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Effect of soil–structure interaction on a masonry structure under train-induced vibrations

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This paper presents an experimental investigation to assess the influence of soil–structure interaction (SSI) on an unreinforced masonry structure under train-induced vibrations. For this purpose, a structure near a railway line was instrumented and monitored when subjected to five railway traffic vibrations. The measured vibrations were used to estimate the modal properties using a frequency domain decomposition technique. The variation in estimated modal properties against varying excitations indicated that the dynamic response depends on the source of excitation. Two finite-element (FE) models were also developed and updated through manual tuning, one with a fixed base and the other with a flexible base accounting for SSI. The modal properties and response time histories measured through experimentation were compared to those predicted by detailed three-dimensional FE models. A comparison between the base input force, base moment and peak displacement of both FE models was also performed. The results indicated that the effect of SSI on the fundamental mode shape and in the prediction of accurate response time histories of the investigated structure was significant. However, the effect on modal frequencies, base input force, base moment and peak displacement of the investigated structure under train-induced vibrations was observed to be very low.

Notation

A	base area
a_m	measured acceleration
a_s	simulated acceleration
B	width
D	depth
G	shear modulus
I	moment of inertia
L	length
N	counts
t	time
V_s	shear wave velocity
ρ	density of soil
Φ_i, Φ_j	i th and j th eigen vectors

1. Introduction

The dynamic analysis of civil engineering structures is generally carried out without considering soil–structure interaction (SSI). The analysis is generally carried out by considering a rigid restraint at the bottom of the structure and neglecting the interaction of soil and foundation. It is now a known fact that kinematic and inertial effects associated with SSI modify the dynamic behaviour of the structure (Behnamfar and

Banizadeh, 2016; Medina *et al.*, 2013). The soil interacts with the structure, deforms and induces its motion to the foundation and the structure above it. The motion of the foundation causes inertial effects in the superstructure that are transmitted back to the foundation and the soil below it. The transfer of inertial forces to the underlying soil creates new waves originating from the soil foundation interface, adding in the previous ones and causing amplification in seismic response (Pecker and Chatzigogos, 2010). These phenomena occur simultaneously and are closely dependent on each other. Moreover, in soft soils, refraction phenomena are observed in which seismic waves are refracted by soil particles and some of them are consumed as they approach the structure foundation, causing a decrease in their intensity. The effect of SSI has been observed experimentally as well as analytically by numerous researchers (Çelebi *et al.*, 2016; Ghaemmaghami *et al.*, 2017; Pitilakis *et al.*, 2018a; Şafak, 1995; Star *et al.*, 2015; Stewart and Fenves, 1998), in which they concluded that soil not only modified the global dynamic behaviour in terms of decreasing modal frequencies and modification in deformation pattern, but also increased the base shear and base moments. Celebi and Şafak (1991, 1992) performed system identification on buildings considering SSI without free field accelerographs and observed rocking type SSI. Lancellotta and Sabia (2015)

investigated the effect of SSI on operational modal properties of a Ghirlandina tower through system identification and numerical modelling and concluded that soil not only increased the time period of the structure but also changed its deformation pattern. Ceroni *et al.* (2014) instrumented the masonry palace in Italy to evaluate the effect of SSI on the natural frequencies of the structure. The natural frequencies were estimated experimentally using cross power spectral density. de Silva *et al.* (2018) investigated the effect of SSI and adjacent buildings on the global dynamic behaviour of a historic Carmine masonry bell tower, using experimental and numerical techniques. The authors concluded that including the effect of adjacent buildings and SSI in finite-element (FE) modelling leads to accurate results. Pitilakis *et al.* (2018b) investigated the influence of SSI on the response of a three-storey masonry structure through shaking table tests. They concluded that it not only changed the modal parameters but also increased displacement and seismic loads. Penna *et al.* (2016) and Marques and Lourenço (2011) discussed the issues related to the numerical simulation of masonry structural components for accurate seismic response and performance assessment.

To investigate a performance of structures in real time, a structure is instrumented with a network of sensors at certain locations to record data, which in turn are used for system identification. Through system identification, modal properties (frequencies, mode shapes, damping ratios, mass participation ratio) are identified. These modal properties are the inherent properties of the structure that represent its global dynamic behaviour. They also vary with the amplitude of excitation as investigated by Ceravolo *et al.* (2017), Satake and Yokota (1996) and Schwer (2007). Moreover, such experiments provide a good opportunity for structural health monitoring, damage detection, FE model calibration and updating and seismic assessment (Butt and Omenzetter, 2012; Cheng *et al.*, 2017, 2018a). Niu *et al.* (2012) instrumented the Canton tower in China with an SHM system for 24 h to obtain wind, acceleration and temperature data to assess its modal properties. Çelebi *et al.* (2014) investigated the rocking behaviour of an instrumented building at Massachusetts Institute of Technology (MIT), USA from ambient shaking data. Zhou *et al.* (2013) performed seismic assessment of a six-storey masonry building, damaged during Wechuan earthquake, which was instrumented for 5 months to detect damage from data acquired through aftershock excitations.

In civil engineering, the FE method is used as a powerful tool for design and analysis of structures. It is extensively used for static and dynamic analysis. The linear and non-linear behaviour of a structure can be simulated with great accuracy once the FE model is properly calibrated (Mortezaei and Motaghi, 2016). The FE method also finds applications in structural control and health monitoring studies in which FE models are first updated to predict the actual response of the structure and then further used for damage detection and assessment of serviceability limit

states (see Cheng *et al.*, 2018b; Wang *et al.*, 2010; Weng *et al.*, 2009; Yousefianmoghadam *et al.*, 2017). Vibration-based FE model updating improves the accuracy of FE models by removing the discrepancies between the experimental and numerical results. Once the model is updated, it can then be used for performance assessment of a structure at a service state. In this technique, the modal parameters of the FE model are matched with those obtained from experimental results.

Generally, the effect of SSI is discussed with reference to earthquake excitations, which are different from train-induced excitations in terms of amplitude and frequency content. Therefore, it is important to investigate the dynamic response of a structure under train-induced excitation along with the effect of SSI to study the consequent modification in this response. The present study is aimed at investigating the effect of SSI on the dynamic response of a single-storey braced masonry structure subjected to train-induced vibrations. For this purpose, the structure subjected to train-induced vibrations was monitored for 12 h. The monitoring system consisted of a total of five triaxial accelerometers, four installed in the building and one free field. From the measured vibrations, the modal properties (mode shapes and frequencies) of the structure were estimated through the frequency domain decomposition (FDD) technique. Two FE models were also developed, one with rigid constraints at the base and the other with translational and rotational dynamic soil springs. These FE models were updated using manual tuning of structural geometry, material properties and boundary conditions based on engineering judgement. After updating had been carried out, the effect of SSI was investigated by comparing the results of measured and numerical modal analysis and also through comparison of experimental and analytical response time histories obtained at the roof sensor location. A comparison between base input forces, base moments and peak displacements of a fixed and flexible base structure was also performed.

2. Experimentation set-up

A real-time vibration-based monitoring was performed on a structure subjected to railway traffic vibrations to investigate its dynamic behaviour. The experiment was conducted on the second week of October 2017 and the response data over 12 h were measured. After the completion of data collection, the recorded signals were processed and chunks of railway traffic-induced vibrations were extracted from the relevant files.

2.1 Structure description

The structure under investigation is a railway signal box/signal cabin constructed in 2001 (Figure 1), and currently in use by Pakistan Railways for signal control. It is situated near Sihala railway crossing almost 20 km from Islamabad, Pakistan. The cabin, 5.18 m × 4.26 m in size, is a rectangular unreinforced masonry structure in which the first floor is a flexible diaphragm consisting of wooden logs placed on five equally spaced iron girders and the roof made of reinforced concrete

(a)

(b)

Figure 1. (a) front view and (b) rear view of east cabin Sihala, Islamabad, Pakistan

slab serves a rigid diaphragm. The thickness of the ground storey and top storey masonry walls are 457 mm and 343 mm, respectively. A reinforced concrete beam ring is also present above the windows to develop a box-like behaviour. The shade is also embedded in the reinforced ring beam. The building is rectangular in plane; two walls facing the railway track are symmetrical and the remaining two opposite walls are unsymmetrical and differ in structural stiffness because of the presence of door openings on one of the walls.

The ground floor consists of a 900 mm × 2100 mm door, six ventilators of dimensions 900 mm × 610 mm, an iron girder 300 mm deep supporting the levers, which itself is placed on two 125 mm deep iron girders. The first floor is a wooden floor placed on five equally spaced iron girders acting as a flexible diaphragm. There are six windows of dimensions 1800 mm × 1500 mm, a door of 900 mm × 2100 mm, a concrete tie beam 343 mm × 300 mm with a cantilevered slab 900 mm wide and 75 mm thick, in a first floor. A reinforced cement concrete slab 125 mm thick in a second storey acts as a rigid diaphragm. The visual inspection of the building

showed no cracks, which indicates that it remained undamaged by any severe events such as earthquakes and strong winds.

2.2 Structure's selection criteria

The reason for selection of a structure close to the rail track was to make sure high-intensity ground motions hit the structure and vibrate it sufficiently in all three directions so that a significant SSI effect could be observed.

The selected cabin is 10 m away from the railway track. The average speed of trains passing near the cabin (Figure 2) ranges from 60 to 80 km/h. The weight of each train, including engine and cabins, ranges from 400 to 500 t. The varying speed of trains together with different weights produced ground accelerations of different magnitude.

2.3 Instrumentation

The dynamic monitoring system consisted of five triaxial low-noise accelerometers with microelectromechanical systems technology, a reading capacity of $\pm 2g$ and a sampling rate of 256 Hz. Each acceleration–time history data file consisted of 5 min and the data are recorded for 12 h. Four sensors were instrumented in the structure to measure accelerations in all three orthogonal directions. Two accelerometers (A3 and A4) were fixed at the base and two (A1 and A2) at the soffit of the top slab. The accelerometers at the base were fixed on the structure below the natural surface level. The fifth accelerometer (A5) was instrumented on a concrete pad 38.4 m away from the structure to account for SSI effects. The instrumentation of the building is shown in Figure 3 and the sensor position layout is given in Table 1. In Figure 3, the notations X , Y and Z represent the global axes of the system, the notation 1, 2 and 3 represent the local axes of each sensor and the direction of the arrows represents the positive direction of wave propagation. In this case, the local axes of the sensors are same as the global axes.

2.4 Data collection

A total of five trains passed during the span of 12 h. The names of the trains, together with their time, duration and maximum recorded peak ground acceleration (PGA), are presented in Table 2. The duration of the trains lay in the range of 25.71 to 31.13 s. From the recorded data, the chunks of train signals were extracted and de-trended. The maximum PGA recorded at the base of the structure in the X -direction was 0.0786g, in the Y -direction was 0.0404g and in the Z -direction was 0.0692g. The minimum PGA recorded in the X -direction was 0.03066g, in the Y -direction was 0.024g and in the Z -direction was 0.0327g. The DN nomenclature represents trains moving in a southwards direction and the UP nomenclature means trains were moving in a northwards direction.

2.5 Vibration based modal parameter estimation

To estimate the modal parameters of the structure under the recorded railway traffic vibrations, the frequency domain

decomposition (FDD) technique was applied. The FDD technique is an enhancement of the classical frequency domain technique in which the limitations of this technique are

Figure 2. Train passing by an instrumented masonry cabin

Figure 3. A three-dimensional schematic diagram of the investigated structure

Table 1. Sensor layout

Position	Measurement direction: cm		
	X	Y	Z
A1	171.08	4.5	640
A2	171.08	428	640
A3	171	4	0
A4	171	430	0
A5	0	−384	0

Table 2. List of recorded train vibrations

Train no.	Time	Duration: s	Name	PGA X-direc.	PGA Y-direc.	PGA Z-direc.
T1	9:22	27.69	Jaffer Exp (40DN)	0.07396	0.0488	0.0692
T2	11:31	27.66	Subak Raftar (101UP)	0.0424	0.03515	0.0389
T3	14:00	31.13	Jaffer Exp (39UP)	0.03066	0.024105	0.0327
T4	17:28	25.71	Hazara Exp (12DN)	0.0699	0.0404	0.0566
T5	18:26	26.07	Islamabad Exp (108DN)	0.0786	0.0388	0.0664

removed while keeping its advantages. In this technique, the modes are estimated using a singular value decomposition (SVD) of a spectral matrix, each corresponding to an individual single-degree-of-freedom system. The maximum values of SVD relate to the modal frequencies and the corresponding singular vectors relate to the mode shapes. The closely spaced modes are easily identified accurately using this technique in case of a lightly damped structure and orthogonal mode shapes. Even if the conditions are not met, the results are still significantly more accurate than the classical approach. More details about this technique can be found from Brincker and Ventura (2015) and Brincker *et al.* (2001).

In the present study, the accelerations measured at roof level by sensors A1 and A2 were used as inputs to estimate the first three modal parameters of the structure using the FDD technique. Before applying this technique, the lag between the two signals was removed and then they were passed through a Butterworth low-pass filter with a cut-off frequency of 25 Hz.

To estimate accurate modal parameters, a FE model of the structure under investigation was developed as a reference. The details of the FE model are presented in Section 3. After modelling, the modal analysis was carried out and the first three modal frequencies obtained were 6.1, 6.34 and 9.84 Hz.

2.6 Results and discussions

Figure 4 presents the SVDs of the spectral densities for both sensors in all the three directions for a train-induced vibration, T5. The SVDs for sensor 1 are presented in black and for sensor 2 are presented in grey. The SVDs of spectral densities in X-, Y- and Z-directions are presented with solid lines, dashed lines and dotted lines, respectively. The first three estimated modal frequencies of the investigated structure as marked by vertical dashed lines are 5.375, 5.625 and 8.125 Hz, respectively.

The first three natural frequencies obtained through FDD for the five different vibrations are shown in Table 3. The first mode for T2 vibration was not clearly identifiable, and therefore is not presented in the results. The average first three modal frequencies of the structure are 5.344, 5.675 and 8.225 Hz, as presented in Table 3. The first three mode shapes of the structure are presented in Figure 5, in a planar view. The solid black lines indicate the undeformed shape and

Figure 4. Singular values of the power spectral density (PSD) matrices obtained from the FDD method

Table 3. Measured natural frequencies of a structure

Mode	Frequency: Hz	T1	T2	T3	T4	T5	Mean
First mode	F1	5.25	—	5.375	5.375	5.375	5.344
Second mode	F2	5.88	5.75	5.625	5.5	5.625	5.675
Third mode	F3	8.25	8.375	8	8.375	8.125	8.225

Figure 5. First, second and third mode shapes of the structure in planar view

the grey dashed lines indicate the deformed shape of the structure. It is clear from Figure 5 that all identified modes were torsional modes. The mode 1 was translationally dominant in the *X*-direction coupled with torsion; mode 2 was translationally dominant in the *Y*-direction coupled with minor torsion; and mode 3 was a pure torsional mode. The reason for all the torsional modes could be the stiffness irregularity between the two walls along the *X*-direction, as one wall contains two door openings, making it more flexible than the other wall.

The correlation between mode shapes for all the three modes of vibration was also calculated using modal assurance criteria (MAC). The MAC between two modes Φ_i and Φ_j can be defined as

$$1. \quad \text{MAC} = \frac{(\Phi_i^T \Phi_j)^2}{(\Phi_i^T \Phi_i)(\Phi_j^T \Phi_j)}$$

The mode shapes correlated through Equation 1 are presented in Figure 6 for all the three modes separately. The MAC for the first mode varied from 78 to 91.67% except between T1 and T3 and T1 and T4, which are 59.6% and 48.7%, respectively. The MAC for the second mode showed good correlation as the first mode as it varied from 75.98 to 99.24% respectively. The MAC for the third mode showed better correlation than the first two modes and varied from 73.41 to 99.81%, respectively.

The first two identified modes are not orthogonal and are also very closely spaced (as visible from Figures 4 and 5); in such cases the bias in mode shape of the weaker mode is stronger, resulting in inaccuracies (Brincker *et al.*, 2001). In the current case, the weaker mode is the first mode of the structure, as clearly visible from Figure 4, which may be the reason for the first mode not being clearly identifiable in the case of T2 vibration. This also explains the relatively greater difference in MAC values of the first mode for all the five vibrations. Little variation in identified modal frequencies for each mode against varying excitations may be attributed to the fact that modal parameters vary with varying excitations (Butt and Omenzetter, 2014; Ceravolo *et al.*, 2017). The results of modal damping ratios are not included in the paper as they are not reliable. The modal damping ratios investigated in the frequency domain are overestimated, the reason being spectral leakage of a digital signal (Brandt, 2011). Spectral leakage causes the modal peaks of the spectral density function to become wider. As each modal peak is proportional to damping, the damping will be increased. For this reason, the results are not included.

3. Finite-element modelling

To evaluate the effect of SSI, a three-dimensional finite element was developed using site measurements and inspections. A macro modelling technique was employed for modelling of the masonry walls, which requires less time and computational effort (Laurenco *et al.*, 1995; Lourenço, 2008; Penna *et al.*,

2014). In this technique, there is no distinction between mortar joints and brick units and a homogeneous isotropic material represents the masonry assemblage. The FE modelling was carried using the CSI SAP2000 software package to simulate the dynamic response of the investigated structure.

The unreinforced masonry walls were modelled as four-noded thin shell elements representing a homogeneous isotropic material (Ceroni *et al.*, 2014). The slab was also modelled as a four-noded thin shell element. Reinforced concrete lintels, tie beam and iron girders were modelled as two-noded frame elements. The wooden logs were not modeled; however, their tributary load was distributed on the adjacent girders. The weight of the levers/gears was also applied as a uniform distributed load on the steel girder. In initial modelling, rigid restraint was applied at the base of the structure.

The mechanical properties of masonry were taken from the literature. The reason for not performing non-destructive tests to estimate the mechanical properties of the structure was that Pakistan Railways did not allow any sort of non-destructive/destructive tests to be performed. The criteria used for the selection of the mechanical properties was to search for values which were obtained through tests on masonry typically available for construction in the subcontinent region in order to minimise the difference in mechanical properties as much as possible. Keeping to this criterion, the mechanical properties were taken from Chourasia *et al.* (2016) and Shahzada *et al.* (2012) in which testing on masonry prisms was performed to identify the mechanical properties using American Society for Testing and Materials (ASTM) procedures. Following the literature and selection criteria, the values used for the initial FE modelling are presented in Table 4. The Poisson ratio, Young's modulus and compressive strength values were taken from Chourasia *et al.* (2016) and the density was taken from Shahzada *et al.* (2012). The FE model developed using the mechanical properties provided in Table 4 and geometric properties stated in the paragraph above served as the baseline model with the aim of providing an idea regarding the dynamic behaviour of the FE model.

After modelling, modal analysis was carried out to investigate the modal frequencies and mode shapes. The results of experimental and numerical modal analyses are presented later in Table 8 for a modal frequencies comparison and Table 9 for a mode shape comparison. The results showed a huge difference

Figure 6. Modal assurance criteria (%) for various train-excited vibrations: (a) first mode; (b) second mode; (c) third mode

Table 4. Elastic mechanical properties of various materials

Mechanical property	Masonry	Concrete	Reinforcement	Iron girder
Density: kg/m ³	1494	2400	7850	7850
Poisson ratio	0.26	0.2	0.3	0.3
Young's modulus: MPa	1560	21 525	1.99×10^5	1.99×10^5
Tensile strength: MPa	—	21	413	248
Compressive strength: MPa	3.32	21	—	—

between experimental and numerical results. The differences in modal frequencies were calculated to be 31.4, 20.91 and 43.3% and the MAC were calculated to be 12.34, 17.49 and 73.4% for the first three modes. This huge difference between experimental and numerical results necessitated the updating of FE model. The FE model updating procedure followed in this paper is given below.

4. Finite-element model updating

The dissimilarity in the modal properties obtained from initial FE model and FDD showed that the dynamic behaviour of the FE model did not coincide appreciably with that observed through the experimental work. The reason for discrepancies in the FE model results was the difference in actual and assumed material properties, boundary conditions and structural geometry used for modelling. To minimise the differences between experimental and numerical results, FE model updating techniques are used. In this paper, a manual FE model updating technique was used which involved manual changes in the structural geometry, material properties and boundary conditions, based on engineering judgement to minimise the difference between the measured and estimated modal properties.

The subject FE model was updated by changing the masonry mechanical properties, mass source and boundary conditions and by adjusting the mesh size. The modal frequencies identified through FDD for all the five excitations were close to each other and the difference between them was very small; therefore, the results of a single input excitation, in this case, T1, were used as a reference for the FE model updating. The comparison of FE and experimental modal frequencies showed that the modal frequencies obtained from the FE model were greater than the experimental ones, indicating that the FE model was stiffer than the actual structure. Therefore, to decrease the modal frequencies of the FE model, the Young's modulus of the material was decreased. The updated values of the Young's modulus (1227.27 MPa) and corresponding compressive strength (3.019 MPa) were taken from Shahzada *et al.* (2012). To further minimise the difference, the mass source was modified to 1.2 and the meshing size was adjusted to 112 mm × 112 mm, the size of a half brick. In the end, the boundary conditions were changed from a rigid restraint at the base of a structure to a flexible joint spring with six degrees of freedom. A spring was provided at each corner of the wall to simulate the effect of SSI. As the aim of this paper is to investigate the effect of SSI, a comparison between two manually updated FE models is made, one with rigid constraint at its

base and the other with dynamic soil springs that provide flexibility at the base. The mechanical properties of the materials used in an updated model are provided in Table 5.

4.1 Modelling of SSI

In this paper, the interaction between soil and structure was modelled by assigning six-degrees-of-freedom joint springs at the base of the structure. The foundation of the structure consisted of a strip footing with four corners, embedded into the ground; therefore, the joint springs were applied at each corner. The impedance functions for these springs were calculated following the procedure presented by Gazetas (1991). The reason for following this procedure was its simplicity, ability to account for frequency dependence in impedance functions and estimation of reasonably accurate results as recommended from the literature (Butt and Omenzetter, 2014; Dutta and Roy, 2002; Dutta *et al.*, 2004).

To calculate the values of static and dynamic spring stiffnesses and dashpot coefficients, the geometric properties of the foundation, mechanical properties of soil surrounding it and the dynamic response frequency of the soil are required. The geometric properties are length (L), width (B), base area (A), depth (D) and second moment of inertia (I) of the foundation. The length of the foundation parallel to the railway line was 5.18 m and perpendicular to it was 4.26 m. The foundation consisted of a three-step masonry strip foundation; each step was 300 mm deep, making a total of 900 mm below the ground level. The thickness of the foundation at ground level was 450 mm and at the bottom of the foundation was 900 mm. In order to obtain the properties of the soil, a geotechnical investigation, which consisted of a standard penetration test (SPT) was performed at the site. An 11 ft deep bore was dug to obtain the number of blow counts (N) at each 1 m interval. The soil was classified as type D soil, which represents soft soil. The values of shear modulus (G) and shear wave velocity (V_s) were estimated from their correlation with SPT N (number of blow counts) values. Their correlation for different types of soil types can be found in the literature. In this study, the correlations used for the estimation are presented in Table 6. The first two correlations are for estimation of shear wave velocity (V_s) from SPT N values and the last two are for estimation of shear modulus (G) from SPT N values. The shear wave velocity and shear modulus are related through the equation

$$2. \quad G = \rho V_s^2$$

Table 5. Updated elastic mechanical properties of masonry and soil

Mechanical property	Density: kg/m ³	Poisson ratio	Young's modulus: MPa	Tensile strength: MPa	Compressive strength: MPa	Shear wave velocity: m/s	Friction angle	Shear modulus: MPa
Masonry	1494	0.26	1227.27	—	3.019	—	—	—
Soil	1506.9	0.4	—	—	—	191.89	28°	55

where ρ is the density of soil and its value is calculated as 1506 kg/m^3 . From Table 6, it is clear that the shear wave velocity (V_s) lies in the range from 188 to 196 m/s and the shear modulus (G) lies in the range from 53.26 to 58.103 MPa. In this paper, the value of V_s is taken as 191.89 and G is taken as 55 MPa to calculate the dynamic soil stiffness. The Poisson ratio was taken as 0.4, taking recommendations from Bowles (1996). The dynamic response frequency is taken as the fundamental frequency of the structure under investigation.

The final values of dynamic impedance functions calculated for each degree of freedom for both walls are provided in Table 7.

5. Effect of SSI

In this section, the effect of SSI on the dynamic response of a structure is investigated through the results obtained from the experimental and numerical analysis. Comparisons between the updated FE model and the experimental modal properties, response time histories, base input force, base moments and

peak displacements are performed in this section. The effect of SSI is evaluated through these comparisons.

5.1 Finite-element model and experimental modal properties

This section presents the comparison of modal properties obtained from FE modelling and the experimental results of T1 vibration. The comparison of natural frequencies between measured, initial and updated FE model (with soil springs and fixed base) is presented in Table 8. The comparison shows that, although overall the difference between fixed and flexible base modal frequencies is small (less than 10%), the modal frequencies captured by updated FE model with soil springs (flexible base) are in better agreement with the experimental results when compared to the fixed base updated FE model. This is due to the additional flexibility added to the structure through these soil springs. The difference between modal frequencies of the measured and flexible base FE model is -2.4 , 4.4 and -3.75% , whereas the difference between the measured and fixed base FE model is -6.6 , 1.3 and -9.6% . A plausible cause for the high difference between the modal frequencies of

Table 6. Correlations between SPT N with V_s and G

Correlation	$V_s = 94.4 \times N^{0.379}$ (Kirar <i>et al.</i> , 2016)	$V_s = 93.67 \times N^{0.389}$ (Marto <i>et al.</i> , 2013)	$G = 13.73 \times N_{\text{average}}^{0.71}$ (Ohsaki and Iwasaki, 1973)	$G = 15.09 \times N_{1(60)}^{0.389}$ (Anbazhagan <i>et al.</i> , 2012)
V_s : m/s	191.896	196.879	188	193
G : MPa	55.19	58.103	53.269	55.979

Table 7. Dynamic impedance functions

Vibration mode	Dynamic impedance function for 5.18 m strip footing		Dynamic impedance function for 4.26 m strip footing	
	Stiffness coefficient: kN/m	Dashpot coefficient: kN s/m	Stiffness coefficient: kN/m	Dashpot coefficient: kN s/m
Vertical (Z)	788 213.8	7807	685 731.53	6512
Horizontal (X) long direction	744 502.9	6958	662 121.47	6014
Horizontal (Y) lat. direction	872 147.4	9660	764 818.02	8164
Rocking rx	852 432.9	2025	734 421.12	1723
Rocking ry	383 6993	11 697	2 598 799.27	7411
Torsion t	33 316.99	2717	37 383.01	1300

Table 8. Comparison between measured and numerical modal frequencies

Mode	Measured frequencies: Hz	FEM frequencies: Hz			Difference between FEM and measured frequencies: %		
		Updated model			Updated model		
		Initial model	With soil springs	With rigid constraint	Initial model	With soil springs	With rigid constraint
First	5.25	6.9	5.38	5.6	-31.4	-2.4	-6.6
Second	5.88	7.11	5.62	5.8	-20.91	4.4	1.3
Third	8.25	11.83	8.56	9.04	-43.3	-3.75	-9.6

Table 9. Similarity correlation between measured and experimental mode shapes (MAC)

Mode	MAC (%) between FEM and measured mode shapes		
	Initial model	Updated model	
		With soil springs	With rigid constraint
First	12.34	88.37	59.21
Second	17.49	75.9	84.33
Third	73.4	89.9	91.2

the third mode as compared to the first two modes may be that higher-order modes typically involve more complex vibration mechanisms governed by local material and structural characteristics, boundary conditions and larger measurement errors.

The comparison of MAC values presented in Table 9 between experimental, flexible and fixed base FE models indicates that the presence of soil modifies the deformation pattern of the structure. The MAC between experimental and flexible base FE model for three modes of vibration is 88.37, 75.9 and 89.9%, whereas the MAC between experimental and fixed base updated FE model is 59.21, 84.33 and 91.2%. The results indicate that the effect of soil is significantly visible on the fundamental mode of the structure and it tends to decrease with an increase in number of modes. At higher modes, the mode shape correlations between the fixed base FE model and experimental mode shapes are in better agreement than for the flexible base FE model. Therefore, it can be said that the effect of SSI is significant on the fundamental mode of the investigated structure in terms of mode shapes and is negligible for higher-mode shapes.

The comparisons of modal mass participation between fixed and flexible base FE model are also presented in Table 10. The huge difference in modal mass participation ratio of the fixed and flexible base system for the first mode again represents the influence of soil in modifying the dynamic behaviour of the structure. It is also observed that the modal mass participation is very much less in the vertical direction for the first three modes as compared to the lateral direction.

5.2 Experimental and numerical response time histories

This section presents the comparison between the acceleration record measured at the roof sensor location A1 and simulated response time history at the same location in the FE model. For this purpose, a time history analysis of both FE models is performed by applying the acceleration record measured at the base level at sensor 3 for a T1 train-induced ground motion. Against this applied time history, the response at the roof at the location of sensor A1 is obtained. This simulated response is compared with the experimental response to quantify the difference. The relative difference was measured using the following equation (Sprague and Geers, 2004)

$$3. \quad \sqrt{\frac{\sum_{t=1}^n a_s^2(t)}{\sum_t a_m^2(t)}} - 1$$

The graphical comparison of flexible base FE model simulated response time history and experimental response time history is presented in Figure 7.

The relative difference in magnitude of the simulated and experimental response time history calculated in the *X*-direction is 3.5%; in the *Y*-direction it is 9.09% and in the *Z*-direction the calculated error is 2.08%, as presented in Table 11. The difference of less than 10% clearly shows that the FE model quite accurately predicts the actual response of the structure.

Similarly, a graphical comparison between the fixed base FE model response time histories with experimental response time histories is presented in Figure 8. The difference in fixed base FEM simulated response time history and experimental response time history is more than the difference calculated from the comparison of flexible base simulated response time history and experimental response time history. The difference in magnitude is 18.8% for the *X*-direction, 23.43% for the *Y*-direction and 5.34% for the *Z*-direction, as presented in Table 11.

From this comparison, it is concluded that the effect of SSI is significantly evident in estimating the accurate response time

Table 10. Modal mass participation ratios of a fixed and flexible base FE model

Mode	Flexible base FE model						Fixed base FE model					
	Translation			Rotation			Translation			Rotation		
	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z
1	0.45	0.03	0.00	0.02	0.33	0.03	0.23	0.24	0.00	0.2	0.2	0.02
2	0.48	0.48	0.00	0.36	0.36	0.03	0.43	0.46	0.00	0.38	0.36	0.06
3	0.53	0.47	0.00	0.37	0.38	0.45	0.51	0.46	0.00	0.38	0.42	0.46
Total mass participation	1.46	0.98	0.00	0.75	1.07	0.51	1.17	1.16	0.00	0.96	0.98	0.54

Figure 7. Time history comparison of flexible base FE model with experimental response TH in (a) X-direction, (b) Y-direction and (c) Z-direction

Table 11. Relative difference in magnitude of FE model and experimental time history response

	X- direction: %	Y- direction: %	Z- direction: %
Flexible base FE model and experiment	3.5	9.09	2.08
Fixed base FE model and experiment	18.8	23.43	5.34

histories for this investigated structure, as neglecting it significantly increases errors.

5.3 Peak displacement, base input force and base moment

The comparison of peak displacement, base input force and base moment for fixed and flexible base FE model showed that the inclusion of SSI did not significantly influence these parameters. Although a slight increase in these parameters was observed for the flexible base system, the overall variation observed was very low. In the X-direction, the peak base input force obtained for the flexible base system was 6.28 kN (1412 lb), which was more than that obtained for the fixed base, that is 4.14 kN (931 lb). In the Y-direction the peak base input force was 10.11 kN (2273 lb) for the flexible base, which was more than that obtained for the fixed base, namely 9.11 kN (2050 lb). In the Z-direction, the peak base input force for the flexible base was 23.29 kN (5237 lb), which was more than that obtained for the fixed base, which was 22.9 kN (5152 lb). The peak displacement obtained for the flexible base FE model in the X-direction was 0.058 mm (0.0023 in), in the Y-direction

was 0.06 mm (0.0024 in) and in the Z-direction was 0.0502 mm (0.002 in). For the fixed base FE model, the peak displacement was 0.053 mm (0.0021 in) in the X-direction, 0.0508 mm (0.002 in) in the Y-direction and 0.049 mm (0.0019 in) in the Z-direction. The capacity curve between the base input force and displacement obtained at the roof at the sensor A1 location is presented in Figure 9 for the Z-direction only, as in the other two directions the base input forces were not prominent.

Similarly, an increase in base moments was observed for the flexible base FE model as the soil is a flexible body and it rotates with the structure, thus increasing base moments. The observed peak base moment for the flexible base FE model in the X-direction was 58.82 kN m (43.39 kip ft), 58.11 kN m (42.86 kip ft) in the Y-direction and 26.15 kN m (19.29 kip ft) in the Z-direction. For the fixed base FE model, the observed peak base moment in the X-direction was 56.5 kN m (41.7 kip ft), in the Y-direction was 57.08 kN m (42.1 kip ft) and in the Z-direction was 18.72 kN m (13.8 kip ft).

It is clear from the above results that no significant difference was observed between the fixed and flexible base FE models in terms of the base moment, global peak displacement and base input force comparison. The reason may be the low amplitude of excitation, height and bracing of the structure. However, a change in the pattern of both capacity curves can be attributed to the presence of soil.

5.4 Summary

The comparison of experimental and numerical modal properties of the investigated structure indicated that the effect of

Figure 8. Time history comparison of fixed base FE model with experimental response TH in (a) X-direction, (b) Y-direction and (c) Z-direction

Figure 9. (a) Capacity curve for flexible base FE model.
(b) Capacity curve for fixed base FE model

SSI was observed significantly only on the fundamental mode shape of the structure. Although the modal frequencies obtained from the flexible base FE model were seen to be in better agreement with the experimental results, the overall difference between fixed and flexible FE models was observed to be very low ($<10\%$). The difference between the modal frequencies of the experimental and flexible base FE model ranged from 2.4 to 4.4%, whereas the difference between experimental and fixed base FE model ranged from 1.3 to 9.6%. The MAC between the measured and flexible base FEM was 88%, whereas between measured and fixed base FE model it was 59% for the first mode. This indicated a considerable effect of SSI. For higher modes, the MAC difference was less between fixed and flexible base FE model, indicating no effect of SSI.

The FE model with soil springs accurately predicted the response time history at the roof of the structure as compared to the FE model with fixed restraints. When compared to the experimental response, the difference in magnitude of the flexible base FE model response time history was calculated as 2.08 to 9.09% in all three directions, whereas the difference in magnitude between fixed base FE model response time history and experimental response time history was calculated as 5.34 to 23.43% in all three directions.

The inclusion of SSI in the FE model increased the base input force, base moment and peak displacement by 1.02 to 1.51% in all three directions. These results indicated a negligible effect of SSI on these parameters under train-induced vibrations. However, a change in the pattern of both capacity curves may be attributed to the presence of soil.

6. Conclusions

The aim of this study was to investigate the effect of SSI on the dynamic response of a masonry structure. For this purpose, a single-storey braced masonry structure near a railway track was instrumented and its response under five train-induced vibrations was measured. From the monitored data, the first three modal properties (mode shapes and frequencies) of the structure were estimated. The variation in identified modal properties against each vibration indicated the fact that they vary with varying excitations.

To investigate the effect of SSI, two FE models were developed and updated. One consisted of a structure with a fixed base and the other consisted of a structure with a flexible base. The results led to the observation that the effect of SSI on the fundamental mode shape of the structure and in the prediction of accurate response time histories was significant. The effect of SSI on modal frequencies, higher-order mode shapes (second and third mode shapes), base moment, peak global displacement and base input force was observed to be almost negligible for the structure under investigation against railway traffic vibrations.

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