INFLUENCE OF SOIL FLEXIBILITY ON THE BEHAVIOR OF EXISTING BRIDGES IN REGIONS OF MODERATE SEISMICITY

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ABSTRACT :

For the assessment of existing bridges a realistic modeling of the system behavior is crucial to capture the relevant influences on the seismic response. This is particularly the case for structures that have not been designed according to modern seismic design principles as they can develop inelastic deformations in parts that would otherwise be capacity protected. This holds also true for the soil-foundation system of existing cantilever piers with spread foundations which may develop significant inelastic deformations and thus even prevent the piers from entering into the inelastic range. As a consequence, the seismic behavior of the bridge structure can change completely compared to a fixed-base pier model, especially with respect to the local member deformation demands. In this paper, a simplified engineering model to estimate the influence of soil flexibility at spread foundations is presented. The model is applied on a sample structure with varying parameters to show the consequences of different conditions and assumptions on the structure response.

KEYWORDS: soil flexibility, spread foundations, bridge assessment, existing bridges

1. INTRODUCTION

Due to intensive research in the past decades significant progress could be made in the development of concepts for the seismic design of new structures. Modern capacity design principles are capable of ensuring that the structure's dynamic behavior is rather predictable and plastic deformations will only occur in predetermined zones which were designed and detailed to control the damage. However, in many countries the majority of bridges has already been built before the establishment of modern seismic design codes, thus not necessarily complying with current standards. As a consequence even in countries of moderate seismicity, like for example Switzerland, a sufficient seismic safety for the existing bridges might not be warranted in every case. For this reason the Swiss authorities established a program to assess all major Swiss highway bridges with respect to their seismic safety.

Generally, in seismic analysis it is desired to model the structure's dynamic behavior in a realistic manner. Whereas for design purposes conservative assumptions in many cases might be sufficient as they do not cause significant extra costs, for the assessment of existing structures a more realistic modeling approach may be justified in order to avoid unnecessary over-conservatism. Also, a structure that had not been designed according to modern capacity design principles may develop significant deformations in regions that would rather be capacity protected in a new design. A modern cantilever bridge pier would be designed to concentrate the inelastic deformations in a well detailed plastic zone at the base while capacity protecting other parts, like e.g. the foundation system, from inelastic deformations. The inelastic behavior of an older existing structure, however, can differ significantly from this desired mode. While an existing pier may have very limited deformation capacity, in some cases even incorporating shear problems, its foundation system may contribute significantly to the total deformation capacity. Eventually, the foundation might even have a lower strength than the pier itself. In this case, inelastic deformations will rather concentrate in the soil-foundation system instead of the concrete pier, thus changing the seismic behavior completely compared to that of a modern bridge structure.

If these differences in the seismic behavior are not taken into account for the assessment of an existing bridge the result may be completely unrealistic and even predict a wrong failure mode. While for a new structure the modeling of the soil-foundation system might be of secondary interest, as long as it is designed to have sufficient strength and stiffness, for the assessment of existing structures an appropriate modeling of the soil-foundation behavior can be crucial. Aside from the soil flexibility at pier foundations, the deformation behavior of the abutments can also significantly influence the global response of the structure as shown by several studies on the measured seismic behavior of real bridges (Wilson and Tan, 1990; Zhang and Makris, 2002, e.g.).
To study the influence of soil flexibility at pier foundations and abutments on the transverse response of existing bridges with typical characteristics of Swiss bridges from the 1960s a set of numerical analyses was conducted by Kuhn (2007). In this study, various bridge geometries, modeling parameters and analysis methods were applied to investigate the sensitivity of the results. In the following chapters, selected results from this study are presented showing the influence of pier foundation size on the global behavior of the structure as well as on the deformation demand of the piers. The presented results stem from inelastic time-history analyses (ILTHA) of one bridge model having varying foundation sizes. The analyses were conducted using seven recorded ground motions which were scaled to represent the moderate Swiss seismicity.

2. STRUCTURAL MODEL

The general geometry of the analyzed structure is shown in Figure 1. The chosen bridge system is based on a 257 m long six span highway bridge with typical characteristics of Swiss bridges from the 1960s and 1970s. The span widths range from 34 m to 57 m and the superstructure consists of a prestressed concrete boxgirder. For the analysis it is assumed that the superstructure remains essentially elastic and for simplicity a constant moment of inertia of 40 m^4 is being modeled. The heights of the piers shown in Figure 1 range from 6 m to 14 m and refer to the distance from the pier bases to the center of gravity of the superstructure thus representing the effective heights for the structural analysis. An unsymmetrical distribution of the pier heights along the length of the structure was chosen to cause some influence of higher modes on the seismic response.

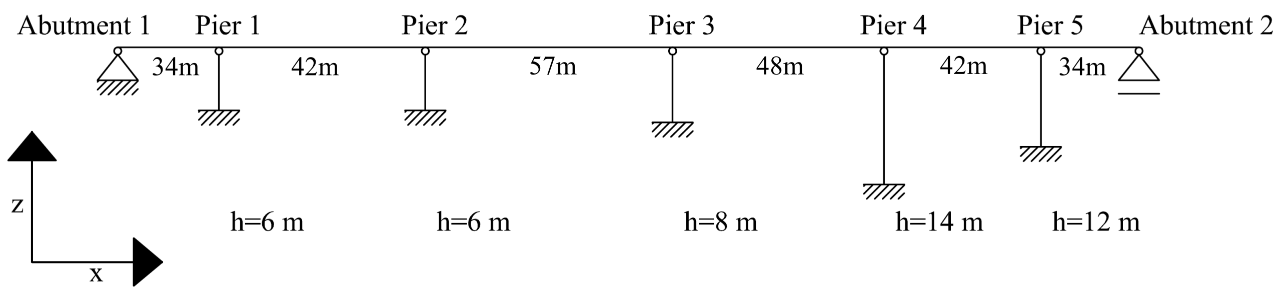

Figure 1 Bridge structural system

2.1. Modeling of the Piers

All piers are modeled to have the same cross-section shown in Figure 2. A concrete compressive strength of 30 MPa and a yield stress of 440 MPa for the longitudinal reinforcement are assumed. The longitudinal reinforcement ratio is 0.32% and the axial load ratios vary depending on the tributary superstructure length between 8.6% and 12.6%.

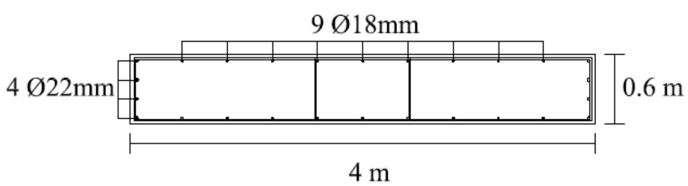

Figure 2 Cross-section of piers

For the given cross-section a monotonic moment-curvature analysis was performed. Based on this, the corresponding force-displacement relationships of the piers were estimated using a plastic hinge model and parameters as outlined in Priestley *et al.* (2007). The result is shown in Figure 3 for the case of pier 3 with an effective height of 8 m. For the inelastic dynamic analyses the member response of the piers was modeled using nonlinear springs with a hysteretic behavior defined by the modified Takeda model. The backbone of the modified Takeda hysteresis was taken as the bilinear approximation for the monotonic force-displacement curve shown in Figure 3. The parameters controlling the large cycle response were chosen to represent "thin" loops adequate for reinforced concrete bridge piers as recommended in Priestley *et al.* (2007). Thus the unloading parameter was chosen as $\alpha = 0.5$ and the reloading parameter as $\beta = 0$.

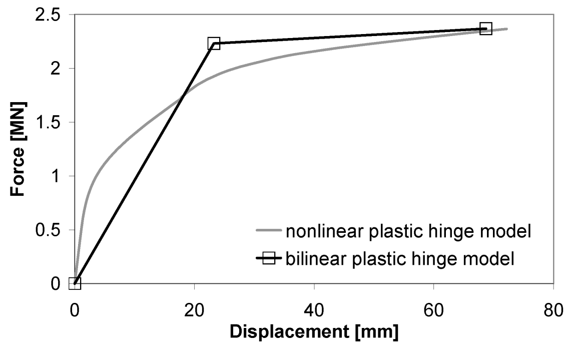

Figure 3 Force-displacement relationship of a 8 m pier

2.2. Pier Foundations

All piers are assumed to have spread foundations of equal dimensions. Although deformations of the concrete foundations themselves are not being taken into account significant additional flexibility can result from elastic and inelastic soil deformations, eventually combined with uplifting of the foundation. The soil beneath the foundation is modeled by means of Winkler springs assuming a bilinear-elastic behavior in compression as shown in Figure 4. Consequently, the soil stress distribution from vertical load and moment can be in one of the four stadiums shown in Figure 5, depending on the foundation characteristics and the loading. An equivalent approach was proposed by Allotey and El Naggar (2003).

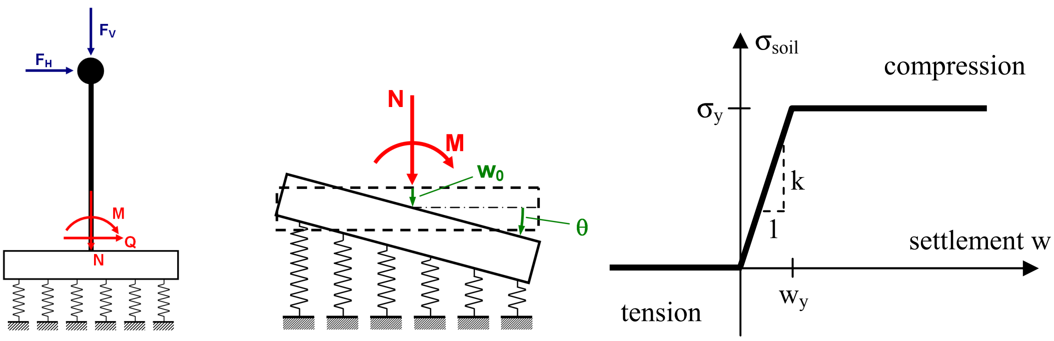

Figure 4 Soil-foundation model with bilinear-elastic Winkler springs

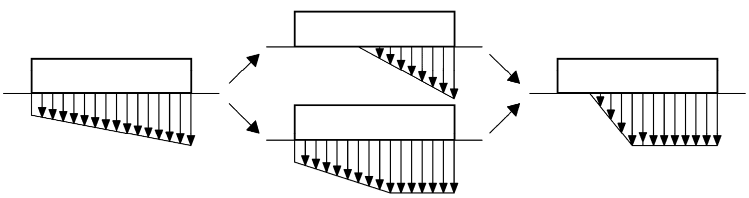

Figure 5 Soil pressure under spread foundations from moment and vertical load

The initial stiffness *k* of the bilinear-elastic soil model shown in Figure 4 can be estimated by means of a settlement calculation based on an elastic half-space and the yield stress σ_y is determined as the bearing capacity of the foundation soil. Thus, the full stress-strain relationship can be developed with standard geotechnical analysis methods.
For the dynamic analyses, three different types of pier foundations are studied. The first type is a medium size spread foundation with plan dimensions of 7.4m x 3.8m, the second one is a larger spread foundation with corresponding dimensions of 10m x 5m, and in the third case the piers are modeled with a rigidly fixed base for comparison purposes, disregarding any soil flexibility. Assuming the concrete foundations not to deform and using equilibrium considerations the moment-rotation relationships of the soil-foundation systems can be calculated for a given vertical load. As soon as the soil leaves the first stress state shown in Figure 5, by either uplifting of the foundation or yielding of the soil, the moment-rotation relationship will become nonlinear developing a yielding behavior. It should be noted that this behavior also develops if the soil itself is infinitely strong (purely linear elastic behavior) from uplifting of the foundation only. For dynamic loading this represents a rocking behavior. The resulting nonlinear moment-rotation relationship for the example of the 10m x 5m foundation is shown in Figure 6 together with a corresponding bilinear approximation.

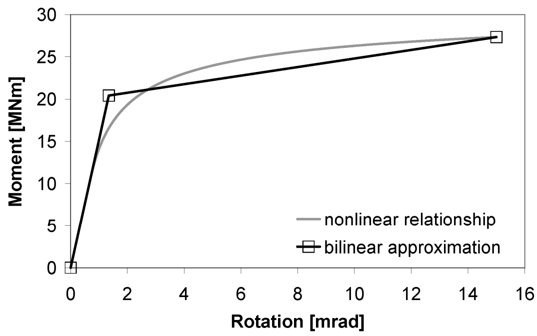

Figure 6 Moment-rotation relationship of a 10m x 5m spread foundation

Dividing the moment by the total pier-foundation height and multiplying the rotation with the same height, a force-displacement relationship can be calculated which represents the deformation behavior resulting from the soil only. Although experimental evidence (Negro *et al.*, 2000) suggests that these inelastic soil deformations can develop significant hysteretic behavior and thus add hysteretic energy dissipation to the system, the presented model is not capable of reliably estimating this hysteretic damping. As the additional displacement capacity from the soil-flexibility can already be considered a beneficial effect it might be unconservative to model an uncertain hysteretic damping which may not be warranted by the real behavior. Therefore, as a cautious approach it was chosen to model the soil by nonlinear elastic springs without any hysteretic behavior.

2.3. Abutments

Several approaches to model the seismic abutment behavior suggested in the literature were studied by Kuhn (2007). As already shown in Zhang and Makris (2002) the modeling uncertainties in terms of variation between different models are very large, making a realistic and reliable estimation of abutment stiffness at the current state almost impossible. To some extent this may be attributed to the complex interaction between abutment and embankment as well as the rather individual nature of abutments. For the given bridge structure the different models considered in Kuhn (2007) resulted in transverse abutment stiffnesses varying between 12 MN/m and 675 MN/m, with some models also defining a yield load.
Considering these uncertainties, at the current state of knowledge it may be recommendable to perform a parameter study calculating the response for a variety of abutment stiffnesses rather than only relying on one single estimate. Such a parameter study was performed in Kuhn (2007) and it was shown that it may not be possible to determine a single stiffness that would be conservative for all local member responses of the bridge structure. In particular, it is not necessarily conservative to only examine the two extreme cases of either rigidly pinned or completely released abutments.

For the analysis results presented in this paper a bilinear elastic spring in the horizontal transverse direction was used, having an initial stiffness of 30 MN/m and a yield load of 1.8 MN. The post-yield stiffness was chosen as 3 MN/m. Hysteretic behavior of the abutment or additional viscous damping was not modeled.

2.4 Dynamic Model of the Analyzed Bridge Structure

Based on the inelastic and nonlinear elastic springs developed in the preceding chapters, a complete 2-dimensional structural model for the transverse bridge response can be developed as shown in Figure 7. Each pier-foundation system is modeled by a combination of an inelastic Takeda spring representing the pier deformations (chapter 2.1) and a bilinear-elastic spring for the soil deformations of the foundation (chapter 2.2). Furthermore, truss members carrying the gravity loads of the piers are attached to allow for P-Δ-effects in the piers. The total mass of the superstructure (and some tributary mass from the piers) is concentrated in single masses at the pier tops and at the abutments. The transverse behavior of the abutments is modeled by the bilinear-elastic springs defined in chapter 2.3.

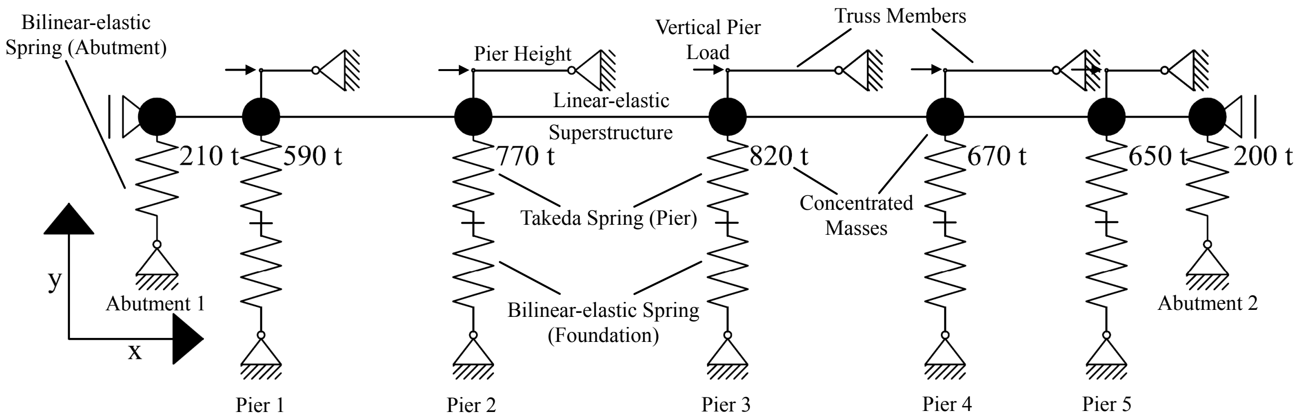

Figure 7 Dynamic system of bridge structure for inelastic dynamic analysis (in plan view)

It should be noted that for each pier-foundation system both springs have an individual yield force while carrying the same load. Therefore it is only possible that the spring with the lower yield force can yield, thus "capacity protecting" the other spring which will remain in its elastic range. As a consequence, even without demand calculations, it can already be determined which part might yield (according to the model) – the pier or the soil – and which one will remain elastic. Depending on the foundation size and the soil characteristics this may result in cases where right from the beginning an inelastic demand for the concrete pier can be ruled out.

3. ANALYSIS METHOD AND SEISMIC LOADING

The results presented in the following chapter were calculated by means of inelastic time-history analysis (ILTHA) using the software code Ruaumoko by Carr (2004) and counterchecked using SeismoStruct by Seismosoft (2006). As discussed in Bimschas and Dazio (2006), the two programs have different implementations of the modified Takeda hysteresis rule with respect to the small cycle response. Especially for the moderate seismic input used in this study, this causes slightly differing results as the system responds in small cycles during large part of the earthquake. However, the results agreed sufficiently well to consider them verified by the double calculation.

The seismic loading was originally defined by an elastic response spectrum according to the current Swiss code SIA 261 (2003). For the highest seismic zone and taking amplification factors for soil and high importance of the structure into account, the code provisions result in a peak ground acceleration of 0.27g. For the dynamic analyses, seven recorded ground motions were chosen which were scaled to meet the spectral acceleration of the elastic target spectrum at the fundamental period of the bridge structure.

As discussed in Bimschas and Dazio (2006), for the elastic viscous damping ratio used in ILTHA of normal bridge structures a range of ζ_{el} = 2% to ζ_{el} = 5% may be suitable. As in the given case the soil-foundation springs as well as the abutment springs are modeled without hysteretic behavior, it may be justified to choose a damping ratio in the upper region of this range. While Priestley *el al.* (2007) propose the use of tangent stiffness proportional damping for ILTHA, which reduces the viscous damping in the inelastic range significantly, for the present study a constant initial stiffness Rayleigh damping was chosen meeting ζ_{el} = 2% at the second and fourth mode. Consequently, a value of ζ_{el} = 2.3% results for the first mode. The overall dissipated energy by viscous damping is likely to be within the range mentioned above even if tangent stiffness proportional damping was used.
For each ground motion, the total peak displacements at the pier tops as well as the corresponding contributions from the piers and the foundations were determined by ILTHA. These results are used to calculate the medium values and standard deviations over all accelerograms.

4. ANALYSIS RESULTS

At first, in Figure 8 the total peak displacements of the pier tops and the abutments are shown for the bridge structure with rigidly fixed pier bases. Aside from the mean values resulting from all seven accelerograms, the variation in terms of the standard deviation is presented. It can be seen that, despite the fact that scaled recorded ground motions were used for the analyses, the standard deviation in this case is only in the order of 1 cm thus suggesting reasonably good agreement between the different responses. The corresponding results for the structures with the two types of spread foundations (7.4m x 3.8m and 10m x 5m) can be found in Figure 9. For the smaller foundation the standard deviation is somewhat higher with a value of approximately 2 cm, while the results for the larger foundation are closer together, again having standard deviations in the order of 1 cm to 1.5 cm.

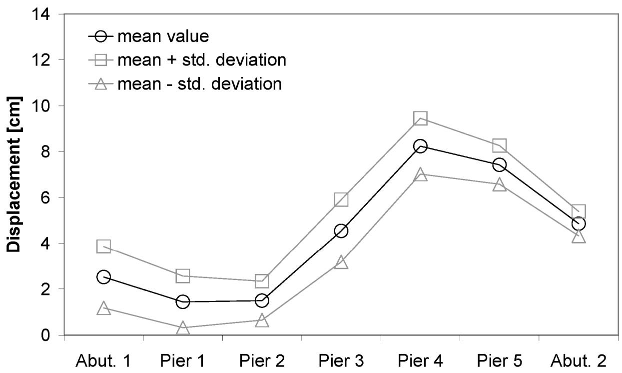

Figure 8 Peak total displacements for rigidly fixed pier bases

In Figure 10, the mean values for the three different systems are compared. Figure 10.a shows the total peak displacements resulting from pier and foundation deformations together. In Figure 10.b the deformations of the piers only are compared among each other and with the corresponding yield displacements, thus showing whether a pier experiences inelastic deformations or not.
It can be seen that the total displacement demand in Figure 10.a shows significant differences between the three systems. The small foundation system has the highest total displacements for most of the piers, while the fixed base system experiences the smallest total displacements. The response of the large foundation system lies between the two aforementioned. This result could be expected as the small foundation system is the softest one and the fixed base system is the stiffest one of the three.

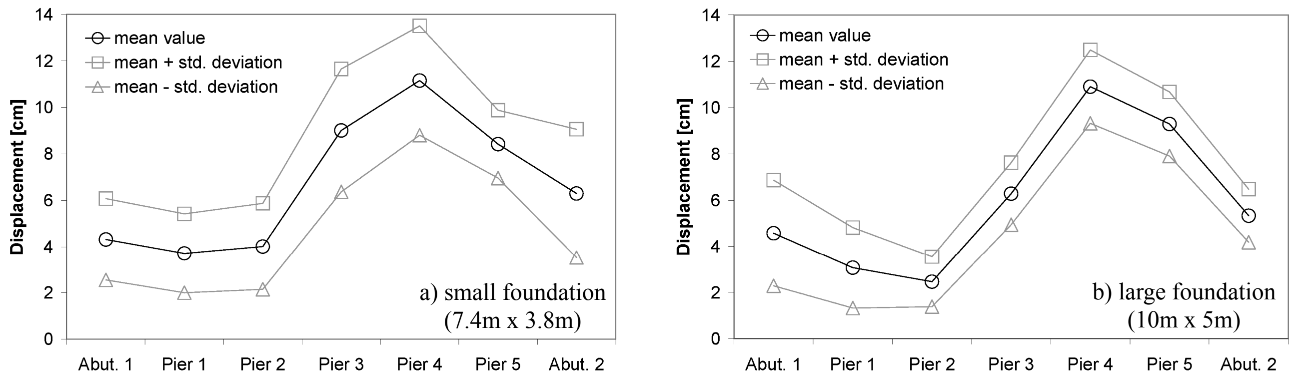

Figure 9 Peak total displacements for piers with spread-foundations

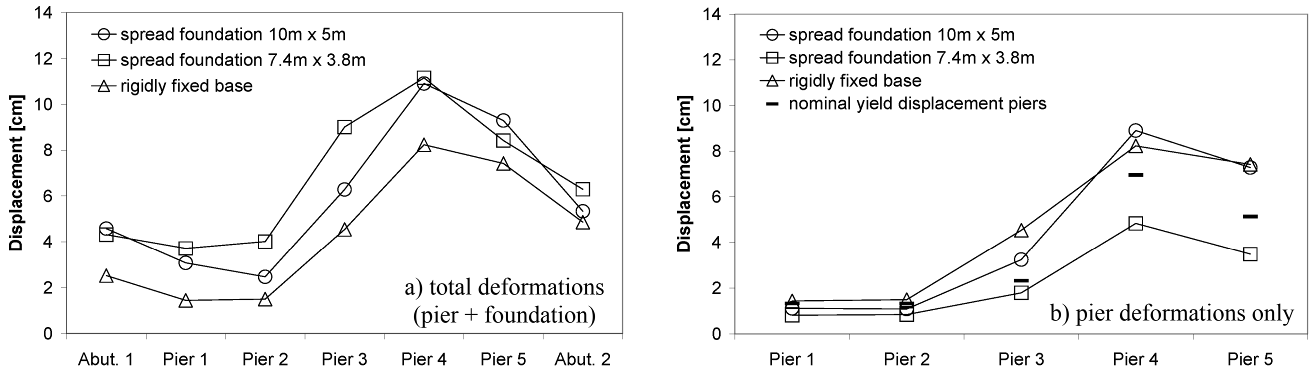

Figure 10 Comparison of peak displacements for structures with different foundation types

Although the various system responses in Figure 10.b showing only the pier deformations differ significantly as well, the relationships are rather contrary to those of the total displacements in Figure 10.a. For the pier deformations, the small foundation system shows the lowest values whereas the fixed base system has the highest values, with the large foundation system being very close to the latter one. The rather small pier deformations for the small foundation case are the result of the low foundation strength which protects the piers from yielding. As a consequence, all piers remain elastic while the yielding foundations contribute a significant part to the total displacements.

Comparing the response of pier 4 for the two systems with spread foundations, it can be seen in Figure 10.a that the total deformations are almost the same in both cases. However, looking at the pier deformations only (Figure 10.b), the demand for the case of the large foundation is significantly higher. Again, this difference is a result of the relationships between pier and foundation strengths. While for the small foundation the soil yields and the pier remains elastic, the strength relation is contrary for the large foundation. In this latter case, the foundation strength is sufficient to cause yielding of the pier. This way the pier has to provide the majority of the displacement capacity while the elastic deformations from the foundation only contribute a minor part.

For the case of the short piers 1 and 2 both spread foundations – the small one as well as the large one – have lower yield strengths than the piers so that the latter ones remain in the elastic range with small deformations. It should be noted that for a given foundation size and the same cross-section of the piers it might be expected that the relation between yield strengths is the same for all pier heights as the moment strengths of the piers and the foundations remain the same, independent of the pier height. However, for the yielding of the pier the moment demand at its base governs, whereas the yielding of the soil depends on the higher moment at the foundation base. For constant foundation heights, the difference between these two moments depends on the pier height and is more pronounced for lower piers having a higher moment gradient.

5. CONCLUSIONS

It was shown that with standard engineering methods the influence of soil-flexibility at spread foundations can be estimated. Especially for the assessment of existing bridge structures where the foundations have not been designed to be capacity protected, it can be crucial to consider the soil behavior for the analysis as it might change the seismic behavior completely, particularly the local deformation demand at member level. The presented results relate to a moderate seismicity with limited total displacement demand. The differences in pier deformation demand between fixed and flexible base response are likely to be even more pronounced for higher seismic loading. It may also be noted that the longitudinal reinforcement ratio $\rho_l = 0.32\%$ of the piers considered was rather in the lower range of typical values. The increased moment capacity for higher reinforcement ratios will lead to an even larger influence of soil flexibility. As a consequence, many existing pier foundations which were mainly designed to support vertical loads may not have sufficient strength to cause yielding of cantilever piers. Very stiff and strong soils do not necessarily change this phenomenon as uplifting of the foundation can still cause nonlinear behavior and foundation dimensions on such soils can be expected to be rather small.
Despite of its value in visualizing and quantifying important phenomena concerning the interaction between the concrete piers and the soil-foundation systems, care should be taken with respect to the implicit uncertainties of the presented models. Aside from the obvious crudeness of the simplified engineering models, a significant uncertainty may exist in determining the appropriate soil parameters for the analysis. Thus, it may be advisable to conduct a sensitivity analysis using a wider range of parameter sets. In any case, not considering the influence of soil flexibility might give a rather mistaken image of the bridge response. Also, disregarding the flexibility of the abutments is likely to result in unconservative results for the displacement demand of the piers.

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