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ABSTRACT

This paper evaluates a simple approximate pseudostatic method for estimating the maximum internal forces and horizontal displacements of pile group subjected to lateral seismic excitation. The method involves two main steps. At first the free-field soil movements caused by the earthquake are computed. Then the response of the pile group based on the maximum free-field soil movements which considered as static movements as well as a static loading at the pile head, which depends on the computed spectral acceleration of the structure being supported is analyzed. The methodology takes into account the effects of group interaction and soil yielding at pile-soil interface. The applicability has been verified by both experimental centrifuge models of pile-supported structures and field measurements of Ohba-Ohashi Bridge in Japan. It is demonstrated that the proposed method yields reasonable estimates of the pile maximum moment, shear, and horizontal displacement for many practical cases despite of its simplicity. Limitations and reliability of the method are discussed and some practical conclusions on the performance of the proposed approach are presented.

INTRODUCTION

During past years, different approaches have been presented to assess the seismic response of piles (single or group) based on both complicated and simplified mathematical or numerical analyses. Therefore, some simplified methods have been developed for practical purposes. In this category, the following methods can be noted:

- Methods based on a Winkler hypothesis initiated by Novak (1974)
- Methods based on a simplified boundary element procedure started by Poulos (1973)

Generally, the main focus of these methods is on the dynamic response of the superstructure and their main goal is to calculate pile head deformation characteristics (Tabesh and Poulos, 2001). On the other hand, if one wants to have a good estimate of maximum pile moment and shear force instead of pile head deflection, the Winkler models may give less accurate results (tabesh, 1997).

Unlike Winkler models, simplified boundary element type models proposed by Poulos (1973) and developed for various static conditions by Poulos and Davis (1980) are essentially oriented to accurate evaluation of both pile internal forces and deflections for practical pile design application.

Recently, pseudostatic approaches for the seismic analysis of pile foundations have emerged. In pseudostatic approaches, a static analysis is carried out to obtain the maximum bending moment and shear force developed in the pile due to earthquake loading. Abghari and Chai (1995) developed a pseudostatic procedure using beam on nonlinear Winkler foundation (BNWF) to evaluate the soil-pile-superstructure interaction. Following the pseudostatic approach, Tabesh and Poulos (2001) presented a method based on simplified boundary element models for “single” pile seismic analysis with linear soil behavior.

In this paper, the pseudostatic method presented by Tabesh and Poulos (2001) is extended to take into account group effects and soil nonlinearity. The proposed method is verified by some centrifuge tests results (Wilson, 1998). In addition, the applicability of the method is shown by comparing the analytical results with those of an instrumented pile-group-supported structure under a real earthquake event (Tazoh et al, 1988). In spite of its simplicity, the proposed pseudostatic approach results are in a good agreement with measured values.

METHODOLOGY

A procedure similar to that of Tabesh and Poulos (2001) is considered here with some modifications and extensions to take into account soil yielding and group effects. In order to take into account both soil yielding and group effects, and to keep the analysis simple, the present method adopts the following approach:

- the earthquake, which is assumed to consist of vertically incident shear waves, is applied at a level below the pile tip and the response of the free-field (soil without the piles) along the pile is obtained,
- the piles are modeled as Eulerian beams and are discretized and modeled by the finite difference method,
- the soil is modeled as an elasto-plastic material; its elastic behavior is modeled via the Mindlin fundamental elastic solution (Mindlin 1935). The use of the Mindlin solution means that the not only one element in the pile has an effect on the other element in the same pile, but also, it can influence all the elements of all the other piles in the group. This is more realistic than the Winkler models in which such influences are ignored or else considered in an approximate manner.
- the maximum values of free-field motion obtained in the first step are applied to each pile as a static external soil movement profile and displacement compatibility is enforced between the pile and soil, as long as the soil is elastic. Whenever soil yielding occurs, the compatibility condition is replaced by the condition that the pressure at all interface elements should remain at or stay below the ultimate lateral pressure of the soil,
- A static lateral force is applied to the pile head, given by the spectral acceleration (related to pile head natural period) multiplied by the cap-mass (including superstructure mass).

Free-field ground response

By assuming that the earthquake consists of vertically incident SH waves, the site response can be obtained using the concept of wave propagation in a layered medium as used in the development of the well-known SHAKE or ERLS programs.

Pile group static analysis

Each pile in a group is assumed to be a thin vertical strip of width d, length L, and constant flexibility E_pI_p, and is divided into n+1 elements, all elements being of equal length δ, except those at the top and tip, which are of length δ/2 (Fig. 1). The soil is first assumed to be an ideal isotropic, elastic material, having a Young's modulus E_s and Poisson's ratio ν_s that are unaffected by the presence of the piles. If purely elastic conditions prevail within the soil, the horizontal displacements of the soil and the pile are equal. In this analysis, these displacements are equated at the element centers. In determining the pile displacements, the differential equation for bending of a thin beam is applied. This equation can be written in finite-difference form as:

$$\frac{E_pI_p}{\delta^4}[D]\{u_p\} = -d\{p_p\} \tag{1}$$

in which $\{p_p\}$=vector of pressure acts on pile, $\{u_p\}$=vector of pile displacements, [D]=matrix of finite difference coefficients.

In the static analysis, the soil displacements can be calculated based on the Mindlin (1936) equation which gives the displacements within a semi-infinite elastic isotropic homogeneous mass caused by a horizontal point load (Poulos and Davis, 1980). The soil displacements for all points along pile 'm' in the group, which arise both from the external source of movement and the pressure caused by the soil-pile (from same pile adjacent elements) and pile-soil-pile (from adjacent piles elements) interaction, may be expressed as:

$$\{u_s\}_m = \{u_e\}_m + [I_s]_{mm}\{p_s\}_m + \sum_{k=1\neq m}^{r\times c}[I_s]_{mk}\{p_s\}_k \tag{2}$$

where $\{u_s\}$ = vector of soil horizontal displacement, $\{u_e\}$ = vector of external soil movement, $\{p_s\}$ = vector of pressure acts on soil, $[I_s]$ = $n+1$ by $n+1$ matrix of soil-displacement-influence factors, r=number of rows in group and c=number of columns in group.

$[I_s]_{mm}$ components (interaction factors from pile 'm' elements on each others) are evaluated by integration over a rectangular area of the Mindlin equation for the horizontal displacement of a point load within a semi-infinite mass while the $[I_s]_{mk}$ component (interaction factors from pile 'k' on pile 'm') are calculated directly from the Mindlin equation (Poulos and Davis, 1980).

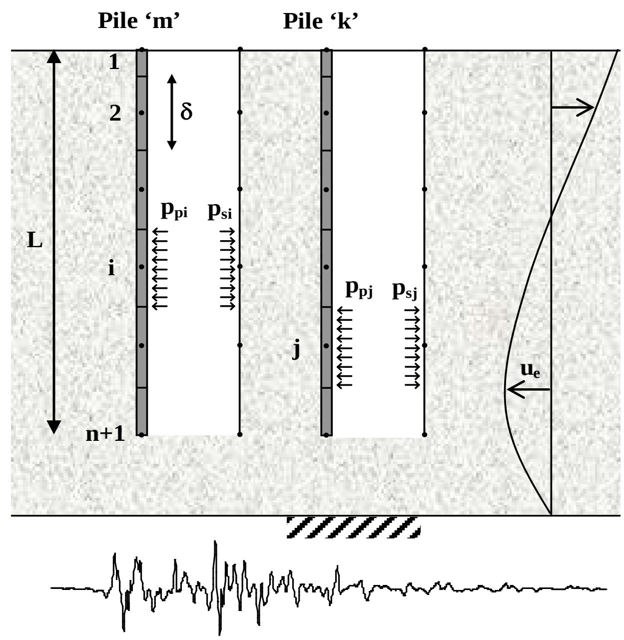

Fig. 1. Specifications for lateral analysis of pile group

A solution to the problem is obtained by imposing displacement compatibility between the pile and the adjacent soil, by combining (1) and (2) which, leads to the following equation:

$$\{u\}_m + \frac{E_pI_p}{d\delta^4}\sum_{k=1}^{r\times c}[I_s]_{mk}[D]\{u\}_k = \{u_e\}_m \tag{3}$$

(3) leads to $n+1$ equations for $n+1$ unknown displacements for pile 'm' in the group. Application of this equation to the end nodes, however, requires two auxiliary points beyond the

each of two ends of the pile; the total unknowns are therefore $n+5$. Four other equations can be obtained from four boundary conditions at the pile ends. For a group with 'r' rows and 'c' columns, $r \times c$ equations [the same as (3)] can be derived.

Soil yielding consideration

The assumption that the soil and pile have the same displacement during the earthquake and imposition of displacement compatibility between the soil and the pile is not correct when soil yielding occurs. In this situation the compatibility equation (3) is replaced by the condition that the pressure at that element is equal to ultimate lateral soil pressure, Py. Therefore, the pressure at all piles elements is recalculated and it is ensured by iteration that at no element of each pile does the pressure exceed P_y. For piles in clay under undrained conditions, it is generally accepted that:

$$P_y = N_c \cdot C_u \tag{4}$$

in which N_c=bearing capacity factor, and C_u=undrained shear strength. N_c can vary between about 8 and 12, but the most commonly used value is 9 (Broms, 1964a) in depths below about 3 to 4 diameters and decrease linearly to a value of 2 at the surface. For piles in sand, Broms (1964b) suggests:

$$P_y = N_p \cdot P_p \tag{5}$$

where N_p=factor which appears to range between about 3 and 5, and P_p is the Rankine passive pressure.
Alternatively, the ultimate lateral soil pressure can be approximated based on the formulae proposed by API (2001) which result relatively similar values for P_y.

Based on the above framework, a computer program named **PSPG** (Pseudo Static analysis of Pile Group) has been developed, which can be used for elasto-plastic pseudostatic analysis of pile group.

VERIFICATION OF THE METHOD

To examine the performance of the proposed pseudostatic methodology for a pile group, two separate sets of recorded data are considered from centrifuge tests and also from an instrumented real pile-supported structure which experienced a real earthquake.

Verification with centrifuge tests results

A series of dynamic centrifuge model tests of pile-supported structures in soft ground is considered. The model included a structure supported by a nine-pile (3×3) group, all founded in a profile of soft clay over dense sand. The model was subjected to nine different earthquake motions having peak base accelerations of 0.02-0.7g from 1995 Kobe and 1989 Santa Cruz records. Test details and the experimental data are available in Wilson et al. (1997a,b). Fig. 2 illustrates the soil profile, structural model, and instrumentation for the tests.

The centrifuge pile group models have been simulated by the PSPG program and the results compared with the measured maximum top moment and maximum cap horizontal displacements.

Also, based on the assumptions of single pile methodology, the calculations were repeated. Results of the above computations are presented in Fig. 3. As can be seen from this figure, in spite of and the relatively simple formulation and quick computation, the calculated and measured results are in reasonable agreement for both moment and horizontal displacement of pile head, for a wide range of input motions. However, for the strong motion of the Kobe earthquake ($a_{max,base}$ >0.6g), the moments were significantly overestimated. As discussed later, this is related to the similarity of the pile cap and ground profile natural periods.

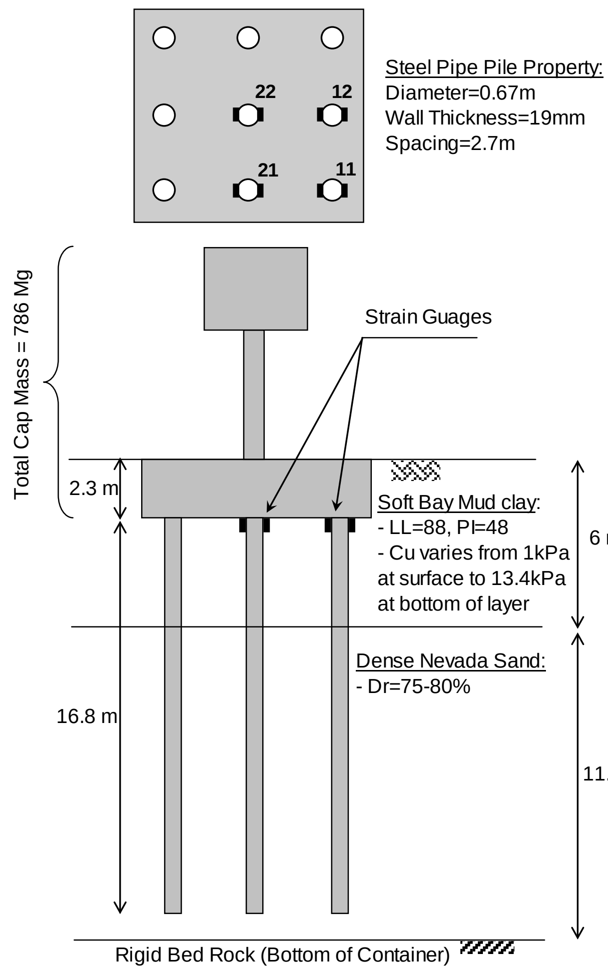

Fig. 2. Specifications of centrifuge models (prototype scale)

Verification via an instrumented real pile-supported structure

The proposed pseudostatic methodology is assessed by estimate the maximum moment developed in the Ohba-Ohashi Bridge in Japan, near Tokyo. The seismic observations at this bridge and one of its pile foundations were conducted between 1981 and 1985 by the Shimizu Corp.

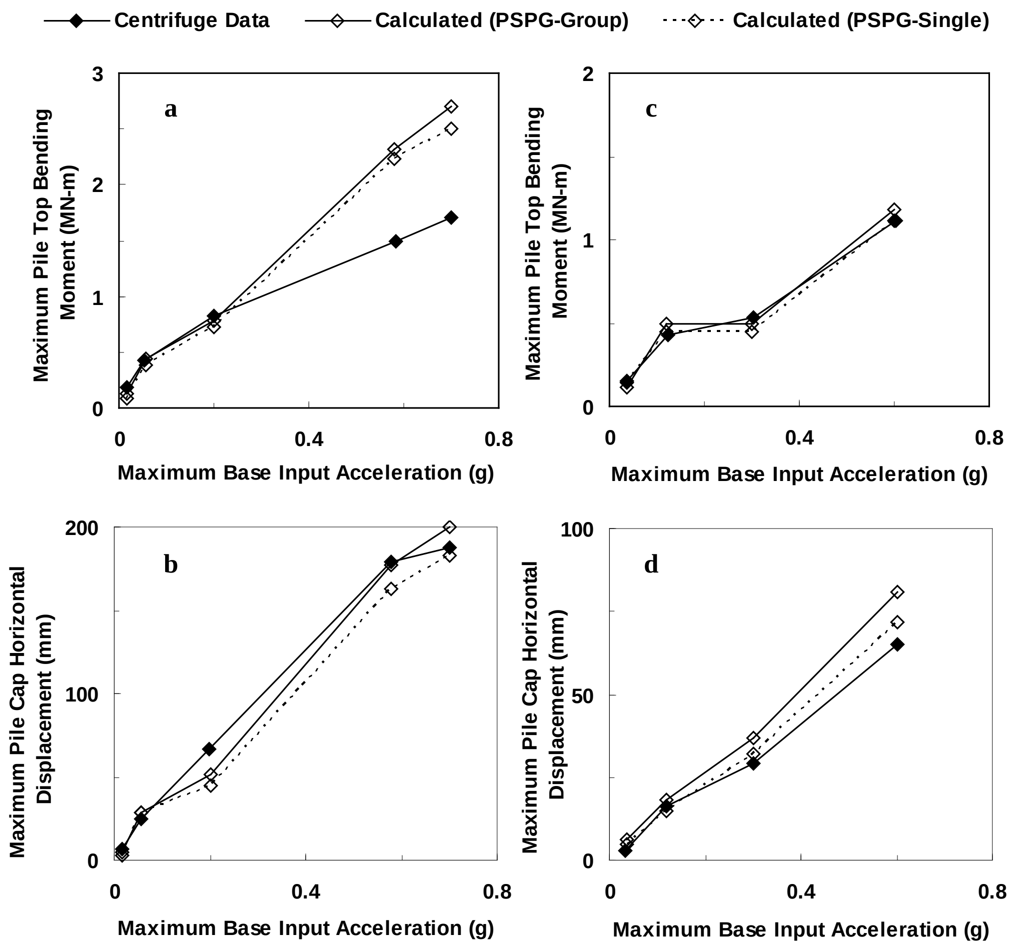

Fig. 3. Comparison of centrifuge data with PSPG single and group solutions: (a) maximum piles' head moment for Kobe earthquake (b) maximum cap horizontal displacement for Kobe earthquake (c) maximum piles' head moment for Santa Cruz earthquake and (d) maximum cap horizontal displacement for Santa Cruz earthquake;

Among the events, the 12th earthquake induced the largest peak horizontal surface acceleration, which was 0.11g. Foundation of the instrumented pier of bridge was a 8×8 pile group included both vertical and battered piles. The soil profile and some of other useful information are shown in Fig. 4. All things about the bridge and its instrumentation can be found elsewhere (Tazoh et al, 1988). The profile of the moment along the pile obtained from the pseudostatic method, along with the maximum moments measured at four locations along the vertical and battered pile, are shown in Fig.5. Despite of complexity of the Ohba-Ohashi Bridge site condition and simplicity of the method, the computed results are in acceptable agreement with the measurements for the vertical pile and good for battered pile. Also, to clarify the group effects, calculations were carried out assuming a single pile only and the results are presented in Fig. 6. It may be noted that in the Ohba-Ohashi case, the inertial effects are relatively small and the kinematic effects are more dominant. Therefore, group effects can be recognized more clearly. As can be seen from Fig 6, in this situation ignoring the group effects may result wrong distribution and amounts of the pile moments. Referring to Fig 6, it is demonstrated that considering the group effects are beneficial when kinematic effects are dominant.

DISCUSSION AND CONCLUSION

A simple approximate pseudostatic method for estimating the maximum internal forces and horizontal displacements of pile group subjected to lateral seismic excitation is evaluated in this paper in comparison with some centrifuge and field data. As explained by pervious researchers [e.g. Tabesh (1997), Abghari and Chai (1995), Wilson (1998)] the pseudostatic approach for pile seismic analysis sometimes overestimates, and sometimes underestimates, maximum moments and shears. However, for a wide range of practical conditions, the pseudostatic method gives results that are reasonable (Tabesh, 1997). The following question may then be asked by practical engineers; "Under what conditions can we rely on a pseudostatic approach for pile seismic analysis?"

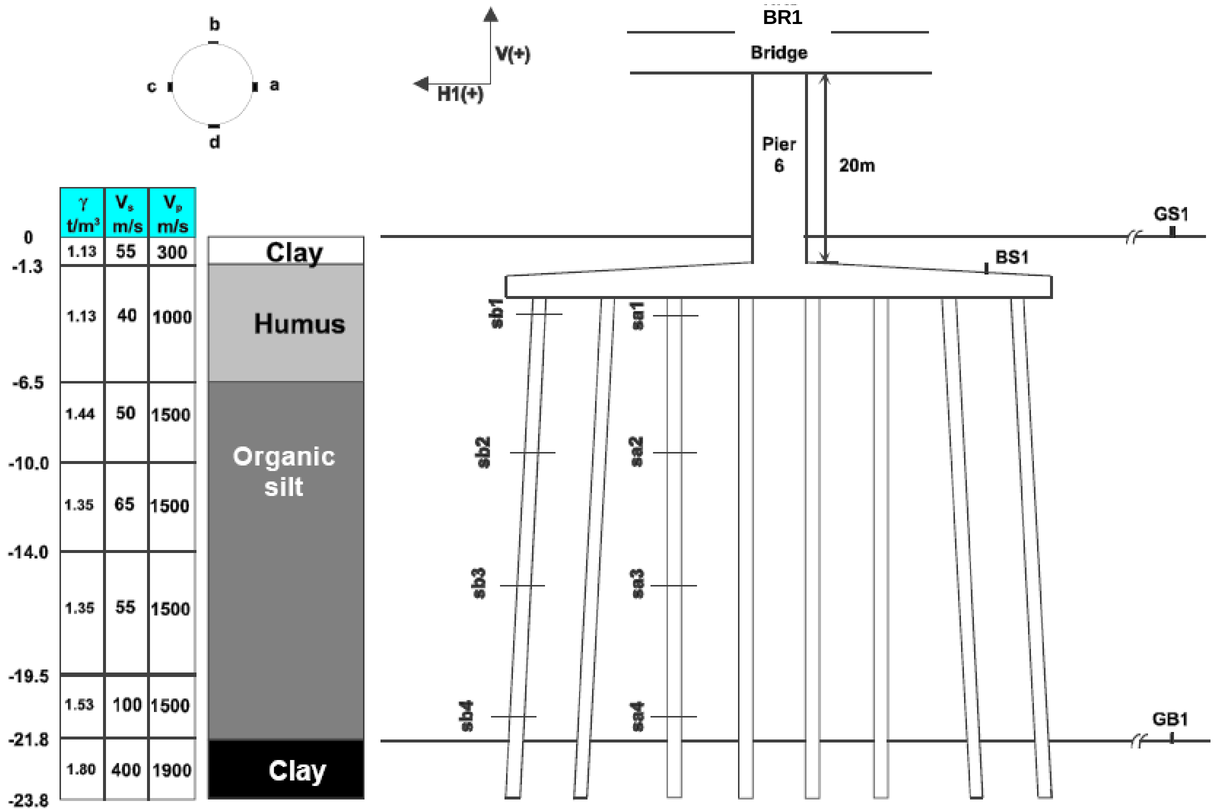

Fig. 4. Ohba-Ohashi Bridge Foundation with 8×8 Pile Group

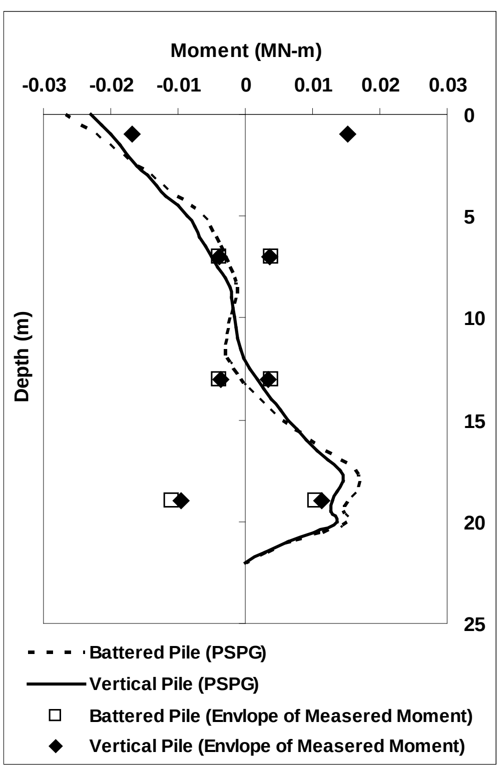

Fig. 5. Distribution of maximum moments along instrumented vertical and battered piles of Ohba-Ohashi Bridge

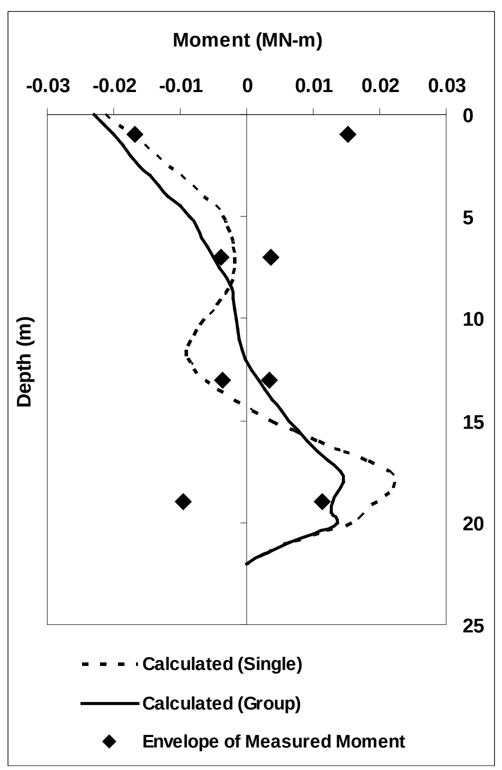

Fig. 6. Group effects on maximum moments along instrumented vertical pile of Ohba-Ohashi Bridge

Referring to the experimental and field examples presented in this paper, the answer is: "when the inertial effects (i.e. cap-mass effects) do not dominate relative to the kinematic effects (i.e. lateral soil movement effects)". In general, inertial effects may become important under the following conditions:

- Large cap-mass (including superstructure mass) or large spectral acceleration of ground surface,
- Small lateral dynamic stiffness of pile group (due to pile and/or soil stiffness) with respect to applied inertial force.

In general, the above conditions are influenced by the following parameters:

- T_{cap}: Pile cap natural period
- T_{max}: Period of maximum spectral acceleration in response spectrum of surface motion
- SA_{cap}: Spectral acceleration related to pile cap period
- SA_{max}: Maximum spectral acceleration in response spectrum of surface motion

Fig. 7 represents a dimensionless diagram of the above key parameters for the cases referred in this papaer. As can be seen, when $SA_{cap}/SA_{max}<0.8$ and $T_{cap}/T_{max}>2$, good agreement is found between measured and calculated maximum values of moment, shear and displacement. In contrast, when SA_{cap}/SA_{max} and T_{cap}/T_{max} approach 1, the pseudostatic method does not perform well, especially for maximum moment and shear. As mentioned by other researchers [e.g. Tabesh (1997)], this may be attributed to assuming that maximum kinematical and inertial forces act simultaneously.

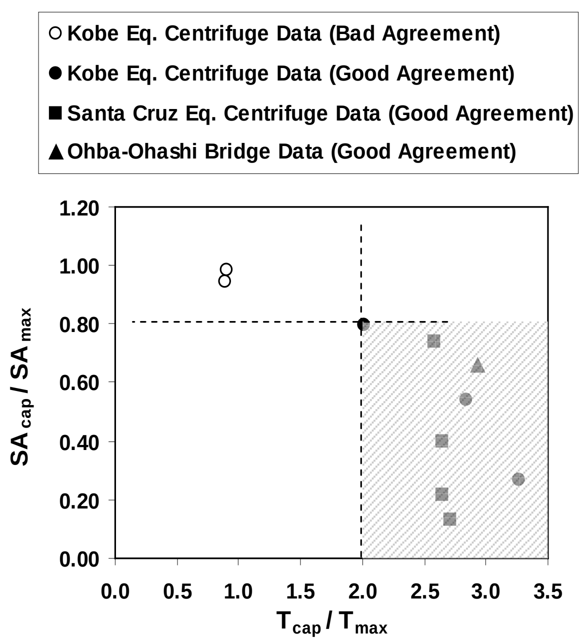

Fig. 7. Good performance conditions for proposed pseudostatic method

ACKNOWELEDGEMENTS

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