Proceedings of the 21st ICSMGE, Vienna, Austria, 14 – 19 June 2026. Pistrol, Adam & Schweiger (eds.)
Published by: ÖGG, Austrian Society for Geomechanics, Salzburg, Austria, ISBN 978-3-9503898-4-5
https://doi.org/10.53243/ICSMGE2026-1777

Approaches for seismic soil-structure interaction analysis of a piled railway bridge abutment with fixed bearings

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ABSTRACT: Seismic response analyses of bridge abutments rigidly connected to the deck should account for the interaction between the soil, the abutment and the inertial forces acting on the structural masses participating in the seismic motion. When a specific soil-structure interaction (SSI) analysis is not performed, the Italian Building Code permits a simplified analysis in which pseudo-static forces are applied to the abutment on the basis of a design response spectrum. This approach, which is widely used in current practice, often leads to an overestimation of seismic actions and a very conservative design.
In this paper, for a High-Speed Railway bridge, results from the simplified approach are compared with those obtained from two- and three- dimensional time-history Finite Element (FE) analyses employing a complete numerical FE model that includes the superstructure, its foundation and the surrounding soil. Such analyses, that belong to the category of "direct methods", explicitly account for all key aspects of SSI: kinematic and inertial effects, input motion propagation and local site response.
Soil nonlinear behavior is coupled and described by elastoplastic stress-dependent constitutive law and the seismic input motion is applied to the model boundaries as an appropriate time-history force excitation.
Advantages and disadvantages of each approach are finally discussed, with a focus on the abutment pile foundations.

KEYWORDS: Bridge abutments, Piled foundations, Soil-Structure Interaction, Time-history FE analysis.

1 INTRODUCTION

Bridge abutments rigidly connected to decks should be analyzed in accordance with the Italian Building Code (MIT, 2018) using a structural model that accounts for interaction effects between the ground, the abutment itself and the effective part of bridge superstructure participating to the motion. The Code also specifies that deformability of both foundation soil and backfill should be taken into consideration for the analyses of bridge abutment response along its longitudinal direction.

A simplified alternative procedure is described in the Code where inertial forces acting on the abutment, the ground resting on its foundation and the bridge deck are calculated using the acceleration related to the "plateau" of the design spectrum, assuming a behavior factor, q, of 1.50. A lower acceleration value, equal to the maximum acceleration at ground surface (a_{max}), is allowed in this approach when the structural system composed by the abutment, the ground resting on its foundation and the bridge deck can be considered rigid (i.e. natural period lower than 0.05s).

Design of rigidly connected bridge abutments relies then on either a complex seismic soil-structure interaction analysis or the simplified approach, with the latter often used as part of current design practice which typically leads to overestimated structural actions; this leads to an uneconomical design and a higher carbon footprint.

This paper presents, for the case study described in the next section, a comparison between two- and three- dimensional seismic soil-structure interaction analyses and the simplified procedure proposed by the Italian building code.

The interaction analyses are based on continuum FE models and a time history approach; such analyses belong to the wider category of "direct methods" as they include the ground volume, the approach embankment, the abutment and its foundations as well as the bridge deck. This method allows for a direct simulation of all key aspects, such as seismic waves propagation, radiation effects, kinematic interactions with foundation piles and inertial interaction with the superstructure.

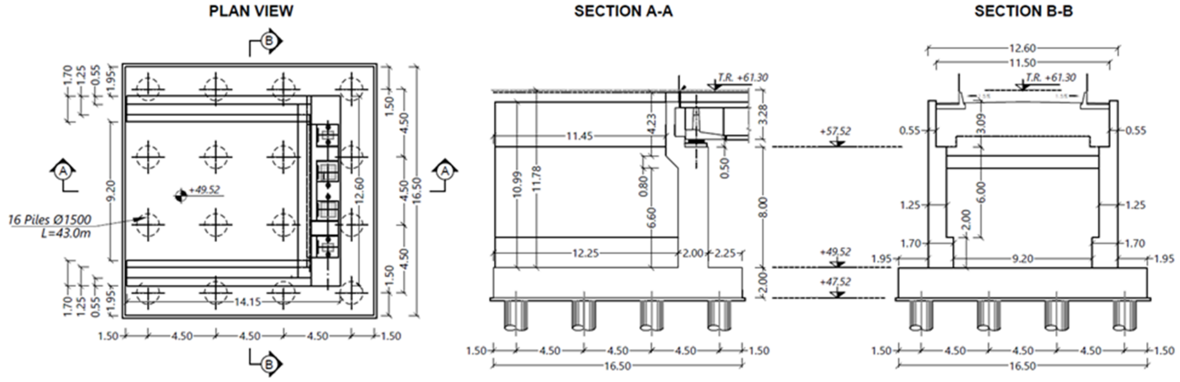


Figure 1. Case study – Abutment plan view, longitudinal and cross section.

2 CASE STUDY DESCRIPTION AND GEOLOGICAL SETTINGS

The case study focuses on a rigidly connected bridge abutment for one of the viaducts under construction on the new Verona-Vicenza-Padova High-Speed Railway.

The abutment is composed of a front wall with 8.0m height and 2.0m thickness, supporting a deck composed of 4no. precast-prestressed beams with a 25m span. The abutment also includes a ballast wall of 3.80m height and 0.60m thickness and two wing walls of variable thickness. The foundations consist of 16no. Ø1500mm bored piles, each 43m long and spaced 4.50m apart, connected to a square cap which has 16.50m side and 2.0m thickness (Figure 1).

About the geological context, the project is located within a Quaternary alluvial deposit in the Venetian Plain, originated by alternations of glacial and interglacial periods, associated to large changes of sea levels and variations of the hydrographic system. Three regions can be identified within this environment – High Plain, where gravelly alluvial conoids are predominant, Middle Plain, characterized by a transition to finer-grained coarse soils interbedded with clayey\silty layers and Lower Plain, reaching the Adriatic Coast, with alternations of granular sandy soil and cohesive sediments. The case study is located in the Middle Plain, in an area where cohesive sediments alternate with comparable thicknesses with gravel layers.

The ground investigations included 2no. boreholes 50m deep with continuous sampling (S52, BH-PE-70), 1no. 10m deep Piezo-Cone Penetration Test (CPTU-PE-39) and 1no. 50m deep Cross-Hole seismic test (CH8).

Gravelly layers were identified at site, alternated with silty--clay layers (Table 1), sometimes containing high percentages of sand and large cobbles (up to 80-100mm). The clayey silts are generally consistent or very consistent and heavily overconsolidated at shallow depths.

Table 1. Design ground profile (OC=Over Consolidated, NC=Normally Consolidated).

Geotechnical Unit	Depth, from (m)	Depth, to (m)
OC silts	0	8.5
Gravels	8.5	17.5
NC silts	17.5	24.5
Gravels	24.5	29.5
NC silts	29.5	33.5
Gravels	33.5	41.5
NC silts	41.5	45.5
Gravels	45.5	100
Bedrock (Seismic)	100	∞

Design ground parameters have been determined on a sitewide basis, using ground investigations result from nearby locations with similar conditions.

Results from Standard Penetration Tests, Cone Penetration Tests, seismic testing (surface and borehole methods) and laboratory testing were interpreted for this scope.

Groundwater levels were estimated from the readings of two piezometers located in the vicinity of the abutment.

As the ground investigations could not directly ascertain the location of the seismic bedrock, its depth was initially estimated to be within the 100-meter to 200-meter range, based on regional geological-hydrogeological settings and ambient vibration measurement (i.e. Horizontal to Vertical spectral ratio, HVSR) seismic tests available in the area. The reference depth was finally set on the basis of sensitivity analyses of local seismic site response, using Linear-Equivalent methods and testing various depths of the seismic bedrock and different ground profiles.

Higher accelerations at ground surface were obtained by assuming shallower bedrock and stiffer soil for bottom layers. Conservatively, the seismic bedrock was thus set at 100m depth and assuming gravelly layers in between the maximum investigated depths and the bedrock.

3 SEISMIC INPUT

Seismic hazard deaggregation was developed on the basis of MPS04-S1 model, whereas historical seismicity was defined using the Parametric Catalogue of Italian Earthquakes (CPTI15) and the Italian Macroseismic Database (DBMI15). Seismogenic sources were identified using the Database of Individual Seismogenic Sources (DISS), version 3.3.0.

Seismic actions were defined from the seismic hazard at rock site for a return period of 1424 years, selecting seven natural and response-spectrum compatible time-history acceleration series, to be used in the longitudinal analyses of the abutment (i.e. the direction parallel to the railway alignment). The vertical component of seismic motion was neglected.

Time-history acceleration series were selected according to current practices; in particular, using the ESM database (https://esm-db.eu), the time histories were manually selected considering magnitude (M = 5.00÷6.50) and distance (R = 2÷62 km) ranges representative of the site's seismicity. Moreover, records were selected on outcropping bedrock (Cat. A) or stiff soil ($V_S \geq 662$ m/s) and scaled for spectra-compatibility matching with moderate scale factors (SF = 0.60÷1.80).

Table 2 summarizes selected acceleration time histories while Figure 2 shows the response-spectrum compatibility achieved, with particular reference to the range of periods T = 0÷1.50s, between the average values of the elastic response spectra (5% damping) of the seven scaled accelerogram selected and target (ground type A) spectra.

The linear-equivalent model implemented in STRATA (Kottke & Rathje, 2009), carried out using total stress analysis in the frequency domain, was used for the deconvolution of the outcrop recorded motion, which was then adopted as input at the base of the model for the time-history analyses.

Table 2. Selected acceleration time-histories and response-spectrum compatibility.

Name	Description and Date	Site and Topographic category	Distance and Magnitude	Scale factor
SRN_E	Greece 2016 (15/10/2016)	A (V_S=1512m/s) T1	62.10km 5.50	0.80
MMO_E	Central Italy (30/10/2016)	A (V_S=891m/s) T1	19.20km 6.50	1.20
MRM_E	Cosenza (IT) (25/10/2012)	A (V_S=1070m/s) T2 (19.7°)	2.40km 5.20	1.80
MSCT_E	Central Italy (18/01/2017)	B (V_S=662m/s) T1	5.50km 5.50	1.10
AQG_E	L'Aquila (IT) (06/04/2009)	B (V_S=696m/s) T1	5.00km 6.10	0.65
SERG_E	Greece 2021 (12/01/2021)	A (V_S=962m/s) T2 (15.7°)	2.60km 5.20	1.30
SVN_E	Sicily (26/12/2018)	B (V_S=686m/s) T1	4.50km 5.00	0.60

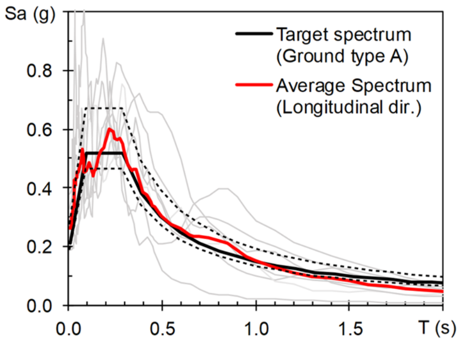


Figure 2. Response-spectrum compatibility plot.

4 NUMERICAL MODELS

4.1 *Geotechnical-Seismic model*

Profiles of shear wave velocities along depth were estimated using the results of two Down-Hole tests carried out near to the abutment. Figure 3 shows a good agreement between in-situ measurements and the trends proposed by Seed et al. (1986) for granular soils and Jamiolkowski et al. (1995) for cohesive soils.

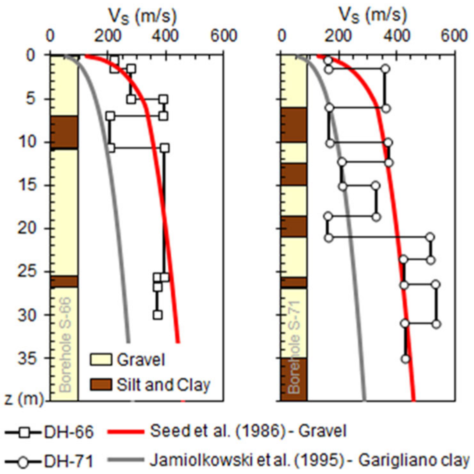


Figure 3. Shear wave velocities – Comparison between Down-Hole test results and two classical literature methods.

Strain dependance of shear modulus and damping were assumed in accordance with curves proposed by Rollins et al. (2020), depending on effective mean stress and particle size distribution of soil, for coarse-grained layers and Vucetic & Dobry (1991), based on soil plasticity index, for fine-grained layers.

The "Hardening soil with small-strain stiffness" (HS-Small) elasto-plastic constitutive model (Benz, 2006) was chosen to simulate soil mechanical response in the soil-structure interaction analyses, which captures nonlinear and hysteretic soil behavior. A one percent fraction of viscous Rayleigh damping was added to account for damping effects associated with small strains vibrations.

Table 3 shows a summary of the constitutive parameters adopted in the SSI analyses. For all analyses, the groundwater level was assumed to be at ground level (i.e. below the embankment base).

Table 3. Geotechnical model – Ground parameters set.

Parameter	OC silts	NC silts	Gravels	Embank.
γ_{sat} (kN/m^3)	19	19	19	20*
p_{ref} (kN/m^2)	40	100	100	5
$E'_{50,ref}$ (MPa)	8	4.8	65	40
$E'_{oed,ref}$ (MPa)	8	3.4	65	40
$E'_{ur,ref}$ (MPa)	24	24	162.5	100
ν_{ur} (-)	0.2	0.2	0.2	0.2
m (-)	0.6	0.6	0.5	0.2
$G_{0,ref}$ (MPa)	43	70	330	150
$\gamma_{0,7}$ (-)	5×10^{-4}	5×10^{-4}	10^{-4}	1.2×10^{-4}
c' (kN/m^2)	30	5	1	1
ϕ' (°)	26	26	39	38
ψ (°)	0	0	0	0
POP (kN/m^2)	220	-	-	-

*Natural Unit Weight

4.2 *Case Study – Detailed Design model*

Detailed Design of the abutment had been developed by means of a structural three-dimensional model, built with the software Midas GEN. Static, both permanent and variable, and pseudo-static loads acting on the abutment had been estimated in accordance with the 2008 edition of the Italian Building Code (MIT, 2008). A code-compliant response spectrum was used to evaluate seismic actions, based on the following assumptions: T1 topographic category, ground type C, and a Lifesaving limit state for a return period of 1424 years.

Seismic earth pressures had been estimated by means of the Mononobe-Okabe theory, using a peak acceleration "a_{max}" equal to 0.295g, also used to compute the inertia forces due to the self-weight of the abutment, the backfill and the deck masses. The effects of permanent displacements on reducing accelerations had not been accounted for (i.e. $\beta = 1.0$).

Furthermore, the structural model adopted in the Detailed Design did not include interaction effects between the abutment raft and the ground (i.e. load carried only by piles). This is a usual design practice assumption for such structures.

A Linear-Elastic spring model had been adopted for the analysis of the pile foundation, with global actions acting at the piles head level. A fully fixed connection between piles and the abutment raft had been assumed and axial loads were calculated based on rigid raft assumption.

4.3 *Two- and three- dimensional FE models*

Soil-structure interaction phenomena were initially evaluated through time-history FE analyses performed on the basis of a two-dimensional plain strain model built in Plaxis 2D v. 2023.2.

The model's dimensions, as reported in Figure 4, are 200 meters in width by 100 meters in height, with an additional 11.80-meter embankment.

The total number of elements is 24'959 and the total number of nodes is 51'269.

Triangular elements with 6no. nodes and quadratic shape functions were used to model the ground, the embankment, the front wall of the abutment and the raft foundation, whereas "embedded-beam" elements were used for the piles. "Dummy" beam elements were added to the key structural parts of the model for a more accurate assessment of structural actions.

An uncracked elastic modulus was used for concrete, together with a 2.50% viscous Rayleigh damping.

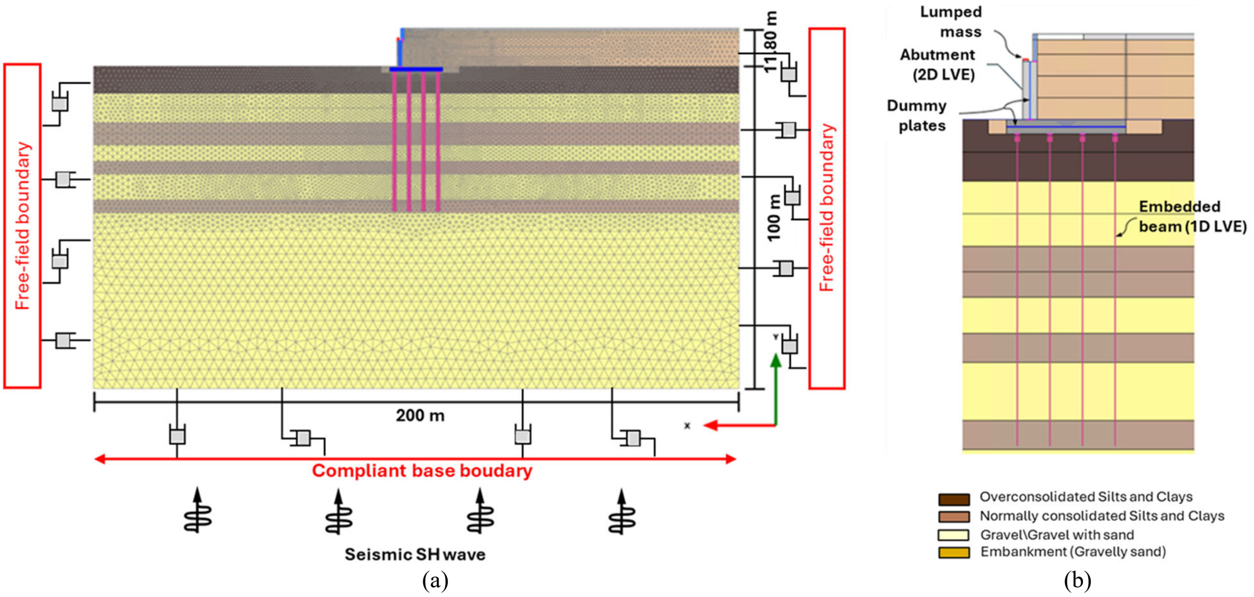


Figure 4. (a) Two-dimensional numerical model and (b) detail of abutment structure and its foundations.

"Free-field" conditions were assigned along the vertical boundaries of the model, in order to simulate radiation effects and mitigate seismic waves reflections. A "compliant base boundary" condition was selected at the base of the model both to absorbing incoming waves and allow conversion of the kinematic input into equivalent nodal forces.

External loads were included in the model to account for viaduct deck and ballast dead weights and trains variable loads.

Numerical analyses were also carried out using a three-dimensional finite element model of the whole soil-foundation-structure system, built in PLAXIS 3D v. 2023.2.

The three-dimensional model is shown in Figure 5. It is composed by 353'783 volume elements and 512'017 nodes. Model dimensions are 300m (width) × 40m (depth) × 100m (height, to bedrock), plus the embankment height, equal to 11.80m. For computational efficiency, only half the system was modeled, thanks to geometric and loading symmetry.

10-node tetrahedral volume elements with quadratic shape functions were adopted to model the ground, embankment, abutment raft and foundation piles, whereas 6-node bi-dimensional plate elements (triangular) were used for frontal, ballast and wing abutment walls.

Similarly to the two-dimensional model, a "compliant base" condition was assigned at the base of the model.

About the lateral boundary conditions, some issues were observed with the combined use of standard "Free field" conditions and HS-small constitutive model, which led in excessive bulging of the vertical boundaries. An efficient correction was identified by means of a "non-collapsible" ground column, added to both ends of the model and capable to limit the permanent deformation of the lateral boundary within an acceptable level, while obtaining close response to the nonlinear one-dimensional site response analysis.

Average stiffnesses values were assumed within these zones (i.e. m = 0) and hysteretic behavior was preserved using the same constitutive model of the adjacent soil. Moreover, a tensile strength and cohesion both of 200 kPa were assumed, for the columns to behave "elastically" and prevent them from collapsing.

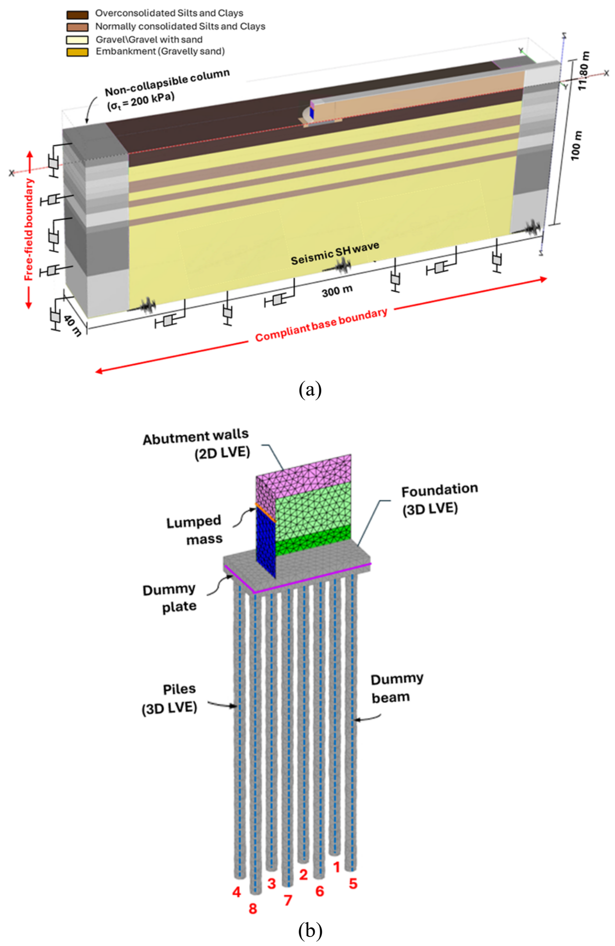
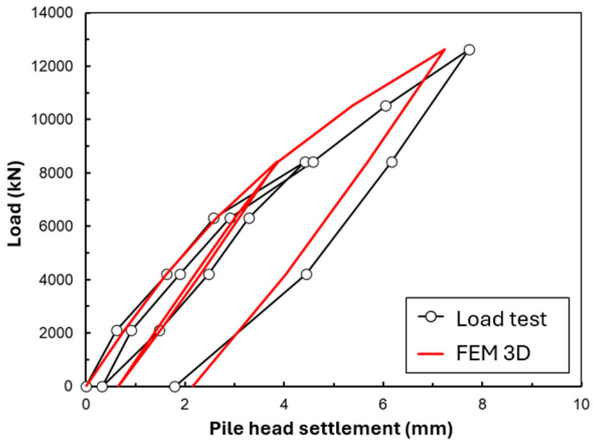


Figure 5. (a) Three-dimensional numerical model and (b) detail of abutment structure and foundations.

The soil parameters for the HS-Small constitutive model (Table 3) and the load-displacement numerical response of the three-dimensional pile model were validated against a preliminary pile load test conducted near the abutment. A close match was obtained, with the results shown in Figure 6.



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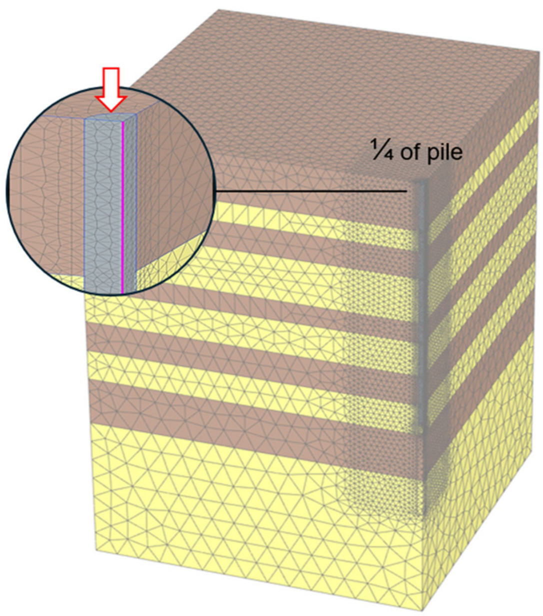


(b)

Figure 6. (a) Comparison between three-dimensional pile numerical model and preliminary pile load-test (b) Detail of three-dimensional pile verification model.

To ensure the most realistic representation of the initial static condition, all construction phases were accurately modeled in both the 2D and 3D simulations before proceeding with the abutment's dynamic response analysis.

Piles are assumed to be wished in place so the soil disturbance associated with their construction is neglected.

Table 4 reports, schematically, the construction sequence.

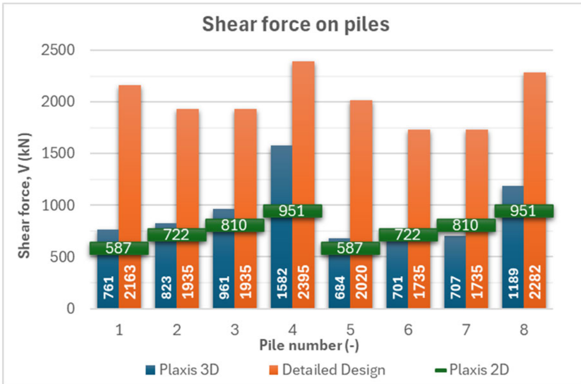
Table 4. Principal phases of the construction sequence adopted in FEM models

Phase no.	Description
1	Initialization of in-situ stresses
2	Realization of the part of the embankment external to the active wedge
3	Piles construction
4	Raft and walls realization
5	Completion of the embankment and backfill
6	Application of the loads transferred from the deck
7	Non-linear dynamic time history analysis

5 RESULTS

Results from the two- and three- dimensional numerical analyses (average of seven time-history analyses, in accordance with §7.3.5 of the Italian Building Code, 2008 edition) were compared with Detailed Design outcomes. The main focus was on foundation piles and, limited to the three-dimensional model, on the front wall of the abutment.

For piles, maximum shear and bending moment calculated with the two-dimensional analyses were found to be respectively 60% and 38% lower than Detailed Design. For the case of three-dimensional analyses, a reduction of 34% was obtained for the maximum shear and 22% for the maximum bending moment (Figure 7).



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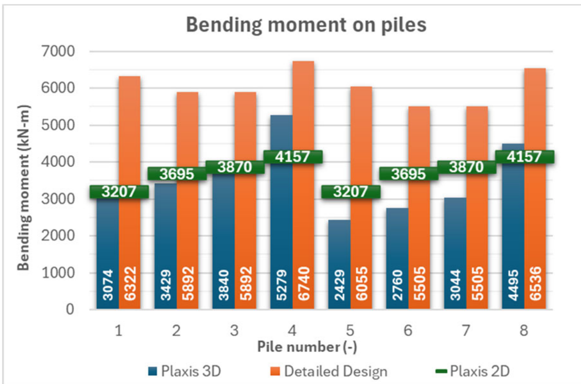


(b)

Figure 7. Maximum calculated actions on single pile (a) shear force (b) bending moment. For Plaxis 2D actions refer to pile rows.

The difference in the actions estimates between two- and three- dimensional analyses was explained with the more realistic geometry that can be accounted by a three-dimensional model. Specifically, a two-dimensional approach cannot accurately take into account the interaction among neighboring piles in the out-of-plane directions or the inertia forces associated with the embankment slopes. For this reason, results obtained from the three-dimensional models should be assumed as the reference for comparison with the simplified procedure adopted in the Detailed Design.

Comparisons were also conducted on the maximum shear force and bending moment at the base of the abutment's vertical wall. The three-dimensional analyses revealed values that were 50% lower than those from Detailed Design, indicating a significant difference in the results of the simplified procedure and the complete numerical analyses.

Key differences to be considered in the comparison between time-history numerical analyses carried out in this work and the Detailed Design include:

a) Detailed Design seismic actions are "pseudo-static" and do not account for either soil-structure interactions or the dynamic response of the system;

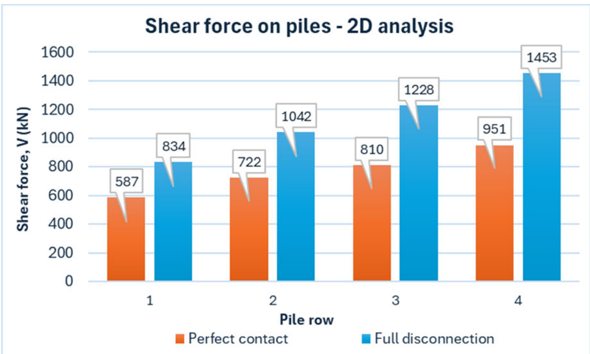
b) Detailed Design seismic input had been based on a code compliant response spectrum, whereas our analyses and the direct approach implemented therein accounted for a re-evaluation of local site response, which in this particular case led to lower accelerations than prescribed by the code;

c) interaction effects between the abutment raft and the ground are modelled explicitly in our direct approach, which lead in the calculations to load sharing between the piles and the raft. This effect is generally neglected in the current design practice.

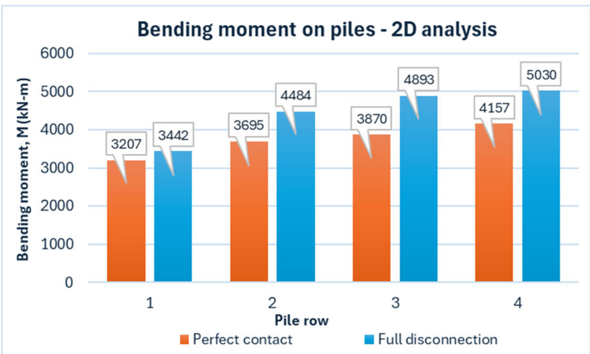
Point c) and in particular the load sharing between abutment raft and piles was investigated through two-dimensional sensitivity analyses of various approaches for the simulation of the interaction between abutment raft and the soil:

1. perfect contact between soil and raft, i.e. no interface included in the model. This was the approach used throughout our two-dimensional and three-dimensional analyses;
2. insertion of an interface element with coupled nodes, assuming first a "rigid" interface capable of mobilising the full strength of the ground (i.e. R_{int} parameter set equal to 1.0);
3. same as point 2, assuming a practically smooth and deformable interface (i.e. R_{int} parameter set equal to 0.1);
4. full disconnection between raft and ground, through implementation of a finite gap (i.e. thin cluster of soil de-activated in the model).

The sensitivity study confirmed the important role of the abutment raft. Figure 8 shows the comparison of cases 1 and 4 (i.e. perfect contact vs. full disconnection) in terms of maximum shear and maximum bending moment on single pile, indicating actions reductions in the order of one third for the case of perfect contact between raft and soil compared to the model featuring a gap.



(a)



(b)

Figure 8. Sensitivity analysis of load sharing between abutment raft and piles (a) shear force (b) bending moment.

6 CONCLUSIONS

A comparison was presented in this paper between the seismic actions calculated from a simplified method and a complete FEM-based time-history soil-structure interaction analyses. The reference case study was a piled railway bridge abutment with fixed bearings, part of a High-Speed railway project currently under construction in Northern Italy.

The pseudo-static simplified method had been adopted for the Detailed Design. The time history analyses were two-dimensional and three-dimensional and made use of a direct approach, in which the numerical model included the ground volume, the abutment, its pile foundations, the approach embankment and the bridge deck masses.

Actions obtained from the direct approach were found to be significantly lower than those calculated by the simplified method used in the Detailed Design.

With reference to the base of the abutment vertical wall, shear forces and bending moments showed reductions in the order of 50%.

For pile foundations, two-dimensional analyses showed reductions of 60% for the maximum shear and 37% for the maximum bending moment, whereas three-dimensional analyses showed reductions of 34% for the maximum shear and 22% for the maximum bending moment.

Three-dimensional analyses were found to be onerous, in terms of model calibrations effort and computing time for the series of seven time-history runs, which are required by the Italian Building Code in case average values of the actions are sought. Their use appears justified for those cases where structural design is more demanding, either for the high seismicity of the site or for the large site amplifications expected from the local ground profile. In these cases, the benefits from design optimisation would outweigh the larger computational cost.

Finally, the important contribution of the abutment raft in sharing the load with the foundations piles was confirmed by a sensitivity study.

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