete were 28.53 MPa, 0.007, and 0.016, respectively, which were determined from the model proposed by Chang and Mander (Chang and Mander 1994). The corresponding elastic modulus, $E_{c}$, from the sample test was $0.867 \times 10^4$ MPa. Steel02 in OpenSees was used to model the reinforcement bar of the column and pile. The yield strength of the reinforcement bar for the column and pile was 461.3 MPa and 566.3 MPa, respectively. The corresponding elastic modulus of the reinforcement bar was $1.06 \times 10^5$ Mpa and $1.75 \times 10^5$ Mpa.

Figure 28. Numerical model of the bridge model including the pile-soil-piers

The shear beam was modeled using the unconventional fiber section with the shear deformation. The specific fiber section was easily achieved by the section Aggregator command in OpenSees. In other words, it combined the conventional fiber section with the material model representing the relation between the shear force and shear angle. Steel02 model was used to resemble the relation between the shear force and shear angle. The yielding shear force and shear angle obtained from the equations (AISC 2005) were 3780 N and $1.5 \times 10^{-3}$ rad, respectively. Other parameters adopted the proposed values from OpenSees.

The uniform excitation was longitudinally imposed on the bridge model for the shaking table test. Therefore, the
outputting acceleration of Table B was taken as an input to apply to the pile tips of the numerical model (Figure 28 (a)). The seismic response of the bridge model was computed using the Newmark method. However, the static load of the bridge model was analyzed before the earthquake load was imposed.

4.2 Comparisons between experimental and numerical responses

Figures 29-30 indicatively show the acceleration histories between the experiment and simulation of the pile-soil-pier with sliding and fixed connections for the bridge model under the El Centro wave with 0.10 g. The duration of the input motion is shorter than that of the experiment and simulation. It is since the data acquisition needs to capture the output information of shaking tables.

As shown in Figure 29, the numerical peak acceleration at the column top of the pile-soil-pier with fixed connections was less than the pile-soil-pier with sliding connections. The variation trend was similar to the experimental results. Therefore, the numerical model could effectively capture the peak acceleration variation caused by the connection configuration change compared to the result results.

The numerical peak accelerations of the pile-soil-piers with sliding and fixed connections were approximately consistent with the experimental counterparts. For instance, the difference of the peak acceleration between the experiment and simulation was 1.8% at the top of the column for the pile-soil-pier with sliding connections and was
1.6% at the bottom of the column of the pile-soil-pier with fixed connections under the El Centro wave with 0.10 g. The numerical peak acceleration at the pile top of the pile-soil-piers with sliding connections was 16.0% less than the experimental results under the El Centro wave with 0.10 g. The numerical waveforms of the pile-soil-piers with sliding and fixed connections approximately matched well with the test responses.

![Experiment vs Simulation Graph](image)

(a) Pile-soil-pier with sliding connections
(b) Pile-soil-pier with fixed connections

Figure 30. Acceleration responses at the pile top between the experiment and simulation of the pile-soil-pier with sliding and fixed connections for the bridge model under the 0.10 g El Centro wave.

Moreover, the numerical displacement history at the column top of the pile-soil-pier with fixed connections matched well with the experimental counterparts except for the pile-soil-pier with sliding connections under the 0.10 g El Centro wave, as depicted in Figure 31. The numerical model could capture the peak displacement variations induced by the connection change. The numerical displacement at the column top of the pile-soil-pier with fixed connections was larger than the pile-soil-pier with sliding connections. The variation trend was similar to the experimental results.

The difference of the peak displacement at the column top between the experiment and simulation was 13.5% for the pile-soil-pier with fixed connections, whereas the variation was 69.6% for the pile-soil-pier with sliding connections under the 0.10 g El Centro wave.
Consequently, the numerical model could approximately reproduce the experimental results and effectively track the displacement variations at the column top induced by the connection change. The numerical model further examined the effects of the boundary connections on seismic responses of the pile-soil-pier. However, the numerical reinforcement strains of the column and pile are not shown in this paper. The numerical model is difficult to describe the experimental reinforcement strain at the microscopic level.

5. Conclusions

The pile-soil-piers in the bridge model were studied using shaking table tests and numerical analyses to estimate the impacts of the inertial mass from the girder on the dynamic characteristics, dynamic responses, and SSI effects. Both connection configurations between the pile-soil-pier and girder, including the sliding and fixed connections, were considered in the cable-stayed bridge model under three representative input waves with different shaking amplitudes. The typical seismic responses and dynamic properties of the pile-soil-piers between the sliding and fixed connections were compared. This paper also explored the influences of the inertial mass of the pile-soil-piers on the inertial, kinematic, and pile group effects. Finally, a numerical model was developed in OpenSees to reproduce the experimental responses. The key finds are summarized as follows.

1. The strains in the reinforcement bars at the column bottom of the pile-soil-pier with fixed connections were
averagely 3.0, 5.0, and 6.4 times than the pile-soil-pier with sliding connections under the synthetic, El Centro, and Mexico City waves, respectively, due to more inertial mass from the girder transferred to the pile-soil-piers. The corresponding strains at the pile top increased averagely approximately 50% compared to the pile-soil-pier with sliding connections under three input motions. Similarly, the pile-soil-pier with fixed connections enlarged the bending moment at the column bottom and pile top compared to the pile-soil-pier with sliding connections because the inertial mass from the girder increased.

(2) The peak displacement at the column top of the pile-soil-pier with fixed connections averagely increased 20%, 54%, and 205%, respectively, compared to the pile-soil-pier with sliding connections under the synthetic, El Centro, and Mexico City waves. The corresponding peak acceleration averagely decreased by 75%, 69%, and 50%, respectively, compared to the pile-soil-pier with sliding connections under the synthetic, El Centro, and Mexico City waves. It is because that the displacement response spectra were larger, whereas the acceleration response spectra became smaller when the connection configurations between the pile-soil-pier and girder changed from the sliding connection to the fixed connection. However, there are insignificant influences on the acceleration of the pile and free-soil.

(3) The inertial mass from the girder significantly amplified the inertial effects on the piles according to the peak strains at the pile top of the pile-soil-pier. However, there are insignificant influences on the kinematic effects on the pile-soil-pier. The pile group effects were observed in the sliding and fixed connections between the pile-soil-pier and girder. The inertial mass from the girder significantly enlarged the pile group effects compared to the sliding connection configuration between the pile-soil-pier and girder.

(4) Compared to the sliding connection between the pile-soil-pier and girder, the predominant frequency of the pile-soil-pier with fixed connections significantly decreased due to the inertial mass from the girder increasing.
The corresponding dominant period of the pile-soil-pier was noticeably elongated. The inertial mass significantly reduced the damping ratios of the first dominant frequency of the pile-soil-pier. Therefore, the predominant frequency and damping were highly dependent on the boundary condition or inertial mass for the pile-soil-pier.

(5) The numerical model could approximately reproduce the representative experimental responses and capture the typical response changes caused by the connection configuration change. Therefore, the effects of the boundary connections or inertial mass on seismic responses of the pile-soil-pier were further validated by the numerical model.

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Declarations

Conflicts of interest: The authors declare that there is no conflict of interest.

Availability of data and material: Data will be made available on request.

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