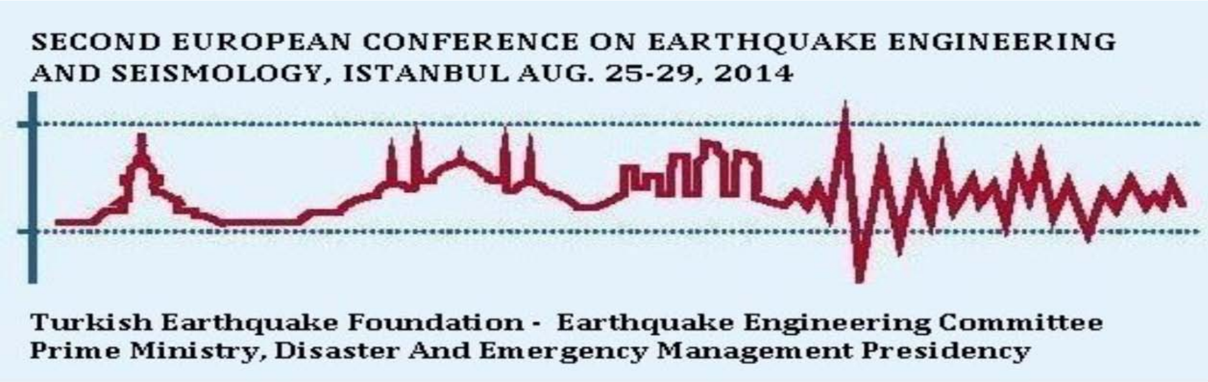


# INCOHERENT SOIL-STRUCTURE INTERACTION (SSI) EFFECTS FOR A 242M LONG CONCRETE BRIDGE FOUNDED ON DEEP PILES

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## ABSTRACT

The paper presents the results of a project of the new research center in earthquake engineering entitled "*Dan Ghiocel International Research Center*" at Technical University of Civil Engineering Bucharest, Romania. The paper presents a seismic SSI study that has been conducted for the newly designed Fartec Bridge in Brasov, Romania. The bridge is a 242m long concrete structure, divided into 8 spans, each of 30m length, and has deep foundations on drilled piles. The structure is typical for Romania and highly used throughout the country. Seismic input was defined by a set of three-directional spectrum compatible acceleration histories generated based on the Romanian seismic design spectrum for the Fartec area. The seismic SSI were performed using the state-of-the-art ACS SASSI software. The SSI analyses were performed for both coherent (synchronous) and incoherent (non-synchronous) seismic inputs. The main focus of the study is to evaluate the effects of the seismic motion incoherency of the Fartec bridge dynamic response. The paper includes comparisons of results obtained using the Eurocode 8 analysis procedures and the state-of-the-art seismic SSI analysis using ACS SASSI. The final conclusions indicate the limitations of the current bridge design SSI modeling and the simplified motion spatial variability modeling in Eurocode 8.

## 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 seismic SSI effects may be neglected for small and light structures build on stiff soils, such as rocks, but becomes very important for larger and heavier structures, such as nuclear power-plants, concrete bridges or even high-rise buildings.

Recent earthquakes have highlighted the necessity to include SSI effects in the design of structures. The 1995 Kobe earthquake has severely damaged bridges along the Hanshin Expressway. This shows that the response of the structure is influenced by the interaction between the foundations and the soil, which, in this particular case, had a negative role (Mylonakis et al., 2000).

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

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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.

## 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. 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.

Figure 1 Fartec Bridge in Brasov, Romania

This type of bridge structure is typical for Romania and highly used throughout the country. Fig.1 presents a picture of the executed bridge.

## 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.

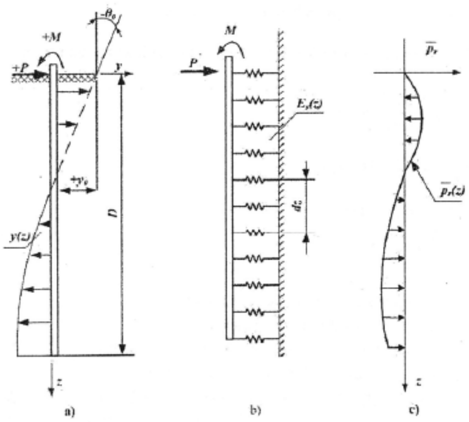


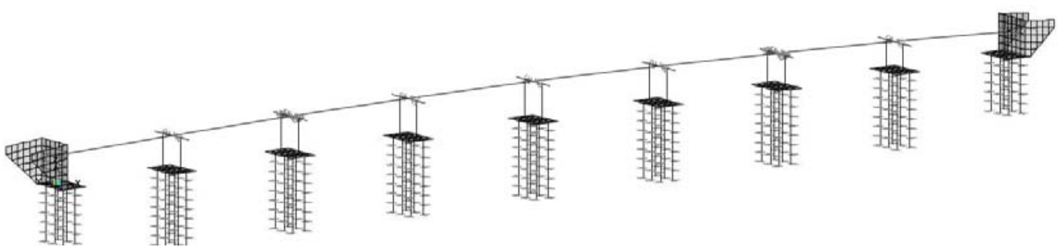

Figure 2. Pile-soil interaction model, as per the Romanian code regarding the geotechnical design of deep foundations on piles, NP 123:2010

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.

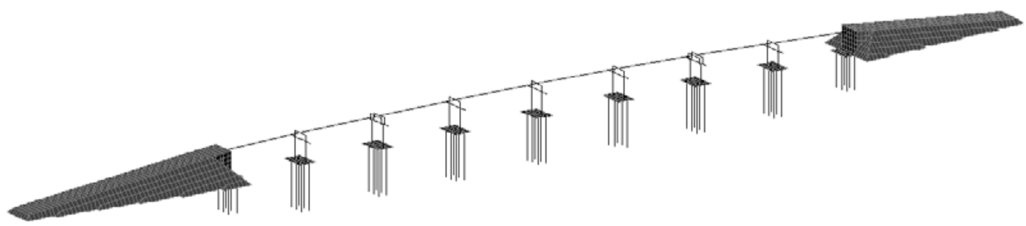
The seismic action of the earth fill behind the abutments has also been considered through the simplified model of constant or linear varying seismic soil pressures, according to the Mononobe - Okabe theory.

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.

From the two sets, the one which generates the maximum response for each particular case shall be considered. These results will be added to the time-history results through SRSS. The calculated values will be presented in the following comparisons.



a. Bridge Design Model



b. Bridge SSI Model

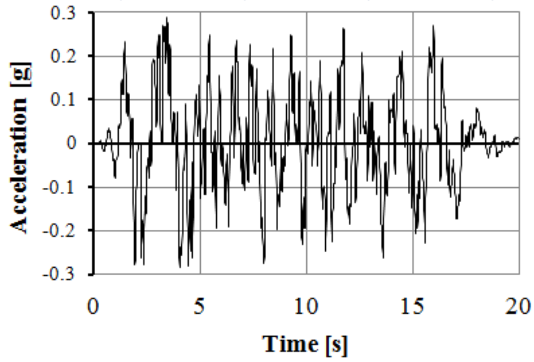
Figure 3. Bridge Models Used in the Study

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).

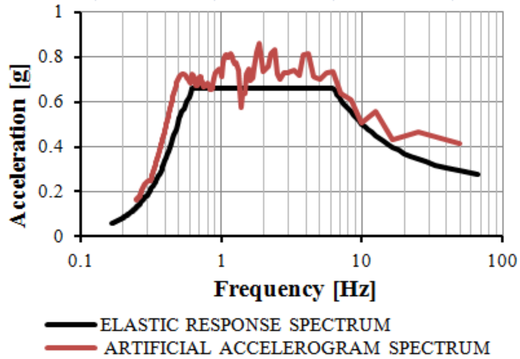
The seismic loads are modelled through different types of wave fields acting in different directions: for the X direction (bridge longitudinal direction) SV-waves have been considered, for the Y direction (bridge transverse direction) SH-waves and for the Z direction (vertical direction) P-waves. The 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. Artificial accelerogram time-history




b. Spectrum comparison

Figure 4. Seismic input

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.

## 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). Additional comparisons will be made between the superstructure absolute displacements and the relative displacements of the foundations.

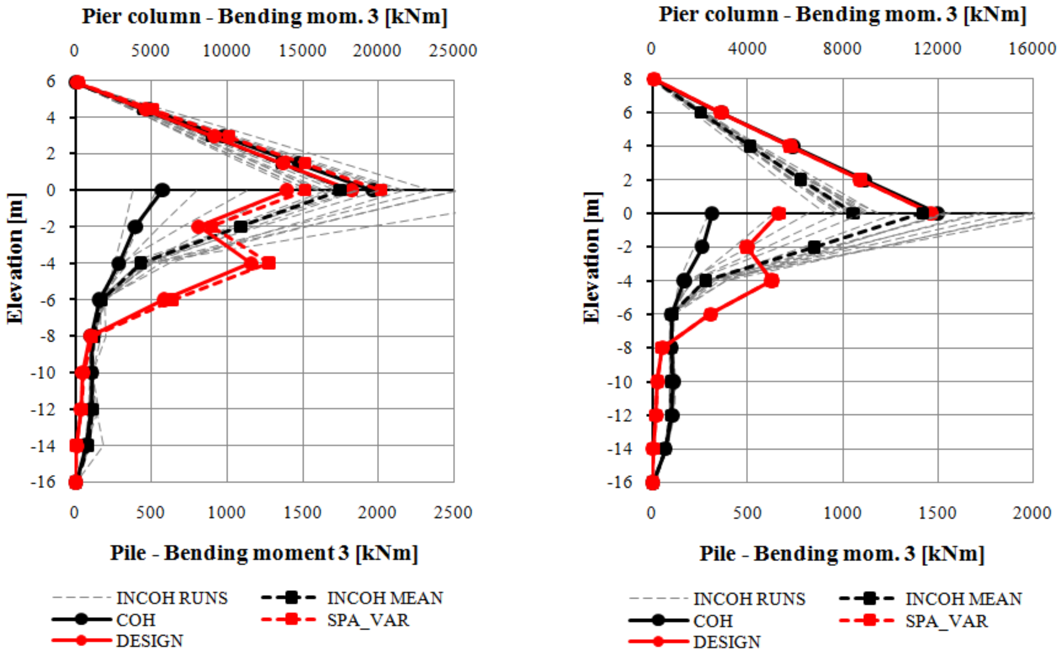



Figure 5. Bending moment 3 for earthquake loading in X direction (left = side pier, right = mid pier)

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.

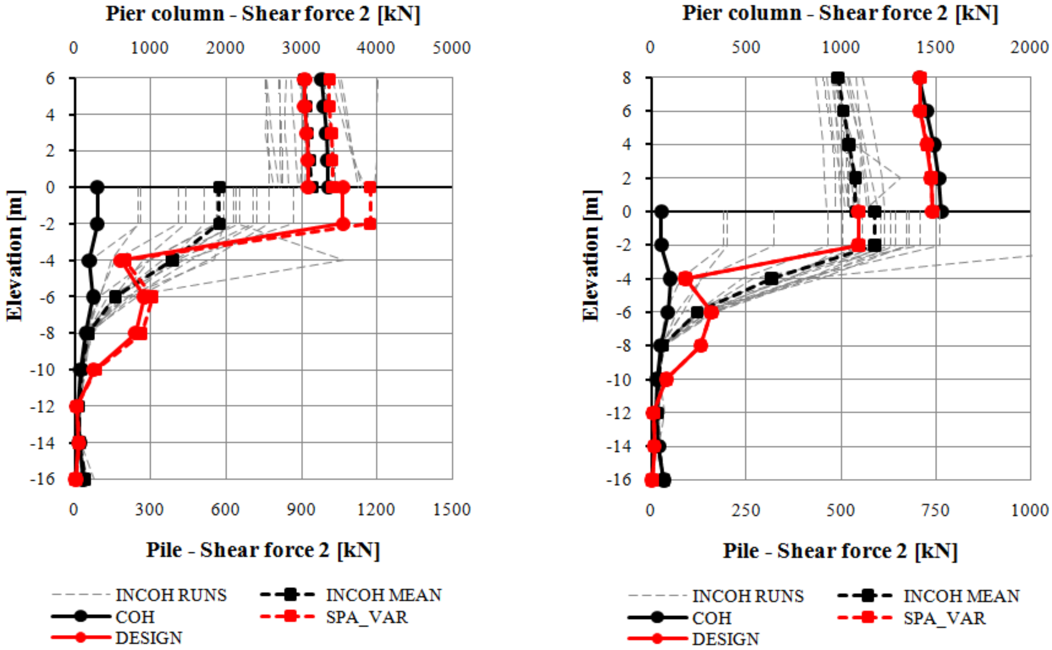



Figure 6. Shear Force 2 for earthquake loading in X direction (left = side pier, right = mid pier)

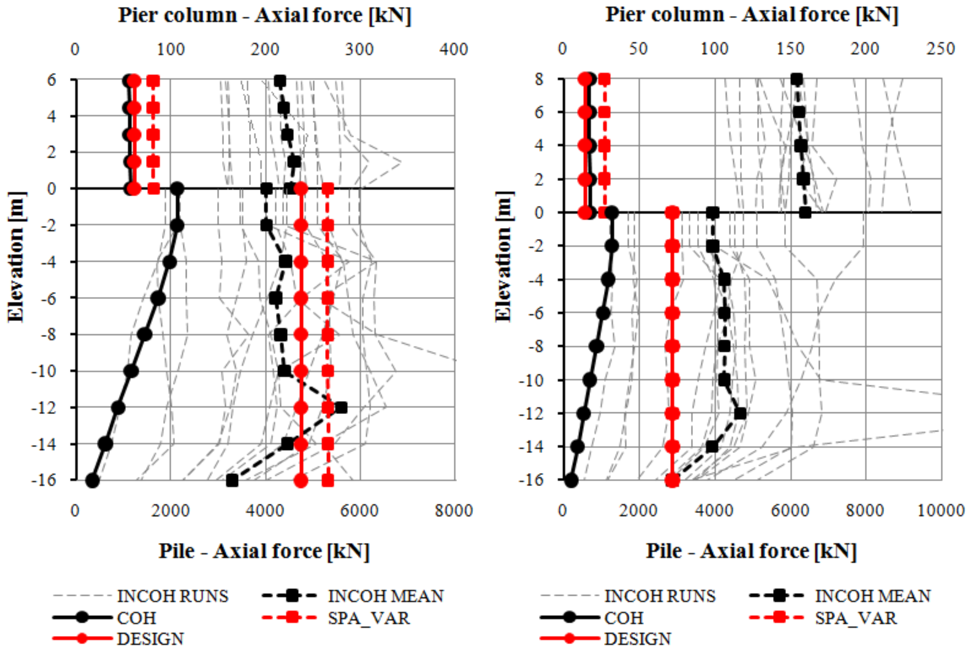



Figure 7. Axial force for earthquake loading in X direction (left = side pier, right = mid pier)

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.

For the side pier, the maximum pile bending moment 3 in the DESIGN case is 1394 kNm and for the COH case 573 kNm; it would seem the current design practice overestimates the structural response for the coherent load cases. When adding the motion spatial variability or incoherency effects, the current design practice generates an increase of approximately 10 %, while for the refined SSI model the bending moment increases about 2 times and the shear force about 5 times. For the mid pier piles, the ratio between the design and coherent cases remains the same; the spatial variability has practically no effects on the pile elastic stresses, but the incoherency produces bending moments almost 4 times larger and shear forces up to 10 times larger. The axial forces in the piles have a similar behavior: the coherent results are reduced, so that the incoherency effects generate a mean increase of about 10 times; the motion spatial variation has a reduced effect through the simplified design method; overall, the SPA_VAR case and the INCOH_MEAN case have similar results.

Table 1. Stress comparison for earthquake loading in X direction

| Pier position | Structural element | Stress | DESIGN | SPA_ _VAR | % INCREASE (SPA_VAR to DESIGN) | COH | INCOH_ _MEAN | % increase (INCOH_ MEAN to COH) |
|---|---|---|---|---|---|---|---|---|
| Side pier | Pier column | M3 | 18247 | 20194 | 10.7% | 19660 | 18226 | -7.3% |
| | | V2 | 3088 | 3412 | 10.5% | 3354 | 3140 | -6.4% |
| | Pile | M3 | 1394 | 1518 | 8.9% | 573 | 1747 | 204.9% |
| | | V2 | 1064 | 1172 | 10.2% | 90 | 573 | 536.7% |
| | | N | 4753 | 5318 | 11.9% | 334 | 3294 | 886.2% |
| Mid pier | Pier column | M3 | 11700 | 11718 | 0.2% | 11970 | 8430 | -29.6% |
| | | V2 | 1482 | 1484 | 0.1% | 1527 | 1075 | -29.6% |
| | Pile | M3 | 663 | 664 | 0.2% | 314 | 1420 | 352.2% |
| | | V2 | 545 | 546 | 0.2% | 27 | 588 | 2077.8% |
| | | N | 2874 | 2880 | 0.2% | 200 | 2870 | 1335.0% |

The pier columns bending moment 3 and shear force 2 are very close in all cases for the more rigid side pier; the DESIGN and COH cases generate very close stress values, and the spatial variation or incoherency has almost no effect. The more flexible mid pier has a different response: motion incoherency reduces the bending and shear stresses by approximately 30 %. The axial forces in the pier columns may be neglected since the piers act as a cantilever beam in the longitudinal direction and the axial force values due to earthquake loading in this direction are negligible.

For the earthquake acting in the X direction, the design model offers good design values for the stiffer side pier (only a slight underestimation of the pile bending moments and overestimation for the pile shear forces). However, for the more flexible mid pier, the design values are almost 50% larger than the actual stresses obtained through the refined SSI analysis.

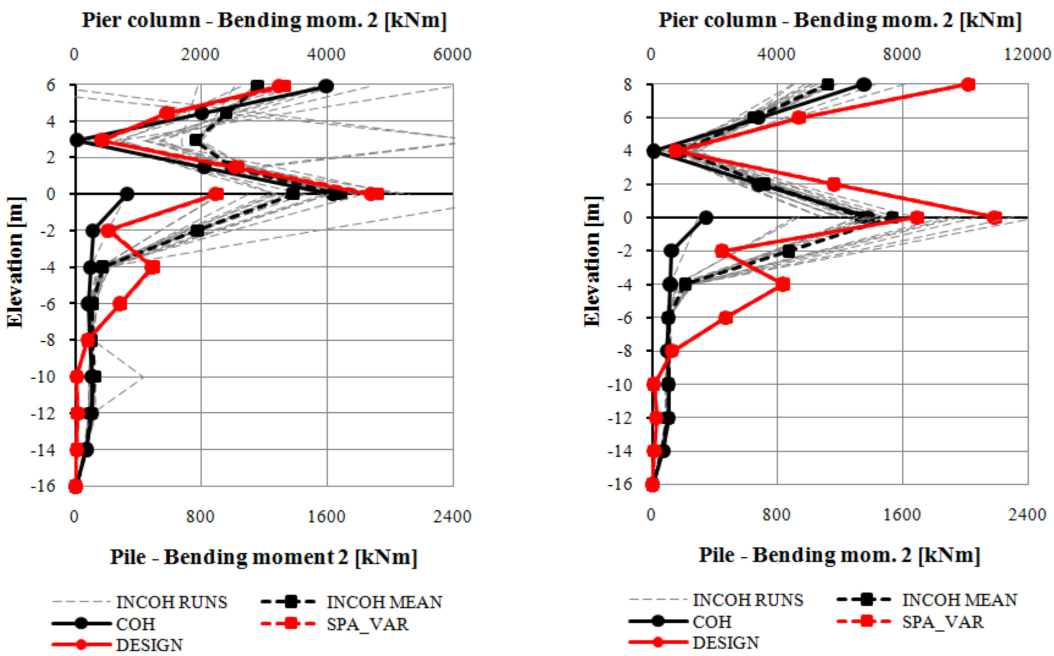



Figure 8. Bending moment 2 for earthquake loading in Y direction (left = side pier, right = mid pier)

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; just like the previous one, Table 2 shows the same comparison.

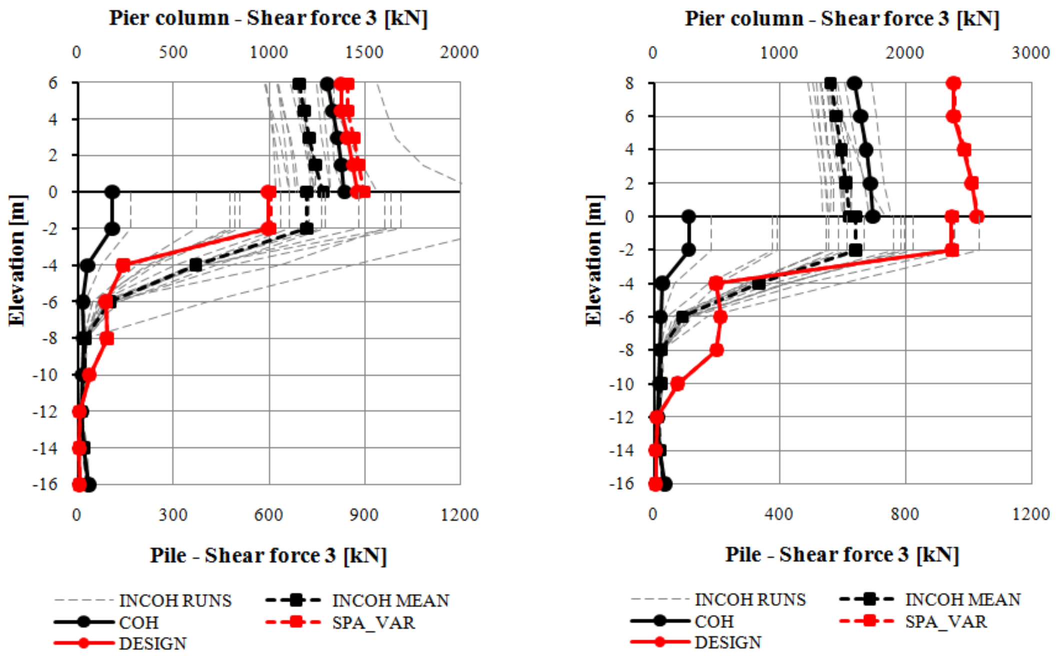



Figure 9. Shear Force 3 for earthquake loading in Y direction (left = side pier, right = mid pier)

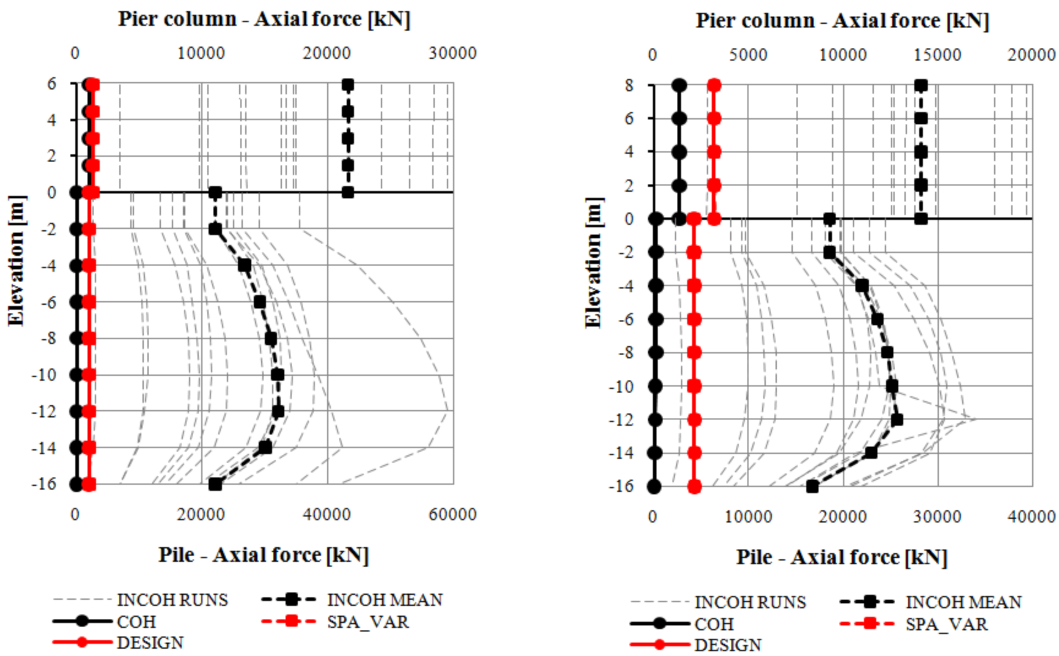



Figure 10. Axial force for earthquake loading in Y direction (left = side pier, right = mid pier)

The Y direction earthquake load generates a much more rigid bridge response; in this direction the foundations are larger and have three rows of piles; the piers columns and the pier cap act together as a frame.

The side pier piles' bending moment 2 and shear force 3 are small in the COH case. The DESIGN and SPA_VAR stress values are 2 – 3 times larger than the COH case, and are practically equal; this shows the simplified EC model for spatial variability generates almost no effects for the Y direction. Both cases slightly underestimate the bridge response by 15% – 40%. The pier column transverse stresses are all very close. One thing should be noted: in this direction, the design models and the COH model act as a frame with a horizontal load in top; this means that the columns have high bending moments at the bottom and at the top and close to 0 values in the middle; the INCOH_MEAN case shows a different behavior, in the way that the bending moments at the bottom of the pier are quite larger than the ones at the top and the middle of the pier does not have null bending moments, but almost 50% of the largest value at the bottom. This possibility needs to be considered in the design, so that the middle of the pier still has enough reinforcement for the given stresses.

For the mid pier, the SPA_VAR load case gives an increase of less than 1%, so its effects can be considered negligible. The design bending moments 2 and shear stresses 3 are a bit overestimated, but overall good results. The pier column design stresses are overestimated by almost 50%. The coherent load case results are already smaller than the design and the incoherency only reduces those values even more. It should be noted that, as stated earlier, the mid pier is more flexible than the rest of the infrastructures.

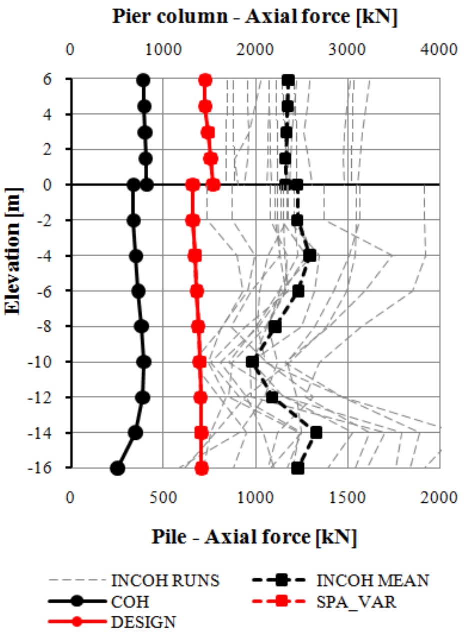
The analyses show a very large increase in the axial forces for both piles and pier columns and for both locations, meaning the side and mid pier. The axial forces in the piles come from both the vertical seismic component, but mostly from the pier bending. These very large values are the result of the 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; Table 3 summarizes the result values.

The axial forces from the vertical seismic component in the piles and pier columns show the same behaviour: the COH analysis gives the smallest results, the DESIGN and SPA_VAR are equal and 2-3 times larger than the COH results, while the INCOH_MEAN is 2-3 larger than the DESIGN. Just like the previous situation, the axial forces for the INCOH_MEAN may be overestimated due to this being a purely elastic pile-soil interaction analysis.

Table 2. Stress comparison for earthquake loading in Y direction

| Pier position | Structural element | Stress | DESIGN | SPA_ _VAR | % INCREASE (SPA_VAR to DESIGN) | COH | INCOH_ _MEAN | % increase (INCOH_ MEAN to COH) |
|---|---|---|---|---|---|---|---|---|
| Side pier | Pier column | M2 | 4683 | 4791 | 2.3% | 4087 | 4213 | 3.1% |
| | | V3 | 1461 | 1494 | 2.3% | 1394 | 1283 | -8.0% |
| | Pile | M2 | 888 | 903 | 1.7% | 327 | 1380 | 322.0% |
| | | V3 | 596 | 604 | 1.3% | 107 | 720 | 572.9% |
| Mid pier | Pier column | M2 | 10913 | 10942 | 0.3% | 6890 | 6676 | -3.1% |
| | | V3 | 2562 | 2569 | 0.3% | 1737 | 1548 | -10.9% |
| | Pile | M2 | 1689 | 1691 | 0.1% | 342 | 1534 | 348.5% |
| | | V3 | 944 | 946 | 0.2% | 110 | 640 | 481.8% |



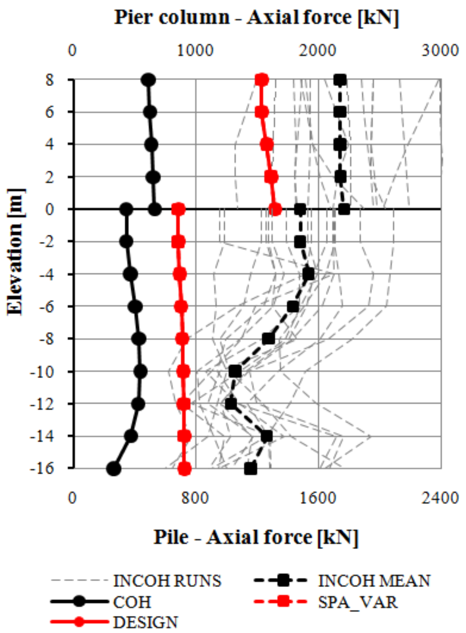




Figure 11. Axial force for earthquake loading in Z direction (left = side pier, right = mid pier)

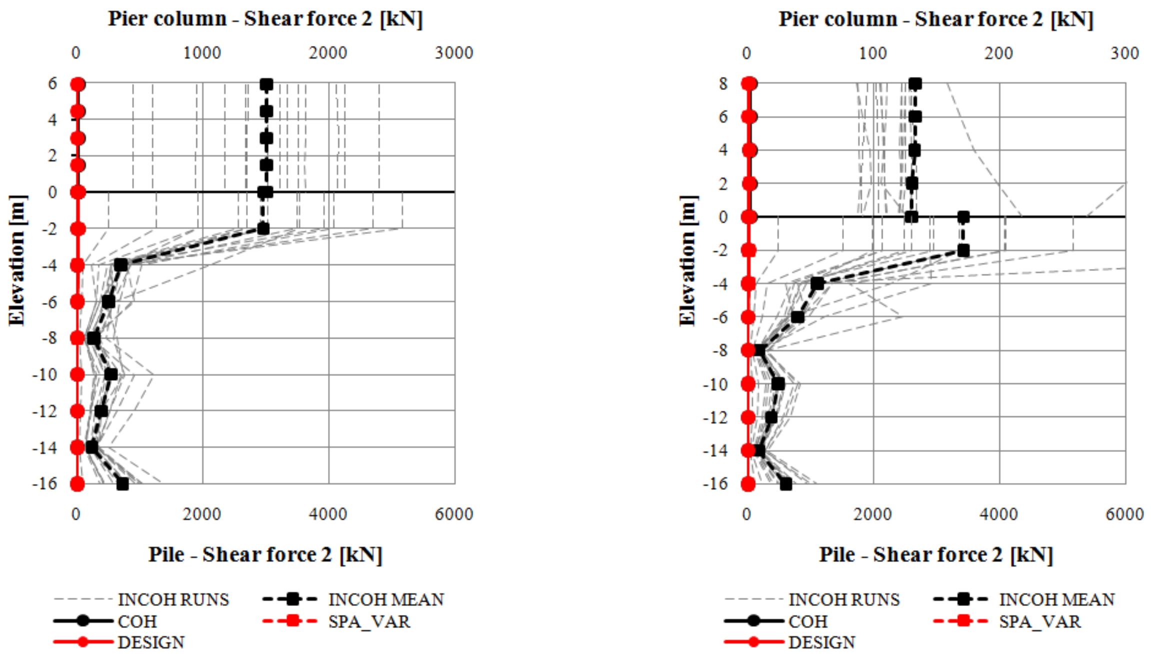



Figure 12. Shear Force 2 for earthquake loading in Z direction (left = side pier, right = mid pier)

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.

Table 3. Stress comparison for earthquake loading in Z direction

| Pier position | Structural element | Stress | DESIGN | SPA_ _VAR | % INCREASE (SPA_VAR to DESIGN) | COH | INCOH_ _MEAN | % increase (INCOH_ MEAN to COH) |
|---|---|---|---|---|---|---|---|---|
| Side pier | Pier column | N | 1538 | 1538 | 0.0% | 820 | 2325 | 183.5% |
| | | V2 | 2 | 2 | 0.0% | 10 | 1503 | 14930.0% |
| | Pile | N | 707 | 707 | 0.0% | 250 | 1230 | 392.0% |
| | | V2 | 13 | 13 | 0.0% | 5 | 2960 | 59100.0% |
| Mid pier | Pier column | N | 1643 | 1643 | 0.0% | 660 | 2206 | 234.2% |
| | | V2 | 1 | 1 | 0.0% | 2 | 130 | 6400.0% |
| | Pile | N | 683 | 683 | 0.0% | 345 | 1480 | 329.0% |
| | | V2 | 14 | 14 | 0.0% | 8 | 3420 | 42650.0% |

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.

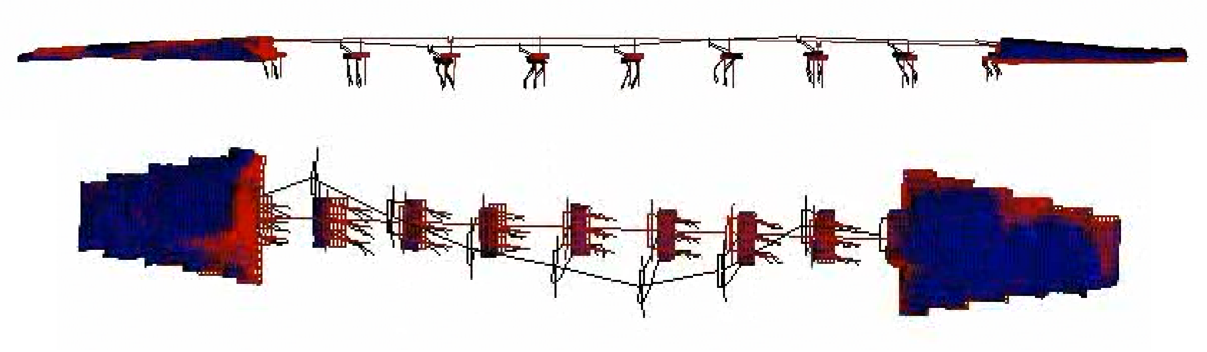


Figure 13. Instant Incoherent Accelerations of Bridge Structure in Elevation and Plan (Blue is deformed shape and Red is undeformed shape)

Fig.14 shows a bridge deformed shape during an incoherent seismic event. The plot scale is large scale and shows the effect of long wavelength wave in moving the entire bridge like a ship on ocean waves. Due to the random incoherent local differential displacements, the piles and pier columns are subjected to large shear stresses that cause them to deform differently along their length.

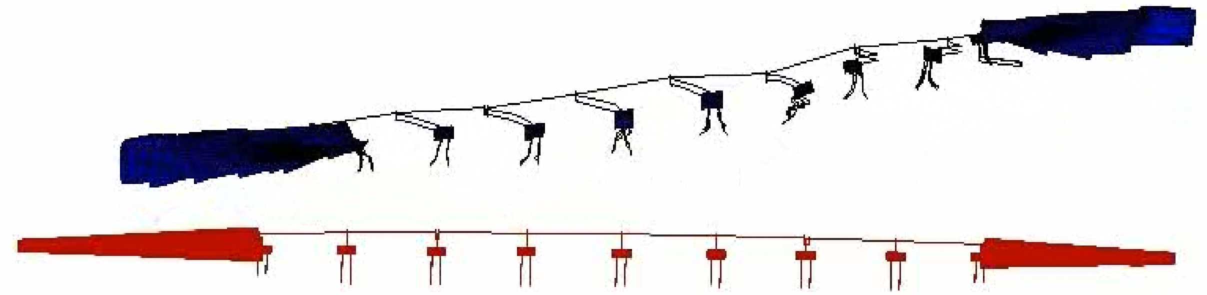


Figure 14. Instant Incoherent Relative Displacements wrt to Free-Field Motion (control point at infinite distance) (Blue is deformed shape and Red is undeformed shape)

Table 4 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.

Table 4. Stress comparison for earthquake loading in Z direction

| | SPA_VAR | INCOH_MEAN |
|---|---|---|
| Consecutive foundations relative displacements [mm] | 7.54 | 14.45 |

## CONCLUSIONS AND FUTURE DIRECTIONS

The scope of this paper is to present results of a seismic SSI study of a newly completed Fartec highway bridge and compare the current practice models with the state-of-the-art seismic analysis models.

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.

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