Ministerul Educației și Cercetării
Universitatea Tehnică de Construcții București
Facultatea de Căi Ferate, Drumuri și Poduri

THE SEISMIC RESPONSE OF THE TERRAIN - FOUNDATION - STRUCTURE ENSEMBLE

RESEARCH REPORT NR. 2

PhD Student
Ing. Mircea Conțiu

Scientific Coordinator
Prof. Dr. ing. Dan Crețu

BUCUREȘTI 2020

1. INTRODUCTION

Most types of civil engineering structures are in direct contact with the soil, and therefore their behavior is affected the soil dynamic properties. This soil-structure interaction (SSI) between the ground and the structure is important during earthquakes, because it may change the seismic behavior of the structure completely. In some cases, the consideration of SSI may be favorable, reducing the forces on the structure and, thus, the execution cost, or it could be unfavorable and not considering it could have devastating results.

The goal of this paper is to compare the current practice in the seismic design of bridges, according to Eurocode 8, and the state-of-the-art seismic SSI analysis methods, developed specifically for nuclear structures, which include sophisticated models for SSI and for the seismic action itself, using measured data to estimate the incoherent behavior of seismic waves. For this comparison, the newly constructed Fartec bridge model is used, as described below.

2. THE FARTEC BRIDGE

2.1. DESCRIPTION OF THE BRIDGE STRUCTURE

The structure being studied is the Fartec Bridge in Brasov, Romania, which has been designed according to Eurocode 8 provisions. The concrete bridge superstructure is a 242 m long continuous girder deck composed of precast prestressed beams connected by a concrete slab and with cast-in-place concrete diaphragms on the supports.

The deck is divided into 3 continuous structures with expansion joints between them: structure 1 is composed of 2 x 29.15 m spans, structure 2 has 4 spans of 29.15 m + 2 x 31.50 m + 29.15 m and structure 3 is the same as structure 1. Even though the deck is not continuous through all of its length, the pier bearings are permitted only a small longitudinal deformation, due to temperature variation. After that, all bearings are fixed, which means that for earthquake loading all bearings may be considered fixed from the start. The effect is the same as a completely continuous structure.

Fig. 1 Podul Fartec din Brașov, Romania

The piers are composed of 2 circular columns of 1.50 m diameter and are connected by a pier cap with a rectangular section of 2.50 m x 1.20 m. The pier height varies from 6.50 m to almost 9.00 m. Both the pier and abutment foundations consist of 6 drilled piles 17.00m long connected by a pile cap. The geotechnical study showed that the soil on site is a mixture of gravel and sand.

This type of bridge structure is typical for Romania and highly used throughout the country.

2.2. STRUCTURAL MODELS AND COMPARATIVE ANALYSES

In order to obtain the results for the intended comparison, two bridge models have been made. For the structure itself, both models are equivalent; linear beam and shell elements are used to model the behaviour of the structural components. The bridge was designed for ductile behaviour, reducing the seismic forces through the dissipation of energy by the formation of plastic hinges at the bottom of the pier columns. The nonlinear behavior of the structure (i.e. formation of plastic hinges) is considered by an equivalent linear analysis through a verified and generally accepted method: the reduction of stiffness by 50% for the elements where the plastic hinges would occur, in this case the pier columns. Due to the high loads an earthquake generates, the other infrastructure elements are considered to work in cracked state, so their elastic stiffness has been reduced to about 80% of the concrete uncracked stiffness by the direct reduction of the modulus of elasticity.

Between the two bridge models, there are differences in the soil – structure interaction methodology and in the way the seismic loads are applied. Detailed explanations are given below.

The first model is the simplified one, which was used for the actual design of the bridge. It was made according to Eurocode methodology and Romania’s current bridge and foundation design practice. The interaction between the foundation and the soil has been considered through the “beam on elastic foundation” model, also known as the Winkler model. Fig.2 is an extract from the Romanian code regarding the geotechnical design of deep foundations on piles, NP 123:2010. The code explains that for the calculation of deformations and stresses along the isolated pile, which is defined in an axis system (Fig.2 a) and loaded transversely (shear forces and bending moments) the soil may be assimilated with a discrete Winkler medium (Fig.2 b) composed of independent springs. The stiffness of the springs varies linearly with the depth. This will generate a transverse pressure diagram similar to the one presented in Fig.2 c.

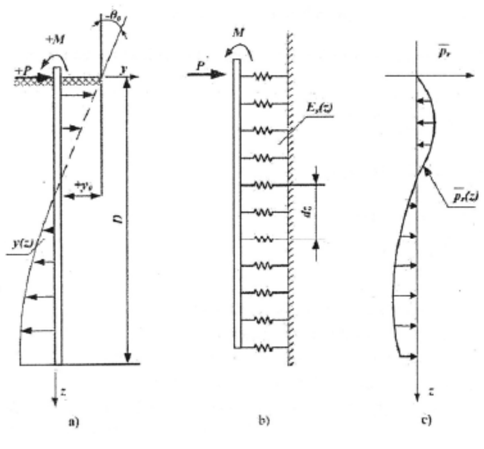
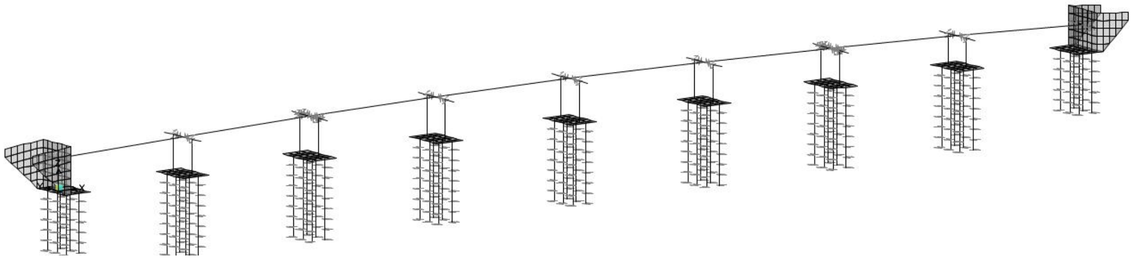


Fig. 2 Modelul de interacțiune pilot – teren, conform Normativului privind proiectarea geotehnica a fundațiilor pe piloți, indicativ NP 123:2010

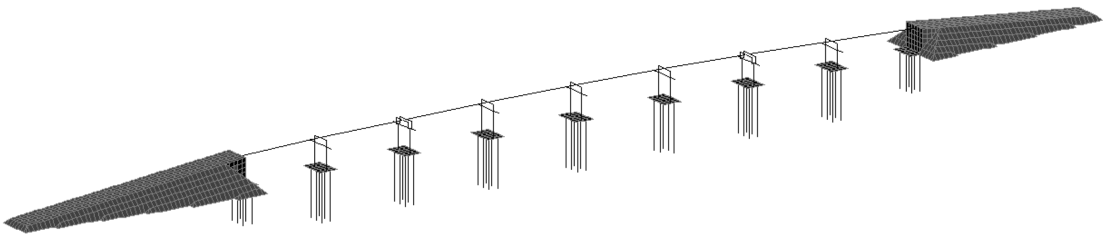
Using the SSI methodology presented, the first model has been made, and is presented in Fig.3 a. The seismic loading acting on the structure is introduced a uniform ground acceleration, acting at all structure support points. For this model, three linear time-history analyses have been made: one for each orthogonal direction.

The seismic action of the earth fill behind the abutments has also been considered through the simplified model of constant or linear varying seismic soil pressures, according to the Mononobe - Okabe theory.

Additionally, as per the simplified model presented in Eurocode 8, the spatial variability of the seismic action is considered thorough the calculation of static differential displacements at each foundation in two sets of values: set A considers a continuously increasing displacement in the same direction at each foundation starting at one end of the bridge and set B considers displacements in opposite directions for adjacent foundations.



a. Modelul de proiectare al podului



b. Modelul SSI al podului

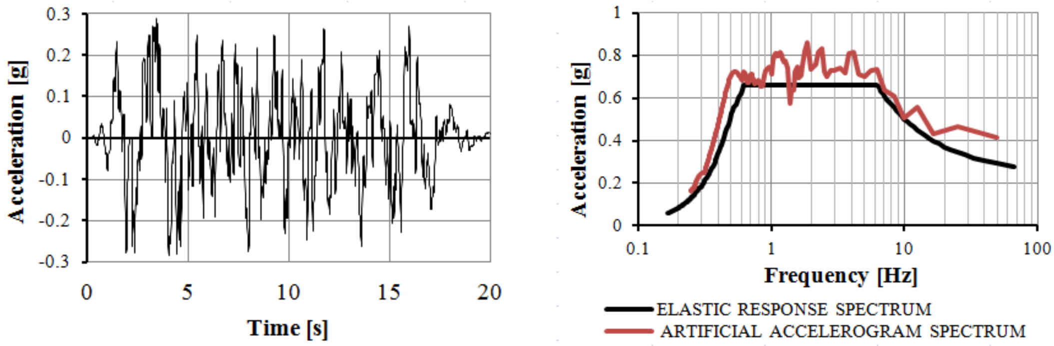
Fig. 3 Modelele folosite în prezentul studiu

Fig.3 b presents the second model, the bridge SSI model. The site is modelled as multiple horizontal layers overlying a uniform visco-elastic half space at considerable depth. All soil material properties are assumed to be visco-elastic. The analysis uses a complex frequency domain solution, so only linearized systems may be used. Still, the non-linear hysteretic behaviour of the soil can be approximated using the Seed – Idriss equivalent linear procedure (Seed and Idriss, 1970), and the soil effective properties (shear stiffness and hysteretic damping) are computed for each layer separately. The ground elevation is considered at elevation 0, which corresponds to the elevation of the pile caps and the bottom of the soil fills behind the abutments. All points at or below elevation 0 are considered interaction points with the site horizontal layers. As can be seen from the picture above, the backfill soil is modelled through solid elements, which have the mechanical characteristics corresponding to the type of soil used (normally gravel).

The seismic loads are modelled through different types of wave fields acting in different directions: for the X direction (bridge longitudinal direction) SV-waves have been considered, for the Y direction (bridge transverse direction) SH-waves and for the Z direction (vertical direction) P-waves. The first set of SSI analyses were completed for a coherent seismic input.

The spatial variability of the ground motion in horizontal plane in the vicinity of the structure is considered through the use of a stochastic field model. Assuming a homogeneous and isotropic Gaussian stochastic field, the spatial variability can be defined by the coherency function. The coherency function direction may be considered arbitrary, due to the assumption of the isotropic stochastic field. The isotropic coherency function is one-dimensional, meaning only the distance between two points is considered, and not the plane location orientation. Abrahamson (1993, 2005, 2006, 2007) has created various statistical database of seismological information recorded in many dense arrays, and generated empirical plane-waves incoherency models for multiple soil conditions and foundation types, through the formulation of the coherency function. The 2005 Abrahamson Abrahamson plane-wave coherency model for all sites was used for this study (Ghiocel 2007, 2013). In addition to incoherency effects, a wave passage effect corresponding to an apparent wave velocity of 1,300 m/s along the bridge was included. Using the ACS SASSI stochastic approach, a number of 15 incoherent simulation SSI analyses were performed for each soil condition.

Both bridge models consider an uniform soil profile with the average characteristics between dense sand and gravel.



a. Accelerograma ariticială **b. Comparaia spectrelor**

Fig. 4 Input seismic

Both models have been subjected to the same input accelerograms, which were generated from the elastic response spectrum of the bridge site. The horizontal accelerogram is presented in Fig.4 a; in Fig.4 b the site elastic response spectrum and the artificial accelerogram spectrum are plotted. For the course of this study, whether the accelerogram is more severe than the actual elastic response spectrum is of no consequence, because it is this same accelerogram that is applied to both models and these results are compared.

2.3. Results and comparisons

The main results that will be compared are the structural stresses for the pier columns and their corresponding piles. A side pier and the mid pier (the one right in the middle of the structure) will be used; they have been chosen because of their height: the side pier is the shortest and stiffest one (aprox. 5.9 m) and the mid pier the highest and most flexible one (aprox. 8.0 m).

To simplify the graphs and explanations in this section, the following notations are used. For the Eurocode design model, the time-history analysis is abbreviated DESIGN and the analysis including the simplified design-oriented spatial variability effects SPA_VAR. As for the SSI analysis model, the coherent analysis is abbreviated COH and the 15 incoherent simulation SSI analyses INCOH_RUNS. From these incoherent runs, average SSI responses were calculated, which are called INCOH_MEAN. When the symbols X, Y or Z are specified, they represent the earthquake input direction.

“M” symbolizes bending moment, “V” shear force and “N” axial force. The local beam directions are as follows: 1 is the axial direction , 2 is the longitudinal direction of the bridge and 3 is the transverse direction to the bridge superstructure axis.

The results are presented in graphs of stresses on the category axis and elevation on the value axis. Elevation 0 represents the passing point between the pier column and the piles; anything above it belongs to the pier column, anything beneath it to the piles. The horizontal axis scale is different for the pier column and pile stresses; they are places above and below the graphs.

Fig.5, Fig.6 and Fig.7 show the bending moment 3, the shear force 2 and the axial force for the earthquake load acting in the X direction; Table 1 shows the same comparison, also giving the percentage increase in stresses for SPA_VAR to DESIGN and INCOH_MEAN to COH.

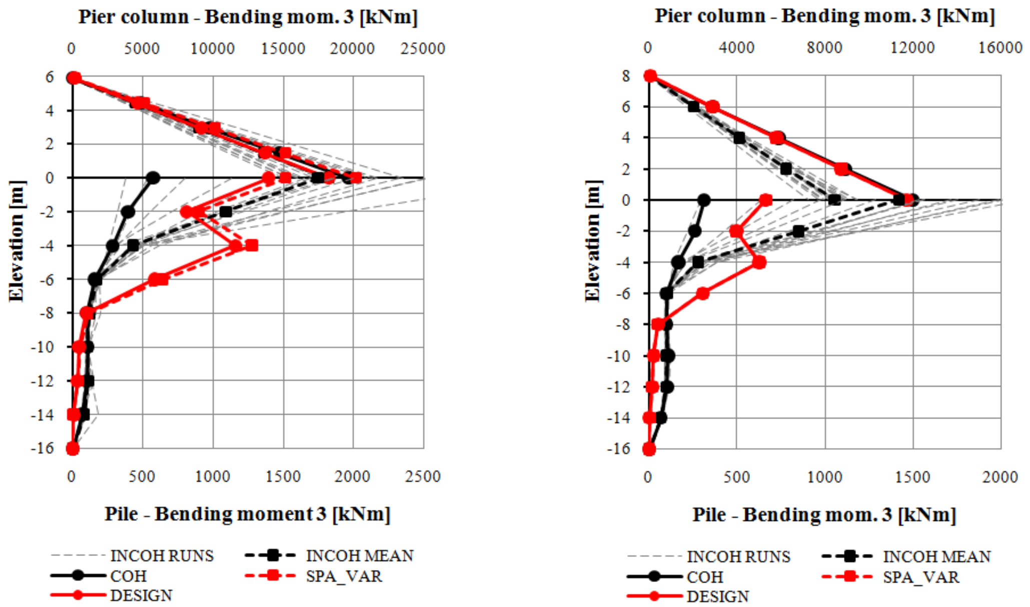


Fig. 5 Momentul încovoietor 3 pentru solicitarea seismică pe direcția X

(stânga = pila de capăt, dreapta = pila centrală)

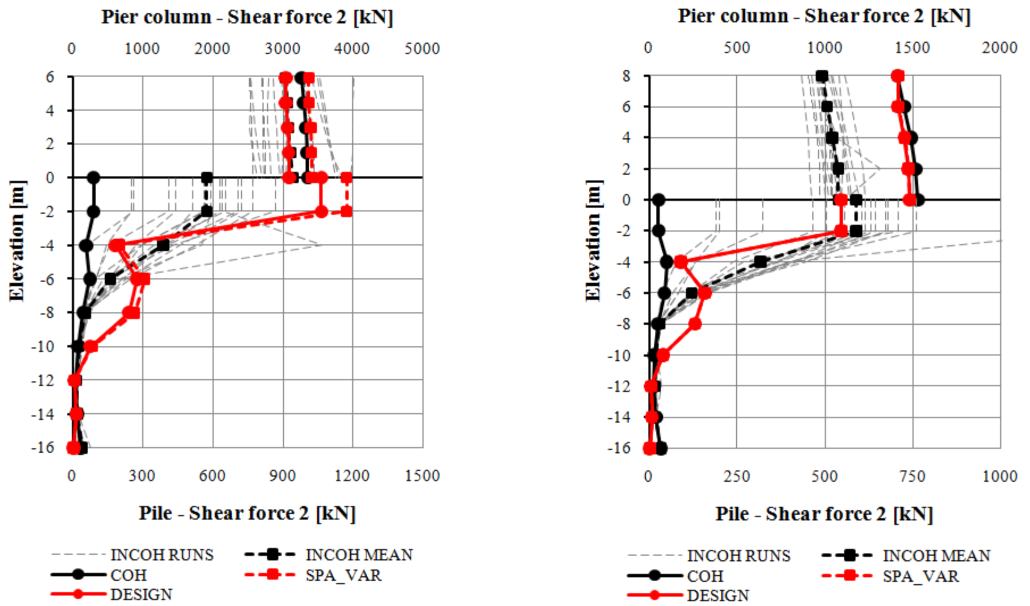


Fig. 6 Forța tăietoare 2 pentru solicitarea seismică pe direcția X

(stânga = pila de capăt, dreapta = pila centrală)

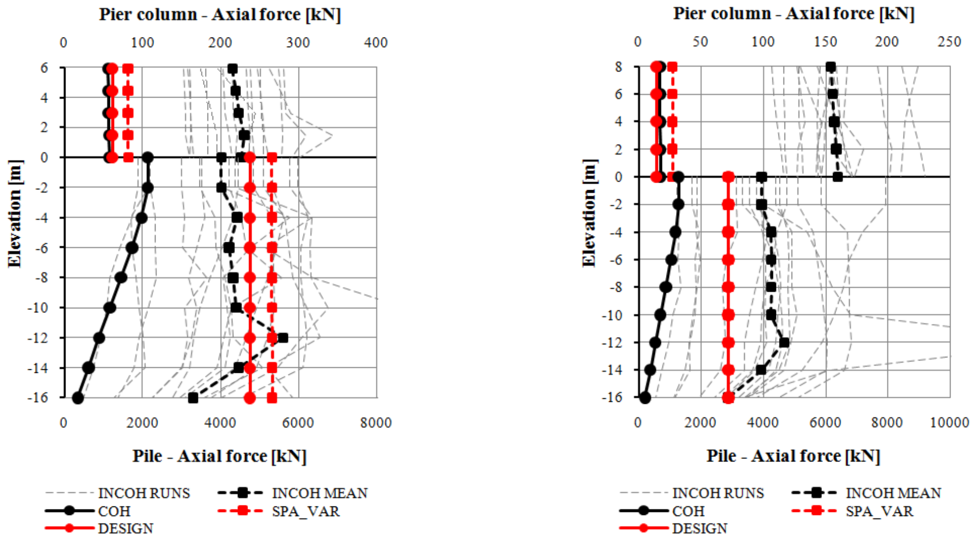


Fig. 7 Forța axială pentru solicitarea seismică pe direcția X

(stânga = pila de capăt, dreapta = pila centrală)

When comparing the bending moments and shear forces for both piers, the first thing that is noticed is that the pile diagrams for the Eurocode 8 SSI model and the refined SSI model are different. Normally, in the current design practice, the pile's maximum bending moments would be located at a depth of about 2 or 3 times the piles diameter. In the refined SSI model, the maximum bending moments are always located at the top of the piles.

The spatial variation simplified approach according to Eurocode 8 does not produce any kind of significant effect on the structure. The incoherent SSI approach may increase structural stresses from a coherent analysis 10- or 20-fold.

For the piles of the shorter, stiffer, end pier, the design model overestimates the bending moments and shear forces, almost 2 times. For the piles of the more flexible mid pier, the response of the design model is underestimated.

The pier elevation shows a different behaviour: for the stidd end pier the results are quite similar, but for the flexible mid pier the design model overestimates the structural response by 30-50%.

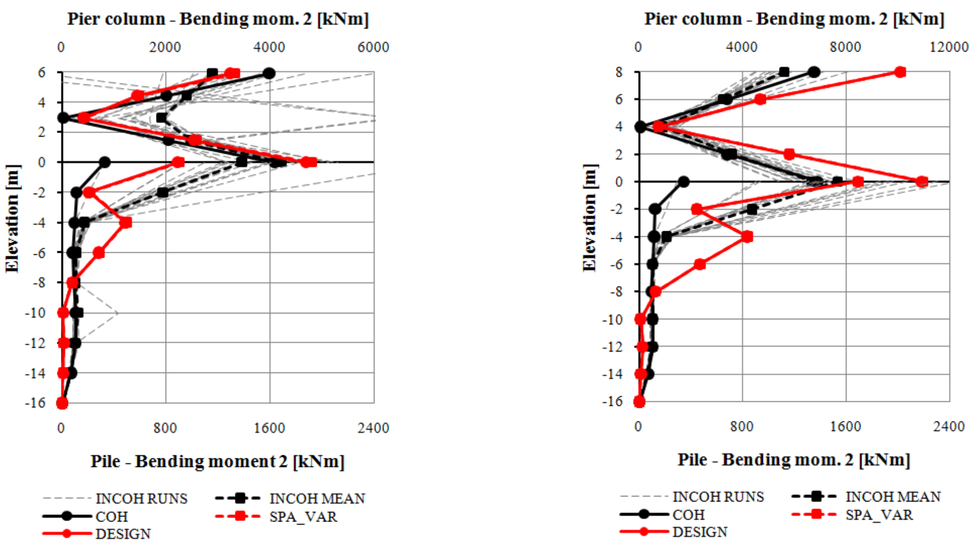


Fig. 8 Momentul încovoietor 2 pentru solicitarea seismică pe direcția Y

(stânga = pila de capăt, dreapta = pila centrală)

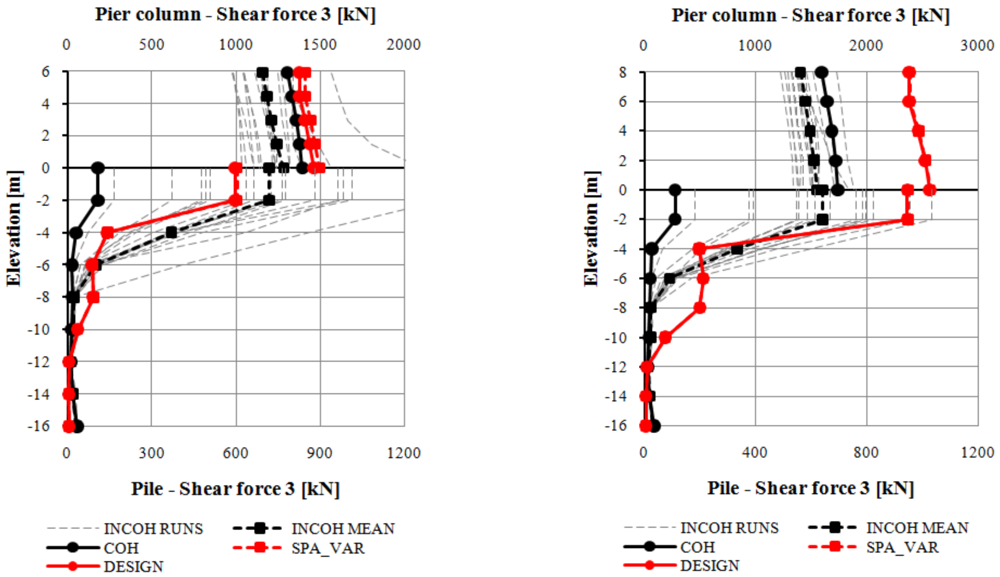


Fig. 9 Forța tăietoare 3 pentru solicitarea seismică pe direcția Y

(stânga = pila de capăt, dreapta = pila centrală)

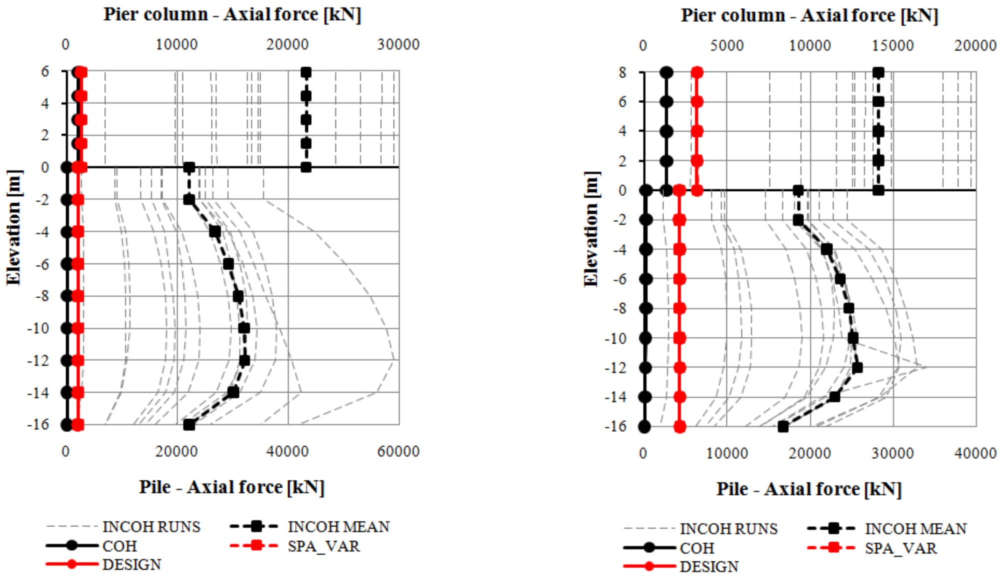


Fig. 10 Forța axială pentru solicitarea seismică pe direcția Y

(stânga = pila de capăt, dreapta = pila centrală)

Fig.8, Fig.9 and Fig.10 show the bending moment 2, the shear force 3 and the axial force for the earthquake load acting in the Y direction. In this direction, the earthquake generates a much more rigid bridge response, as the foundations are larger and have three rows of piles; the piers columns and the pier cap act together as a frame.

In this direction, the simplified method for spatial variability of the seismic action generates an increase of only 2-3%, which is nothing compared to the increase due to motion incoherency in the SSI model.

In this direction, the difference in stresses between the design and the SSI model are not so large: the design model underestimates the stresses in bending by 30% for the end pier piles, while, at the same time, overestimating the shear forces in the piles of the center pier.

The analyses show a very large increase in the axial forces for both piles and pier columns and for both locations, meaning the side and mid pier. The axial forces in the piles come from both the vertical seismic component, but mostly from the pier bending. These very large values are the result of the elements being very stiff in the axial direction and the adjacent soil model behaving completely elastic. Any vertical displacement in the soil would generate extreme stresses in the elements in the axial direction. This problem will be treated better in the next step of our study, but for now these results may be ignored.

Fig.11 and Fig.12 show the axial force and the shear force 2 and the shear force 3 for the earthquake load acting in the Z direction. The axial forces from the INCOH_MEAN analysis show a 2-fold increase, but the values are not large enough to actually cause any problems for the structure.

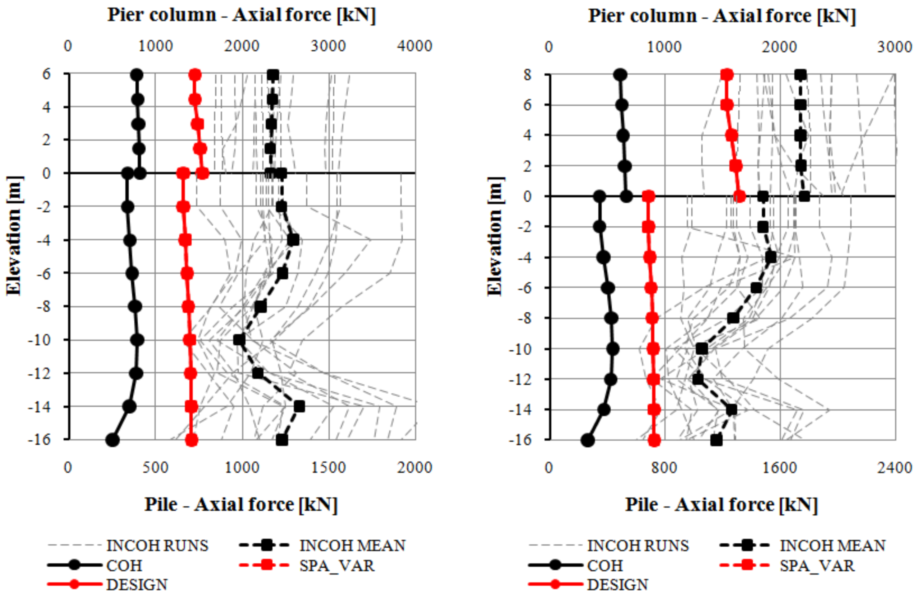


Fig. 11 Forța axială pentru solicitarea seismică pe direcția Z

(stânga = pila de capăt, dreapta = pila centrală)

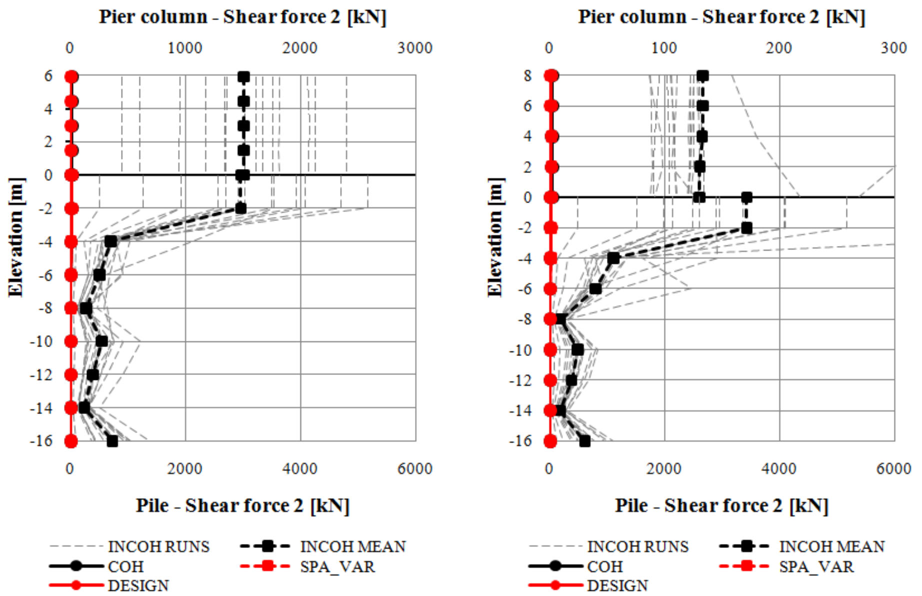


Fig. 12 Forța tăietoare 2 pentru solicitarea seismică pe direcția Z

(stânga = pila de capăt, dreapta = pila centrală)

An aspect of concern is that the coherent input assumption, for both the current design model and the refined SSI model produces unconservative results for the seismic motion component in the vertical direction. In reality, due to spatial variability of soil motion and presence of other waves than vertically propagating waves simulated by incoherent motion simulations, the shear forces and moments in the structure, i.e. in the piles and pier columns, are much larger. The DESIGN and SPA_VAR stress results were virtually zero forces and moments for the coherent vertical seismic loading. Realistically, due to motion incoherency there are some rotations in the isolated foundations that are not considered if only coherent, vertically propagating waves are considered. However, it should be noted that the incoherent SSI results are obtained assuming full pile-soil contact and elastic concrete pile and soil properties. If some local pile damage or local soil plasticity effects occur, then, the large shear forces and moments may go considerably down. Such an nonlinear SSI investigation is planned as a future study.

Fig. 13 shows two acceleration plots of the structure for the incoherent motion. The first one is an elevation view of the bridge and the second one a plan view. It can be seen that the bridge is not subjected to uniform acceleration in the same direction. The shape of the plots seems “broken”, especially for the piles. Piles are subjected to variable accelerations along their length due to the the differential soil motions along the piles produce by incoherent waves. It can be noticed that the superstructure itself is subjected to a strongly variable acceleration along its length, both in horizontal and vertical direction. Another interesting effect is the fact that due to the long wavelength seismic waves for which the bridge moves almost rigidly.

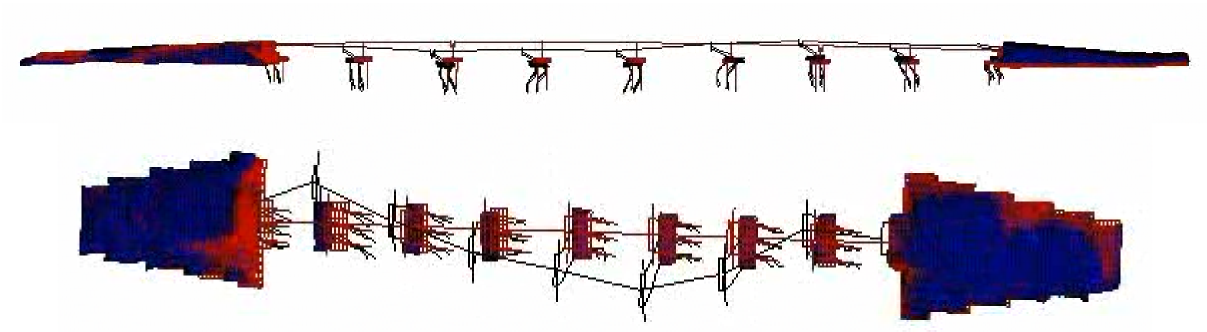


Fig. 13 Accelerații incoerente instante ale structurii podului în elevație și plan (albastru este forma deformată, roșu cea nedeformată)

The following table compares the maximum relative displacements for two consecutive foundations between the simplified spatial variation model in the Eurocode and the average value obtained from the incoherent analyses. It is clear that the simplified model underestimates the actual relative displacements that occur between the structures foundations. The INCOH_MEAN value represents the difference between the absolute maximum displacements of each foundation.

	SPA_VAR	INCOH_MEAN
Consecutive foundations relative displacements [mm]	7.54	14.45

2.4. Conclusions and future directions

The overall conclusion is that the design can generally be considered acceptable if coherent seismic inputs are used. In most cases, the Eurocode 8 design was conservative, and when the design was exceeded, it was only by a small amount, which can be included in the safety factors.

However, the refined incoherent SSI model results show a different behavior for the piles than the simplified Eurocode design model. As shown in the previous analyses, for the Eurocode based model the maximum bending moments in the pile are obtained at a depth of approximately 2-3 times the diameter of the pile; for the refined incoherent SSI model, the maximum value was always located at the top of the pile. It seems that the code-based models overestimate the stiffness at the top of the pile (near the pile cap).

The simplified earthquake spatial variation model given in the Eurocode 8 influences the structure very little. While it may give strong responses to very stiff structures, for this bridge, that is quite flexible, it’s effect was at most 10% for the X direction and at most 2% for the other directions; the effect in the longitudinal direction was larger because the superstructure transfers the load axially and gives a very stiff response. Another issue of the simplified design model is that the procedure fails to capture the strong shear forces that the piles are subjected to due to the incoherent differential displacements in the soil. The procedure is to introduce a single displacement values per each foundation, which means that the whole foundation will move at the same time generating no stresses along the length of the pile.

The current design bridge model gives quite a large response for coherent seismic action than the refined SSI model, especially for the piles. These values are slightly influenced by the simplified spatial variation model, whereas the refined SSI model responds strongly to incoherent seismic input. Due to the large coherent response, the design model values are relatively close to the incoherent SSI model. No clear answers for generic situations, as to whether the design model overestimates or underestimates the structural stresses, can be given based on the limited investigated case study results. In this study, the refined SSI model produced a reduced response for the more flexible structural elements, e.g. the mid pier, and was very close to the design model for the more rigid elements. The only thing that can be clearly stated is that the behavior of the current design practice models does not capture all the effects displayed by the state-of-the-art SSI model. The seismic bridge design community should seriously consider the incoherent SSI aspects to avoid tragedies or reduce construction costs.

As a future direction of the study, a more refined bridge model will be used, that will better simulate the behavior of the bridge, e.g. for the superstructure. Other high priority issue to be addressed is related to the local soil nonlinear SSI effects around the piles, including the local plastification of the soil material in the immediate vicinity of the piles.

3. A DETAILED STUDY OF THE FOUNDATION

After the initial study on the complete Fartec bridge, two clear directions have been highlighted:

- The use of a more refined model, especially for the superstructure
- Taking the nonlinear soil behaviour in the immediate vicinity of the structure into account

The first task doesn’t require any further studies. The second one is undoutably more complicated. Due to its high complexity, the issue will be divided into several steps.

Because the model of the whole bridge would have been extremely large, a single bridge pier will be studied in detail. Of course, the corresponding superstructure mass has been added at the top of the pier. The results that will be compared are:

- Response spectra at superstructure level

- Displacements at superstructure level
- Piles and pier columns internal forces (bending moments, shear forces)

The first modelling decision is the substructuring method. The flexible volume method (FV) is the most precise, but due to the large number of interaction nodes also has the largest analysis time. The use of the flexible interface method is taken into consideration (FIT), because of its numerical efficiency, but it has to be validated by comparing it to the FV method.

In order to include the nonlinear effects of the soil in the immediate vicinity of the foundation, the a soil volume has been added to the structural model. There are two modelling possibilities:

- A “belt” of soil around each pile = “Local” models;
- A common soil volume around all piles = “Global” models;

Both of these solutions were applied in different models, in order to compare their results.

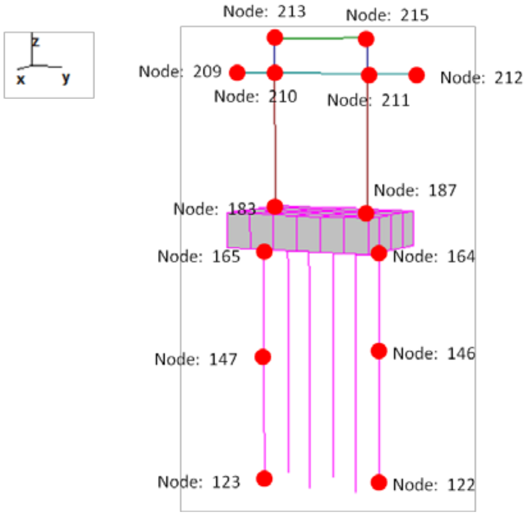
Another important variable is the size of the FEM mesh. Three mesh sizes have been chosen: (S, M and L); only the last two were used in the end, as the first one generated a model that was to large and had extremely long analysis durations.

3.1. LINEAR ANALYSIS – MODEL TESTS

Before going into nonlinear behaviour, the model has to be validated with linear analysis. The models being considered use multiple modeling techniques for both the structure and the near-field soil. In this first stage of the study (testing phase) only coherent seismic input will be used.

3.1.1. BEAMS

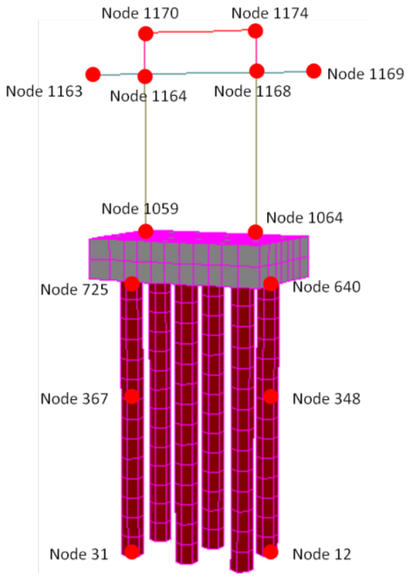
This model is equivalent to the original model of the whole bridge. The only difference is in the modelling of the pile cap: the shell elements from the original model have been changed to 3D solid elements.

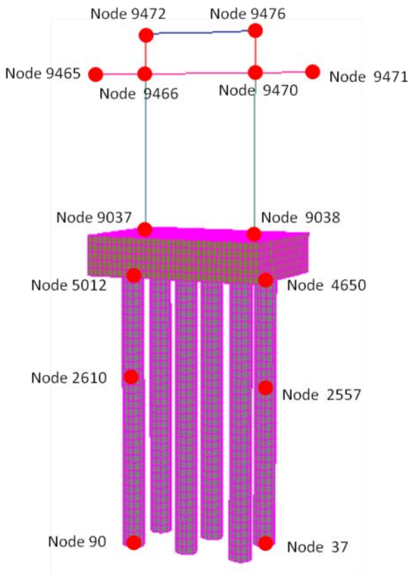


3.1.2. NoSoil_L and NoSoil_M

The NoSoil model uses 3D solid elements for the whole foundation (both piles and pilecap), but does not include any kind of near-field soil. This was introduced to benchmark the stiffness of the first model (Beams).

The model is shown in the underlying figure: the one on the right (NoSoil_M) uses a finer mesh, while the one on the left (NoSoil_L) a coarse mesh.

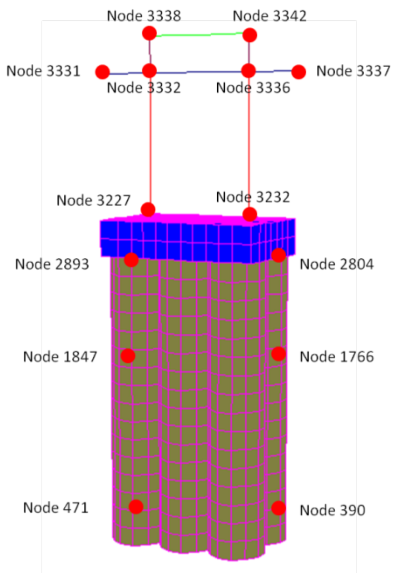


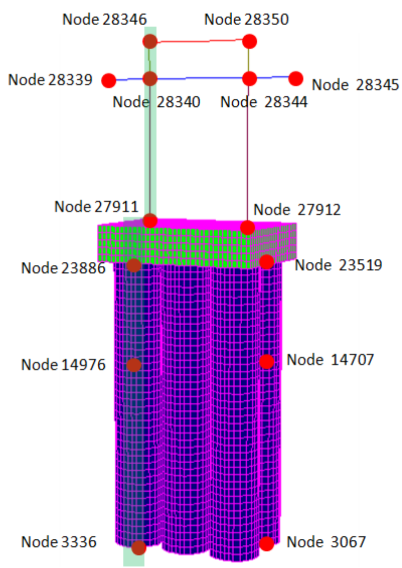


3.1.3. Local_L and Local_M

The Local model contains a belt of soil around each pile with interaction points on its boundry. Novak (2003) recommends using a belt thickness of about the diameter of the pile. In this case, this led to a point contact between the soil elements of each pile. Still, the difference from the Global model lies in the fact that there are more interaction points in the FIT model.

The Local_L (left) and Lolcal_M (right) models are shown below. The circular belt of soil around the piles can be seen.

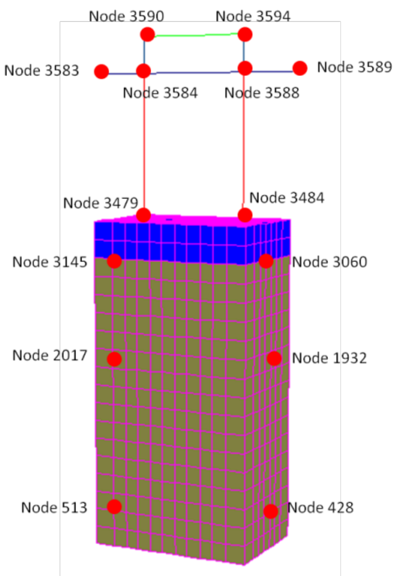


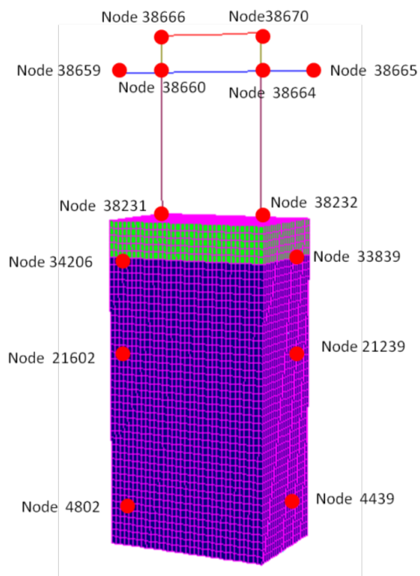


3.1.4. Global_L and Global_M

The Global model uses a continuous larger soil volume where the foundation is sunk.

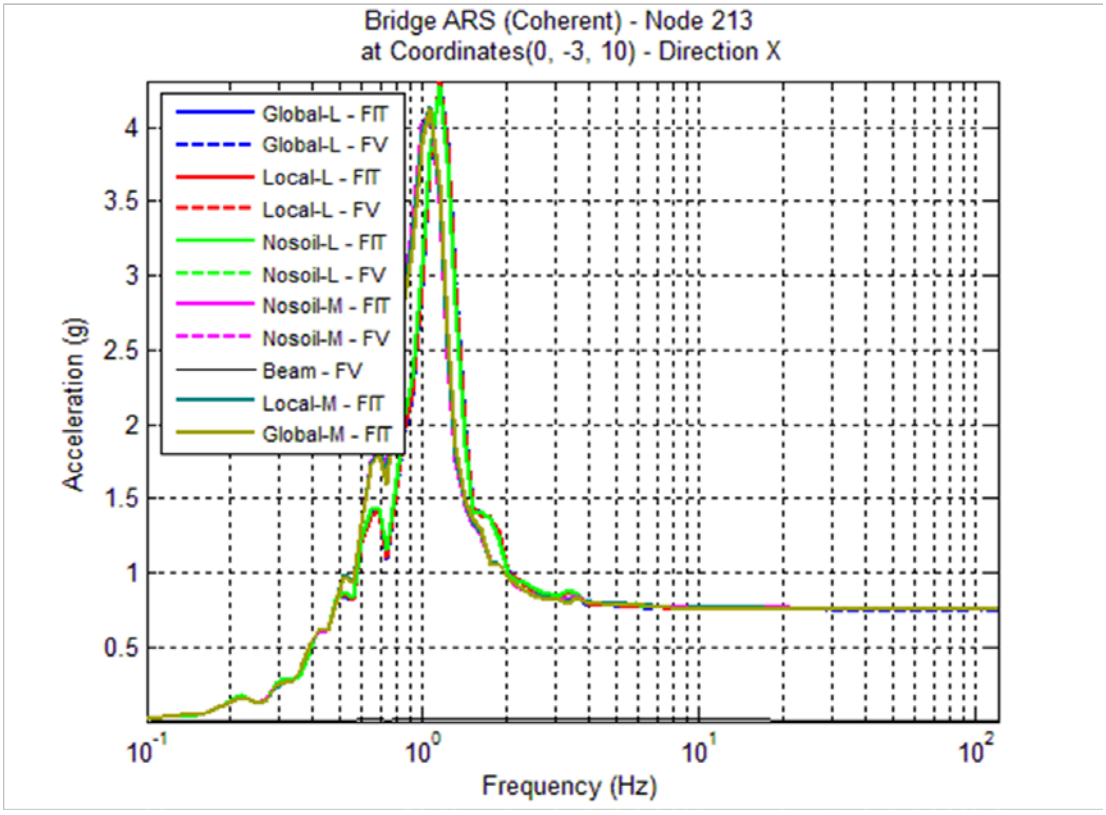
The Global_L (left) and Global_M (right) are shown below. The large soil volume can be observed.

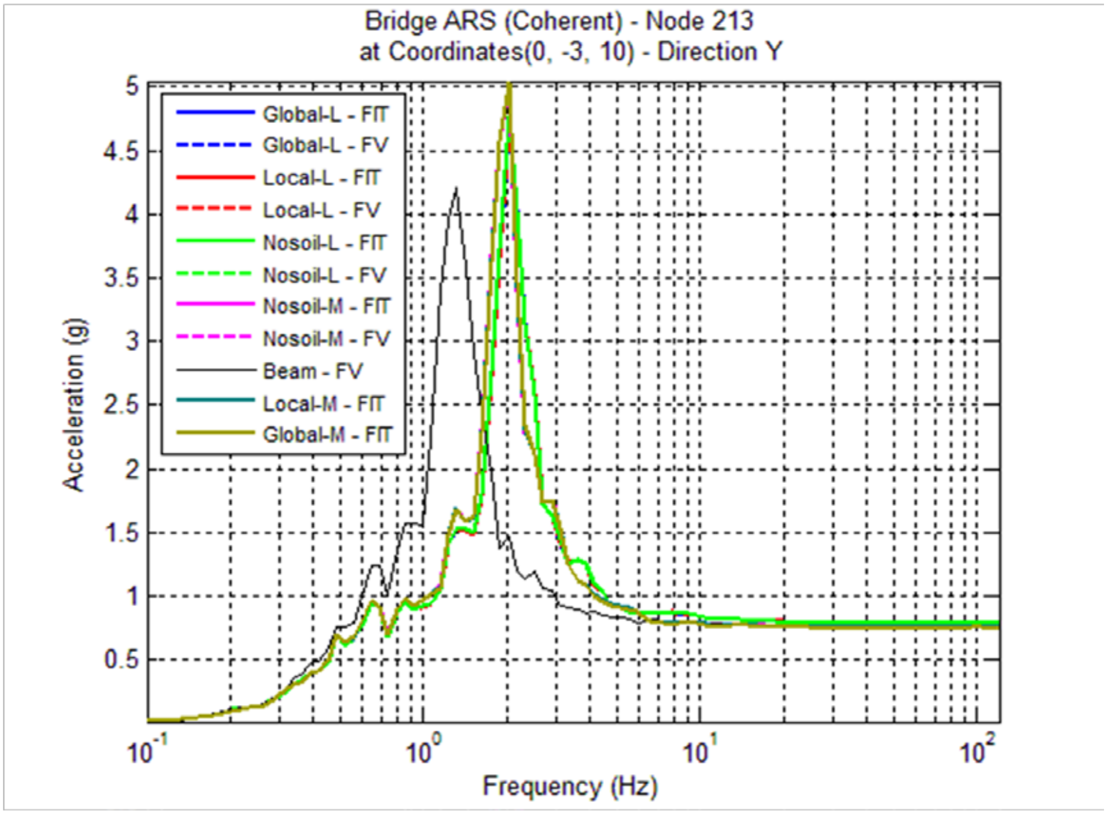


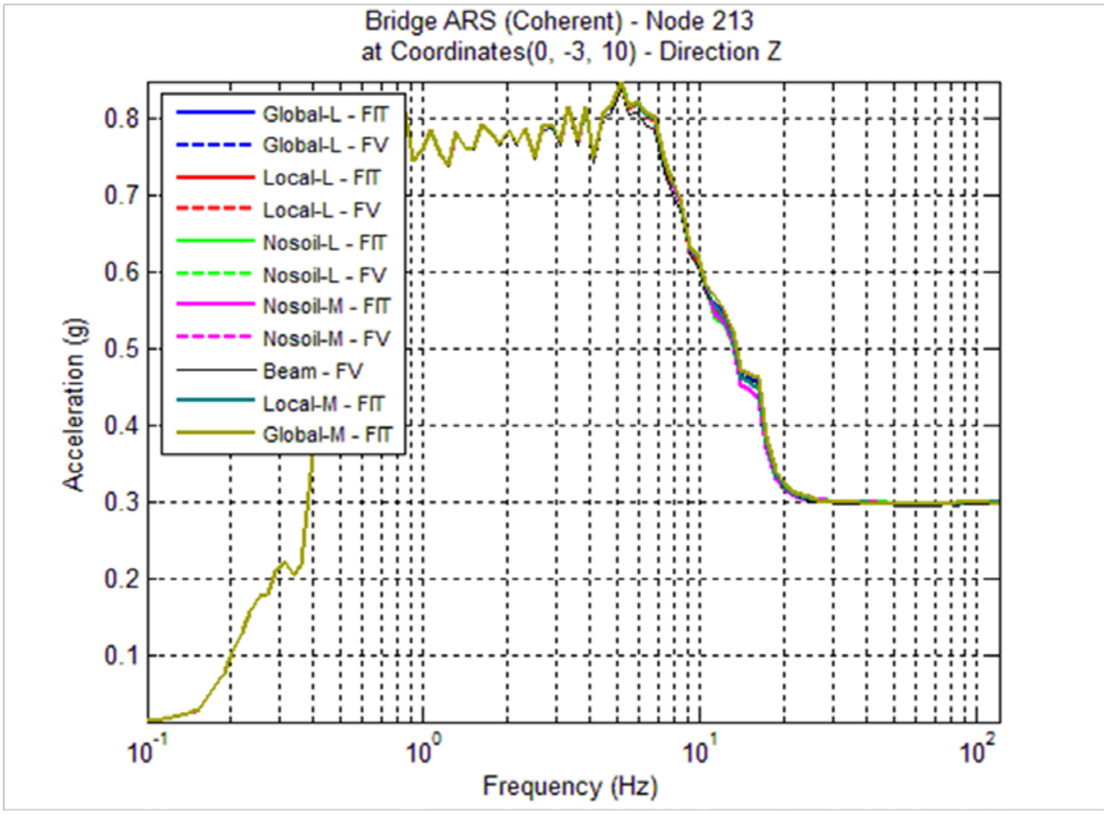


3.1.5. Results and comparison

The following figures show the acceleration response spectra at superstructure level, on the three ortogonal directions.





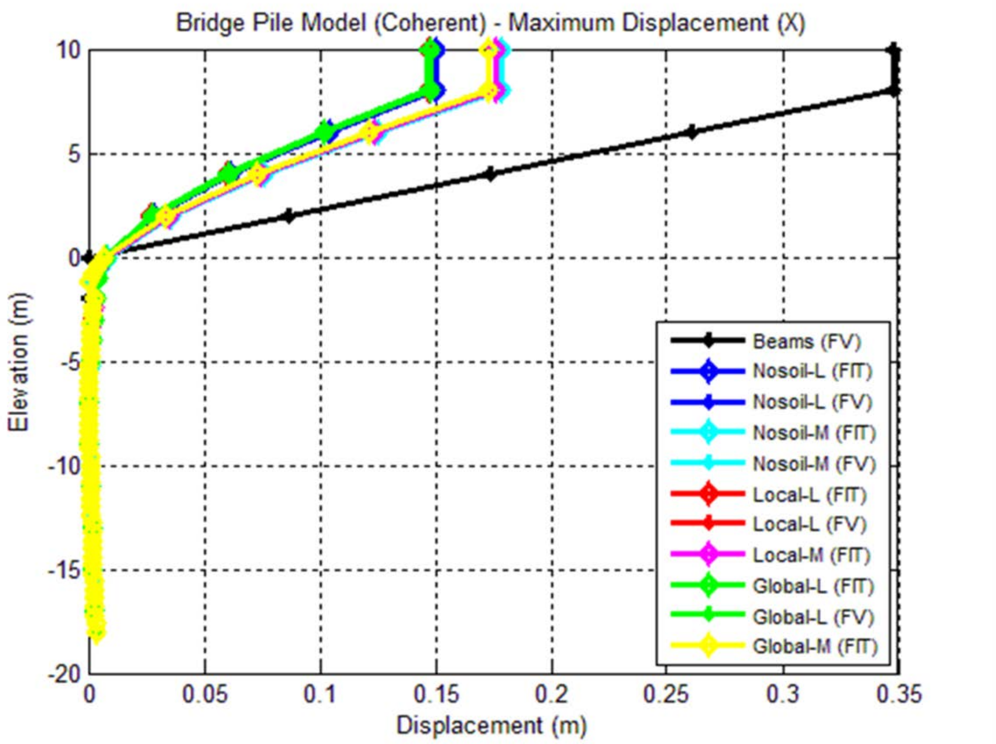


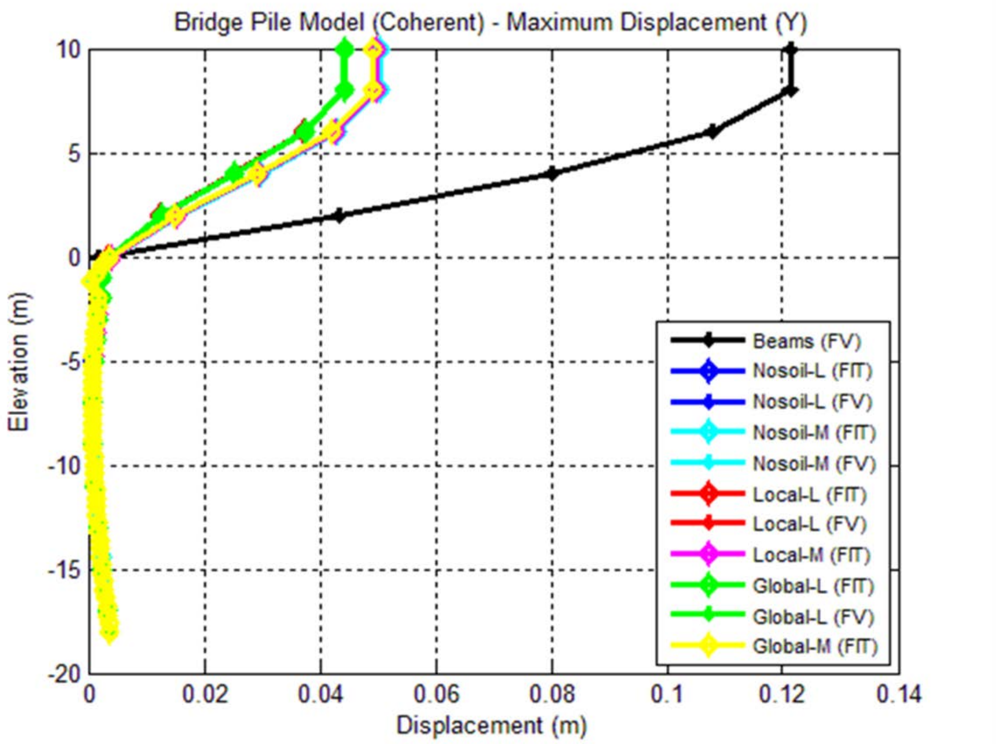
All the models have similar results, except the Beams model. In the Y direction, the stiffness is not correctly estimated.

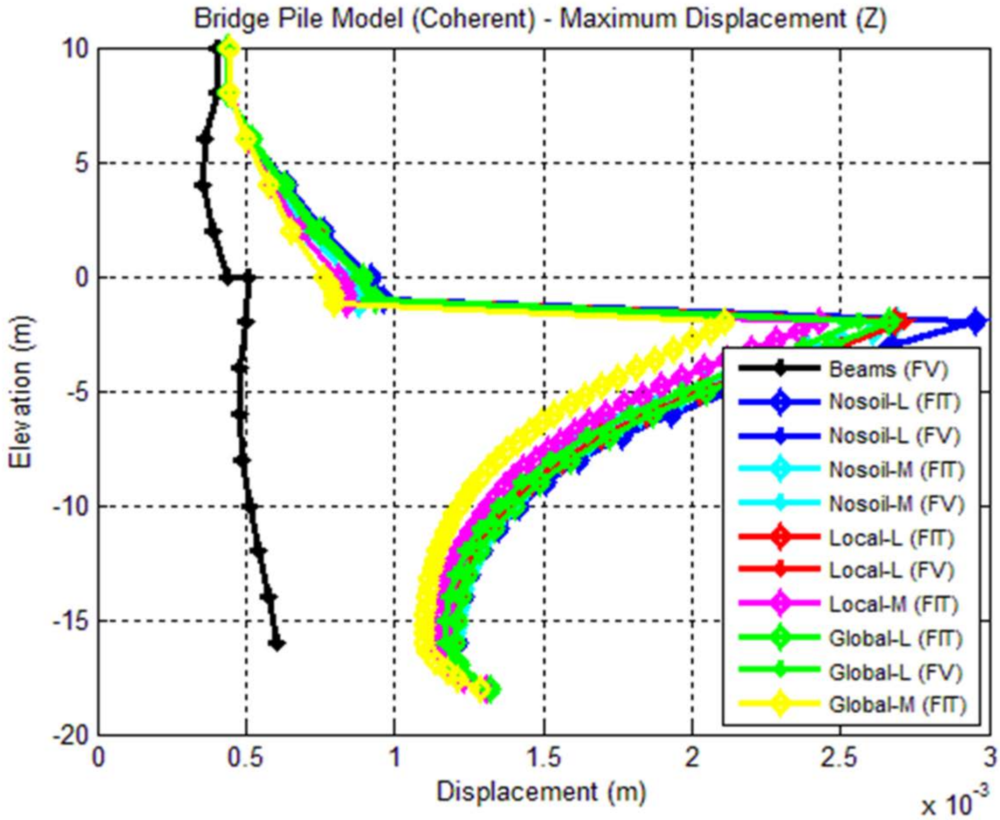
The introduction of soil elements around the piles does not have a very important effect. This was expected, as without the near-field soil nonlinearity, the soil elements behave exactly like the free-field.

A very good correlation between the FV and FIT models is observed. This validates the use of the flexible interfeace method, which considerably reduces the analysis duration.

The figures below show the maximum deformation of the structure subjected to the seismic action.







The Beams model has very large deformations when compared to all the other ones. The fact that it is too flexible was also observed from the response spectra which showed a smaller vibration frequency. The rest of the models show similar behaviour.

3.1.6. CONCLUSIONS

The first stage of the pile study proved that the Beams model does not estimate the stiffness correctly and will not be used in the following stages.

The rest of the models showed similar behaviour. Because the use of the flexible interface substructuring was validated, it will be used in the rest of the study. Also, the larger mesh did not show a big difference in results; the L models will be used from now on.

As for the near-field soil model approach, the Global model has been chosen. It has a smaller number of interaction nodes and it will be able to catch the nonlinearity of the soil between the piles better.

3.2. ANALIZA NELINIARĂ A PILEI

The second stage of the study starts where the linear analysis left off. The Global_L_FIT model is used:

- Flexible interface substructuring method;
- A common near-field soil volume that incorporates the whole foundations (both piles and pile cap);
- The mesh has larger elements;

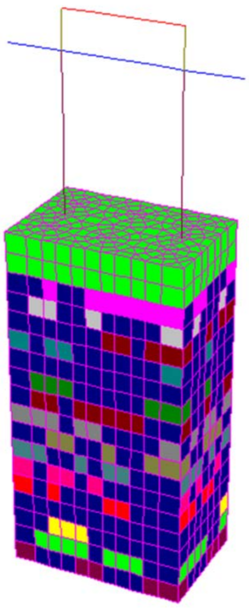
In order to simplify the model even more, the use of square piles is studied in stead of the hexagonal piles the previous models used. The size of the square cross section is chosen to have the same moment of inertia as the actual circular piles.

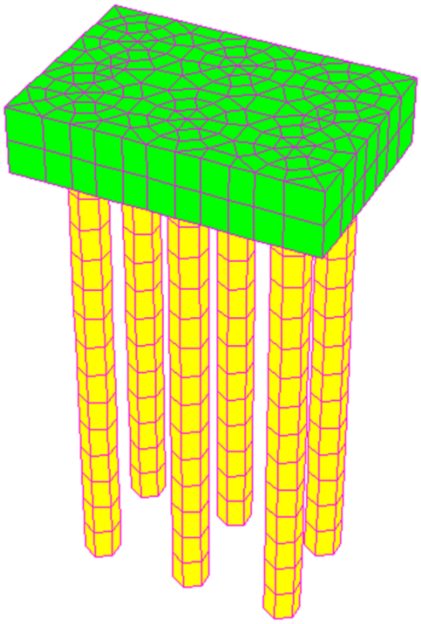
3.2.1. GLOBAL_L_FIT WITH HEXAGONAL PILES

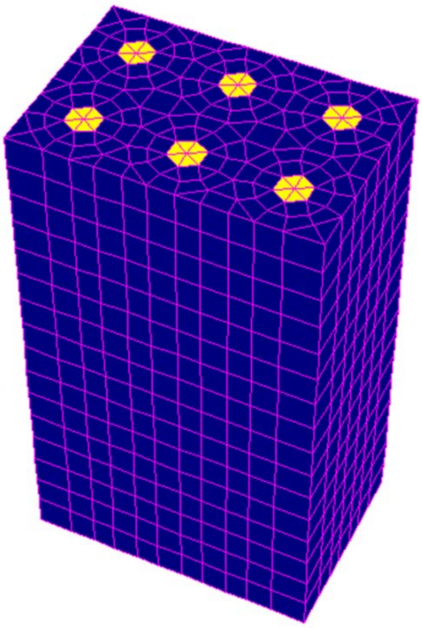
The figures below show the more sophisticated version for the finite element mesh, where the piles have a hexagonal cross section.

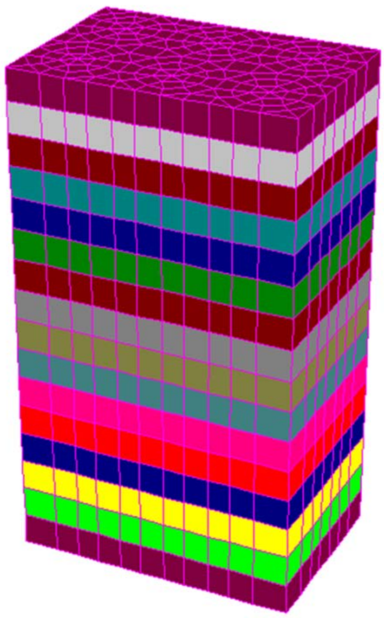
The first picture shows a global view of the model, the next one only the structure itself (piles and pile cap), then the near-field soil volume with the piles and the last one is the excavation model which contains a different material for each soil layer.

This model will be called HEXAGON.







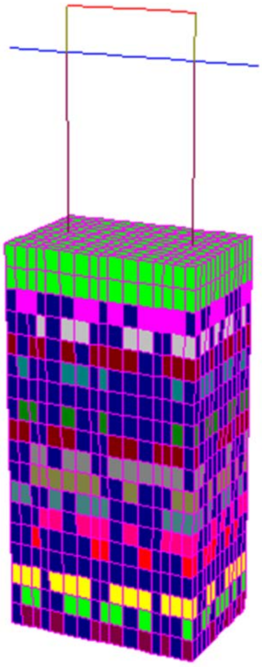


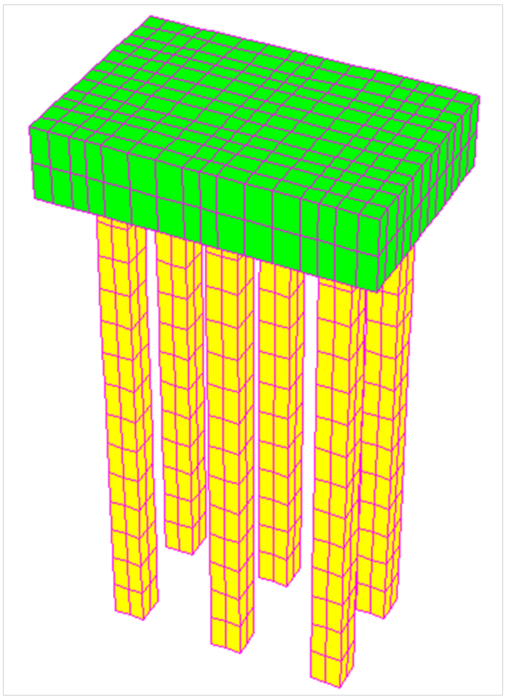
3.2.2. GLOBAL_L_FIT WITH SQUARE PILES

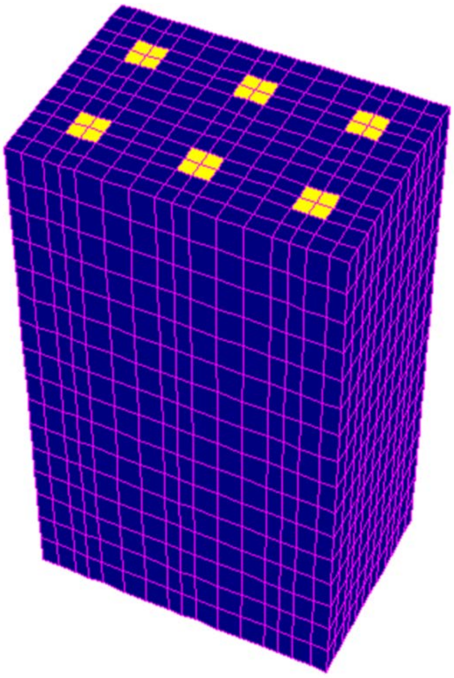
The figures below show the simplified Global_L_FIT model, where the piles are modeled as thier equivalent square shape.

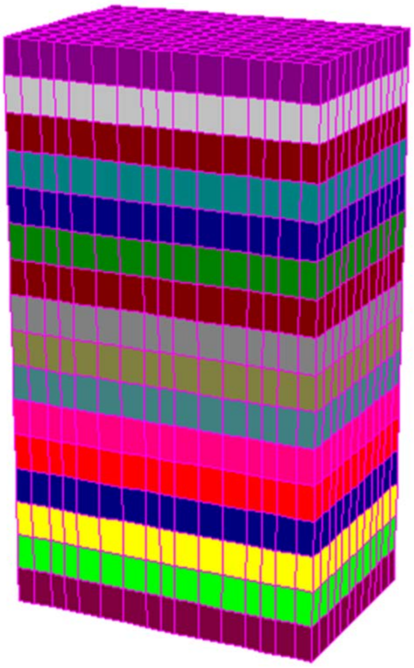
The first picture shows a global view of the model, the next one only the structure itself (piles and pile cap), then the near-field soil volume with the piles and the last one is the excavation model which contains a different material for each soil layer.

This model will be called SQUARE.



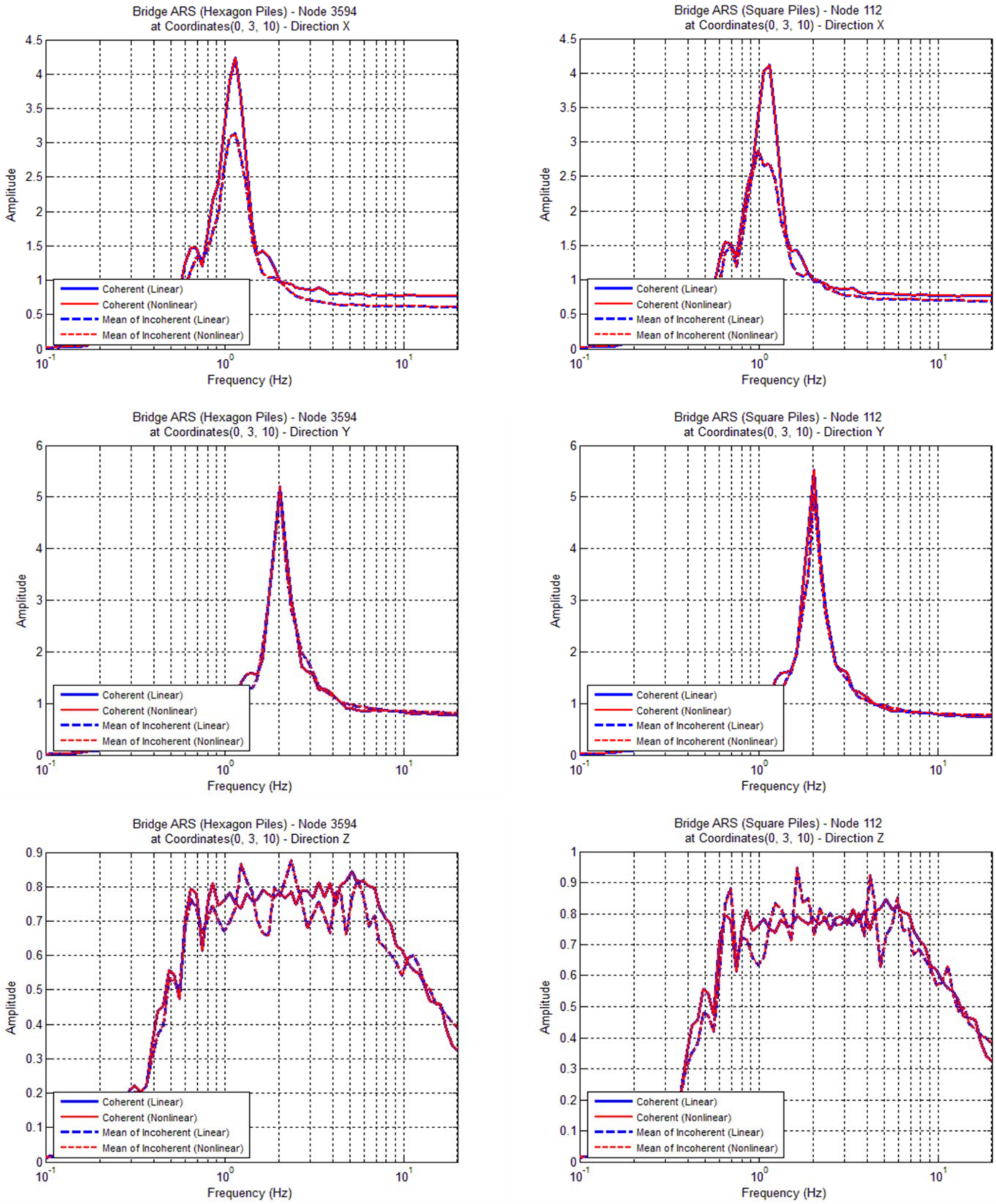






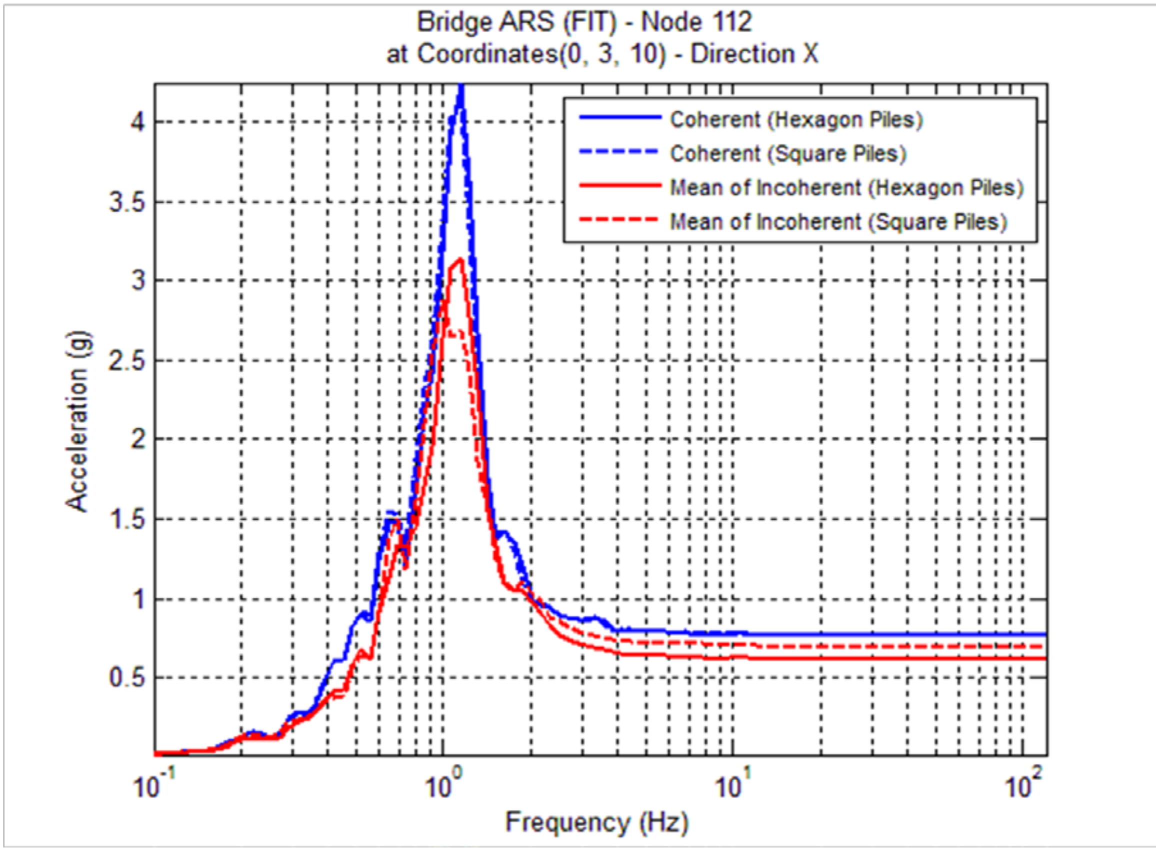
3.2.3. RESULTS AND COMPARISON

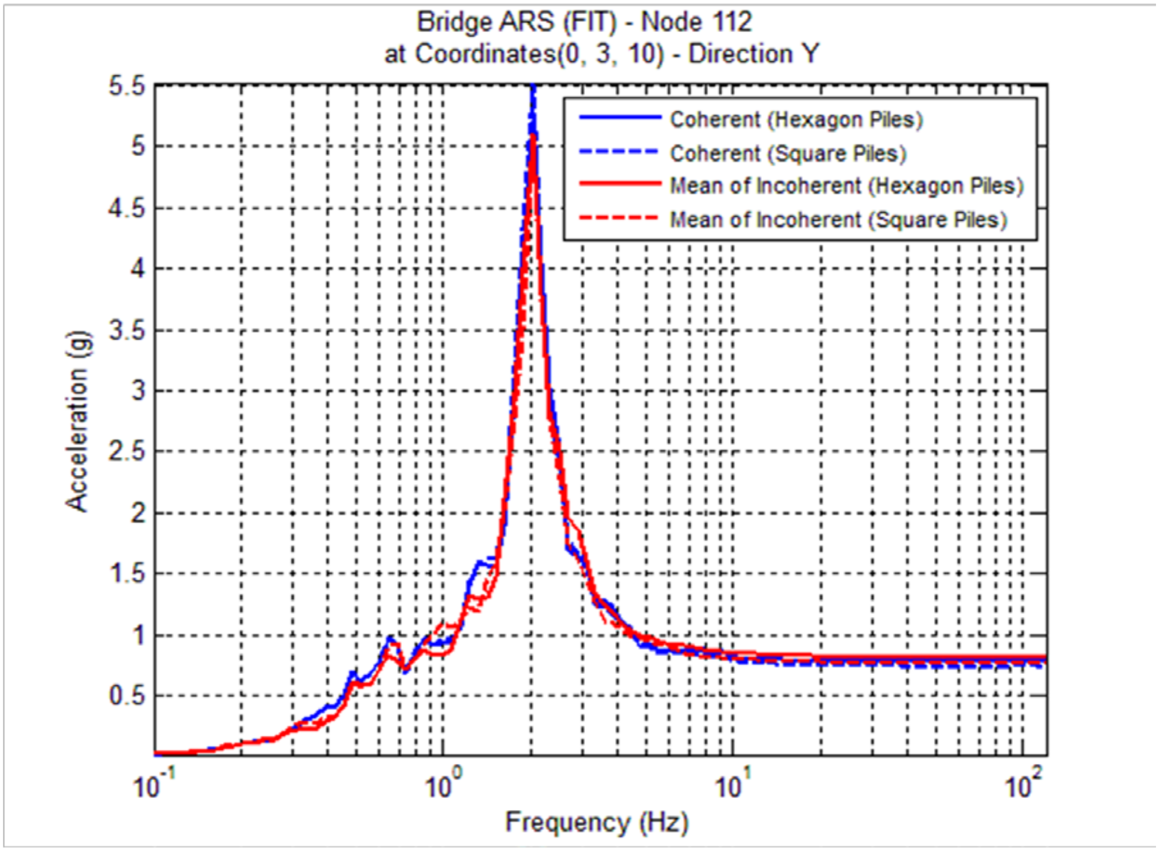
The following graphs represent the acceleration response spectra at superstructure level for the X, Y and Z directions; the results for the HEXAGON models are on the left and the SQUARE models on the right.

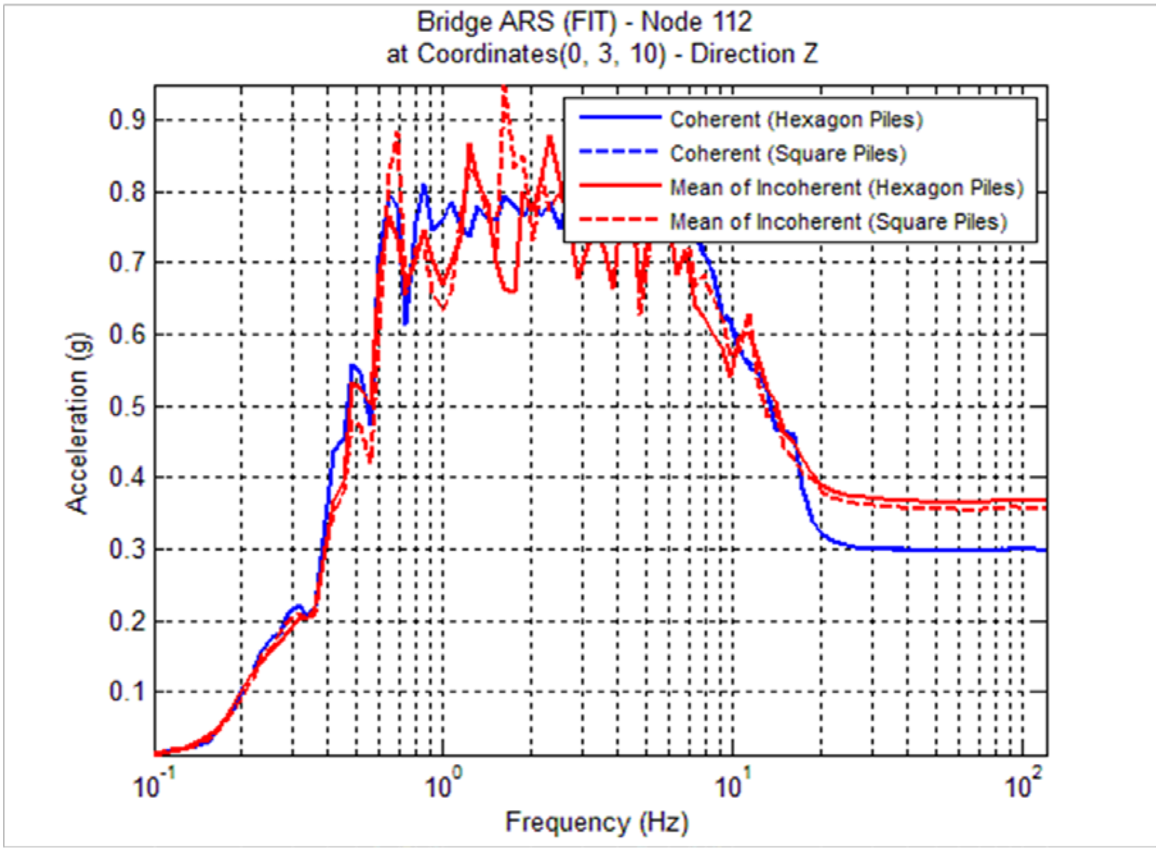


The nonlinear analysis do not show any significant change in the global behaviour of the structure. This may be due to the fact that the interaction points on the pile cap edges connect to the free-field and the forces are transmited directly through the stiffer elements and not thorugh the softer near-field soil elements below.

The following figures compare the results fro the nonlinear analyses of the two models for coherent and incoherent seismic input, on the three ortogonal directions.



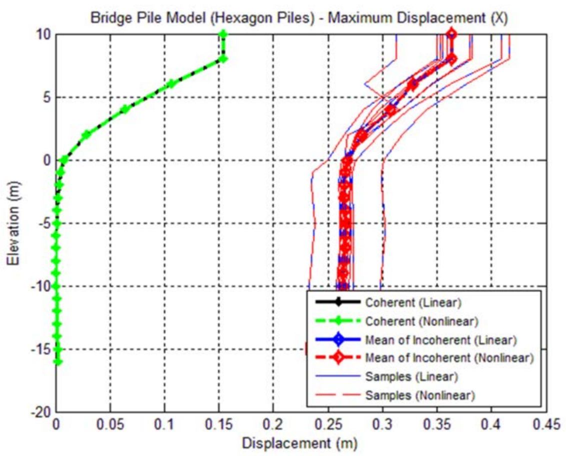


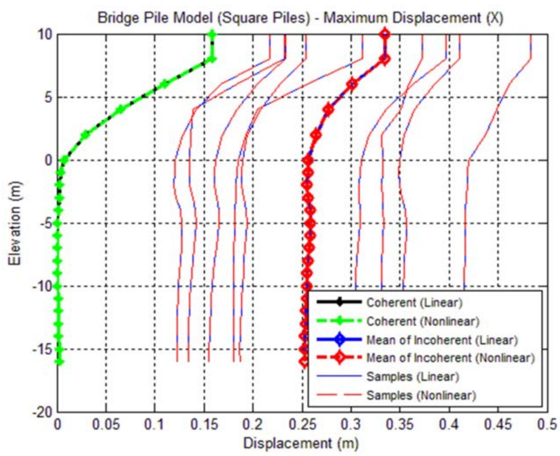


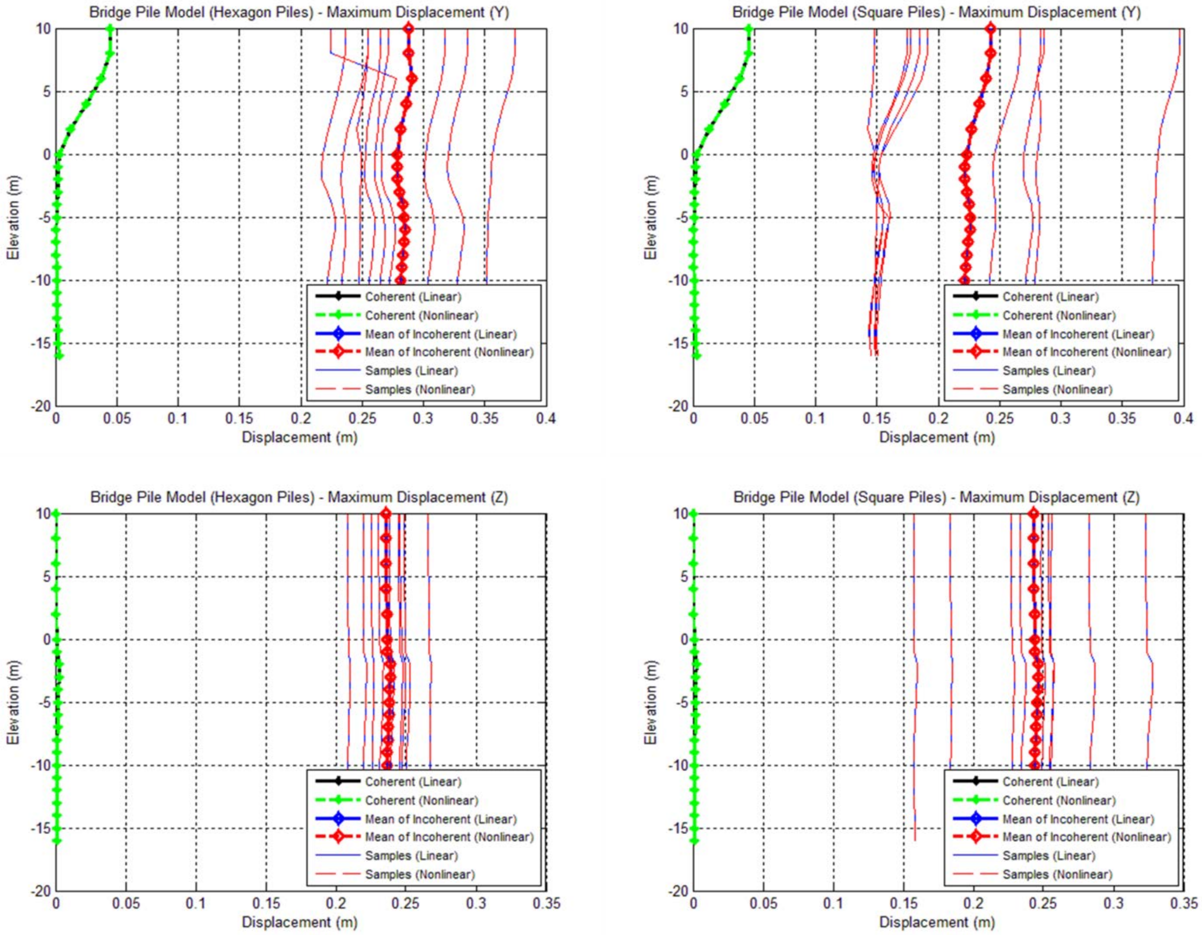
The incoherent analysis reduces the structural reponse by about 25% on the X direction. On the Y direction, the results are similar on both models, and the incoherency does not produce any significant change. In the vertical direction (Z), the effect of the incoherency is minimal.

From the presented results, we can conclude that the two models HEXAGON and SQUARE both prove good results and the use of the SQUARE model in the following stages is justified.

The next figures show the displacements along the height of the model (piles and pier columns) from coherent / incoherent and linear / nonlinear analyses. The left side corresponds to the HEXAGON model, and the right to the SQUARE model.



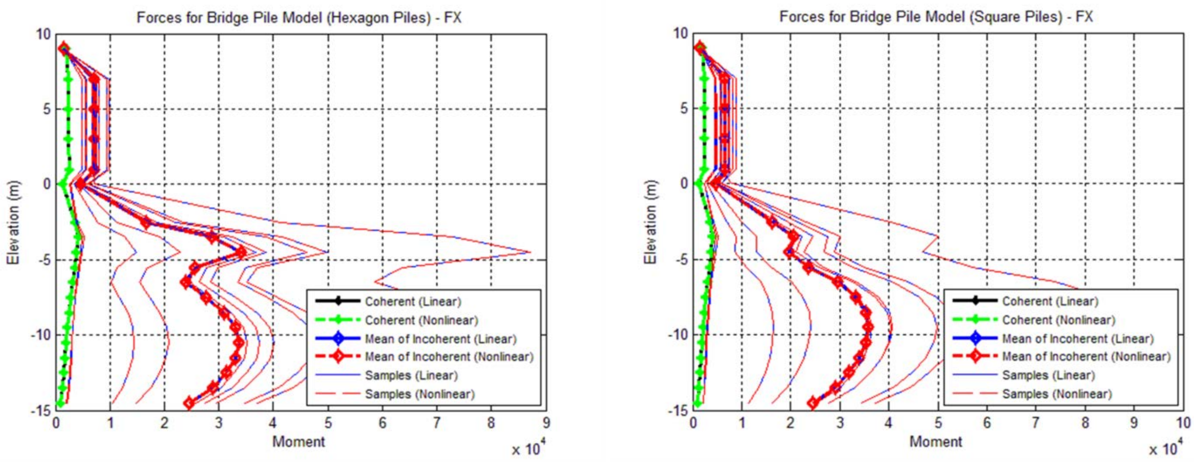




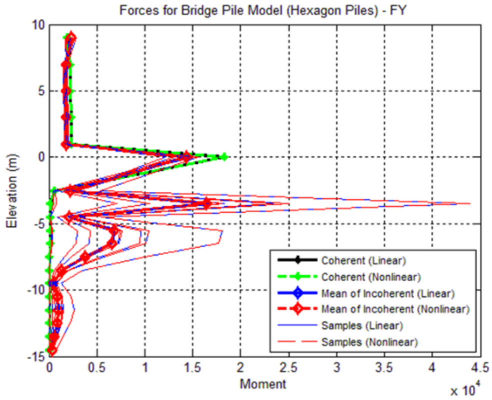
From all the incoherent analyses, the „rigid body” displacements can pe observed, that are produced by seismic waves with a very large wave length. The relative displacements along the pier columns is reduced, but for the piles it increases.

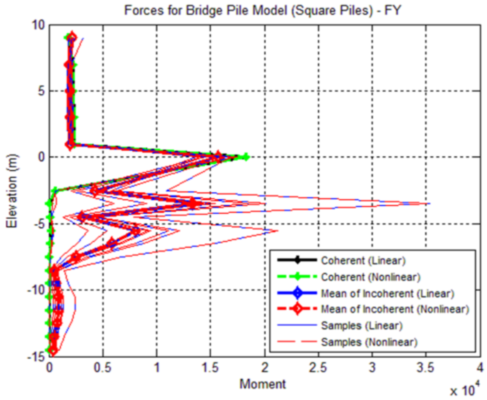
The HEXAGON and SQUARE models have similar displacement results, which once again proves that the use of the simlified square mesh is acceptable.

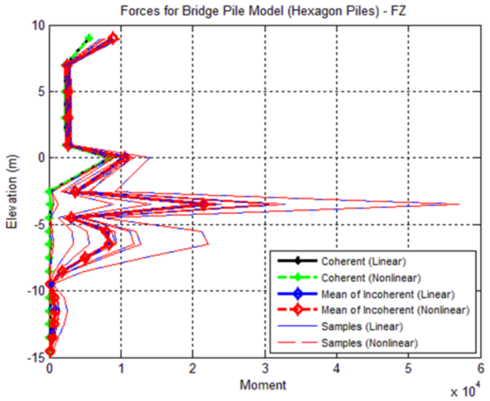
The following graphs show the maximal stresses in the piles from coherent and incoherent analyses with linear and nonlinear behaviour. The left side corresponds to the HEXAGON model, and the right to the SQUARE model.

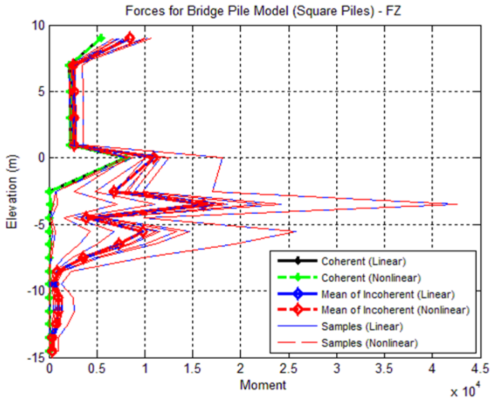


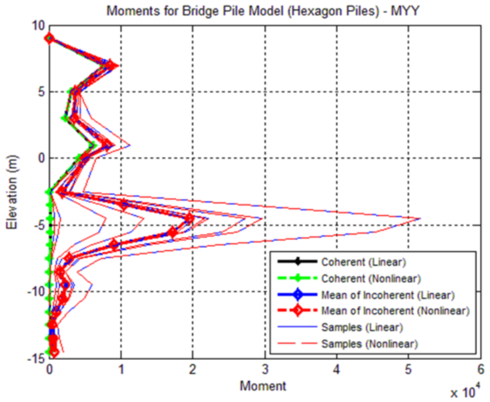
The axial forces in the piles still have very large values and the nonlinear analyses do not reduce them as it would have been expected.

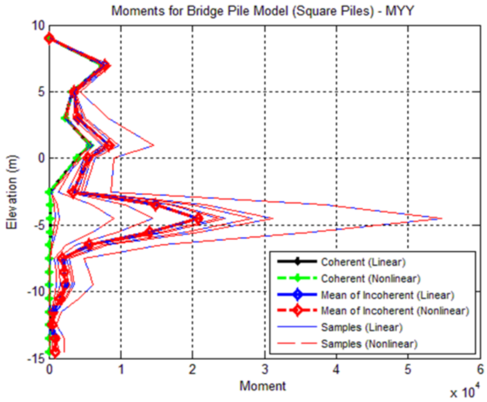


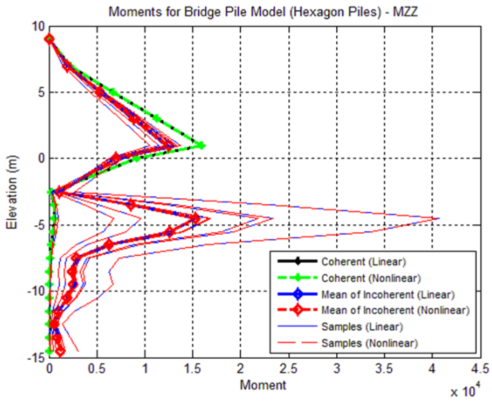


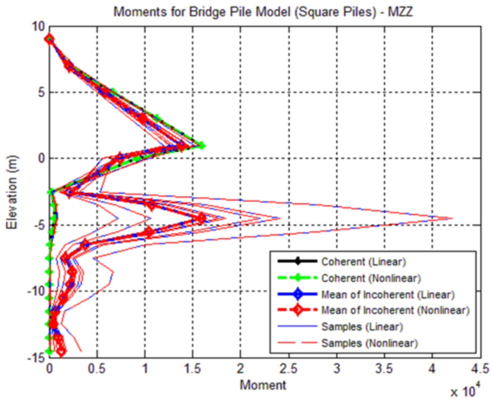












The transverse forces in the piles show a considerable increase due to motion incoherency. For the pier columns, the incoherency produces little changes.

3.2.4. Conclusions

The simlified mesh model SQUARE has shown a very good ans stable behaviour in all analyses. For the rest of the study, this solution may be applied for the beneficial reduction in the size of the model and analysis times.

The high axial forces issue is still present even after introducing the near-field soil nonlinear bahaviour. In the last stage, the modelling of relative slip between the piles and the near-field soil will be added.

4. References

Contiu M, Ghiocel DM, Cretu D (2014) Incoherent soil-structure interaction (ssi) effects for a 242m long concrete bridge founded on deep piles, Second European conference on earthquake engineering and seismology (2ECEES), Istanbul, Turkey

Contiu M, Ghiocel DM, Cretu D (2018) Soil-structure interaction and motion incoherency effects on a concrete bridge with deep foundations, IOP Conference Series: Materials Science and Engineering, Volume 399, 3rd China-Romania Science and Technology Seminar (CRSTS 2018), Brasov, Romania

Ghiocel DM (2007) Stochastic and Deterministic Approaches for Incoherent Seismic SSI Analysis as Implemented in ACS SASSI, Appendix C, Electric Power Research Institute, Palo Alto, CA and US Department of Energy, Germantown, D, EPRI Report TR-1015111, USA

Ghiocel DM (2013) ACS SASSI NQA Version 2.3.0 - An Advanced Computational Software for 3D Dynamic Analysis Including Soil Structure Interaction, User Manuals, Technical Reports, GP Technologies, Inc., Rochester, New York, USA

Abrahamson N (2006) Spatial Coherency for Soil-Structure Interaction, Electric Power Research Institute, Palo Alto, CA and US Department of Energy, Germantown, MD, EPRI TR-1014101

Abrahamson N (2007) Effects of Seismic Motion Incoherency Effects, Electric Power Research Institute, Palo Alto, CA and US Department of Energy, Germantown, MD, EPRI TR-1015110

Seed HB, Idriss IM (1970) Soil Moduli and Damping Factors for Dynamic Response Analysis, Earthquake Engineering Research Center, Berkeley California, Report EERC 70-10, USA

Mylonakis G, Gazetas G, Nikolaou S, Michaelides O (2000) “The role of soil on the collapse of 18 piers of the Hanshin expressway in the Kobe earthquake”, *12th World Conference on Earthquake Engineering*, Auckland, New Zealand, 30 January - 4 February, 1074