BEHAVIOR OF PILE GROUPS IN THE VICINITY OF SURFACE FAULT RUPTURES

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ABSTRACT: The 1999 ChiChi earthquake, Taiwan, did great damage to a number of bridges along the trace of the surface rupture. These bridges crossing some major rivers have foundations deeply or shallowly embedded in deposits of sands, gravels and other suspended matters that these rivers have carried over centuries. In this paper, behavior of a pile group subjected to soil deformation caused by faulting at its bedrock is numerically studied using Material Point Method (MPM), and its cap motions are discussed for different cases of its stiffness, location etc.

Key Words: Thrust fault, surface rupture, MPM, dilatancy, pile group, upright beam

INTRODUCTION

The trace of the surface rupture that appeared in the 1999 ChiChi earthquake closely followed the frontal slope of the local mountain range where the range trends north south (Chen et al., 2003). Some major rivers cut this range, and bridges crossing these rivers were seriously damaged by large deformations of soils caused by the fault rupture. A discussion on this issue must be based on a quite different scenario from those for ordinary designs, in which ground accelerations and/or velocities are crucial factors. Many foundations supporting the damaged bridges were embedded in deposits of sands, gravel and other suspended matters that rivers have carried over centuries. Therefore due attention should be directed to deformation buildup in soil deposits that cover hidden faults. When a base rock comes steadily up into a soft soil deposit, strains will be distributed over some wide zones, which extent depends largely on the material properties, dip angle, etc. Consequently an embedded foundation will be shifted from its original location, and deformed even though it is located off the major rupture zone. For analyzing this problem, two phenomena should be discussed simultaneously; deposit rupturing and pile-soil interaction. Some researches have been conducted both for soil deformations caused by dip-slip and strike-slip fault dislocations. Most of them were experimental works with numerical verifications (see e.g. Bray 1990, Stone 1988); but there are few studies on structures affected by fault ruptures. A material point method (MPM) is used herein for numerical modeling of fault rupture effects on structures. The MPM is categorized as one of the finite element methods formulated in an arbitrary Lagrangian-Eulerian description of motion. In MPM, a body to be analyzed is described as a cluster of material points. The material points, which carry all Lagrangian parameters, can move freely across cell boundaries of a stationary Eulerian mesh. This mesh, called a computational mesh, should cover the virtual position of the analyzed body. The computational mesh can remain constant for the entire computation, thus the main disadvantage of the conventional finite element method related to the problem of mesh distortions is eliminated. Its main drawback, however, is that any localization, heterogeneity and boundaries that can exist within one cell are not sharply outlined (see **Figure 1**). In other word, a cell size determines the resolution of MPM.

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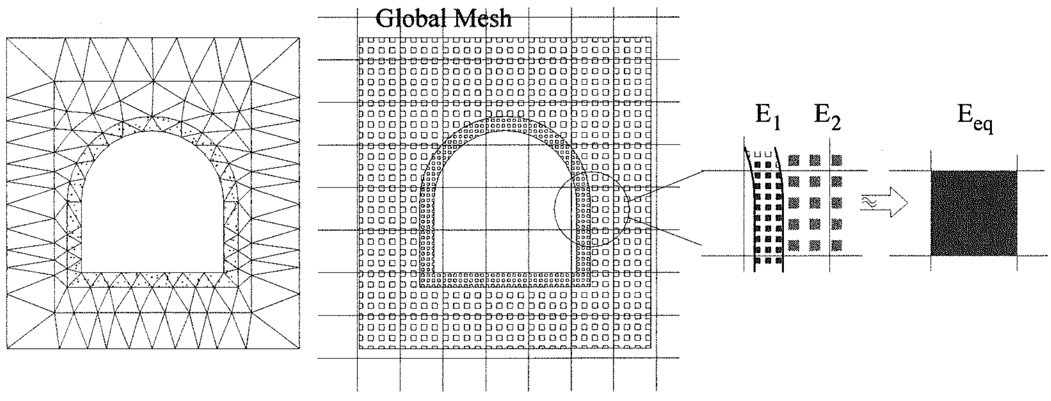


Figure 1. FEM and MPM: Resolution of MPM greatly depends on cell size.

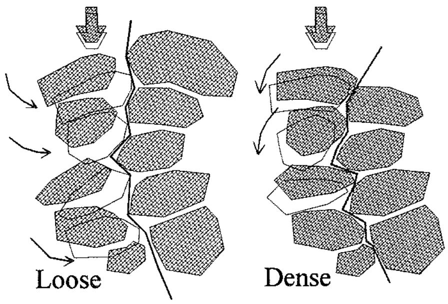


Figure 2. Dilating and contracting behavior of granular assemblage

SOIL-STRUCTURE MODEL

Soil

Soils in nature are often rich-graded granular assemblages. When a soil is sheared, it keeps dilating without showing any clear sign of contraction (**Figure 2**), and reaches its maximum volume when the shearing displacement reaches two to three times of its shear band thickness. The soil discussed herein is thus assumed to be a homogenous and isotropic material with constant elasticity properties. Mohr Coulomb criterion with Associated flow rule describes its plastic behavior. Taking into account that natural soil deposit includes large boulders among other finer matters, its shear band is assumed to dilate over the entire shearing process.

Pile group

Piles, grouped beneath a superstructure, interact with the surrounding soil, and the pile-soil-pile interaction often affects the motion of its superstructure to a considerable extent. Straightforward evaluation of the pile-soil-pile interaction, however, is cumbersome especially in dealing with tens or hundreds of piles grouped together. Hence a simplified approach for the evaluation of such pile-soil-pile interaction is highly desirable for the purpose of treating the behavior of an entire soil-foundation-structure system. Recently, the second author developed a further simplified approach in which a group of piles is viewed as an equivalent single upright beam (Konagai et al., 2000, 2002 and 2003), the idea based on the fact that a group of piles often trap soil among them as observed when pulled out (Railway Technology Research Institute, 1995).

The following assumptions were taken to derive the stiffness matrix of the equivalent single beam:

(1) Pile elements within a horizontal soil slice are all deformed at once keeping their intervals constant, and the soil caught among the piles moves in a body with the piles.

(2) Frictional effects due to bending of piles (external moments on each individual pile from soil) are ignored.

(3) The top ends of piles are fixed to a rigid cap.

(4) All upper or lower ends of the sliced pile elements arranged on the cut-end of a soil slice remain on one plane (Note this assumption does not necessarily mean that each pile's cross-section remains in parallel with this plane. See **Figure 3(b)**).

The stiffness matrix includes two stiffness parameters, EI_{sway} and EI_{rock} (see Konagai et al. 2000, 2003). The first parameter EI_{sway} that governs the sway motion of the beam is the product of the bending stiffness of an individual pile EI_{single} and the number of piles n_p. The second stiffness parameter EI_{rock} is most strongly associated with the rocking motion of the beam.

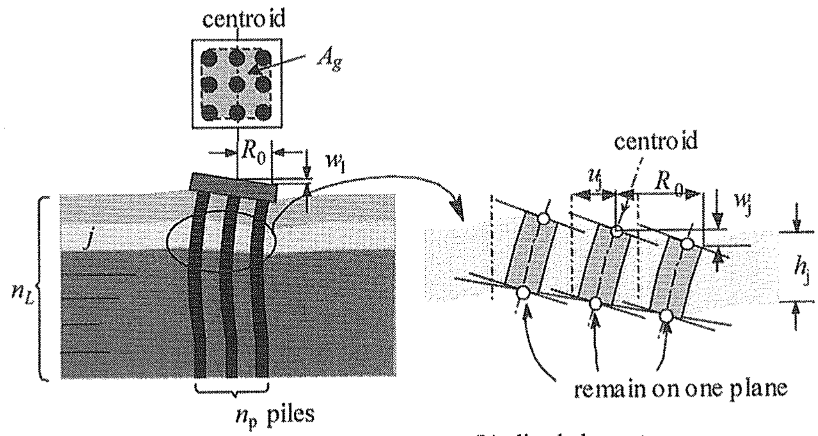
In the following discussion, it is necessary to introduce the idea of active pile length. Under lateral loading at the pile cap, the horizontal deflection of a pile decreases with increasing depth. In practice, most laterally loaded piles are indeed 'flexible' in the sense that they are not deformed over their entire length L. Instead, pile deflections become negligible below an active length (or effective length) L_a. The active length, an important parameter in the design of a pile foundation (Wang MC, Liao WP, 1987), depends largely on the ratio of the pile stiffness EI_{sway} for flexural deformation and the soil stiffness μ, and is given by:

$$L_a = \alpha_0 L_0 \tag{1}$$

where, the parameter α_0 reflecting, in theory, only different soil profiles rationally excluding the pile group effect, and

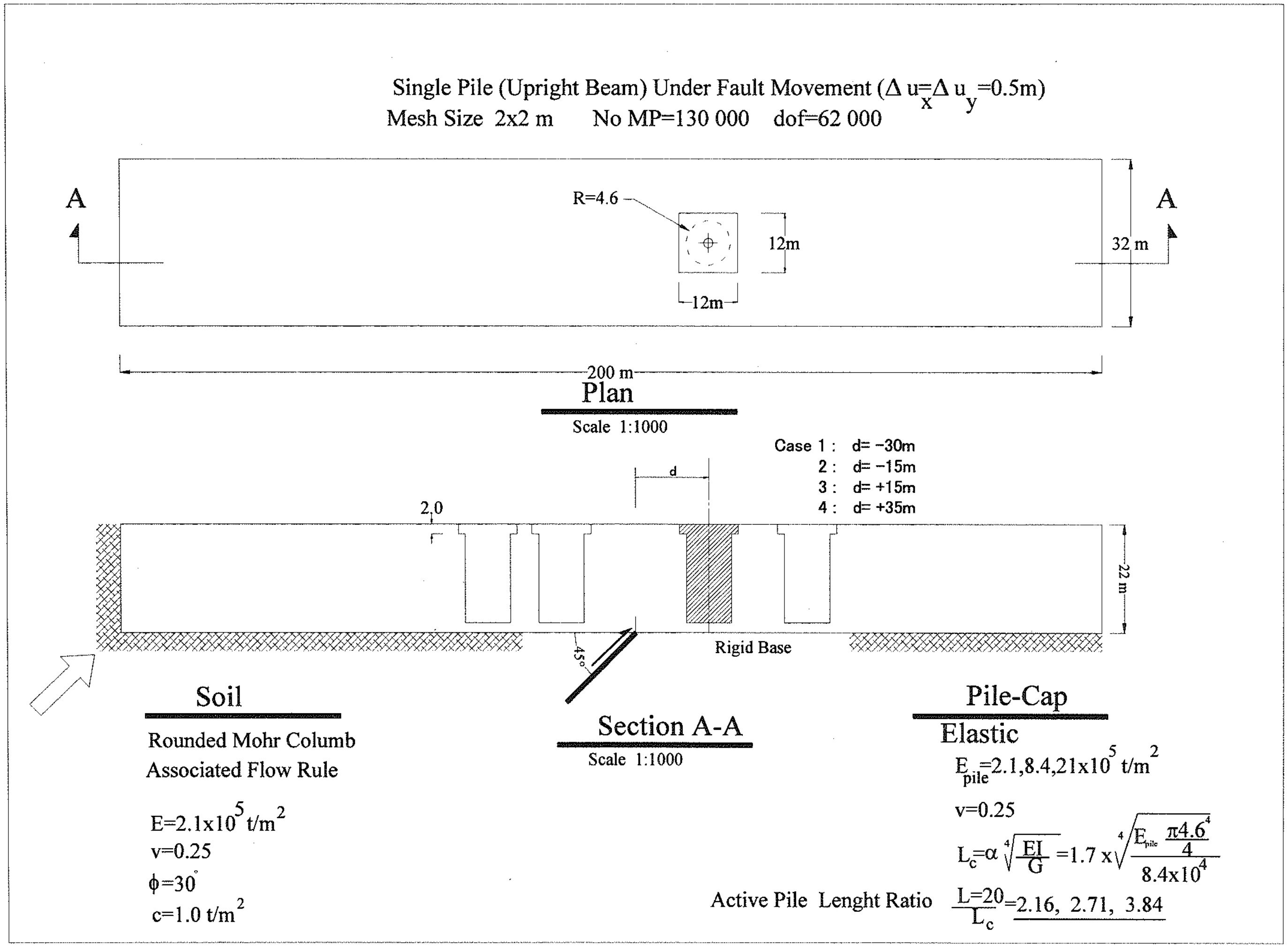
$$L_0 = \sqrt[4]{EI_{sway} / \mu}. \tag{2}$$

For the present study, the ratio of pile length L to L_0 is set at 2.16, 2.71 and 3.84. These values correspond to pile-soil stiffness ratios E / E_{soil} = 1, 4 and 10, respectively.



(a) soil- pile-group system **(b)** sliced elements

Figure 3. Assumptions for evaluation of equivalent single beam



$$L_c = \alpha \sqrt[4]{\frac{EI}{G}} = 1.7 \times \sqrt[4]{\frac{E_{pile}\,\frac{\pi 4.6^4}{4}}{8.4\times10^4}}$$

$$\frac{L=20}{L_c} = 2.16,\ 2.71,\ 3.84$$

Figure 4. Fault geometry

Fault geometry
A reverse fault movement is given at the mid bottom of a 200m-long, 22 m-deep and 32m-thick surface soil deposit (**Figure 4**). Dip angle is set at 45 degrees. Two rigid walls retaining both sides of the surface soil deposit move with the bedrock. The walls were made slippery so that their presence has little affect on the numerical results. A pile group (equivalent upright beam) is located –30m, -15m (on the hanging wall side), +15m and 35m (on the footwall side) off the point of the bedrock rupture for CASE 1, 2, 3 and 4, respectively. For keeping pile group stresses below allowable range, a thin layer of soil is put between pile head and bedrock.

SOIL DEFORMATION

Deformation of the surface soil deposit is first analyzed by excluding the pile group. The deformation is then compared to that with a pile group. This procedure allows a rational evaluation to be made for the effect of the pile-group inclusion in the vicinity of the fault rupture zone. In addition, the result allows the verification of a 2D MPM, which can be used for this particular case in place of the 3D MPM decreasing drastically the number of material points.

Figure 5 shows the distribution of the maximum shear strains. Since the range of the strain was too wide to describe detail features of strain distribution pattern, they were mapped with gray halftones in logarithmic scale. Two conjugate shear bands propagate up through the soil deposit, and one in the direction of the fault dip is clearer than the other. **Figure 6** shows spatial distributions of both horizontal and vertical displacements.

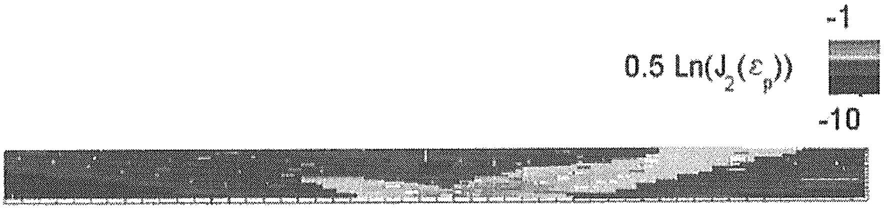


Figure 5. Maximum shear strain distribution

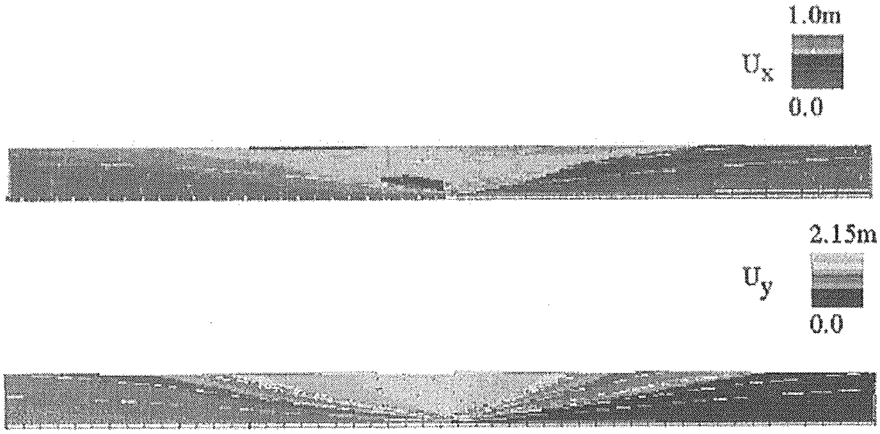


Figure 6. Horizontal and Vertical Displacement Plot After Fault 1 m 45° offset

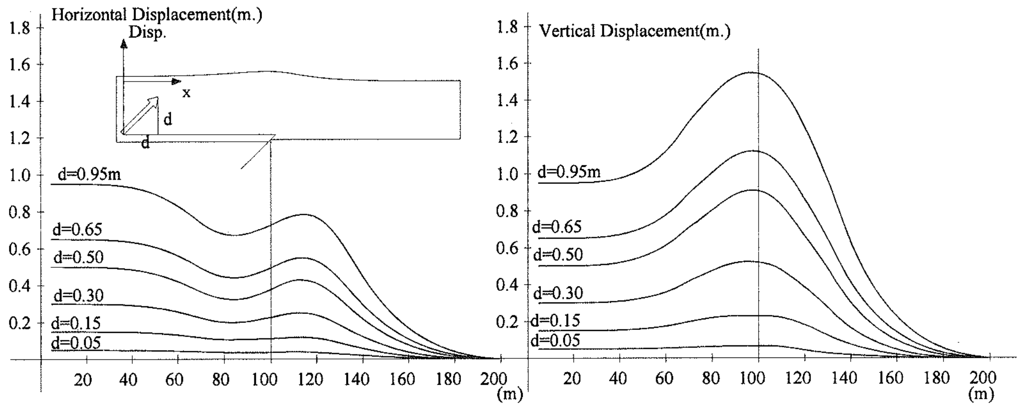


Figure 7. Horizontal and vertical displacements after a 45° fault offset of 1 m.

For a thorough discussion, variations of ground surface displacements are shown in **Figure 7** at different bedrock dislocations with respect to the distance along the bedrock. Parameter d in this figure denotes either lateral or vertical component of the bedrock dislocation. It is noted here that vertical displacement reaches its peak exactly above the point of bedrock dislocation, and is larger than the vertical component of dislocation d. Dilative feature of soil may have caused part of this upheaval, but it seems mainly that the thrusting movement of the fault pushed up the soil block in between the two conjugate shear bands. All curves showing horizontal soil displacements (left chart of **Figure 7**) go down gently oscillating towards right. These oscillations have their first bottom values appearing at around 80m lateral distance. This means that there is a lateral compressive movement between the leftmost soil mass and that at the 80m distance.

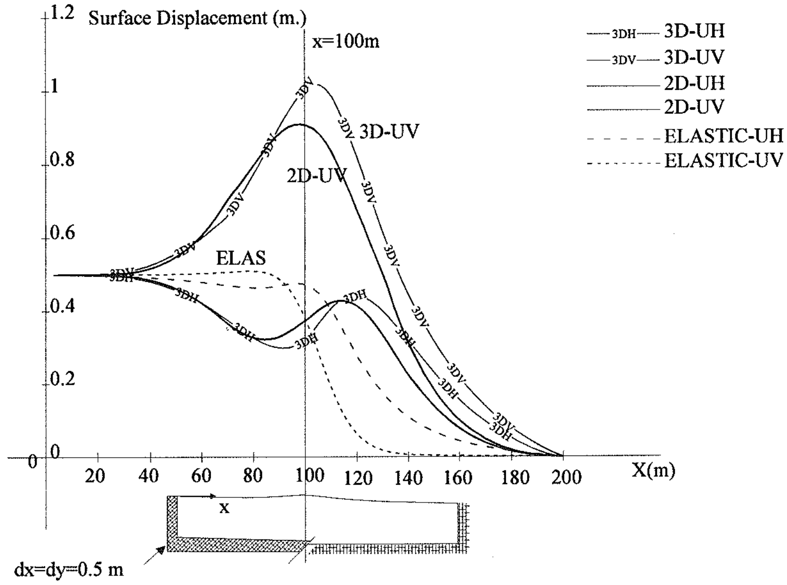


Figure 8. Results from 2D and 3D MPM analyses

Figure 8 compares the surface soil displacements from 2D and 3D MPM analyses when d reaches 0.5m. Slight difference seems to have caused by the plane-strain assumption for the 2D model, while out-of-plane motions of material points are not completely restricted in the 3D MPM analysis. **Figure 8** also shows the variation of displacements for an elastic soil deposit. There is no clear soil upheaval appearing in this figure because the soil does not exhibit any plasticity.

The surface soil deformation was than calculated putting a pile group in the surface soil deposit (see **Figure 9**). **Figure 10** shows surface soil displacements calculated for different locations of a pile group, +15m and +35m (on the footwall side) and –15m, -30m (on the hanging wall side) off the point of bedrock fault rupture. The presence of the pile group certainly caused the displacement distribution to change in the vicinity of it. However, no serious difference can be seen among cases for different pile-soil stiffness ratios examined (E/E_{soil} = 1, 4 and 10). As for horizontal displacements, the flexural pile group followed rather closely the motion of the surrounding soil, while clear changes in vertical displacements indicate that the pile group, though flexible in its lateral direction, is stiff enough to pull down the heaving soil.

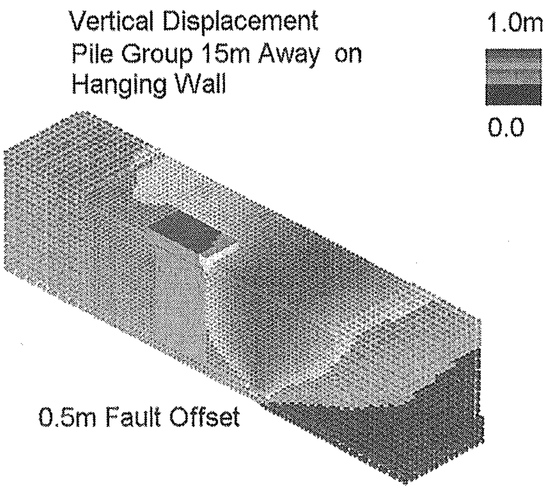


Figure 9. Spatial distribution of vertical soil displacement

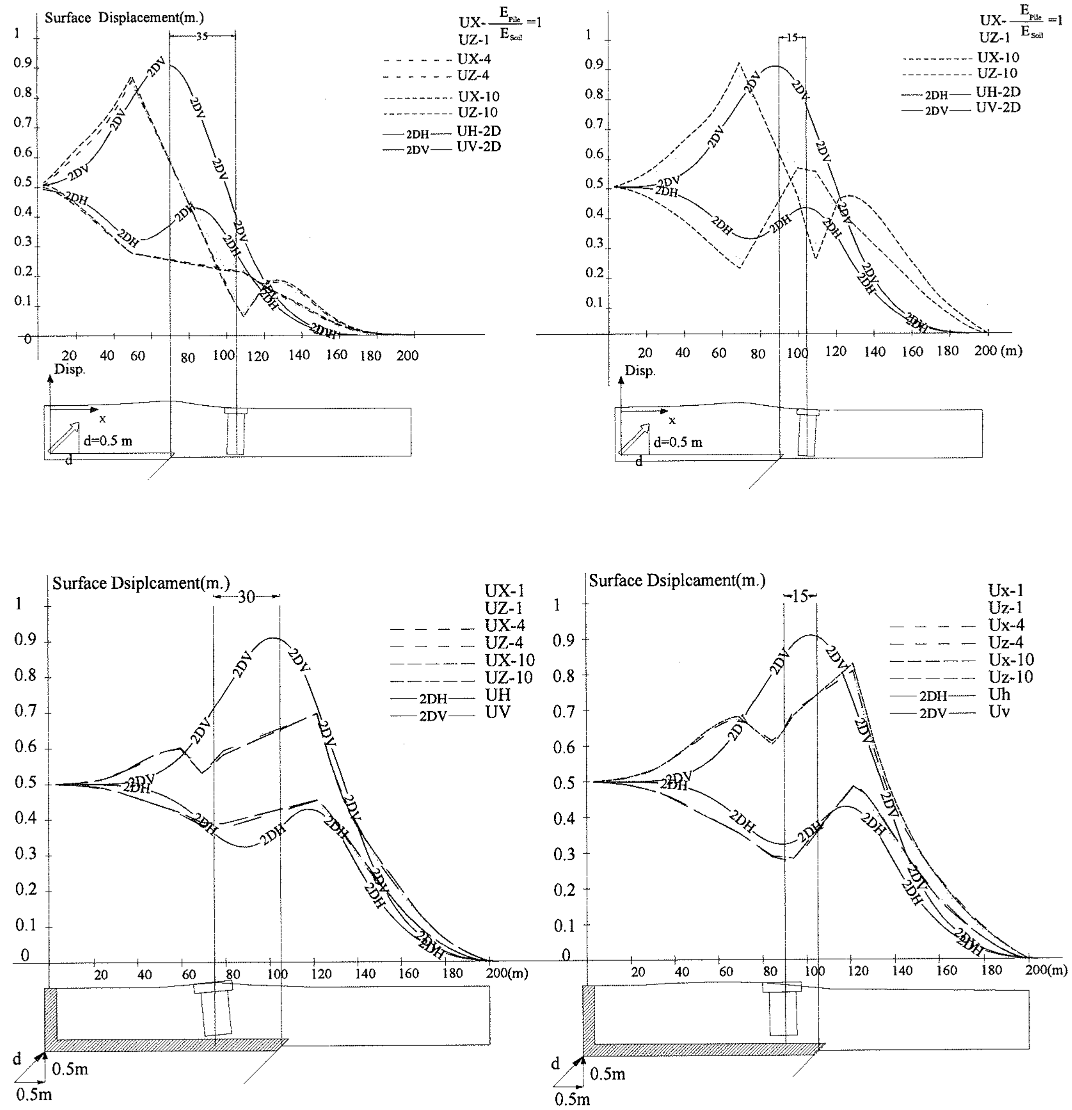


Figure 10. Surface soil displacements calculated for different locations of pile group, +35m and +15m (on the footwall side) and –30m, -15m (on the hanging wall side) off the point of bedrock fault rupture.

Figure 11 shows the increasing rotation angle of the pile cap with the increasing bedrock dislocation. No remarkable difference can be seen among cases for different pile-soil stiffness ratios examined. Kinks appeared at around 0.42 m vertical offset probably because the bedrock offset exceeded one cell size

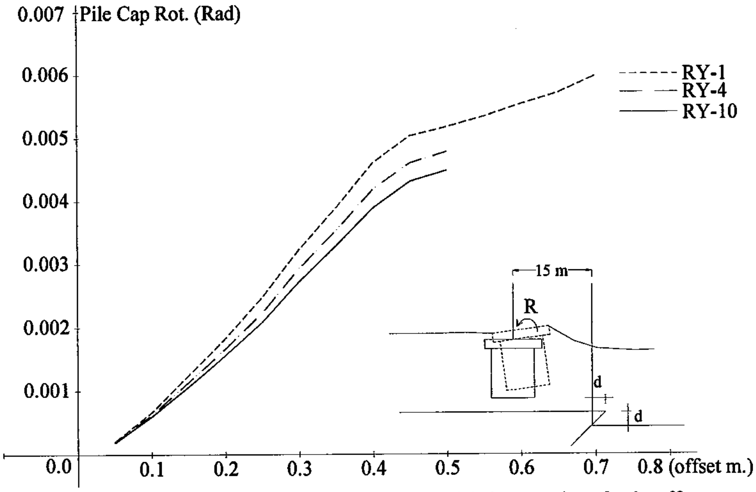


Figure 11. Pile Cap rotation history with increasing fault offset

CONCLUSIONS

Behavior of a pile group subjected to soil deformation caused by faulting at its bedrock is numerically studied using Material Point Method (MPM). Conclusions obtained through the numerical examinations are summarized as follows:

1. Deformation of the surface soil deposit is first analyzed by excluding the pile group. Two conjugate shear bands propagate up through the soil deposit, and one in the direction of the fault dip is clearer than the other. The vertical component of displacement reaches its peak exactly above the point of bedrock dislocation. Dilative feature of soil may have caused part of this upheaval, but mainly the thrusting movement of the fault seems to have pushed up the soil block in between the two conjugate shear bands.
2. The presence of the pile group certainly caused the displacement distribution to change in the vicinity of it. As for horizontal displacements, the flexural pile group followed rather closely the motion of the surrounding soil, while clear changes in vertical displacements indicate that the pile group, though flexible in its lateral direction, is stiff enough to pull down the heaving soil.

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