EXPERIMENTAL AND NUMERICAL INVESTIGATION OF AN INTEGRAL ABUTMENT BRIDGE MODEL

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Abstract

In the last years there has been renewed interest on Integral Abutment Bridges (IABs). These designs are characterized by the absence of bearing supports and expansion joints, leading to a reduction in construction and maintenance cost over ordinary bridges. In IABs, complex Soil-Structure Interaction (SSI) phenomena develop between bridge and backfill under both static and dynamic conditions, due to thermal expansion and earthquake action. The use of IABs is common in many countries, but there is a lack of design guidelines especially regarding seismic design. The behavior of IABs has been investigated in a number of experimental and numerical studies. Along these lines, an experimental campaign was conducted on the Earthquake Simulator of the University of Bristol, to explore SSI effects between a scaled aluminum IAB model and the backfill/foundation soil under earthquake loading. The tests were performed for five different configurations, by varying i) the head constraints of foundation piles (connected or disconnected from the cap), and ii) the conditions behind the abutment wall by inserting one or two "*compressible inclusion*" layers with the purpose of mitigating the earth pressures on the abutment walls. Different types of instruments were used to monitor the dynamic response of the soil-structure system: accelerometers, LVDT's to record displacements and strain gauges. The effectiveness of the designed measures can be checked using the experimental results. A preliminary numerical model of the experimental setup was developed and is discussed in the paper.

Keywords: Integral Abutment Bridge, Shaking Table, Soil-Structure Interaction, Experimental test

1. Introduction

Bridges are the most vulnerable elements in highway networks. Poor maintenance and ageing can result to a deterioration in mechanical properties. A recent report published by the American Road & Transportation Builders Association [1] states that there are more than 47,000 bridges in the United States (US) which are rated “structurally deficient” and need urgent repairs. Similar studies in Europe [2] have indicated that different levels of maintenance and damage exist in European highway networks, with many examples of bridges being in bad conditions from a structural point of view.

Integral Abutment Bridges (IABs), known in some countries as jointless bridges, have no bearing supports and joints between the bridge girder and the abutments. IABs have received increasing interest in the recent past due to a number of advantages over conventional bridges [3]. The use of IABs is quite common worldwide. IABs are very common in the United States (US) [4]; according to Maruri and Petro [5] over 13,000 such bridges were built by the mid 2000’s. Also, many European countries are adopting IABs, including Austria, France, Germany, Luxembourg, Switzerland, and the UK [6]. Also, Japan, Australia and China have started adopting this design for their highway bridges [7]-[10].

The main advantage of IABs over conventional bridges (CB) is their durability and low maintenance cost. Moreover, IABs have a higher redundancy with respect to CBs, which on one hand improves the resistance of the bridge, but on the other hand more complex Soil-Structure Interaction (SSI) effects between the abutments and the backfill soil. SSI horizontal actions can result in an abrupt increase in earth pressure on the abutments. In literature there are various numerical studies about the contribution of the embankments and the backfill to the dynamic behavior of the soil-structure system [11]-[12].

On the other hand, experimental data on SSI effects related to IABs are scarce. Recently, a consortium of US universities carried out a joint research project on the seismic response of seat-type abutment bridges (University of Nevada-Reno, University of California – San Diego, University of California - Berkeley). As an example, Saiidi et al. [13] performed shaking table tests on large scale models of two-span and four-span bridges at the University of Nevada, Reno.

A number of novel solutions to improve the behavior of IABs under seismic conditions have been proposed. For example, Horvath [14] interposed layers of a compressible material between the abutments and the backfill soil. The materials are typically geosynthetic, as for example Expanded Polystyrene (EPS) Geofoam, which is available in blocks to be positioned behind the abutment wall. The benefits of compressible inclusion layers were demonstrated by some numerical studies, which highlighted a reduction of earth pressures in the backfill and on the abutments [15].

Project SERENA (Seismic Response of Novel Integral Abutment Bridges) was developed under the Horizon2020 SERA (Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe www.sera-eu.org). The project aimed at performing shaking table tests on a scaled model of Integral Bridge, using a 5m – long soil container (“Shear Stack”) reproducing natural soil conditions [16],[17]. Five different configurations were tested on the shaking table, varying the number of compressible inclusion layers placed behind the abutment wall, and disconnecting the foundation piles from the foundation [18]. In the following, the test set-up and protocol are presented. Some results regarding the dynamic response of the model are shown and discussed.

2. Test Setup

The experimental campaign was carried out in the Earthquake Laboratory of the University of Bristol. The EQUALS-BLADE shaking table is 3 x 3 m and weighs 3.8 tons. The maximum payload is 15 tons within a frequency range of 0-100 Hz. As displayed in Fig.1, the soil container consists of eleven rectangular aluminum rings, which are stacked alternately with rubber sections. It is 4.80 m long, 1 m wide and 1.15m deep [19],[20].

The tests are performed using strain gauges and Linear Voltage Differential Transducers (LVDTs) and accelerometers. These latter were placed on the shaking table, on the exterior walls of the shear stack, inside the backfill soil and on the bridge deck, abutments and footings. Strain gauges are placed on the abutment wall, the model piles, bottom and top part of the abutment wall footing. LVDTs are located on the East side of the abutment wall, monitoring both horizontal and vertical displacements. The configuration of LVDTs and accelerograms are depicted in Fig.1. The accelerometers on the shear stack are colored in magenta, the accelerometers network in the backfill soil in blue, in free field conditions in red, those placed on the bridge model and around the foundation piles in black. This figure does not show the strain gauges located on the abutment and piles; their response is not reported in the present paper. The layout of instrumentation was designed based on similar shaking table tests performed in the Earthquake Laboratory of the University of Bristol [20].

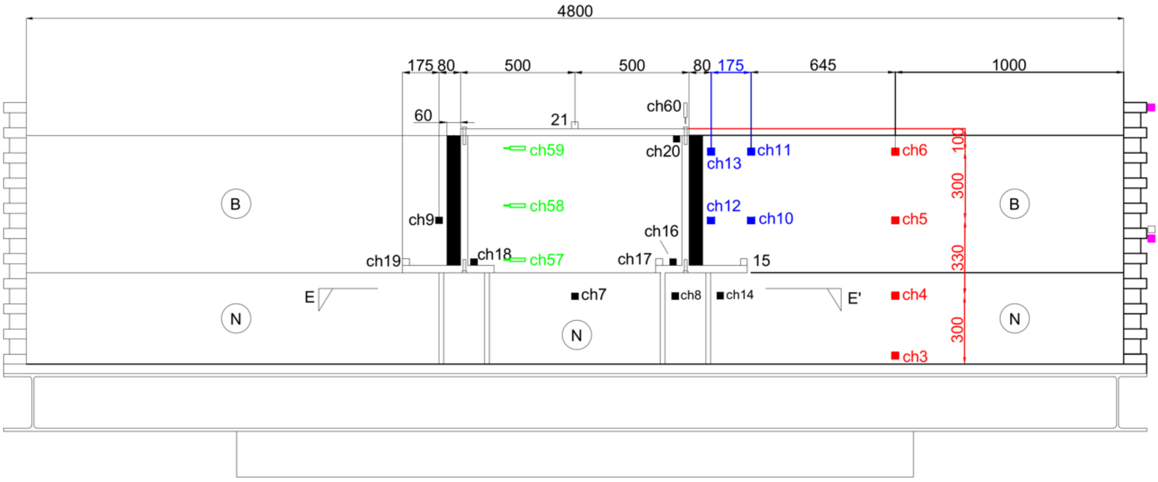

Fig.1 – Layout of the instrumentation placed in the soil container. (From [17])

The bridge scaled model is displayed in Fig.2a. The deck was realized using four steel beams with length, width, and depth of 1000, 100, and 30 mm, respectively. The abutment and footings of the bridge are realized using 32 mm thick aluminum sheets, connected by M12 bolts. Appropriate scaling laws based on the equivalence among specific mechanical properties are under development to extend the results obtained in these tests to real existing or new bridges.

As depicted in Fig.2b, the abutment wall footings have a two-line pile foundation. Sixteen model piles are included in the model bridge. Piles have hollow aluminum sections being 400 mm long and having an outer diameter and thickness of 22.24 and 1.3 mm, respectively. A nylon plug was included at the top of each pile to screw a steel riveted bar to obtain the different connection configurations between the base of the abutment wall and the piles.

Leighton Buzzard Sand Fraction B is used [21] to model the backfill soil. The soil was deposited in the shear stack in two different steps. Initially, a 400 mm thick layer is deposited around the model piles in the shear stack. The 400 mm thick layer is then densified with vibrations applying a white noise vibration with the shaking table. After the placement of the bridge model into the soil container, an additional 600 mm thick sand layer is placed behind the abutment walls.

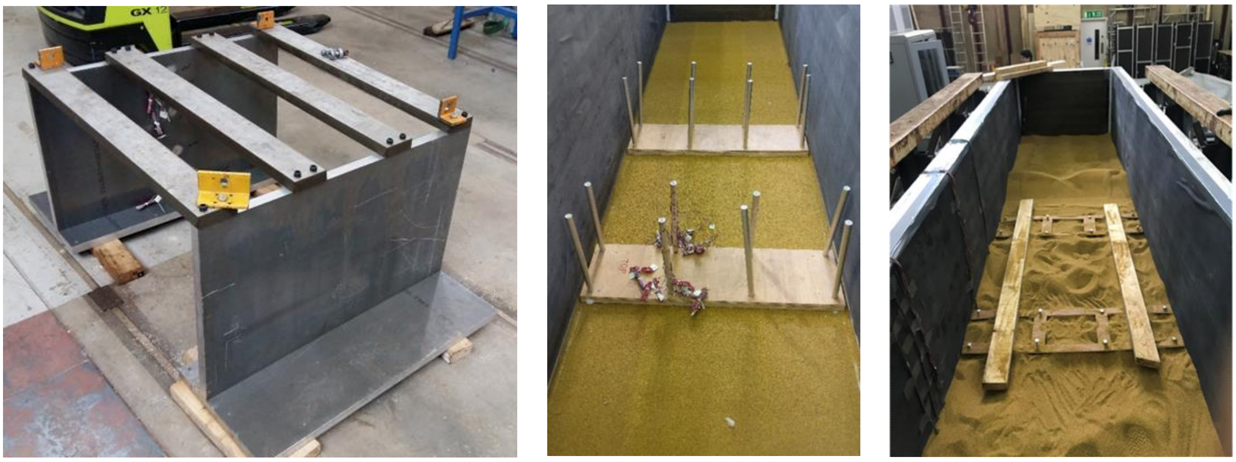

Fig.2 – (a) bridge model used in experimental campaign; (b) Position of foundation piles in the Equivalent shear beam container before filling with sand; (c) Soil container with foundation piles after

As mentioned above, five different test configurations were tested. Polyurethane (PU) foam was chosen with a relatively low stiffness, in order to maximize the benefits and simulate the effect of EPS in a typical IAB prototype. The choice of PU foam is motivated by the fact that it easily workable in the laboratory. Materials tests were carried out to assess its mechanical properties in the Laboratory. All the tested configurations are reported in Table 1. In the first test configuration two layers of PU foam are interposed between the abutments and the backfill soil. Then, a vacuum pump is used to remove the sand behind the backfill soil and realize the configuration with only one PU foam layer and with no compressible inclusion, respectively.

Subsequently, all the backfill soil sand (across the whole length of the soil container) is removed in order to modify the pile connection and simulate a hinged constraint between the base footing of the abutments and the piles (i.e., noCP configurations). First, noCP configuration is run without any compressible inclusion and one layer of EPS is added for the final configuration.

Table 1 – Configurations tested within SERENA project.

Configuration	**Date**	**EPS Layers**	**Pile Heads**
EPS2-CP	02.07.2018	2	Connected
EPS1-CP	04.07.2018	1	Connected
noEPS-CP	05.07.2018	0	Connected
noEPS-noCP	09.07.2018	0	Disconnected
EPS1-noCP	10.07.2018	1	Disconnected

The acceleration time-histories used as input for the shaking table are selected among the seismic records of the Italian Accelerometric Network (RAN), which are openly accessible and can be downloaded by the website of the European Strong Motion Database (www.esm.mi.ingv.it, [22]). Only strong motion stations for which the shear wave velocity V_S is available were taken into account. For three seismic intensities, corresponding to return periods of the seismic action of 50, 100 and 500 years, the elastic response spectra defined by the Italian seismic code (NTC 2018), for L'Aquila (Italy) site were evaluated. For each return period, depending on the hazard disaggregation, different Magnitude M and source-to-site Distance R intervals (in this case the epicentral distance) were used to search the database.

Unscaled natural records were compared with the code spectrum to verify the spectral compatibility. In selecting signals, specific care is taken to the frequency content, excluding records with frequencies above 30 Hz. The records are scaled in the frequency domain by a factor of 5 to fit the experiment scaling. This value has been chosen based on the development of scaling laws specifically derived for the soil-bridge model developed during the project. Table 2 reports the parameters of the ground motion records reported in the present study.

Table 2 – Selected accelerograms from European Strong Motion Database

Return period (years)	Station	Date	Comp.	M	R (km)
50	CSC	26/10/2016	NS	5.4	20.2
100	CSC	30/10/2016	NS	6.5	15
500	AMT	24/08/2016	NS	6.2	8.5

3. Results

Fig.3, Fig.4 and Fig.5 show some preliminary results in terms of maximum accelerations recorded by the accelerometers positioned in the backfill soil and on the bridge model, for 50, 100 and 500 years return period, respectively. In particular, the figures show the maximum acceleration response corresponding to the instant in which the maximum acceleration peak in the west direction occurs for each of the horizontal accelerometers.

The first observation that can be done on the vertical “free field” array (i.e. sensors 3, 4, 5, 6, see Fig.1), is that there is amplification of the motion from the bottom to the top of the shear stack. In fact, the free field motion should be substantially undisturbed by the different configurations adopted for the abutments and the foundation piles. The good behavior of the soil container is confirmed by the accelerations recorded on the box itself, similar to those recorded by the accelerometer placed at 0.4 m from the bottom of the box.

It is interesting to compare the evolution of maximum acceleration from the case with smallest input acceleration (Fig.3) to the ones with higher acceleration (Fig.4 and Fig.5). Indeed, in Fig.3, if one considers as an example the configuration noEPS-CP, the sensors located just behind the backfill (i.e. sensors 10, 11, 12, 13, see Fig.1) have recorded acceleration values which are very similar to those recorded in the Free Field array: sensors 10 and 12 have recorded an acceleration of 0.05g, which is equal to the one recorded by sensor 5 in the Free Field. The amplification in the upper layer is slightly higher comparing sensors 11 and 13 (0.07g) with sensor 6 (0.05g). The results for the same input motion are very similar also for all the other configurations.

When the results of the test obtained with an input motion with a return period of 100 years (Fig.4) are considered, the situation seems to change. In fact, while the trend at 400 mm from the soil surface is similar to the previous motion, the values obtained for certain configurations in the upper sensors of the backfill (i.e. 11 and 13) are sensibly higher than in the Free Field. For example, in the configuration noEPS-CP the acceleration is 0.23 and 0.24g for sensors 11 and 13, while is 0.17g for sensor 6, respectively. The acceleration seems to drop down when the compressible inclusion is behind the backfill. For the same input motion, the accelerations are 0.21, 0.21 and 0.17g for configuration EPS1-CP and 0.23, 0.24 and 0.17g for EPS2-CP, respectively. If the input motion with 500 years return period is considered, for noEPS-noCP, the values are 0.29, 0.3 and 0.24g and for EPS1-CP are 0.3, 0.31 and 0.24g, respectively while for EPS2-CP configuration the reduction effect is higher, the acceleration values being 0.25, 0.26 and 0.21g. Therefore, it seems that for this level of input motion the reduction of acceleration is high only when two layers of compressible inclusion are present. Regarding the effect of piles disconnection, this does not seem to have a very strong effect on the seismic response of the soil-structure system.

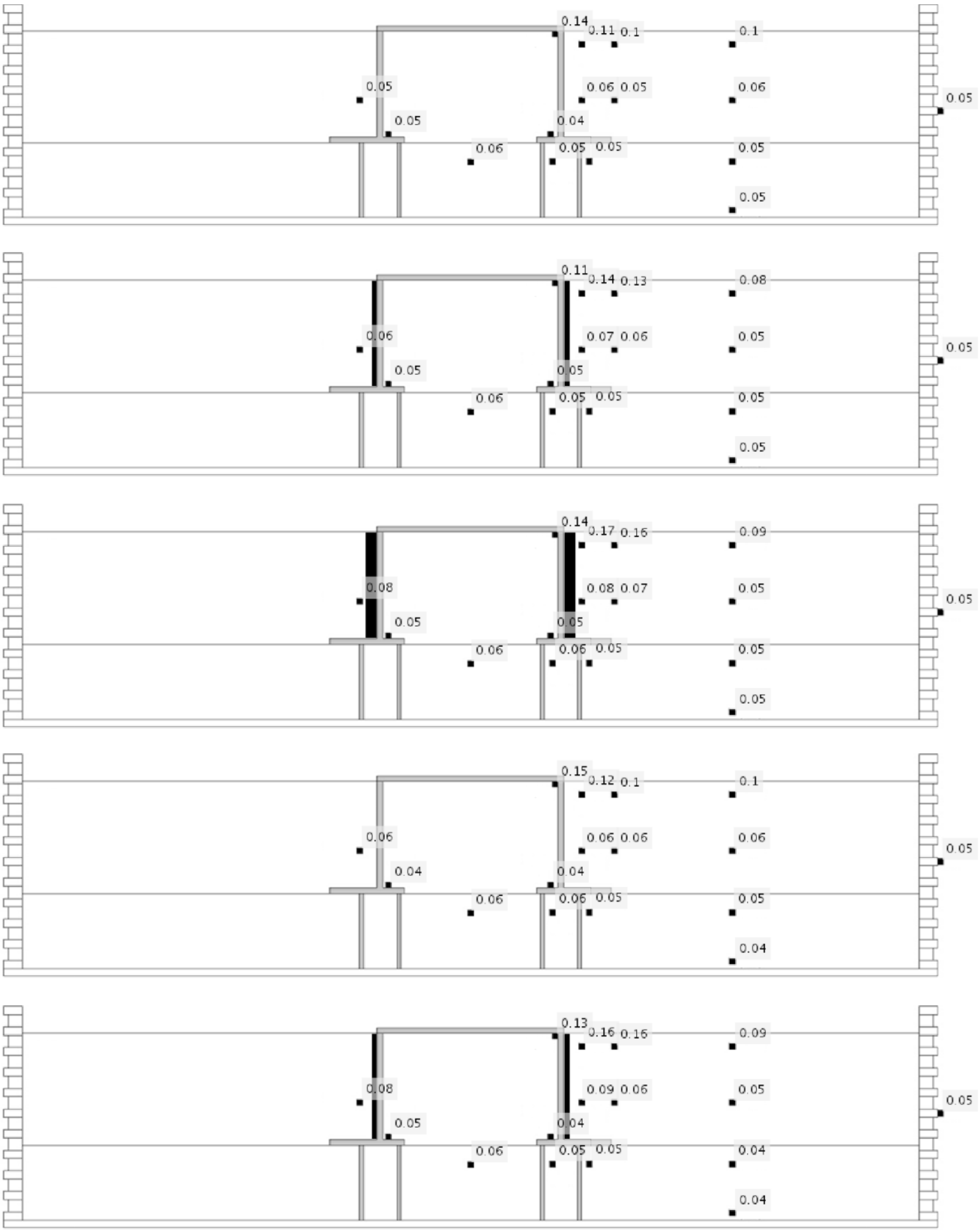

Fig.3 – Maximum acceleration responses for all the five configurations – 50 years return period of the seismic input.

Soil settlements were monitored during the experimental program for all configurations. Settlements are very important in the design and maintenance of real bridges as they are used to design important elements such as approaching slabs [23]. The changes in the depth of the soil surface after each shaking event was measured on a grid of points. Three alignments were used for the measurements. The grid is denser in close proximity of the abutment wall where the settlement of the backfill is commonly encountered in field applications. The sand level was made uniform with the abutment wall every time a new configuration was tested and the changes in the sand level according to a horizontal plane were monitored and recorded. The measured settlements are displayed in Fig.6.

The largest settlement in close proximity of the abutment wall occurred in EPS2-CP configuration which suggests that the decrease in the volume of the compressible inclusion was compensated with the settlement of granular material.

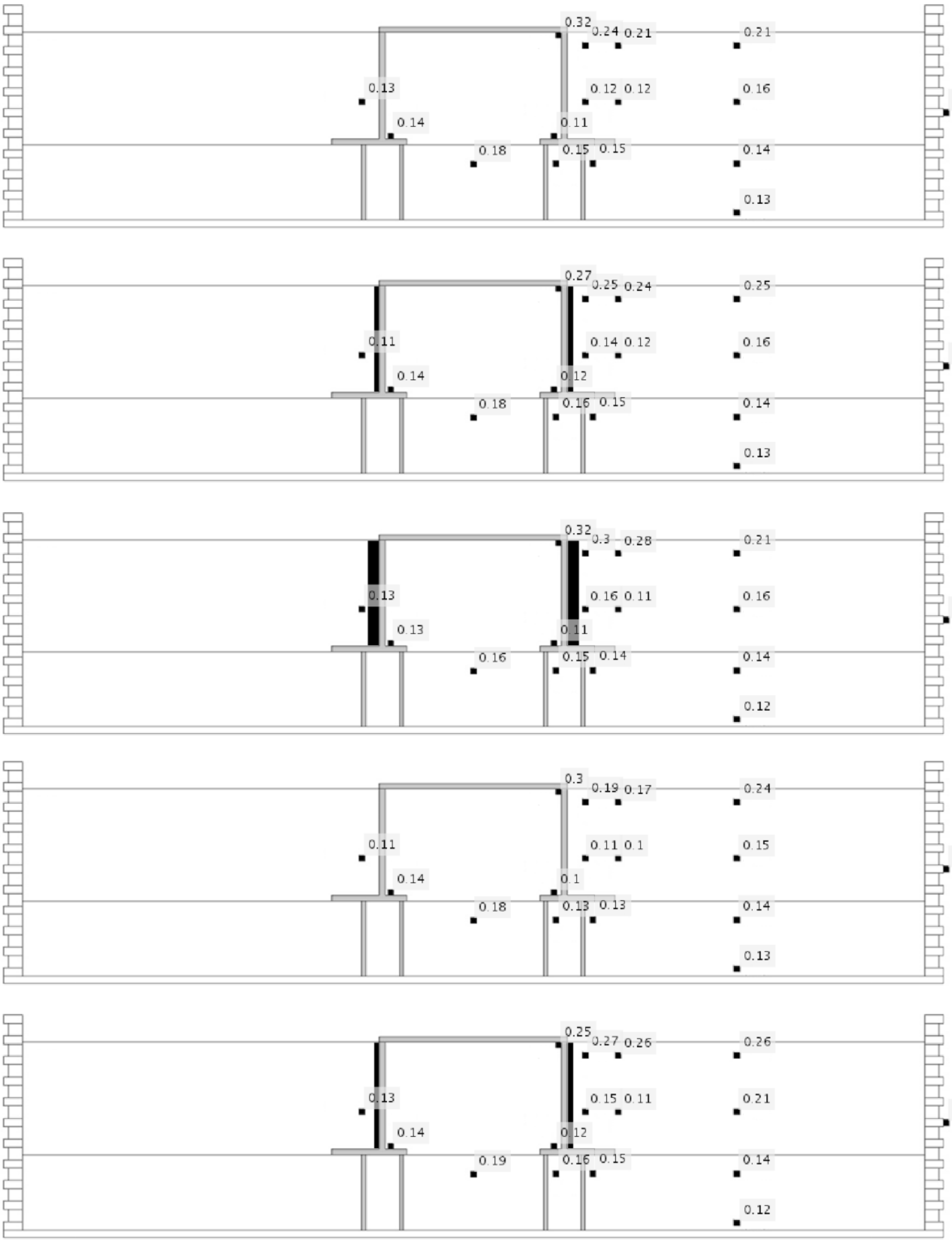

Fig.4 – Maximum acceleration responses for all the five configurations – 100 years return period of the seismic input.

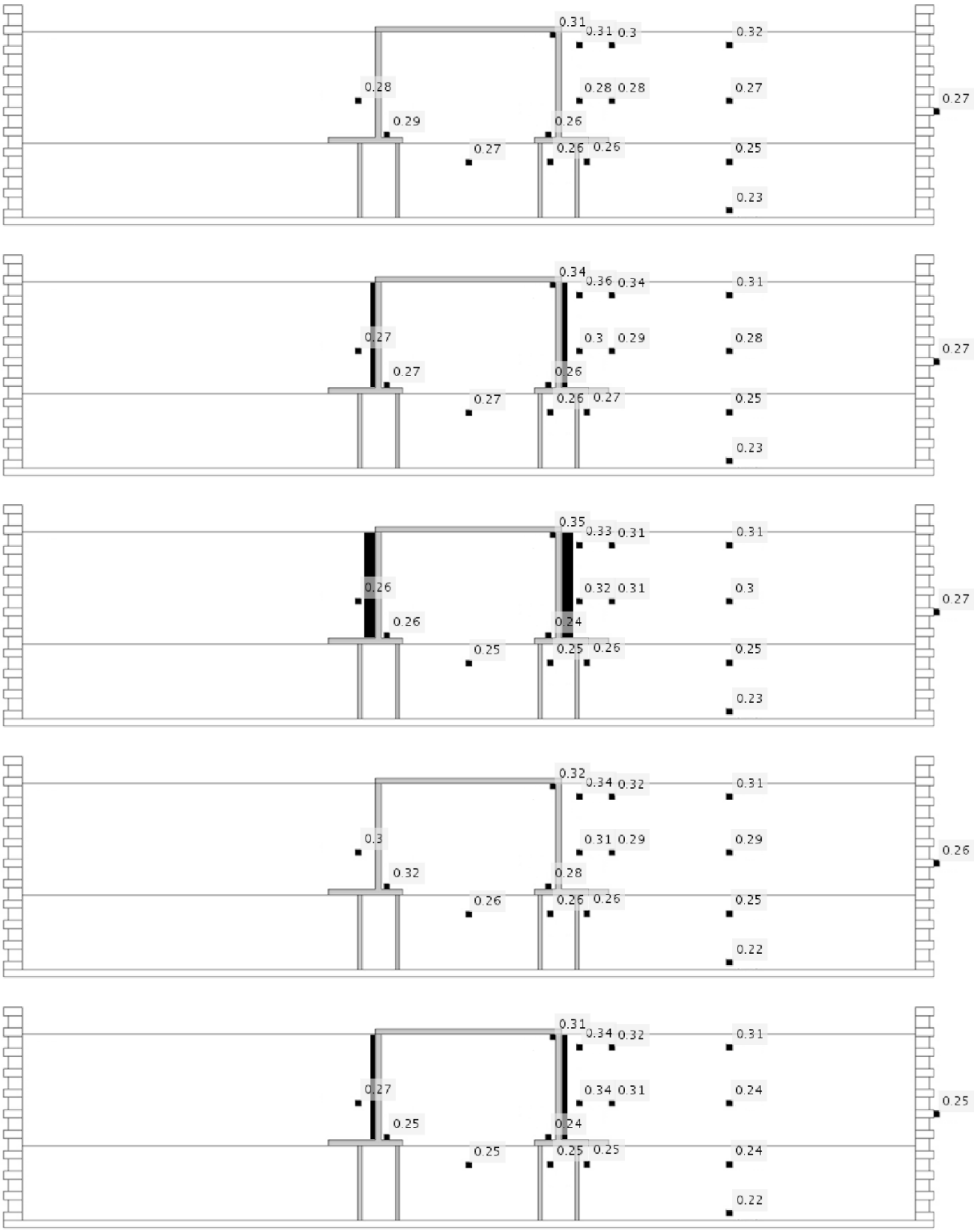

Fig.5 – Maximum acceleration responses for all the five configurations – 500 years return period of the seismic input.

A first, draft simulation of the shaking table tests is performed using the commercial software Plaxis 2D, as shown in Fig.7. The code allows to generate a 2D Plane strain Finite Element model. The mesh is composed by 15-node triangular elements. As a preliminary assumption, a Mohr-Coulomb criterion was used to model the soil behaviour. Preliminary results show a good matching between the numerical analyses and the experimental ones in terms of bending moments on the instrumented abutment wall. The development of more accurate numerical models, performing analyses in the time domain, together with the use of optimization techniques, will allow to carry out a model-updating taking into account all the relevant parameters and improving the accuracy of the simulation.

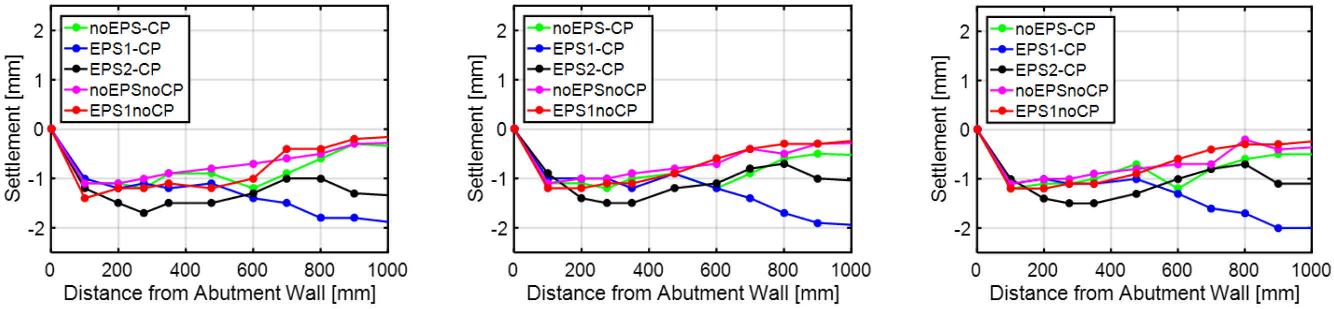

Fig.6 – Settlements measured in the backfill after tests S3 (a), S5 (b), S6 (c).

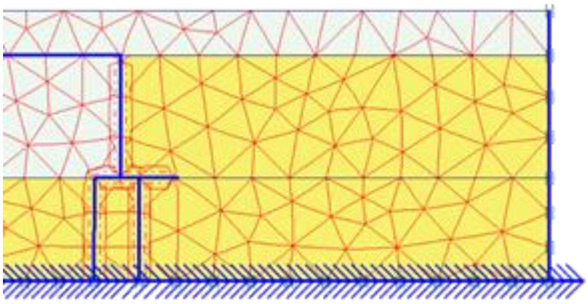

Fig.7 FEM Model in Plaxis 2D

4. Conclusions

The issue of Soil-Structure Interaction is particularly complex for Integral Abutment Bridges, due to the monolithic nature of the connection between the deck and the abutments. SERENA project aimed at testing a model of an Integral Bridge, inserted in a soil container, on a shaking table in order to study the dynamic response of the soil-bridge system. The tests were carried out for different configurations of the abutment (with/without compressible inclusion layers behind the abutment wall) and foundation piles (connected/disconnected), with the objective to study the effectiveness of these interventions as mitigation strategies. The strong motion records used in the tests were selected from the European Strong Motion Database.

Preliminary results show that the mitigation strategies, and in particular the use of compressible inclusion layers between the backfill soil and the abutments, seem to have a beneficial effect on acceleration recorded on the bridge. This effect is appreciable for stronger seismic input motions (i.e. 100 and 500 years return period) with respect to seismic input with a return period of 50 years. The measurement of settlements in each configuration are reported as it is an important parameter to inform the design of elements of the IABs such as transition slabs. A preliminary FEM model in Plaxis 2D is presented showing a reasonable matching with the simulation in terms of bending response of the wall. Further numerical optimizations of the model are necessary to replicate the experimental results for further use in design applications.

5. Acknowledgements

This research was conducted under the auspices of Seismic Response of Novel Integral Abutment-Bridges (SERENA) project. SERENA project was funded by the EU H2020 under grant agreement number 730900 [SERA]. The authors gratefully acknowledge the DPC-ReLUIS consortium for the financial support within the framework of the 2014-2018 and of the 2019-2021 Research Projects.

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