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# BRIDGE PIER FOUNDED ON PILE-GROUP: DUCTILE DESIGN AGAINST FAULTING

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**ABSTRACT:** This paper explores the feasibility of ductility design for bridge pile group foundations subjected to tectonic deformation. Its potential effectiveness is investigated utilizing a typical bridge structure as an illustrative example. It is shown that allowing plastic hinging at the piles can be an effective way of obtaining vertical offsets of greater magnitude. The penalty to pay is larger rotation at pier base due to pile yielding.

**KEY WORDS:** Soil-structure interaction; foundation; faulting; ductility.

## 1 INTRODUCTION

In many large magnitude earthquakes the causative fault rupture may extend all the way to the ground surface. Structures on top of the resulting surface fault scarp may undergo significant differential displacements that could lead to foundation and/or superstructure distress. Thus, seismic codes have invariably prohibited construction in the “immediate vicinity” of active faults. However, this demand is not very easy to comply with, especially for long structures such as bridges.

Modern seismic codes do not prohibit construction of structures near active seismic faults, but only after a special study is conducted, typically assuming elastic foundation response. Such requirement may be quite reasonable, since any damage to the foundation is typically difficult to discover and even more difficult to repair. However, such a requirement is particularly stringent and overly conservative in the case of piled foundations subjected to faulting-induced deformation.

To begin with, experience has shown that demanding elastic pile response in such an adverse case of loading leads to excessive levels of reinforcement that may be costly and even difficult (if not impossible) to construct. Most importantly, the hazard associated with a structure being subjected to faulting has a relatively low probability of occurrence, compared to strong seismic shaking.

First of all, experience has shown that the probability of a fault rupture outcropping all the way to the ground surface is relatively low. Even in such a case, the probability of a structure being “hit” by the rupture is substantially lower compared to shaking. As schematically illustrated in Figure 1, in a crudely simplified manner, in the worst case scenario (i.e., if the fault outcrops at the ground surface throughout its entire length) the area affected by the fault rupture will be a narrow zone (no more than 100 m wide) along the fault outbreak. For example, for a fault of length $L$ = 30 km, an area $E_{rupt} \approx 0.1$ x 30 km = 3 km$^2$ will be affected. In stark contrast, the area that will be affected by the vibratory component of the earthquake will be substantially larger : an eclipse covering an area $E_{vib} \approx 1500$ km$^2$. Evidently, if ductility design is acceptable for the shaking component of the earthquake, it is more than reasonable to be at least acceptable for the faulting-induced deformation component.

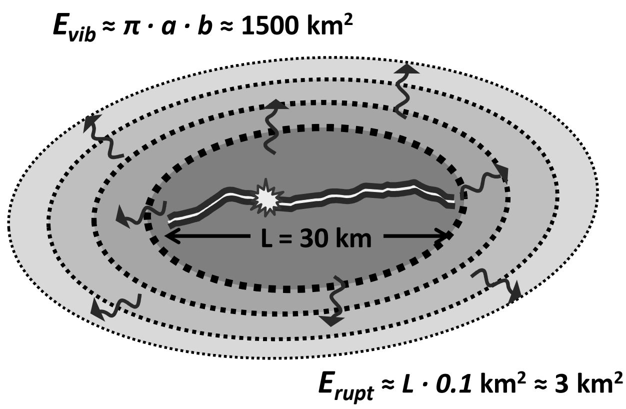


*Figure 1*. Schematic illustration of the areal effect of the two components of earthquake, fault rupturing and seismic oscillation.

This paper explores the feasibility of ductility design for bridge pilegroup foundations subjected to tectonic deformation. Its potential effectiveness is investigated utilizing a typical bridge structure as an illustrative example. The selected bridge structure is part of a new highway in Southern Greece, running through an area full of active seismic faults.

As shown in Figure 2, the bridge is 75 m long, with 3 simply supported decks on elastomeric bearings, having piers of height $H$ = 8 m. The foundation system is rather typical as well, consisting of 2 x 4 pilegroups with piles of diameter $d$ = 1.2 m and length $L$ =18 m.

Although piled foundations have been shown to be quite vulnerable to faulting induced deformation (Anastasopoulos et al., 2008; Gazetas et al., 2008), they still remain the most preferred solution for bridges combining transmitting superstructure loads to healthier soil strata, decrease of settlement and dynamic rotations.

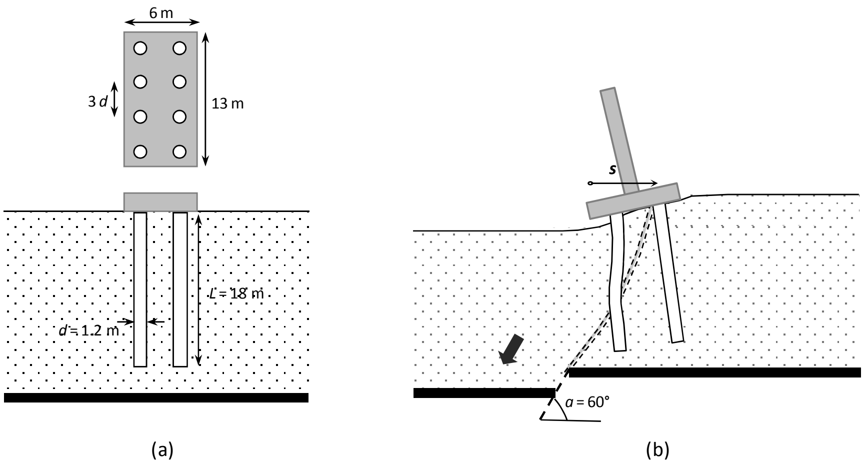


*Figure 2*. Schematic illustration of : (a) the piled foundation, and (b) the problem explored herein.

## 2 FINITE ELEMENT MODELING

To realistically simulate the response of the piles, 3D finite element modeling is required. The analysis is conducted employing the FE code ABAQUS. Figure 4 illustrates the geometry and the main features of the FE mesh. Half of the model was simulated, taking advantage of symmetry conditions.

The piles are modelled with beam elements circumscribed by 8-noded hexahedral "dummy" elements (i.e. of zero mass and stiffness). The central beam elements are rigidly connected to the appropriate circumferential element nodes of the same height. The non-linearity of the piles is introduced through moment-curvature relations, computed through cross-section analysis using XTRACT. The pile cap is modelled with hexahedral brick elements, assuming elastic reinforced concrete response ($E$ = 25 GPa). An appropriate interface was used to model the contact between the pile and the surrounding soil, to realistically simulate sliding and detachment between the piles or the pile cap and the corresponding soil.

The soil is modelled with hexahedral brick-type elements of dimension $d_{FE}$ = 1 m, with the mesh refined closer to the piles ($d_{FE}$ = 0.3 m). The non-linear behaviour of the soil is modelled with an elastoplastic constitutive model with strain softening (Anastasopoulos et al., 2007). The Mohr–Coulomb failure criterion is used to define failure accompanied by an isotropic strain softening law, which degrades the friction ($\varphi$) and dilation ($\psi$) linearly with the increase of plastic octahedral shear strain $\gamma^{pl}_{oct}$. The soil material utilized in the analyses is idealized dense sand with the following material properties: $\varphi_p = 45^o$, $\varphi_{res} = 30^o$, $\psi_p = 15^o$, and $E = 4000 \div 84000$ kPa. The dip angle of the fault plane was set to $a = 60^o$, while the maximum fault offset at the bedrock was set to $h_{max}$ = 1 m.

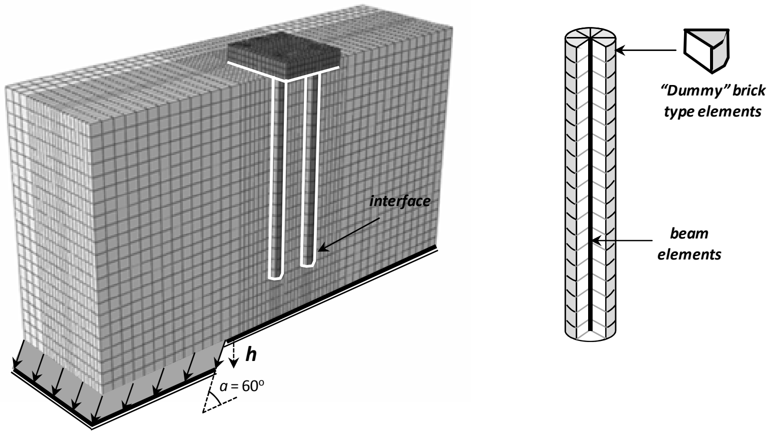


*Figure 3*. Outline of finite element model employed for the analysis of the pilegroup.

## 3 ELASTIC ANALYSIS

To argue for the irrationality of demanding elastic response of the foundation in such an adverse case of loading such as tectonic deformation, an example is presented in this section. To that end, the piled foundation of the selected typical bridge is analyzed, subjected to a bedrock offset of merely $h$ = 10 cm. The piles are assumed linear elastic.

First, fault rupture propagation through free field is analyzed. Then, knowing the exact location of fault rupture emergence, the foundation is subjected to fault rupturing at various distances $s$ from its hanging-wall (left) edge. The worst case scenario, both in terms of displacement/rotation and pile distress, corresponds to $s$ = 8 m in this specific case.

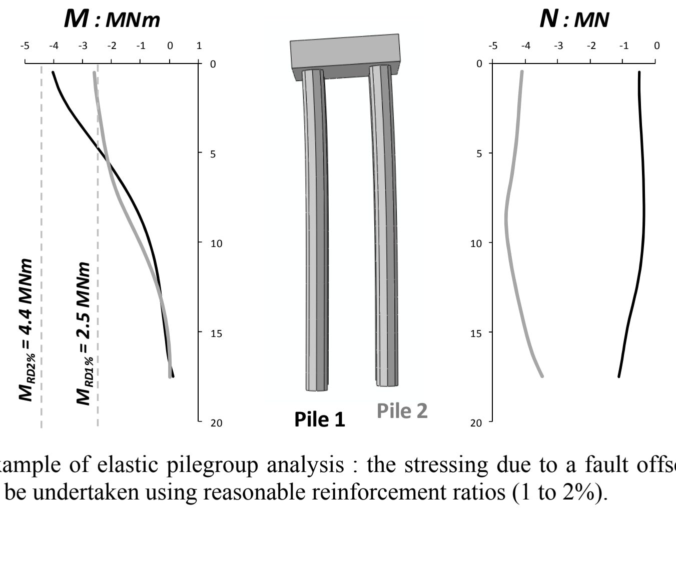


*Figure 4*. Example of elastic pilegroup analysis : the stressing due to a fault offset $h$ of merely 10 cm cannot be undertaken using reasonable reinforcement ratios (1 to 2%).

Figure 4 depicts the distribution with depth of pile bending moments *M* and axial forces *N*. Even a fault offset *h* of merely 10 cm, the developing *M* can only be undertaken with a rather "heavy" reinforcement ratio of the order of 4%. With more realistic reinforcement rations (1% to 2% at most, resulting to ultimate moment capacity $M_{ult}$ = 2500 kNm and $M_{ult}$ = 4400 kNm, respectively), the piles would unavoidably yield developing plastic hinges. In conclusion, very "heavy" reinforcement ratios are required to undertake elastically even minor bedrock offsets of the order of few centimeters. Such reinforcing can be unreasonably expensive and most of the times very difficult to realize.

## 4 NON LINEAR ANALYSIS

### 4.1 The effect of longitudinal reinforcement ratio

Allowing plastic hinging at the foundation may lead to a significant increase of the dislocation the piles may sustain. To that end, the nonlinearity of the piles is introduced in the analysis. Figure 5 depicts the moment curvature curves of the pile for two characteristic cases of reinforcement ratio: a lower bound of 1% corresponding to the current code of practice, and an upper bound of 4% (corresponding to the maximum reinforcement that may be installed in a pile). As depicted in Figure 5, ultimate capacity and ductility capacity are two contradicting concepts. As expected, the increase of the reinforcement ratio from 1% to 4% leads to a (3 times) larger bending moment capacity of the pile. However, this also results to a decreasing of its ductility capacity (by a factor of 2 in this particular case).

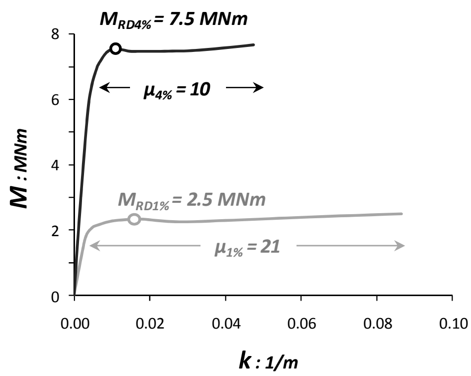


*Figure 5.* Moment curvature relations for the *d* = 1.2 m piles, as calculated through XTRACT.

Figure 6 compares the two reinforcement ratios in terms of ductility demand over ductility capacity $\mu_{demand}/\mu_{capacity}$ with respect to the imposed bedrock offset *h* and fault outcropping location *s*. When $\mu_{demand}/\mu_{capacity}$ exceeds 1, the pile has reached its ultimate bending capacity, plastic hinges have developed, and the response is nonlinear. Pile failure is reached when the ductility capacity is exhausted, meaning that the pile has not only developed plastic hinges, but has suffered significant damage being essentially destroyed.

Evidently, the performance of the heavily reinforced piles is favorable. The lightly reinforced piles fail at $h$ = 45 cm, while those the heavily reinforced can endure vertical bedrock offset of the order of $h$ = 70 cm. In both cases, however, exploiting the plastic region of the piles increases spectacularly the levels of bedrock offset the piles may endure (compared to the elastic analysis).

The advantageous performance of the heavily reinforced piles though is not concentrated only on enlarging the ultimate bedrock offset. Not only there are fewer lines above the failure line in the case of heavy reinforced piles, but the width of the zone where the piles fail is substantially narrower. In other words, not only a greater bedrock offset is needed to lead the heavily reinforced piles to failure, but the fault has to outcrop at a very specific location, minimizing substantially the possibility of failure due to tectonic deformation.

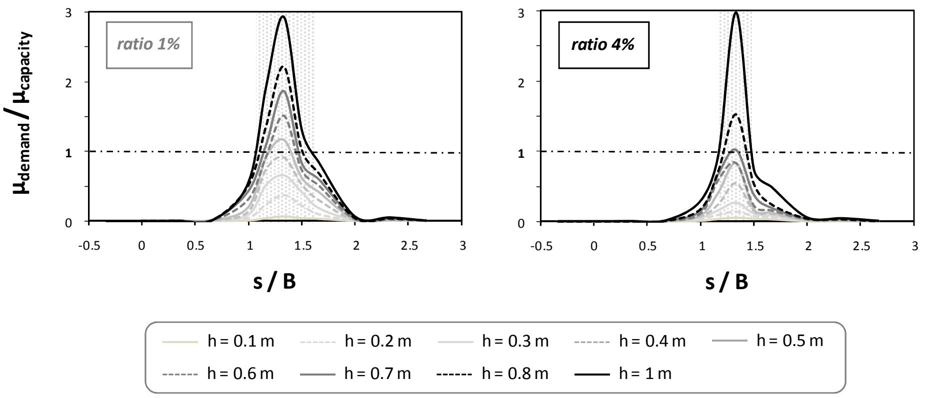


*Figure 6.* Evolution of ductility demand to ductility capacity with respect to the fault outcropping location *s,* and the fault offset *h*.

## 4.2 Consequences to the superstructure

Allowing the formation of plastic hinges at the foundation level seems to be a quite effective way to obtain larger values of vertical bedrock offset, that could not be obtained otherwise, and therefore increase the design margins. However, there is always a price to pay. Exploiting the ductile response of the foundation unavoidably generates adverse consequences to the superstructure. Pile yielding inevitably leads to an increase of the rotation they develop due to rupture imposed bending.

Figure 7 compares the two reinforcement ratios (1% and 4%) in terms of pilecap rotation $\theta$ with respect to the normalized outcrop distance $s/B$ for two representative levels of bedrock fault offset, $h$ = 30 cm and $h$ = 50 cm. The increase of $\theta$ is directly associated with the level of pile yielding. For $h$ = 30 cm (Figure 7a), the heavily reinforced pilegroup (4%) develops almost half the rotation of the lightly reinforced system (1%). Observe that there is practically no difference between the heavily reinforced and the elastic pilegroup. For $h$ = 50 cm, the differences between the "heavily" and the "lightly" reinforced system becomes larger, since, now, both pilegroups have entered their plastic region and behave nonlinearly.

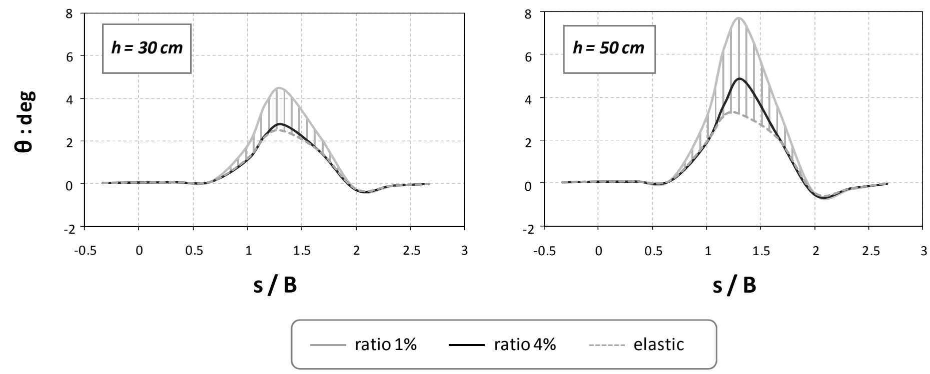


*Figure 7.* Rotation at the pier base with respect to the normalized fault outcrop location *s/B* , for $h$ = 30 cm and 50 cm. Pile yielding leads to an increase of pier rotation, that has to be taken into account.

In summary, the increase of pier rotation due to pile yielding may be significant. Although it may decrease with heavier reinforcement, it may still be substantial, and has to be taken into account in design.

## 5 CONCLUSIONS

The key conclusions can be summarized as follows:

a) The demand for elastic response of a piled foundation of a bridge pier against large tectonic deformation is unreasonably conservative and sometimes even meaningless. It is shown that even a minor bedrock offset of a few centimetres may cause pile yielding. With elastic design, this unavoidably leads to a requirement for excessive reinforcement that can be, apart from expensive, impossible to construct.
b) Allowing plastic hinging at the piles can be an effective way of obtaining vertical offsets of greater magnitude and therefore increasing the design margins.
c) Heavier reinforcement may lead to a more favourable performance. In spite of reducing the ductility capacity, the increased bending moment capacity results to large deformation margins before the exhaustion of the ductility capacity. Apart from the increase of fault offset they can sustain, using larger

reinforcement ratios leads to decrease of the width of the area where the piles are really rupture sensitive.

d) Although the adoption of a ductile pilegroup design can be an effective means to design for larger fault offsets, it is unavoidably associated with an increase in pier rotation. This is something that has to be taken into account in design, in order to avoid falling of a deck.

## ACKNOWLEDGMENTS

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