On the basis of Isola della Scala Bridge (see Fig. 1 and Fig. 2) considering the abutment strengths and piers capabilities, therefore, the analytical length limit for integral abutment bridge is investigated as following.

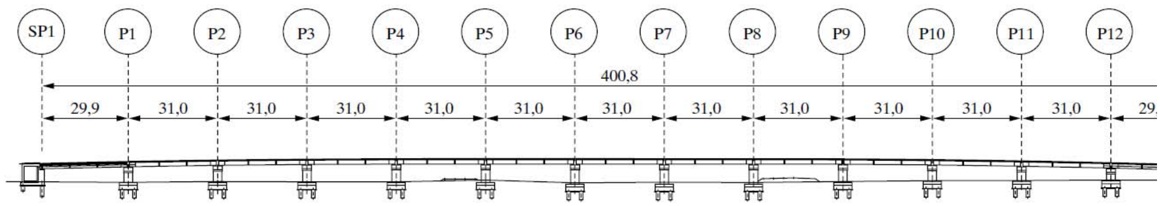


Figure 1. Elevation plans of Isola della Scala Bridge.

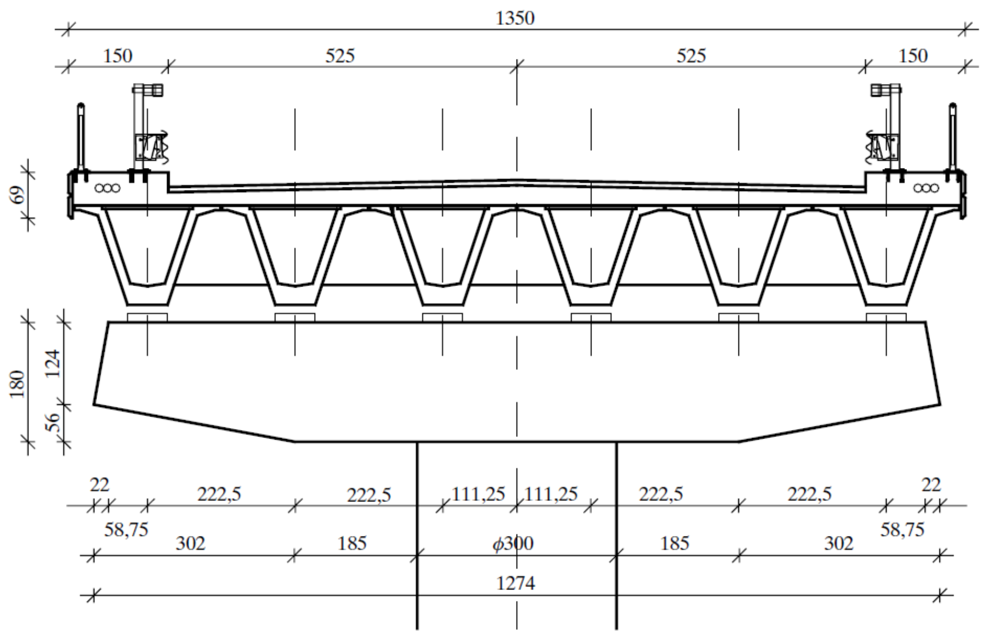


Figure 2. Typical cross section of Isola della Scala Bridge (unit: 10^{-2} m).

For a general case, the following parameters are considered: E = elastic modulus of the girder concrete; A = area of the cross section of girder, the same L = span length for all spans; K_i = stiffness of i-th pier top.

Firstly, the case of an odd number of spans is considered. The results obtained will be then extended to the case of an even number of spans.

For an odd number of spans, namely $n_s = 2n - 1$, considering the symmetry of the bridge and starting from the central span , the forces and the deformations involved in the equilibrium and compatibility conditions for all the spans are listed in.

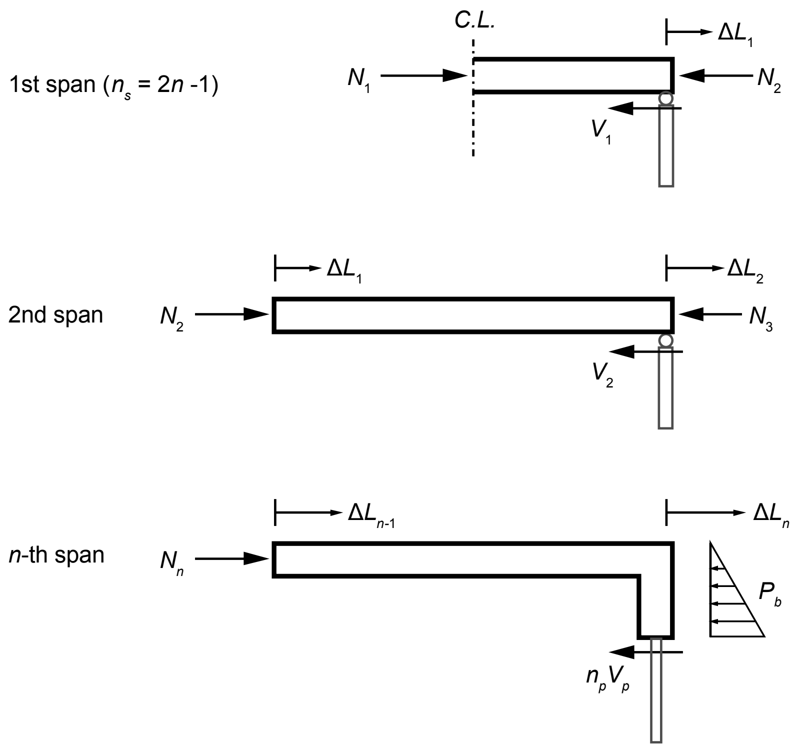


Figure 3. Horizontal Forces of Each Span.

For the 1st central half span, from compatibility equation and horizontal force equilibrium the following equations can be derived:

$$N_1 = EA\left(\alpha\Delta T - \frac{2\Delta L_1}{L}\right) \tag{1}$$

$$N_2 = N_1 - V_1 = EA\left(\alpha\Delta T - \frac{2\Delta L_1}{L}\right) - K_1\Delta L_1 \tag{2}$$

Similarly for the 2nd span:

$$N_2 = EA\left(\alpha\Delta T - \frac{\Delta L_2 - \Delta L_1}{L}\right) \tag{3}$$

$$N_3 = N_2 - V_2 = EA\left(\alpha\Delta T - \frac{\Delta L_2 - \Delta L_1}{L}\right) - K_2\Delta L_2 \tag{4}$$

From Eqs. (2) and (3), the relation between the displacements of 2nd and 1st pier tops can be expressed as:

$$\Delta L_2 = \left(3 + \frac{K_1 L}{EA}\right)\Delta L_1 \tag{5}$$

And for i-th span ($3 \le i \le n$), similarly it is found:

$$\Delta L_i = \left(2 + \frac{K_{i-1} L}{EA}\right)\Delta L_{i-1} - \Delta L_{i-2} \tag{6}$$

Therefore, for span number $n_s = 2n - 1$:

$$\Delta L_i = \begin{cases} \Delta L_1 & i = 1 \\ \left(3 + \frac{K_1 L}{EA}\right)\Delta L_1 & i = 2 \\ \left(2 + \frac{K_{i-1} L}{EA}\right)\Delta L_{i-1} - \Delta L_{i-2} & 3 \le i \le n \end{cases} \tag{7}$$

For an even number of spans, namely $n_s = 2n$, similarly it is found:

$$\Delta L_i = \begin{cases} \Delta L_1 & i = 1 \\ \left(2 + \frac{K_1 L}{EA}\right)\Delta L_1 & i = 2 \\ \left(2 + \frac{K_{i-1} L}{EA}\right)\Delta L_{i-1} - \Delta L_{i-2} & 3 \le i \le n \end{cases} \tag{8}$$

Equations（7）and （8）can be generalized as:

$$\Delta L_i = c_i \Delta L_1 \tag{9}$$

Where, c_i is the constant function of K_{i-1}, L, EA: $c_i = f_i\ (K_{i-1}L\ /\ EA)$ for different i and n_s. Creep and shrinkage can be considered modifying elastic modulus E as well as concrete cracking can be taken into account locally acting on the same parameter. The parameter expressing the longitudinal horizontal stiffness of the piers K_i needs to be considered properly as described in Lan, 2012.

And, from the compatibility condition on the n-th span, it is obtained:

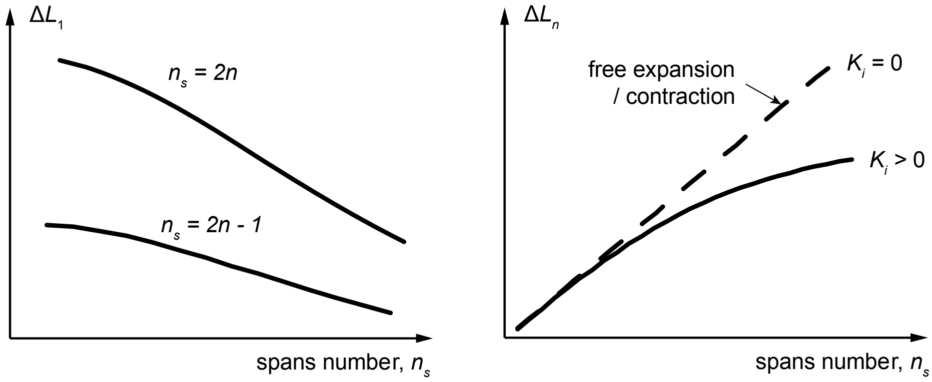
$$N_n = EA\left(\alpha\Delta T - \frac{\Delta L_n - \Delta L_{n-1}}{L}\right) = EA\left(\alpha\Delta T - \left(c_n - c_{n-1}\right)\frac{\Delta L_1}{L}\right) \tag{10}$$

Therefore, given a certain temperature load ΔT and span number n_s, using Eqs. and substituting, the displacement ΔL_1 can be obtained as for Eq.(11) and ΔL_n then can be calculated as Eq.(12)

$$\Delta L_1 = \frac{EA\alpha\Delta T - n_p V_p - \frac{1}{2}K_0 \gamma H_b{}^2 w_b}{EA\left(c_n - c_{n-1}\right) + \frac{1}{2}k_p c_n \gamma H_b L w_b} \cdot L \tag{11}$$

$$\Delta L_n = \frac{EA\alpha\Delta T - n_p V_p - \frac{1}{2}K_0 \gamma H_b^2 w_b}{EA\left(c_n - c_{n-1}\right) + \frac{1}{2}k_p c_n \gamma H_b L w_b} \cdot c_n L \tag{12}$$

The relation curves between displacements and the total number of spans are calculated and shown in a qualitative way in.



a) ΔL1 v.s. ns b) ΔLn v.s. ns

Figure 4. Relations between Displacements and Number of Spans.

However, for large displacements in abutments, although the plastic hinges have formed in piles in earlier stages they still can channel remarkable internal forces, especially for the case of integral abutment bridges. It's possible that the critical shear or bending moment in abutments would be reached under the movement of abutments and earth pressure on them (Bozorgzadeh, 2007), before piers reach their rotation capability. From the force equilibrium in the abutment, assuming the section positions of maximum shear and bending moment as shown in (Erhan and Dicleli, 2009), the critical internal forces can be obtained as:

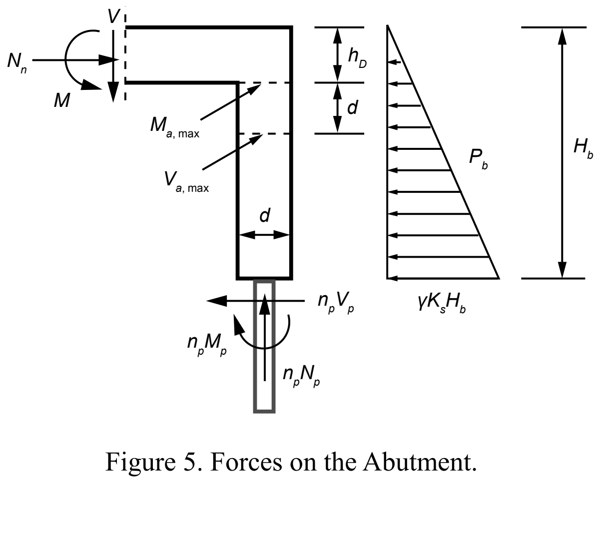


Figure 5. Forces on the Abutment.

$$V_{a,cr} \geq V_{a,\max} = \frac{1}{2} K_s \gamma \left[H_b^{\ 2} - \left(h_D + d \right)^2 \right] w_b + n_p V_p \tag{13}$$

$$M_{a,cr} \geq M_{a,\max} = K_s \gamma \left[\frac{\left(H_b - h_D \right)^3}{3} + \frac{h_D \left(H - h_D \right)^2}{2} \right] w_b + n_p \left[M_p + V_p \left(H - h_D \right) \right] \tag{14}$$

FromEqs.（13）and（14）, taking the example of positive temperature variation before the backfill passive pressure would reach the plastic phase, the limits of ΔL_n, on the basis of abutment strength capability, are expressed as follows:

$$\Delta L_n \leq \frac{2\left(V_{a,cr} - n_p V_p \right) H_b}{k_p \gamma \left[H_b^{\ 2} - \left(h_D + d \right)^2 \right] w_b} - \frac{K_0 H_b}{k_p} \tag{15}$$

$$\Delta L_n \leq \frac{M_{a,cr} H_b - n_p H_b \left[M_p + V_p \left(H_b - h_D \right) \right]}{k_p \gamma \left[\frac{\left(H_b - h_D \right)^3}{3} + \frac{h_D \left(H_b - h_D \right)^2}{2} \right] w_b} - \frac{K_0 H_b}{k_p} \tag{16}$$

Therefore, controlling the limits of ΔLn the corresponding ns can be found from the curves in 4b. Hence, the overall length could be easily obtained as: Ls = ns L.

The search for the limit length of an integral abutment bridge has to be referred to the boundary conditions and the geometrical and mechanical properties of the structure considered. In the present study the parameters affecting the design of the 400 meters long Isola della Scala Bridge were taken as a reference being this bridge, as for the knowledge of the authors, the longest integral abutment bridge ever built.

Therefore, besides the parameters of Isola della Scala Bridge, the following reference parameters were considered: pier rotation capacity of θpr = 0.0050 rad, abutment flexural capacity of Ma, cr = 50 MNm, and shear capacity of Va, cr = 10 MN. The mentioned values were used in the previous equations as limit values. And the temperature variation of +20°C as ultimate load condition was considered (passive earth pressure on the abutment). The solution obtained from the modified analytical formulation is presented in and compared to the case of free expansion.

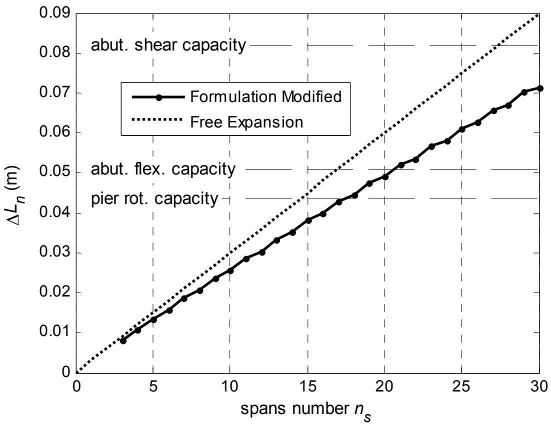


Figure 6. Thermal Displacements of Different Span Numbers Based on Isola della Scala Bridge.

It can be seen that, due to the boundary conditions affecting the analysis, the maximum number of spans would have been limited to ns = 18 due to the pier rotation capacity. Hence, under the assumptions and the approximations introduced, the overall length limit of this type of bridge would be L = 540 m.

Despite the common practice to design IABs to cover rather limited lengths, it has been proved that this kind of structures is suitable also for longer bridges. IABs must be nevertheless still considered as has always been done, as low-cost bridges built with common building materials and possibly with the use of pre-fabricated elements: with reference to these characteristics, super-long IABs are defined as those close to the limits deriving from the use of conventional building materials under the design boundary conditions given by the mechanical properties of the soil.

The introduction of parametric analyses in the design practice to take into account the presence of uncertainties related to the thorough understanding of the soil-structure interaction leads to more reliable solutions with positive effects in terms of overall maintenance costs during the lifespan of the bridge.

An analytical formulation suitable to assess the overall limit length for an IAB given the single span length and the general boundary conditions was proposed in this paper for the case of positive temperature variation. Similar results can be obtained in the case of negative temperature variations. The solution is highly affected by the characteristics of the bridge and the method proposed is still under development: the exact definition of the coefficients ci = fi(Ki−1L/EA) for different i and ns has to be thoroughly investigated for different cross sections, abutment and pier layouts, type of piles, soil properties and other factors (skew, and so on).

Nevertheless it has been proved that for a very conventional and low-cost bridge such as the Isola della Scala Bridge in Verona, Italy, built with prefabricated pre-stressed beams and common construction materials, an overall maximum length of more than 500 m can be theoretically reached.

Optimal Pile Shape

In integral abutment bridge, piers and abutments generally need not be designed to resist horizontal loads applied to superstructures (Burke, 2009). In fact, the horizontal loads are distributed complicatedly due to the interactions of soil-abutment and soil-pile, where the backfill earth pressure may take the larger proportion of 74%~88% of the applied load while shear force at the top of pile may take the small proportion of 12%~26% (Arsoy et al., 1999). This distribution might be caused by the geo-phenomenon of "ratcheting" (Horvath, 2004). Meanwhile, the displacement at the top of the pile nearly equals to the displacement at the top of stub-type abutment. Therefore, the piles should be designed to have the ability to accommodate the certain (lateral) shear force and large thermal-induced displacement. The current design optimization on piles shows that lateral loaded pile could be optimized for better performance with less material. With this purpose, the piles of integral abutment bridge, especially concrete piles, with fixed or pinned connection to the abutment, are going to be optimized with less material without failure in materials through a proposed design optimization approach.

For the laterally loaded piles, the beams surrounded by soil and laterally loaded at the top, are used as foundation structures usually with no or negligible axial loads, like, for example, in bridge abutments, retaining walls, off-shore wharfs, and the foundation piles of wind generators (Bowles, 1996). One of the simplified models in technical fields is to consider the laterally loaded beams surrounded by different materials to behave as beams in a Winkler medium, where earth pressure can be expressed as Terzaghi's equation Eq. when kh is constant.

Through fully stressed design (FSD) method with finite element method (FEM), the iterations of structural analysis and mass distribution would found out the optimum solution with minimum weight. And this kind of heuristic method has been used to optimize the beam length and section dimension as shown in .

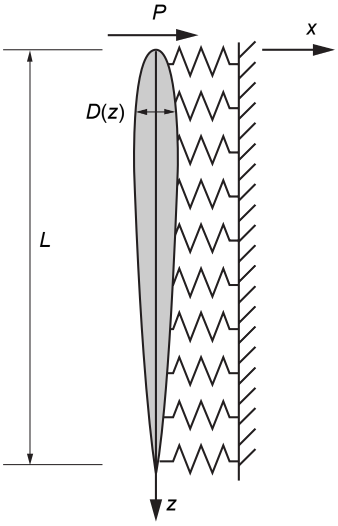


Figure 7. Shape of Fully Stressed Beam with Optimum Length.

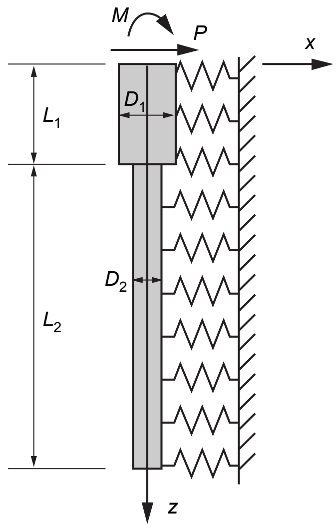


Figure 8. Model of the R/C Bored Pile Made Up of Two Parts.

However, in fact, the best mass distribution of laterally loaded piles is not achievable for real piles due to the difficulties in fabrication and shape casting as such a distribution should continuously change along the pile. Thus, for R/C bored piles, for example, the best mass distribution that can be achieved, by realizing bored piles with two parts with two different diameters. The problem to be solved is then how to design it, that is which diameter and length chose for the upper and lower part respectively.

Besides, piles in different conditions and different structures, the design principles or objectives would be different. Most of the piles are required to have less displacement of pile head under the same loading (or saying stiffer) with the same or less pile length or volume, in purpose of making better use of materials. Based on the global optimization algorithm, the frame of optimization approach is illustrated as following flowchart in. In the presented procedure, it is collaborated with main coding program of MATLAB and finite element modeler and solver of OpenSees. The finite element modeling in OpenSees and its advantage in saving computation time will be introduced in next section.

In this procedure, based on the optimization description, firstly, the design variables were assigned. Then their values were wrote to file and passed to subroutines that computes the objective function and constraint function, where, the finite element modelling and solving part might be called according objective and constraint required. Based on these loops of creating design variables (DV) then calculating constraints (SV/CON) and objectives (OBJ), the sampling of the optimization problem was passed to the "Global Optimization" setup. Then, by setting some control parameters of global search, such as variables tolerances, maximum iteration number and search step size, coupled local minimizers would be launched, until the converged optimal solution was found, or an unconverged results if tolerances or maximum iteration numbers were reached.

The most common method used for the numerical analysis of piles under lateral loads is the p-y method. In this method, the three-dimensional (3D) laterally loaded pile problem is analysed by using a beam on nonlinear Winkler foundation (BNWF) approach in which uncoupled one-dimensional (1D) springs are used to describe the soil-pile interaction. Therefore, despite known shortcomings inherent to the method, the p-y approach can be used successfully in many types of lateral pile analysis. To this purpose, Finite element simulations were conducted using the open-source FE framework of OpenSees (The Open System for Earthquake Engineering Simulation) (OpenSees, 2011). Embedded in this open-source software, the soils materials models are based on API recommendation and fibre section is available for building pile concrete sections. Besides, the important advantage of OpenSees is that it can easily interact and exchange the data with MATLAB through file operations during the process of optimization. The model was adapted from a simple example of OpenSees - a beam on a nonlinear Winkler foundation (BNWF) (McGann and Arduino, 2011; McGann et al., 2011), and then developed by applying fibre section and non-linear materials properties. Besides, in the example of optimization of integral abutment bridge pile design, the pile of Isola della Scala Bridge was taken from the global model; the forces obtained on the pile head were applied as the loads on pile; the same soils condition was considered; the geometry and section properties were all taken as starting values for optimization. In the model of the laterally-loaded pile problem, displacement-based beam elements are used to represent the pile and a series of nonlinear springs to represent the soil. The soil springs are generated using zero-length elements assigned separate uniaxial material objects in the lateral and vertical directions. An idealized schematic of the laterally-loaded pile model is provided in Fig. 10, and the optimized profile comparisons are listed in Fig. 11 and Fig.12.

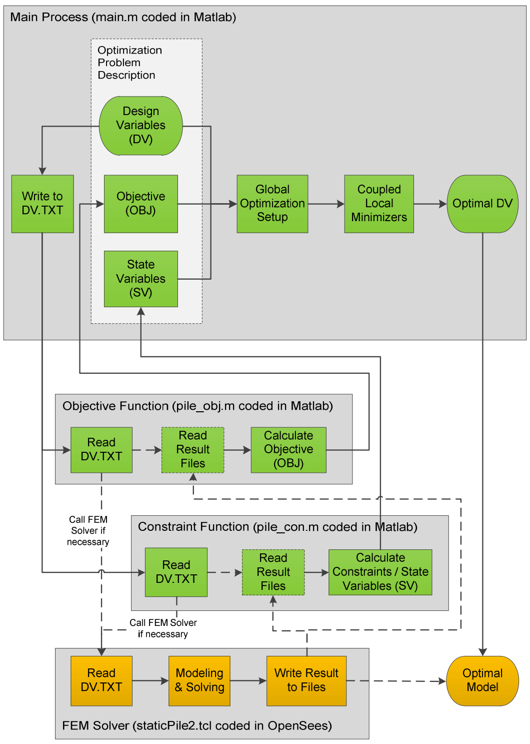


Figure 9. Frame of the Optimization Programming Thermal.

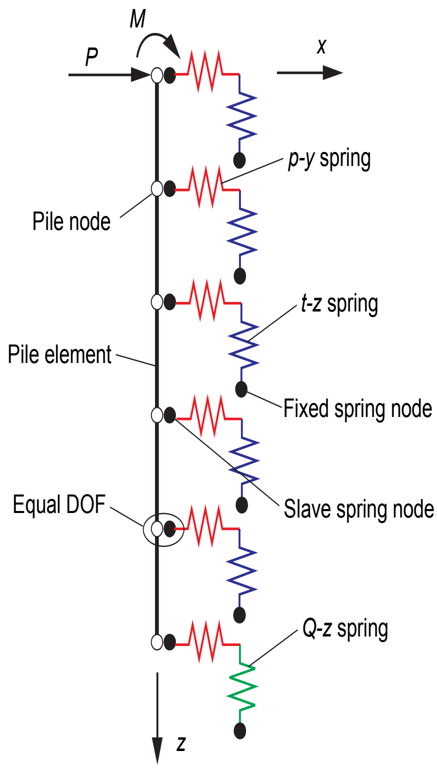


Figure 10. Pile Modeling in OpenSees.

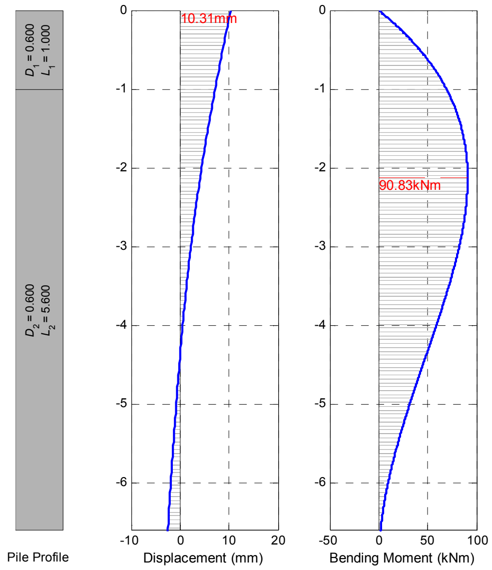


Figure 11. Stating Profile of Lateral Loaded Pile.

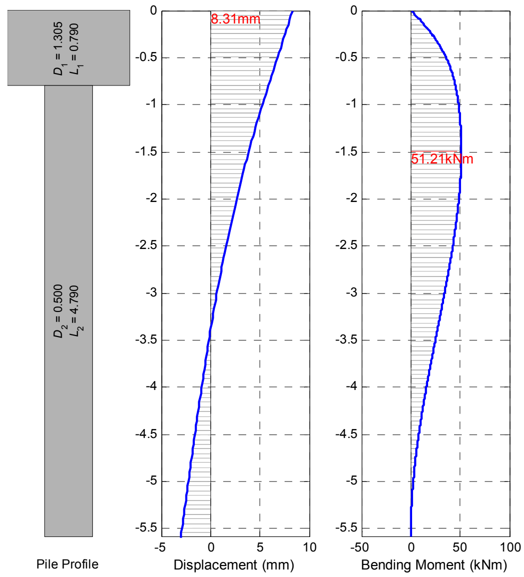


Figure 12. Optimized Profile of Lateral Loaded R/C Pile.

SHAKING TABLE TEST ON RIGID SOIL CONTAINER WITH ABSORBING BOUNDARIES: SSI

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Past earthquake events demonstrate (Chau et al., 2009) that damages in piles (see Fig. 13) are commonly induced during moderate to strong earthquakes. Mizuno (Mizuno, 1987) prepared the earthquake induced damages of piles reported in Japan from1923 to 1983, including those of the great Kanto earthquake. Damages in pile have been observed during the 1964 Niigata earthquake, the 1964 Alaska earthquake, the 1985 Mexico City earthquake, and the 1989 Loma Prieta earthquake (Meymand, 1998). More recently, severe damages in piles were also reported during the 1995 Kobe earthquake (Matsui and Oda, 1996) and the Tohoku earthquake.

Figure 13. Piles failures during earthquakes (Tohoku earthquake).

Civil structures are usually large in size and their response and performance under strong earthquakes cannot be meaningfully tested in an ordinary lab or in the field. Testing should be of large-scale, if possible of real-scale specimens. It often has to be combined with heavy advanced computations, integrated with the large-scale experiments to complement them and extend their scope. An important consideration in laboratory dynamic soil structure interaction study is to replicate the semi-infinite boundary condition in the soil container (Lombardi and Bhattacharya, 2012). This is usually done by using a rather expensive arrangement of the side walls of the box (Fig. 14).

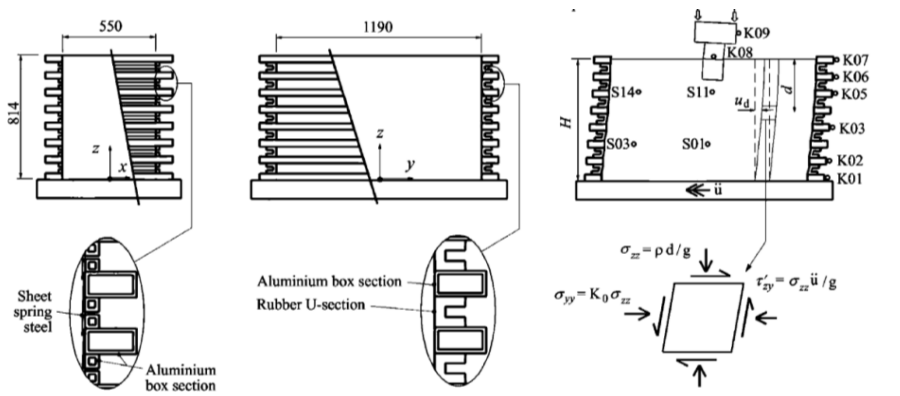

Figure 14. Shear box: (a) cross-section; (b) long section and (c) long section during testing and instrumentation layout (Pitilakis et al, 2008).

In the proposed experiment, a simplified economic layout is proposed following a rigid soil container with absorbing boundaries used at the Bristol Laboratory for Advanced Dynamics Engineering (BLADE) (Fig. 15). The model is designed to replicate real stratum of sand subjected to one-dimensional shaking applied to the bedrock. In order to minimize the reflection and generation of P waves from the artificial boundaries, absorbing materials will be placed at the end-walls. The set-up of the test will be very simple and the conclusions will be taken step by step, so it'll be possible order to study the effective beneath of using absorbing materials. A two meters in length steel pipe fixed at the bottom of the box will simulate one pile embedded in the bedrock.

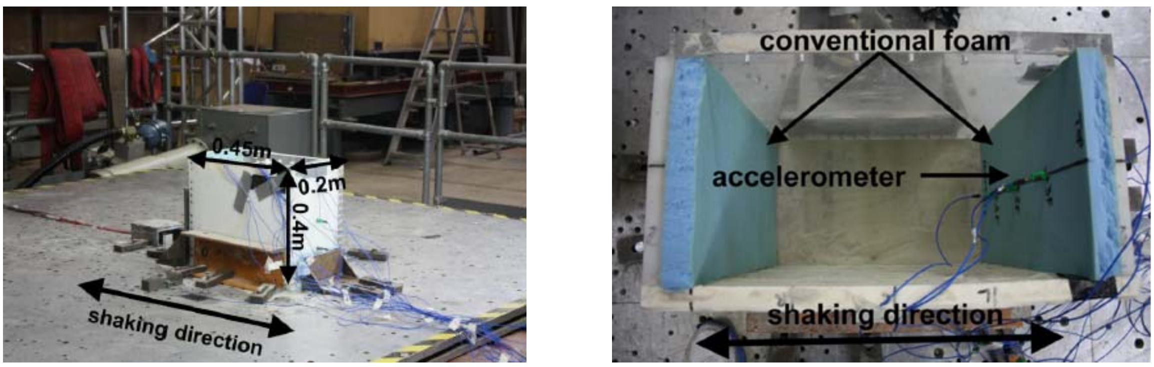

Figure 15. Rigid soil container with absorbing boundaries used at the Bristol Laboratory for Advanced Dynamics Engineering (BLADE). A soft material, namely conventional foam was applied at both end-walls of a small soil container (Lombardi and Bhattacharya, 2012).

The Pile-Soil-Structure Interaction (PSSI) can be computed (Mylonakis et al., 1997) as the superposition of two effects: (1) a so-called kinematic interaction effect (sometimes also referred to as "wave scattering" effect), involving the response to base excitation of a hypothetical system in that the mass of the superstructure is set equal to zero; (2) an inertial interaction effect, referring to the response of the complete pile soil structure system to excitation by D'Alembert forces associated with the acceleration of the superstructure due to the kinematic interaction.

To study the kinematic and inertial interaction effect a shaking table test on a pile backfilled in sand inside a box will be done. In the first phase (1) the kinematic interaction and far field condition will be studied in order to know the behavior of absorbing boundaries (foam) and the conditions of the soil inside the box. In the second phase (2), different masses will be added on the pile and the inertial interaction will be investigated. In the third phase (3), the setup of the experiment will be changed in order to study the beneath of using damping materials near the top of the pile in dissipation energy during earthquakes.

Test Set-up

The following tests phases will be performed:

I) PHASE I (Fig. 16)

With this set up is possible to study:

-Beneath of foam sheet in order to simulate the semi-infinite boundaries condition in soil container:

i) comparing accelerations data inside, near and outside the soil;

ii) shaking in orthogonal direction comparing displacement with and without absorbing foam -Far field condition without disorder of pile:

iii) calibrating the model with literature values;

iv) doing seismic and preliminary analysis on sand; -Kinematic interaction (only to pile-sand interaction):

v) evaluating the bending moment values on the pile that are not linked to superstructure.

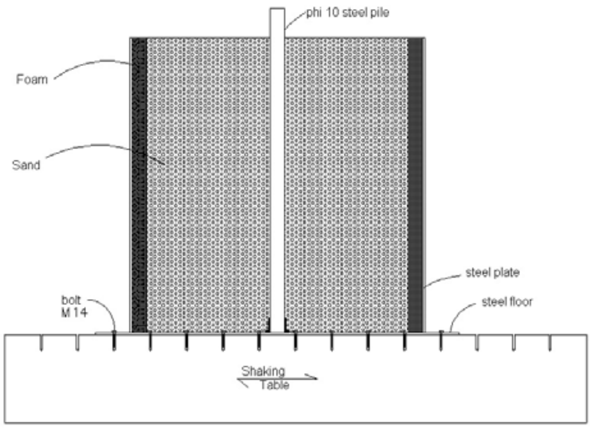

Figure 16. Phase I set-up

II) PHASE II (Fig. 17)

With this set up is possible to study:

ii) studying displacements near the pile;

-Comparisons between different loading condition:

iii) effects on pile deformation;

iv) soil reaction under increasing of system stiffness;

-P-y curves in order to evaluate the dynamic behavior of the soil:
v) dissipation of energy during the seismic loading (reduction of damping curves)
vi) shear modulus and shear wave velocity degradation.

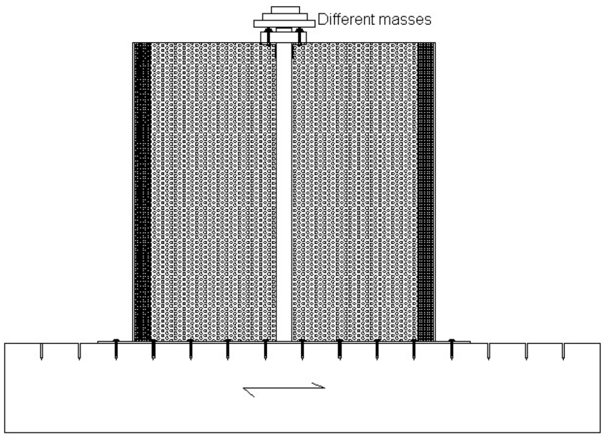

Figure 17. Phase II set-up

III) PHASE III (Fig. 18)

In PHASE III the setup of the test will be little changed digging a hole around the pile and put inside some damping materials. With this set up is possible to study:

-Benefits of distribution of different materials on the soil damping curve;

-Different shapes and depth of hole optimization.

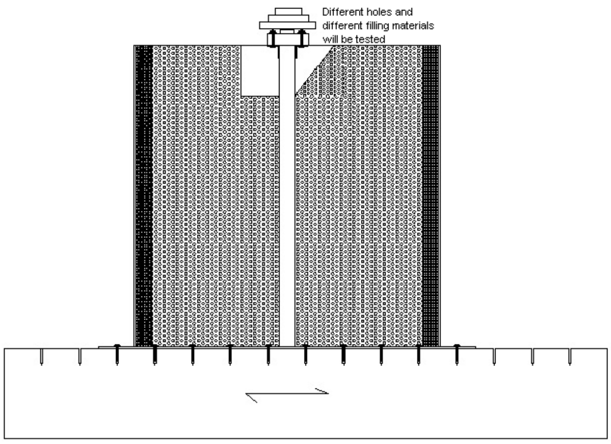

Figure 18. Phase III set-up

IV) PHASE IV

It seems to be important to study the behaviour of the box in direction of the motion in order to validate the FEM model and to get some important information for the following tests.

V) PHASE V

Finally all the collected data of the tests will be checked and compared with software like FEM and spreadsheet in order to have a full picture of what happen along the development of tests.

CONCLUSIONS

The research on Joint-less bridges developed inside the "Sustainable and Innovative Bridge Engineering Research Center" (SIBERC) of Fuzhou University has been briefly introduced in the paper. In particular the maximum length and the optimal piles shape for a typical integral abutment beam bridge has been discussed. More research is needed to investigate the influence of different parameters as: a) bridge geometry in plan: b) spacing of piers (span) and typologies of piers; c) shape of the deck; d) types of foundation; e) soil-structure interaction on the maximum length.

Finally a rigid soil container with absorbing boundaries developed to study the Pile-Soil-Structure Interaction (PSSI) with the so-called kinematic interaction effect and the inertial interaction effect though shaking table tests has been introduced.

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