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NONLINEAR INELASTIC SEISMIC RESPONSE OF SLENDER BRIDGE PIER ON SURFACE FOUNDATION

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Abstract. *The paper highlights the results of a numerical study of the seismic response of a tall bridge-pier type structure supported on surface foundation. Both the structure and the soil are treated as inelastic materials — the former capable of developing plastic hinging at its column base, the latter capable of mobilizing bearing-capacity type "failure" mechanisms. Moreover, the geometric nonlinearity arising from the uplifting of the foundation due to the large overturning transmitted moment is treated rigorously. Exciting a typical such system with two actual earthquake records, it is shown that whereas a conventionally-designed system may suffer substantial structural plastic permanent rotation at the column base (and thereby even collapse in a very strong event), a more-daring unconventional design which specifically allows inelasticity/nonlinearity in the soil and the soil–foundation interface to take place suffers only permanent settlement and in some cases permanent tilt of the foundation without suffering any structural distress.*

1. INTRODUCTION : CONVENTIONAL WISDOM AND THE NEED FOR CHANGE IN THE DESIGN PHILOSHOPHY

Seismic design of structures recognises that highly inelastic material response is unavoidable under the strongest possible shaking for the specific underlying soil where the structure is founded. "Ductility" levels of the order of 3 or more are usually allowed to develop under seismic loading, implying that the strength of a number of critical bearing elements is fully mobilised. In the prevailing structural terminology "plastic hinging" is allowed as long as the overall structural stability is maintained.

By contrast, a crucial goal of current practice in seismic "foundation" design, particularly as entrenched in the respective codes (e.g. EC8), is to avoid the mobilisation of "strength" in the foundation. In *structural* terminology : *no* "plastic hinging" is allowed in the foundation, below the ground surface.

Thus the conventional approach to seismic foundation design introduces factors of safety against sliding and against exceedance of ultimate capacity, in a way similar to the traditional static design. This approach involves two consecutive steps of structural and foundation analysis :

(a) Dynamic analysis of the structure is first performed, in which the soil is modelled as an elastic medium represented by suitable translational and rotational springs (and, sometimes, with the associated dashpots). The dynamic forces and moments transmitted onto the foundation are derived from the results of such analyses, after the column forces have been reduced by dividing with a *ductility-capacity* depended factor.

(b) The foundations are then designed in such away that these transmitted horizontal forces and overturning moments, increased by *"overstrength"* factors, would not induce sliding or bearing capacity failure.

The use of *"overstrength"* factors is necessitated by the so-called *"capacity design"* principle, under which plastic hinging is allowed only in the structural elements — not in the below−ground (and hence uninspectable) foundation and soil. Therefore, structural yielding of the footing and mobilisation of bearing capacity mechanisms is not allowed. However, there is a growing awareness in the profession of the need to consider soil-foundation inelasticity, in analysis and perhaps even in design [1-10]. This need has emerged from :

- The very large accelerations and velocities recorded in earthquakes, which would impose enormous inelastic demands to structures if soil−foundation "yielding" would not effectively limit the induced accelerations.
- Seismic retrofitting of a structure increases the shear capacity of some elements and hence the forces onto their foundations ; it might not be feasible to undertake them elastically. A (stiff) concrete shear wall inserted to upgrade a frame carries most of the inertia-driven shear, and thereby transmits a disproportionately large horizontal force and overturning moment onto the foundation. If uplifting, sliding, and mobilisation of bearing capacity failure mechanisms are correctly taken into account, the shear wall "sheds" off some of the load onto the columns ; the opposite is erroneously the case when such inelastic action is disallowed.
- Many slender historical monuments have apparently survived several strong seismic motions in their (often long) life. While under static conditions they would have easily toppled or otherwise failed, it appears that sliding at, and especially uplifting from, their base during oscillatory seismic motion has been a key to their survival. These phenomena can not therefore be ignored.

- Compatibility with state of the art structural earthquake engineering is another reason to compute the complete inelastic lateral load–displacement or load–rotation response of the foundation system, to progressively increasing loads up to collapse. Otherwise the *"performance-based"* structural analysis, will be incomplete.

It is therefore logical to extend the inelastic analysis to the supporting foundation–soil system.

2. ALLOWING "PLASTIC HINGING" IN THE SOIL-FOUNDATION SYSTEM

Excluding structural yielding in the isolated footing or the foundation beam, three types of nonlinearity can take place and modify the overall structure–foundation response :

(a) *Sliding at the soil–foundation interface*. This would happen whenever the transmitted horizontal force exceeds the frictional resistance. As pointed out by Newmark in his 1965 Rankine Lecture, thanks to the oscillatory nature of earthquake shaking, only short periods of exceedance usually exist in each one direction ; hence, sliding is not associated with failure, but with permanent irreversible deformations, as it will be shown in the subsequent section of this paper.

(b) *Separation and uplifting of the foundation from the soil.* This happens when the overturning moment tends to produce net tensile stresses at the edges of the foundation. Thanks again to the oscillatory nature of the seismic shaking, the ensuing rocking oscillations in which uplifting takes place do not lead to overturning of the structure. There is not detriment to the vertical load carrying capacity and the consequences in terms of induced vertical settlements may be minor. Moreover, in many cases, footing uplifting is beneficial for the response of the superstructure, as it helps reduce the ductility demands on columns.

(c) *Mobilisation of bearing capacity failure mechanisms in the supporting soil.* Such inelastic action is almost unavoidable with uplifting of the foundation. In *static* geotechnical analysis large factors of safety are introduced to ensure that bearing capacity modes of failure are not even approached. In *conventional seismic* analysis, bearing capacity is avoided thanks to an *"overstrength"* factor of about 1.40.

The oscillatory nature of seismic shaking, however, allows again the mobilisation *(for a short period of time)* of the maximum soil resistance along a continuous ("failure") surface. No collapse or overturning failure occur, as the applied causative moment quickly reverses, and a similar bearing-capacity mechanism may develop under the other edge. The problem again reduces to computing the inelastic deformations, i.e. the *permanent rotation*.

3. RESULTS OF A COMPARATIVE STUDY

To illustrate the interaction between soil, foundation, and structure under strong seismic shaking mobilising inelastic deformations in the soil we have selected the system portrayed in Figure 1. A single-column concrete bridge pier, 3m in diameter and about 12 m high, carries a deck with a mass of 1200 Mg). It is founded with a surface square foundation (side B) on a 25 m thick stiff clay deposit *(S_u = 150 kPa)*.

Two foundation solutions are examined :

(a) B = 11m. This derives from a conventional slightly-conservative design, which leads to a large static bearing–capacity factor of safety ($FS_V \approx 5.6$) and a pseudo-static bearing-capacity factor $FS_E \approx 2$, for a code-specified design spectrum having A = 0.24 g, corresponding to soil category "B", and an estimated *"behaviour"* or *"q"* factor of 2. Although the value of 2 is high for a seismic FS, it was chosen in the anticipation of much-stronger accel-

eration histories than those specified by the A = 0.24 g code spectrum. It is thus quite possible that during a *design* or *stronger–than–design* event structural (bending) plastic hinging will develop at the base of the column, with a rather minimal inelastic action in the soil or the soil–footing interface — a typically prudent conventional design.

(b) B = 7 m. This corresponds to the new design concept, where significant plastic deformation is allowed to take place in the foundation–soil system, to the point of mobilisation of the bearing–capacity failure mechanisms. These may develop alternatingly on either side under the footing, as large cyclic overturning moments arise during shaking. This design is barely adequate under static vertical loads ($FS_V \approx 2.8$) ; under the design earthquake it leads to a pseudo-static $FS_E = 0.50$ — well below unity to be acceptable within convential engineering thinking. It is therefore expected that during shaking by a design−level, and especially by an above−design−level, ground motion the soil "failure" mechanisms will develop. The question is what the consequences will be for the foundation and the superstructure. And how the computed response of the two systems of Fig. 1 differs from one-another.

The numerical finite-element modeling of the problem using ABAQUS is highlighted in Fig. 2(a). The hysteretic Mohr-Coulomb constitutive low describes soil behavior while suitable gap elements are attached at the soil–foundation interface. More detailed description of the modeling aspects can be found in Refs 9,10, 11. The two different real accelerograms used as excitation are given in Fig. 2(b). Specifically :

- The Kalamata Administration Building record of the 1986 $M_s = 6.2$ earthquake. With an A ≈ PGA ≈ 0.26 g but a response spectrum with values smaller than those of the design code spectrum at the period range of interest, this motion will be referred to as *"Small" Intensity* excitation.
- The Takatori record of the 1995 $M_{JMA} = 7.2$ Kobe Earthquake, which in addition to its high PGA, 0.63 g, has spectral values in excess of 1.5 g over a very wide period range (0.30 sec – 1.20 sec) . It thus undoubtedly constitutes a *"High" Intensity* excitation — substantially larger than a design excitation.

Figures 3 and 4 show the results for the *"Small" Intensity* shaking. Specifically, Figure 3 plots the time histories of the main components of superstructure displacement.:

- the displacement of superstructure mass due to foundation rocking : u_θ
- the (additional) displacement of the superstructure due to the bending of the column : u_{str}
- the algebraic sum of the above two : $u_{tot} = u_{str} + u_\theta$

The lateral displacement of the foundation is not shown, as it is quite secondary for the particular chosen slender system.

Figure 4 refers to the response of the foundation. It plots the dynamic moment-rotation, M − θ, relationship and the dynamic settlement–rotation, w − θ, relation for each of the two foundations. The conclusions emerging from the figures are clear :

- The conventional foundation (B = 11 m) experiences very small nearly-elastic rotation, with a small accumulation of cyclic settlement (≈ 2.5 cm). By contrast, the column experiences large distortion, almost 10 cm, as if the structure were responding on a fixed base.
- The "daring" foundation design, B = 7 m, experiences very large rotation, with u_θ reaching 12 cm. The significant inelastic action in the soil is reflected in the highly-hysteretic M – θ relationship, as well as in αn appreciable accumulated foundation settlement (≈ 6

cm). By refreshing contrast, the structural distortion has been limited to merely 2 cm — indicative of almost elastic column response.

In similar fashion, Figures 5–6 compare the results for the *"High" Intensity* (Takatori) excitation. The following is a summary of important conclusions from these plots :

- The conventional foundation design, B = 11 m, with too little help from the ("unyielding") foundation, can not cope with the huge accelerations of the Takatori record. Plastic structural (bending) deformations begin to accumulate after 5 sec of motion to non-converging large values exceeding 1.5 m — indicative of structural failure. (It already corresponds to an enormous ductility demand of the order of 30.) Although some inelastic action in the soil is evident in the $M - \theta$ and especially the $w - \theta$ plots, the response of the soil–foundation system is *more-or-less* as anticipated in the pseudostatic design.
- Diametrically different is the response of the B = 7 m design. Huge inelastic soil deformations, manifest themselves in the $M - \theta$ plot with uplifting and repeated alternating mobilisation of bearing-capacity failure mechanisms. The maximum rotation θ_{max} reaches 0.036 rad, corresponding to a substantial deck displacement $u_\theta \approx 50$ cm. Accumulated settlement : $w_{max} = w_{res} \approx 26$ cm. The superstructure, however, hardly deforms and thus remains safe despite the *much-larger-than-design* ground shaking. With a warning: the above significant displacements (26 cm and 50 cm) may imply such large differential settlements between adjacent foundations and differential displacements between adjacent piers that indirect structural damage is unavoidable.

One might arguably consider the above foundations deformations as excessive. Notice, however, that these are peak values ; the residual rotation and displacement appear to be very small — in fact, for this particular excitation (undoubtedly by sheer coincidence) they almost vanished !

In conclusion, inelastic "failure" mechanisms could be allowed to develop in soil-foundation systems designed for strong seismic excitation. Mobilisation of such mechanisms does not usually lead to failure ; it may in fact prove quite a beneficial way to save the structure.

Two supporting case histories are mentioned here : (a) the settlement of slender buildings in Adapazari during the 1999 Kocaeli Earthquake, despite mobilisation of bearing-capacity "failure" mechanisms (as tentatively explained in Ref. [12]).. (b) the behaviour of the Kobe harbour breakwaters with no overturning and only small permanent lateral displacements and rotations; whereas by contrast, the uni-directionally moving soil-supporting quaywalls [caissons identical to the breakwaters] suffered huge seaward displacements and rotations [13, 14].

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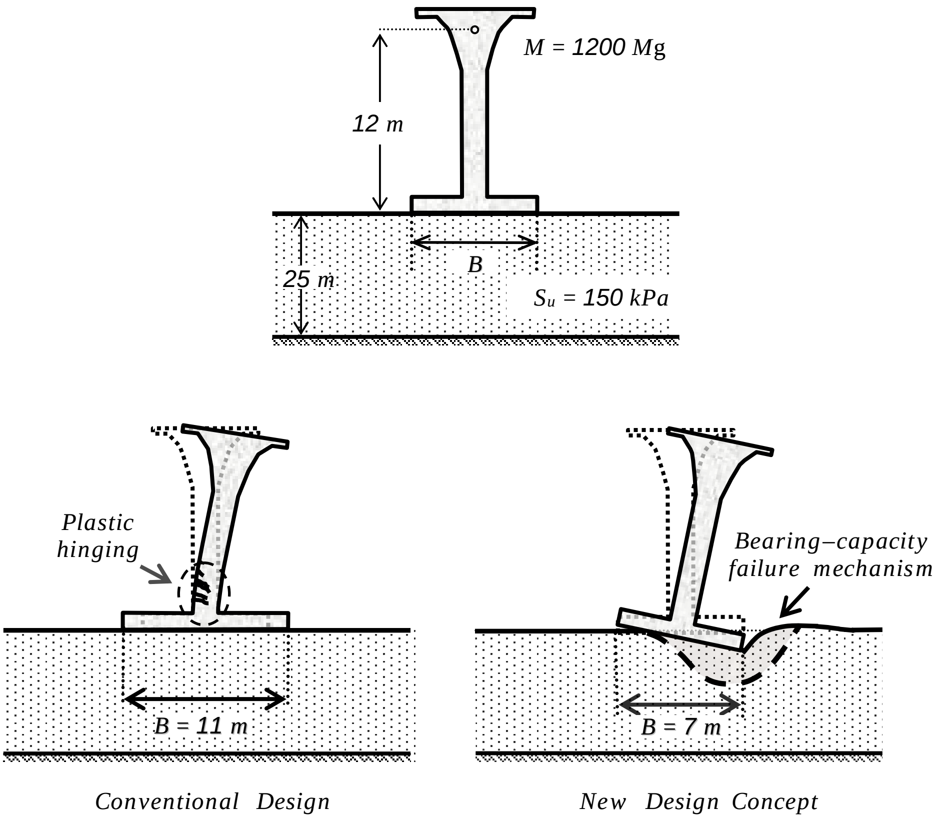

Figure 1 : Top: problem geometry. Bottom: the two studied foundation schemes, representing the conventional and the new design philosophies.

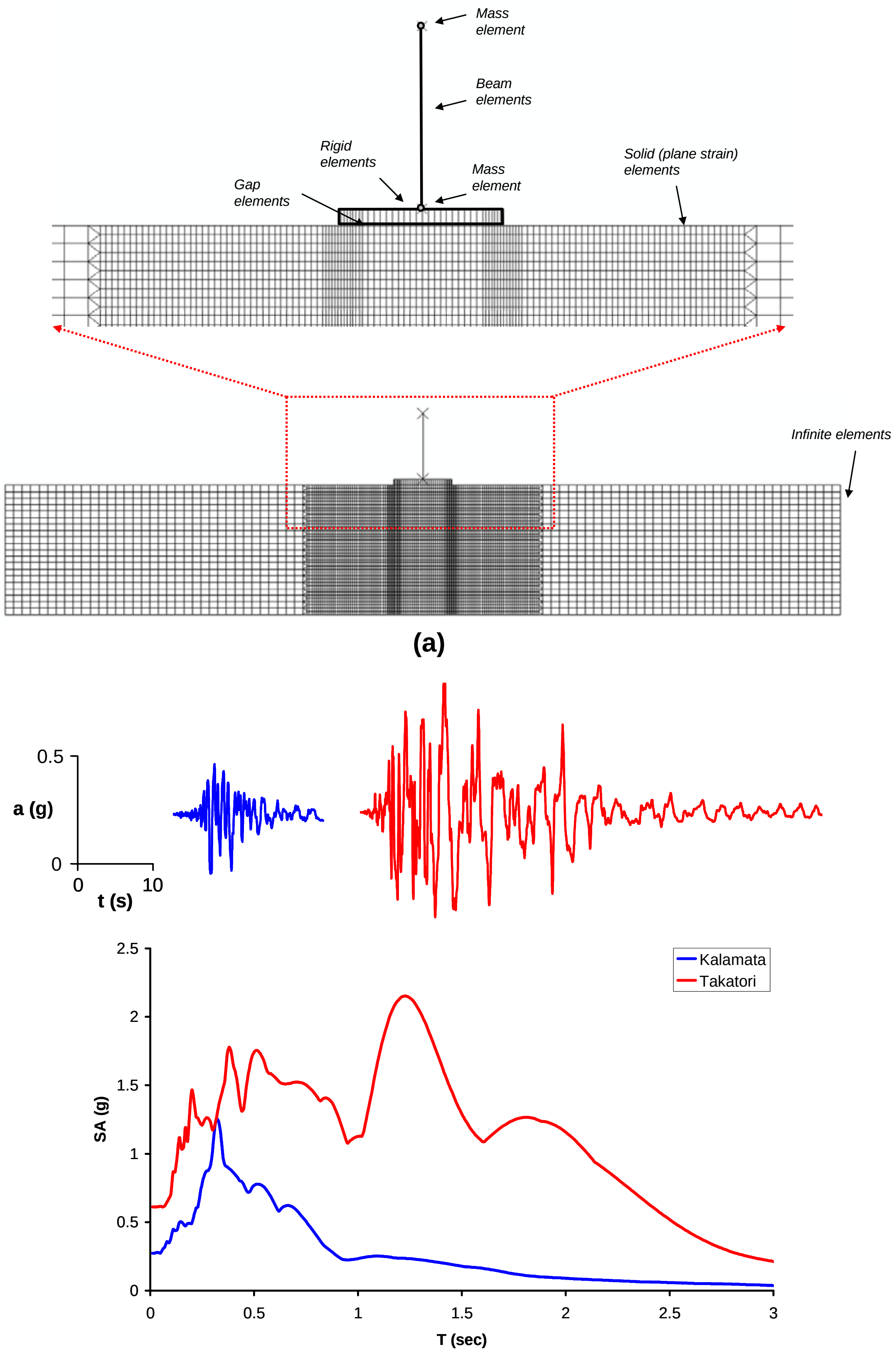

Figure 2 : (a) Finite element discretisation with the types of utilized elements. (b) The two earthquake records utilized as base excitation with their response acceleration spectra. : Kalamata (1986), Takatori (1995).

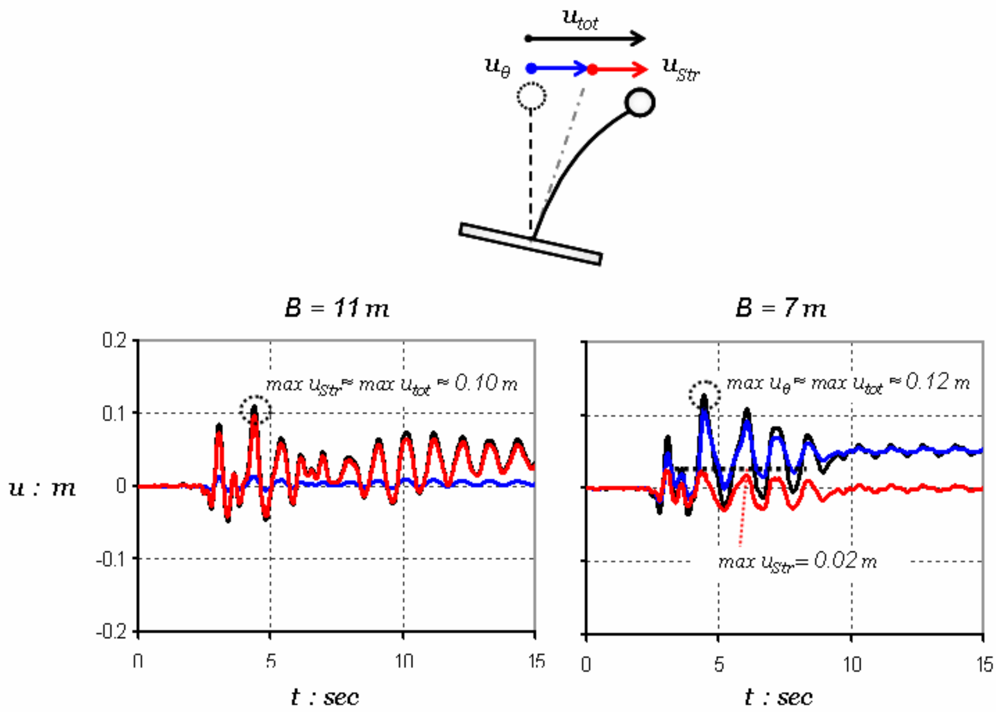

Figure 3 : Computed displacement response time-histories for the two design schemes of Figure 1, for the *"small" intensity* (Kalamata 1986) motion.

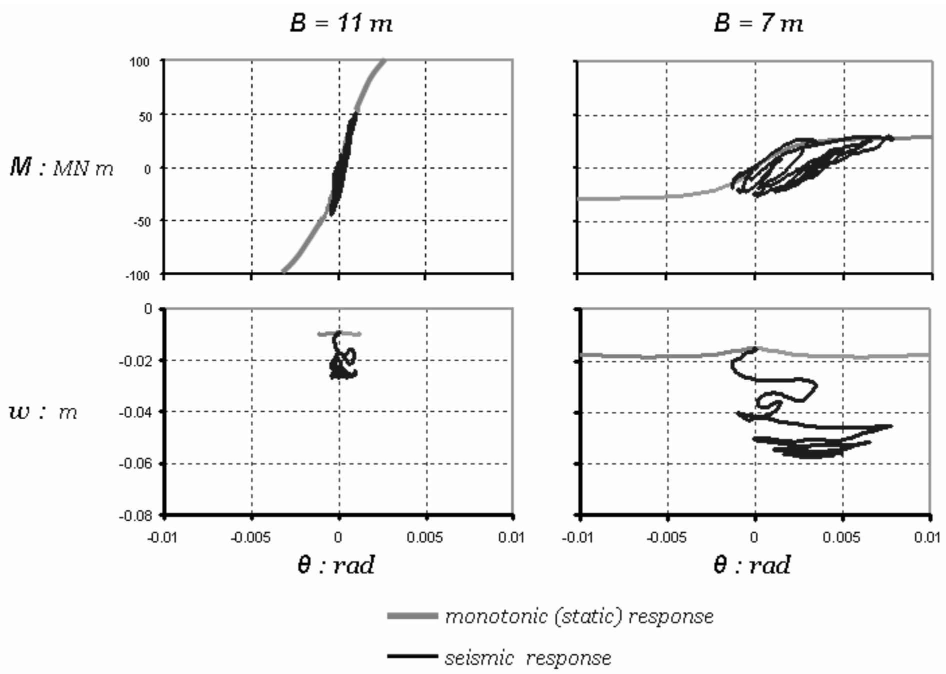

Figure 4 : Computed moment–rotation and settlement–rotation hysteretic response of the two foundations of Figure 1, for the *"small" intensity* (Kalamata 1986) motion.

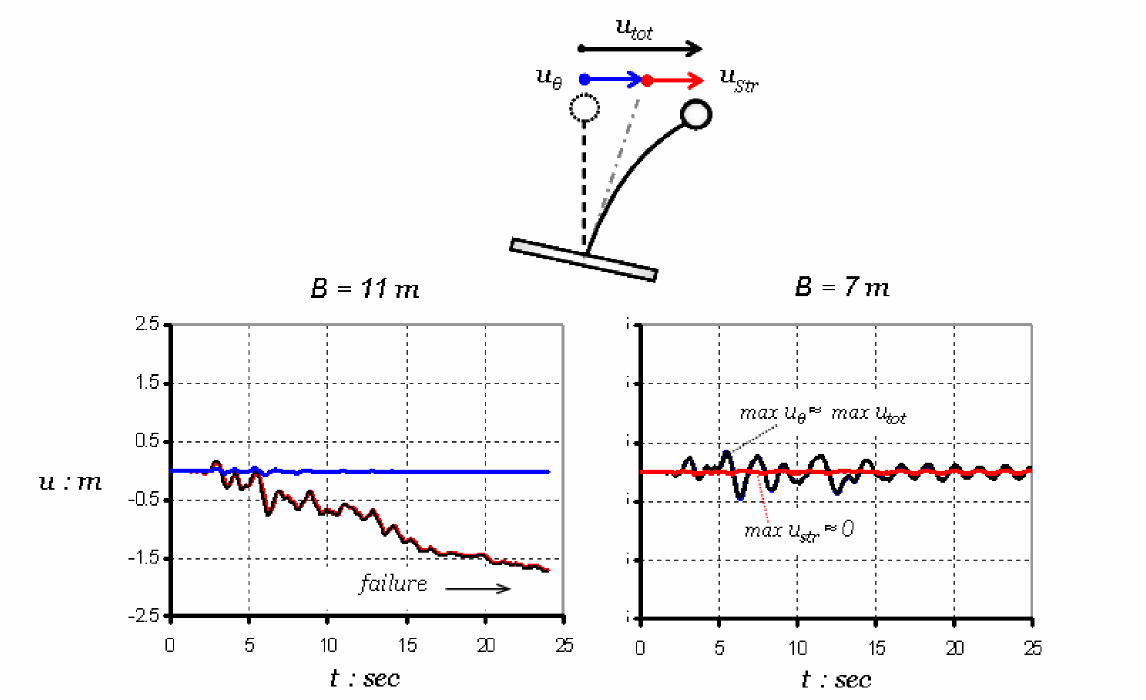

Figure 5 : Computed displacement response time-histories for the two designs of Figure 1, for the *"high" intensity* (Takatori 1995) motion.

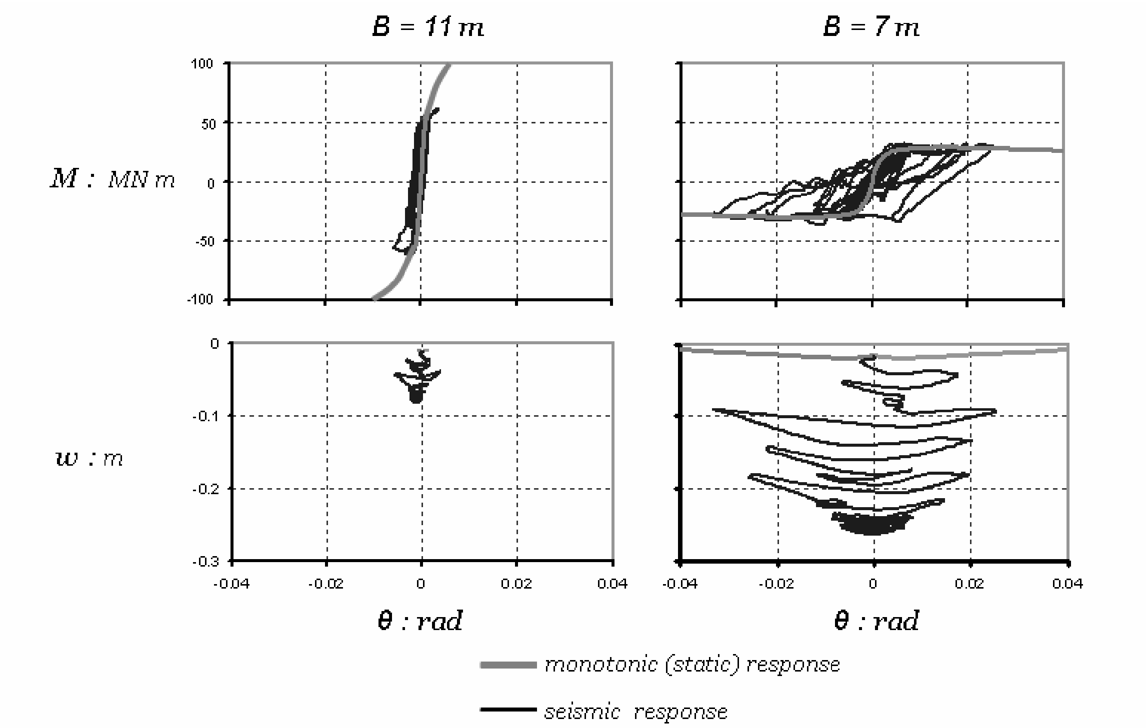

Figure 6 : Computed moment–rotation and settlement–rotation hysteretic response of the two foundations of Figure 1, for the *"high" intensity* (Takatori 1995) motion.

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