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5ICEGE
5th International Conference on Earthquake Geotechnical Engineering
January 2011, 10-13 | Santiago, Chile
Paper No. SROGH

SEISMIC RESPONSE OF PIER FOUNDATIONS OF AN EXISTING ROADWAY BRIDGE

Priyanka GHOSH[1], Filippo SANTUCCI de MAGISTRIS[2], Giovanni FABBROCINO[2]

ABSTRACT

Recent earthquakes in Italy pointed out the sensitivity of existing infrastructures to hazardous environment. This applies not only to buildings, but also to road networks and other infrastructures. Attention is generally focused on above ground components and on the soil response in free-field conditions to strong ground motion, but relevance of performance of foundations in dynamic field is generally not completely investigated and established. This is the background of the paper, which analyze the behaviour of an existing bridge deep foundation, using numerical tools.
The study takes advantage of structural and geotechnical data derived from field investigations and review of design reports and drawings. In particular, the foundation Trigno VIII bridge, along the Trignina Highway in the Molise region, Italy is considered here. The foundation, that follows a typical scheme adopted for several bridges along the road, is constituted by group of piles connected with a hexagonal cap. It is designed basically to carry the dead as well as live loads and resting on the river bed with layered soil deposit of 11.7 m deep underlying the strong bedrock. The geotechnical layout is determined based on the SPT data available for that location. The analysis is carried out with and without water table, to explore the effect of water table on the seismic response of the foundation system. The main objective is to determine the seismic response at the base of the pile cap. The seismic analysis is performed in the presence of both horizontal and vertical acceleration obtained from a recorded Italian strong motion earthquake. The 3D seismic analysis is performed numerically by using the Fast Lagrangian Analysis of Continua (FLAC^{3D}) software.

Keywords: Bridge foundations; Dynamic response; Pore pressure development; Seismic vulnerability.

INTRODUCTION

The assessment of seismic response of any civil engineering structure such as bridges is very much essential as the damage of such massive structures may lead to catastrophic failure (Pinto et al., 2009). Several investigations have been performed by different researchers to estimate the seismic risk of the superstructure of a bridge (see for instance Priesley et al., 1996; Pinho et al., 2007; FIB, 2007).

However, it is often found to be more challenging to obtain the seismic response of the whole bridge including super and sub structure, which can be effectively handled by employing the method of sub-structuring, where the structural and geotechnical aspects of the structure are decoupled to facilitate the complicated seismic analysis. In such a decoupled geotechnical seismic analysis of a soil-foundation

[1] Assistant Professor, Indian Institute of Technology Kanpur, Dept. of Civil Eng., email: priyog@iitk.ac.in
[2] Structural and Geotechnical Dynamics Lab., StreGa, SAVA Department, University of Molise.

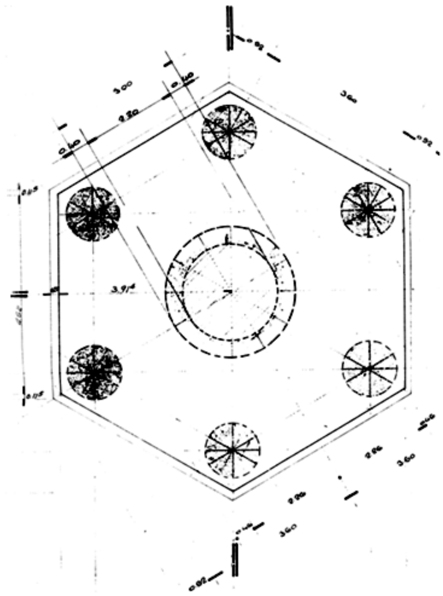
system, several issues related to geotechnical engineering such as soil-structure interaction, scouring near bridge foundation can be addressed quite efficiently.

The dynamic response of pile foundation was studied by various researchers (i.e., Gazetas et al., 1993; Wu & Finn, 1997). Recently, Thavaraj et al. (2010) proposed a quasi-3D continuum method for the dynamic nonlinear effective stress analysis of pile foundation under earthquake excitation. The behaviour of the different components in a piled-raft foundation is quite clear under static loads (Mandolini & Viggiani, 1997; Mandolini et al., 2005) but it is an interesting research issue in seismic areas, particularly addressing the important practical problem of kinematic interaction (Gazetas, 1984; Fan et al., 1991). Yegian et al. (2002) reported 3D dynamic soil-foundation analyses of the new Woodrow Wilson Bridge to demonstrate the effect of site conditions and pile foundations on the design seismic motions. It was observed that in the presence of scour, pile foundations on the river bed were found to move significantly more than the free field, whereas for foundations outside the river, pile cap motions were found to be smaller than free-field motions.

As well known, the seismic response of soil-pile-structure can be obtained by referring to the method of substructures, which requires that the study of soil-foundation-structure can be divided into the following steps (Gazetas & Milonakis, 1998; Dente, 2005):

- A_0. Kinematic analysis of the interaction that develops under the influence of the earthquake motion on a simplified scheme that includes the foundation and the soil, while the mass of the above-ground structure is set equal to zero.
- A_1. Seismic analysis of the superstructure subject to the actions of point A_0;
- A_2. Analysis of the interaction between the foundations, loaded by the forces of the phase A_1, and the soil, with determination of the characteristics of stresses in piles. These stresses are added to those of phase A_0, assuming valid the principle of superposition of the effects. In this phase, the limit states of the foundation are also computed.

(a)



(b)

Figure 1. View of the Trigno VIII Bridge (a) and plan view of a pier foundation after the original design table (b)

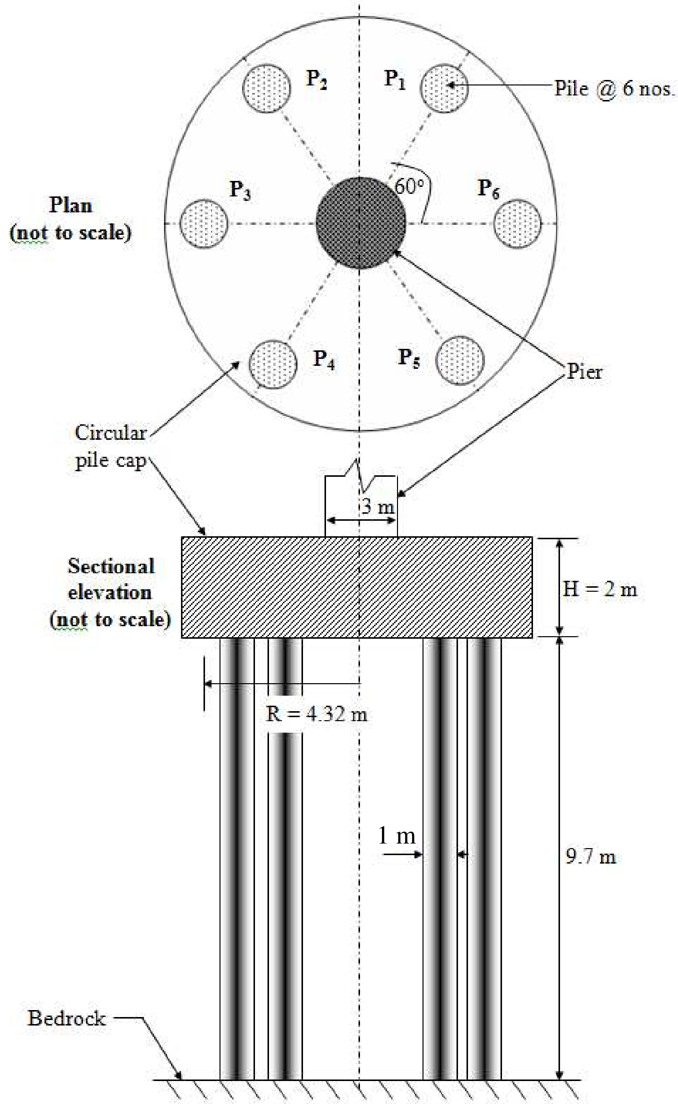


Figure 2. Schematic diagram of pile foundation system

This paper, in a broader programme aimed at defining the seismic vulnerability of existing key infrastructures and strategic buildings of the Molise Region, Italy (Di Carluccio et al., 2009; Evangelista et al. 2011), presents a numerical study on the seismic response of a pile group foundation under the pier of an existing viaduct using the commercially available finite difference code (FLAC^{3D}) (Itasca, 2006), according to point A_0 of the previous list.

The viaduct, Trigno-VIII (Figure 1a), is located in the Molise region, Italy. The foundation consists of a pile group of six piles along with a hexagonal pile cap. To avoid stress concentrations in the corners, in the present analysis, the hexagonal pile cap (Figure 1b) is converted to an equivalent circular cap with six piles oriented at 60^{o} angular spacing between two piles (Figure 2). The whole foundation system rests on a layered soil bed whose corrected SPT N values are obtained from the standard penetration test performed in the actual location. The horizontal and vertical acceleration-time histories obtained from the 1976 Friuli earthquake are considered as the input excitation at the bedrock level. The effect of water table on the seismic response is also explored in the present study. The shear forces and moments induced at the different location of piles and pile cap due to the seismic excitation have been critically studied both in absence and presence of the water table.

NUMERICAL ANALYSES

Trigno VIII viaduct is located in the east part of the Molise region, Italy. The bridge crosses the Trigno River near the city of Trivento in the Campobasso prefecture. The viaduct is constituted by a single carriageway having width of 11.60 m divided into 8 spans for a total length of 272 m. Each span has constant length of 34.00 m, and an average slope of about 1.83%. The decks are girder, with three longitudinal beams prestressed and four prestressed beams, two head and two in the span. The beams have a double T section current with no bulb symmetrical. The structural type is uniform. The devices supports are Steel-Teflon and therefore the static scheme of the structure is simple beam supported at the ends. All columns have circular hollow section. Foundations are homogeneous piled rafts. The pile foundation under the pier of Trigno-VIII viaduct consists of a group of six circular piles of 1 m diameter along with a circular pile cap placed on a layered soil strata lying over the bed rock at depth of 11.7 m. The piles are spaced at 60^o angular spacing between each other and the center of the piles are 1 m away from the periphery of the pile cap. The pier gives a vertical permanent static load intensity of $9.9 \cdot 10^5$ N/m^2 on the top of the pile cap over a circular area of 3 m in diameter.

The goal of the analyses is to assess the seismic response of soil-foundation system in terms of the acceleration-time history at the base of pile cap, forces and moments in the piles and pile cap, and settlement behavior of foundation due to the application of horizontal and vertical seismic excitation at the bed rock level.

Materials

The pile group with six numbers 9.7 m long and 1 m diameter circular piles are considered to rest on the layered soil deposit as found in the location of Trigno-VIII viaduct. The piles are rigidly connected with the circular pile cap of 8.64 m in diameter (D) and 2 m in depth (H). The pile cap is completely embedded in the soil with the top surface flushing with the ground surface.
In the surrounding area alluvial soils together with cemented sandy materials and marls and stiff clays are found. The geotechnical model adopted for the soil deposit, based on specific tests and previous available investigations for the seismic zonation of the area comprising of four layers resting on strong bedrock at 11.7 m below the ground surface. The thickness of top three layers is considered as 3 m each, whereas the bottom layer is 2.7 m thick. The different properties and strength parameters of each layer are obtained empirically from the available field standard penetration test (SPT) results. The variation of corrected SPT N values along with the details of each layer is shown in Figure 3.

The elastic modulus (E), angle of internal friction (ϕ) and undrained cohesion (c_u) are obtained from the SPT N values by using empirical correlations recalled in Evangelista et al. (2011). The Poisson's ratio of soil is considered as 0.3.

Due to the lack of reliable piezometric measurement, in the following analyses water table has been fixed both at the ground surface or at great depth (i.e., dries conditions). The bulk and shear modulus of the embedded concrete pile cap are considered as $1.39 \cdot 10^7$ kN/m^2 and $1.04 \cdot 10^7$ kN/m^2, respectively; whereas the elastic modulus and Poisson's ratio for concrete pile are taken as $3.0 \cdot 10^7$ kN/m^2 and 0.2, respectively.

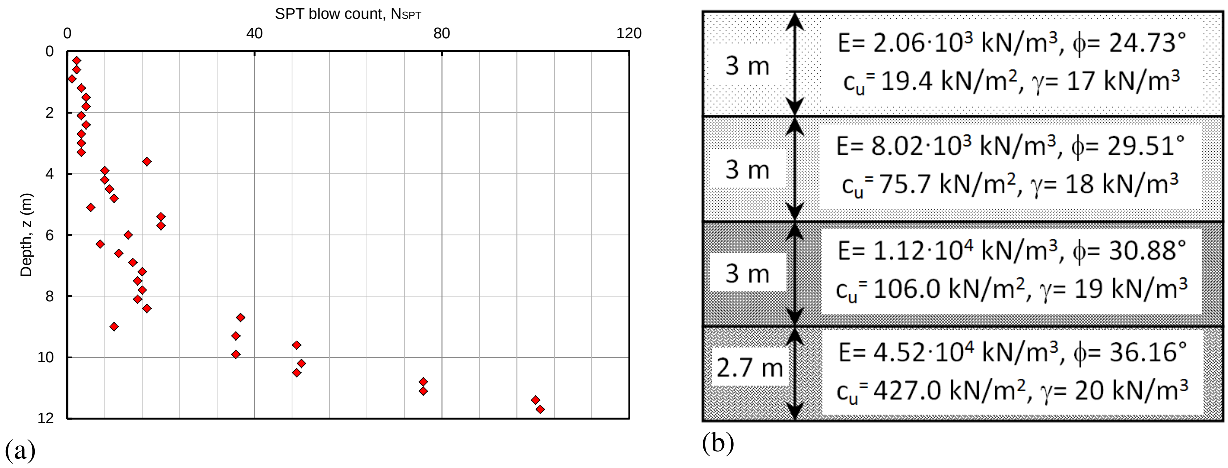


Figure 3. SPT N values (a) and geotechnical model (b) adopted for the numerical analyses

FLAC^{3D} model

The three dimensional FLAC^{3D} mesh generation with brick elements for the soil-foundation system along with pile group and circular pile cap is shown in Figure 4. It is important to note here that the piles are rigidly connected at the base of the pile cap and no separation is considered at the interface of pile cap and piles. The pile cap is considered as perfectly rough and there is no slippage and separation at the cap-soil interface. Piles are modeled with the pile elements available in FLAC^{3D} and the skin friction and end bearing properties are obtained from the surrounding soil properties. First, the static analysis of the whole soil-pile foundation system has been carried out with a static load intensity of $9.9 \cdot 10^5$ N/m^2 acting on the top of the cap over a circular area with diameter of 3 m. A sensitivity analysis has been performed to determine the optimum domain size with different distances such as 5*D*, 8*D* and 10*D* from the center of the pile cap along both x and y directions, where *D* is the diameter of the cap. Beyond the distance of 8*D* from the center of the pile cap, no significant changes in the results are observed at the cost of very high computational time.

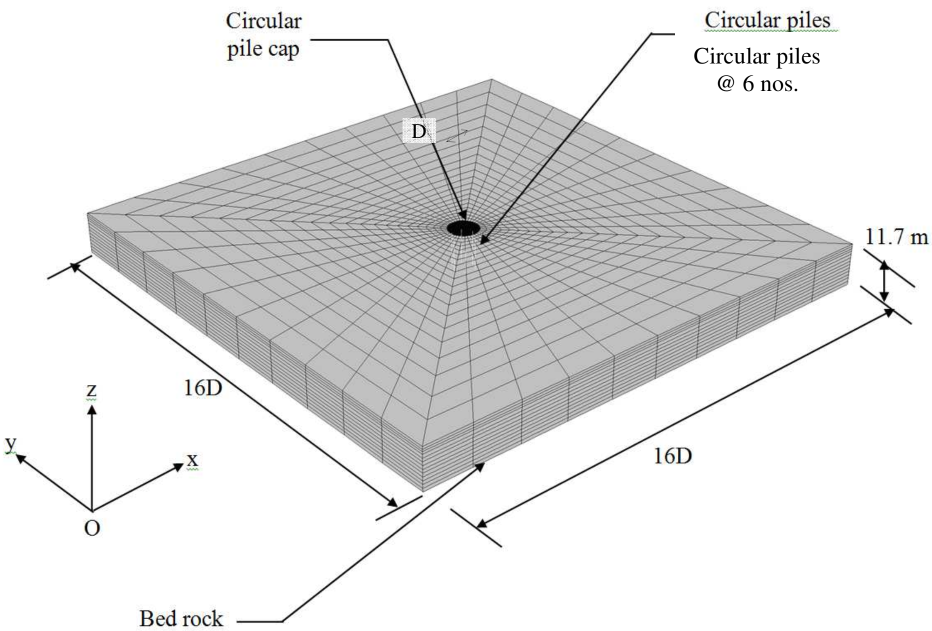


Figure 4. Domain width and meshing used in the present model

Therefore, in the present study the failure domain with 8*D* distance from the center of the pile cap is considered for the analyses. However, the depth of failure domain along negative z direction is extended up to the bedrock level, which is 11.7 m below the ground surface.

Rather than using a proper set of design acceleration of the area, in this case history the horizontal (N-S) and vertical acceleration-time histories obtained from the accelerometer registration at Tolmezzo station for the main shock of the earthquake of Friuli (Italy) on May 6th, 1976, are used as the horizontal and vertical input excitations along x- and z-directions, respectively at the base of the failure domain, i.e. at the bedrock level and the present seismic analysis has been carried out in the time domain. The data (WG ITACA, 2010) were sampled at 200 Hz for a total number of 7279 registration points. For the N-S component, the peak acceleration, equal to 0.339 g, was reached at the time t = 4.03 s. Most of the energy is included into a frequency range between 0.8 and 5 Hz, with a predominant frequency of 1.5 Hz. The Arias intensity is 0.78 m/s and the significant duration is 4.25 s.

In the present analysis, the soil is assumed to follow the Mohr-Coulomb failure criteria with non-linear failure envelope available in FLAC^{3D}, whereas the piles and the cap are assumed to obey the non-linear elastic model. The analysis has been carried out in the following three steps:

- The model is solved under the gravity loading only.
- The model is solved with the static load intensity coming from the superstructure and acting on the pile cap.
- The model is solved with the application of horizontal (N-S) and vertical acceleration data of the Friuli earthquake as input excitation applied at the bed rock level.

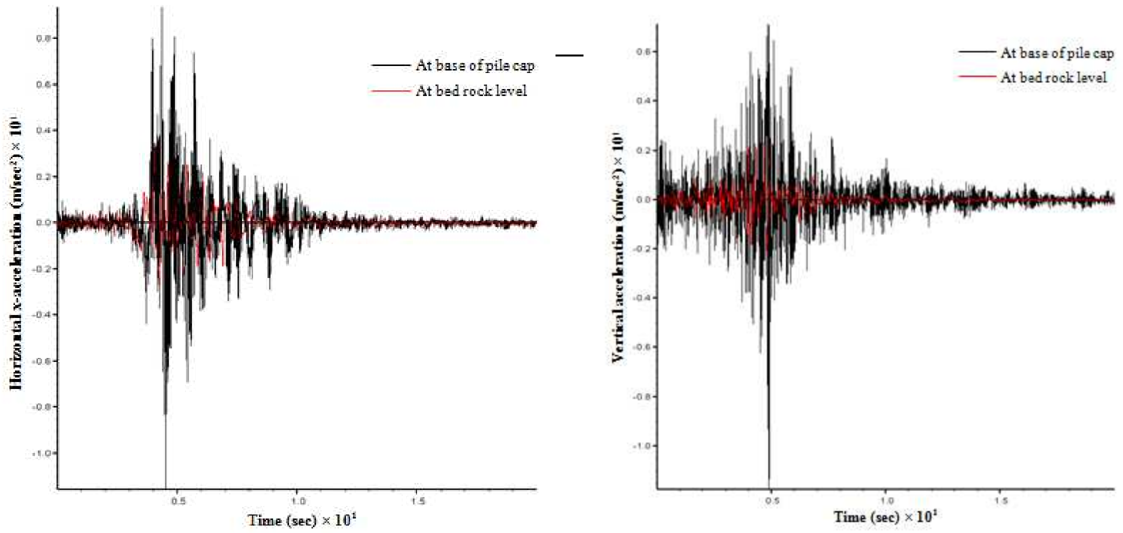
During the static analysis under steps 1 and 2, the extreme side boundaries of the failure domain are kept fixed in the direction normal to the respective plane, whereas the bottom boundary is considered fixed along x, y and z directions. However, during the seismic analysis all the side boundaries are considered as quiet boundaries, available in FLAC^{3D}, to avoid the reflection of seismic waves back to the domain again, whereas the bottom boundary is kept free to apply the input horizontal and vertical seismic accelerations along x- and z-directions. In the present analysis, 5% local damping is assumed for the soil strata and pile cap, whereas the piles are analyzed with combined damping available in FLAC^{3D}.

RESULTS AND DISCUSSION

In presence of input base excitation along x- and z-directions, the horizontal (x-direction) and vertical seismic acceleration response at the base of the pile cap and at different locations of the failure domain is obtained with and without water table. In presence of water table, the development of pore pressure at different locations is also determined. Occurrence of liquefaction is not considered here.

Dry soils

The x- and z-acceleration responses at the base of the pile cap along with the actual input excitation at the base are shown in Figure 5. It can be seen that the acceleration response at the base of the cap is much higher than that at the bedrock level, which eventually leads to an average amplification factor of more than 3. The maximum values of horizontal (x) and vertical accelerations at the cap base are obtained as 11.55 m/sec^2 and 11.75 m/sec^2, respectively; whereas those for the input excitation are found to be 3.36 m/sec^2 and 2.57 m/sec^2, respectively. The variation of vertical settlement of the pile cap with time is shown in Figure 6. It is worth mentioning here that Figure 6 shows the settlement under seismic excitation



(a) (b)

Figure 5. Horizontal - x direction – (a) and vertical (b) acceleration response at base of pile cap and at bedrock level without water table

only. However, the settlement due to the application of static load is not exclusively shown in this paper as it is assumed that the existing foundation system is stable under the static condition.

It can be observed from Figure 6 that the settlement of the cap continuously increases as the time of earthquake increases. The maximum settlement of the cap under seismic condition only is found to be 9.25 cm, which is observed at the central location of the cap base. The magnitude of this cap settlement is 21.9% higher than that observed under the application of static load. Therefore, it is quite clear that the bridge might suffer serviceability failure caused by excessive seismic settlement. In the pile cap, the maximum shear stress occurs at the base of the cap, whereas the piles experience the maximum shear stress at the cap-pile junction. The magnitude of maximum moment increases with the increase in the depth of piles.

Saturated soils – water table at ground level

The x- and z-acceleration responses at the base of the pile cap along with the actual input excitation at the base are shown in Figure 7. It can be observed that the acceleration response at the base of the cap is much higher than that at the base of the model, which ultimately leads to an average magnitude of amplification which is little higher than 3 times the input excitation.

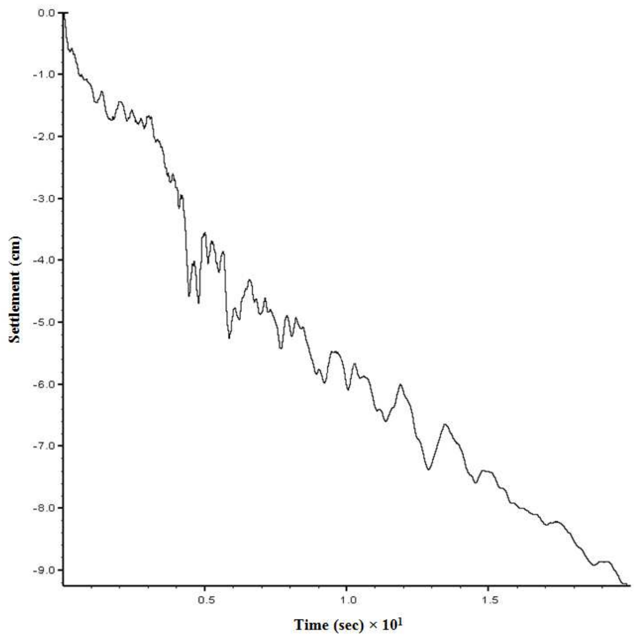


Figure 6. Vertical settlement response of pile cap under seismic condition without water table

The maximum values of horizontal (x) and vertical accelerations at the cap base are obtained as 10.26 m/sec^2 and 10.56 m/sec^2, respectively. The variation of vertical settlement of the pile cap with time is shown in Figure 8. It can be noted that the settlement of the cap continuously increases as the time of earthquake increases and the maximum settlement of the cap under seismic condition only is found to be 12.92 cm.

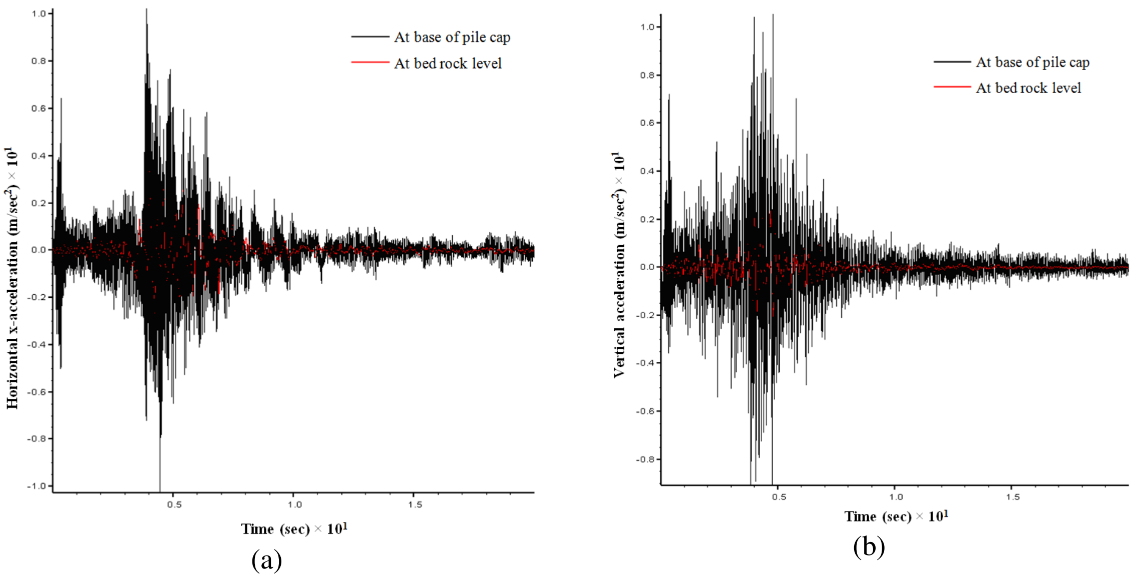


Figure 7. Horizontal - x direction – (a) and vertical (b) acceleration response at base of pile cap and at bedrock level with water table

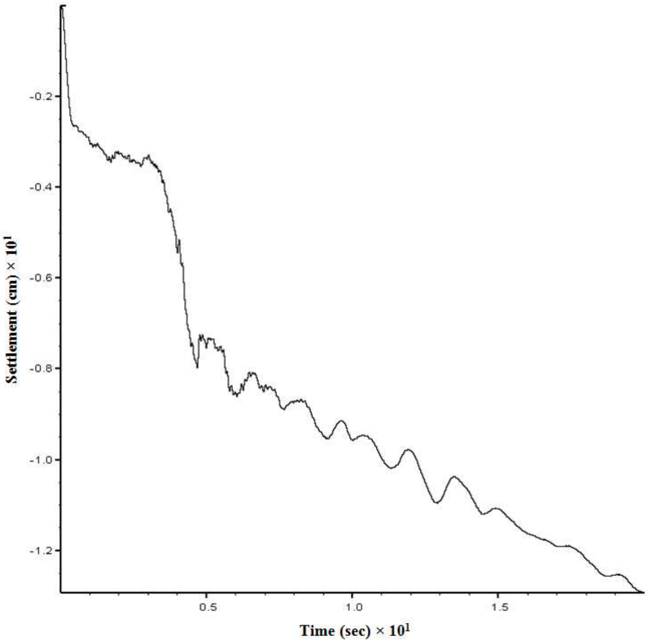
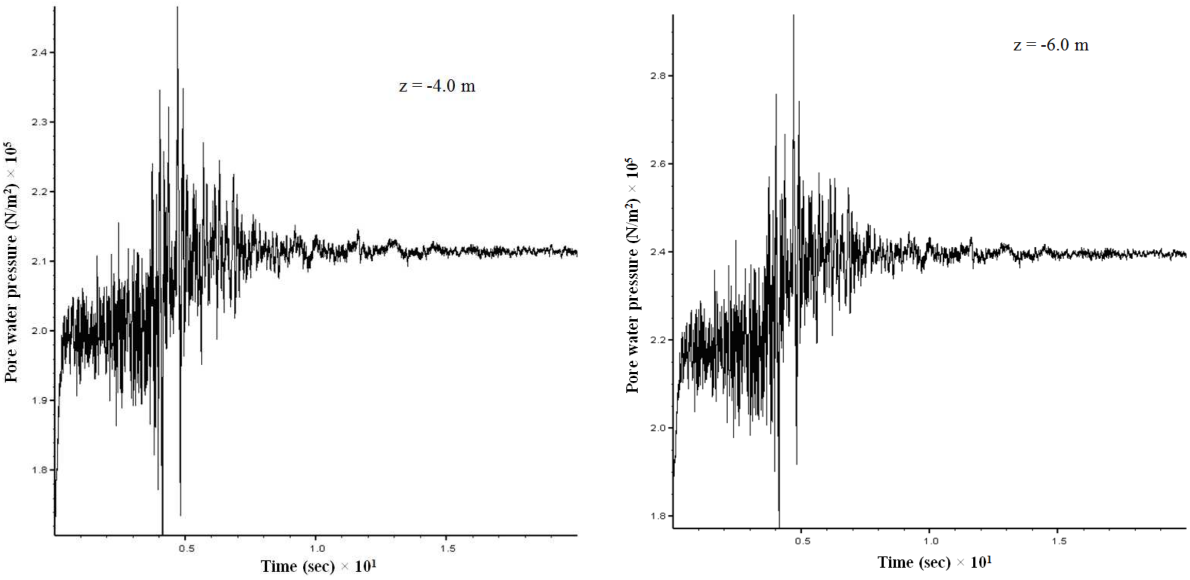


Figure 8. Vertical settlement response of pile cap under seismic condition with water table



(a) (b)

Figure 9. Pore water pressure developed below the cap at (a) z = -4.0 m g.l., (b) z = -6.0 m g.l.

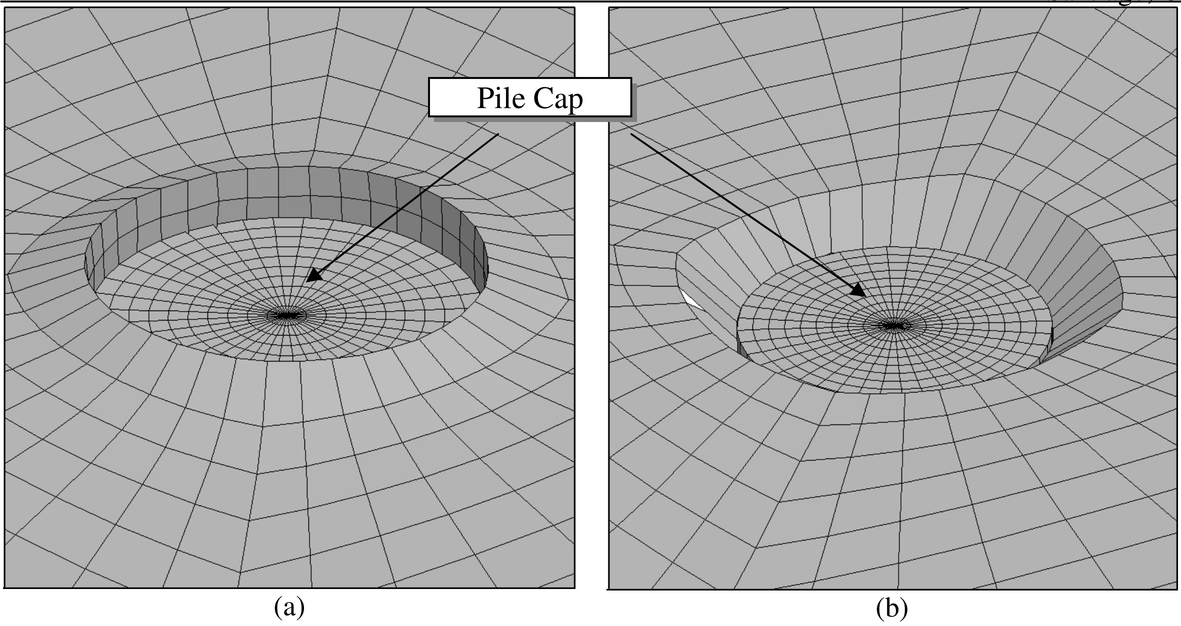


Figure 10. Deformed mesh near the pile cap (a) without water table and (b) with water table

In the pile cap the maximum shear stress occurs at the base of the cap; whereas the piles experience the maximum shear stress at the cap-pile junction, which is similar to the trend observed without water table. Similarly, the magnitude of maximum moment generally increases with the increase in the depth of piles. The pore water pressure developed at different locations of the soil deposit is shown in Figure 9. It can be seen that the pore pressure continuously increases as the time increases and becomes at most stable beyond 10 seconds. The magnitude of pore pressure also increases with the increase in depth.

The magnified deformed mesh near the pile cap is shown in Figure 10 both in presence and absence of the water table. It can be observed that the cap experiences a significant amount of settlement for both the cases. However, the soil mass surrounding the pile cap experiences a greater disturbance in presence of the water table.

CONCLUSIONS

In the present study the seismic response in terms of acceleration and settlement with and without water table are obtained for the pile foundation system of the pier of Trigno-VIII viaduct situated in the Molise region of Italy. The horizontal (N-S) and vertical acceleration-time history of the Friuli earthquake is used as the input excitation at the bedrock level.

It is observed that a significant amount of amplification occurs both in the horizontal (x) and vertical (z) acceleration response at the base of the pile cap, which is generally little higher than three times the input acceleration applied at the bed rock. However, the magnitude of accelerations with water table is generally found to be lower than that without water table.

The maximum vertical settlement of the cap is found to be 12.92 cm and 9.25 cm with and without the water table, respectively. In the pile cap, the maximum shear stress occurs at the base of the cap irrespective of the presence or absence of the water table. The shear stress in the piles generally decreases with the increase in depth keeping the maximum value at the cap-pile junction; whereas the reverse trend can be observed for the maximum moment distribution in the piles. The distorted mesh near the pile cap shows that the foundation settles significantly inside the ground keeping a little heave around the cap.

The present seismic analysis would be very much useful to assess the vulnerability of the existing Trigno-VIII pier foundation and also the results reported here could be used by the design engineers to design and assess the behavior of such complicated foundation system under seismic condition. It is clear that the results provided in the present paper are not exhaustive, but confirm the feasibility of a so complex numerical approach. Further analyses are needed to confirm some aspect of the dynamic response of the foundation and provide guidelines for seismic vulnerability assessment of existing bridge foundations.

AKNOWLEDGMENT

Dr. Ghosh was on sabbatical leave at the University of Molise under the program "Scholarships for young Indian researchers" funded by the Italian Ministry of Education, University and Research. The joint support received by the Ministry and by the University of Molise is greatly appreciated.
This research is a part of project funded by the Italian Highway Agency ANAS. The fruitful cooperation with the management of ANAS Molise Department is really acknowledged.

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