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NONLINEAR SOIL-PILE GROUP INTERACTION IN THE VICINITY OF SURFACE FAULT RUPTURES

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SUMMARY

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 materials 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

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. Some major rivers cut this range, and bridges crossing these rivers were seriously damaged by large deformations of soils caused by the fault rupture Chen[1] ,Kosa[2] .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[3], Stone[4]); 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

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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

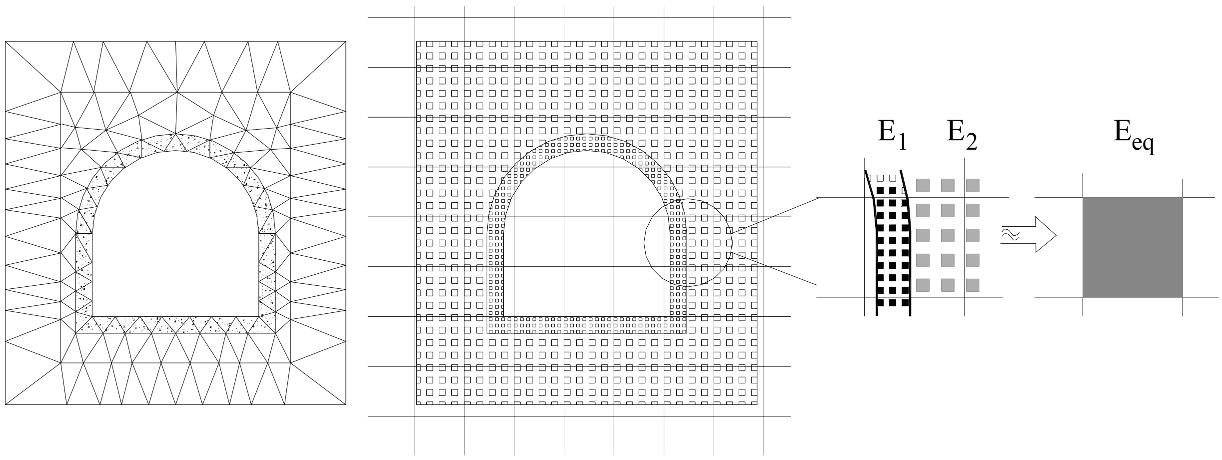

Figure 1. FEM and MPM: Resolution of MPM greatly depends on cell size

SOIL-STRUCTURE MODEL

Bed Rock

Here the definition of bedrock is slightly different than conventional seismological ones which is based on wave speed. The bedrock is the layer where the earth crust dislocation can continue in the form of major ruptures.

Soil

With the fault motion, the overburden soil deposit deforms to keep compatibility with the induced dislocation. One key factor for displacement (strain) field compability is dilatancy (volumetric) behavior of soil which is depended on deposit grain size and compaction degree.

On **Figure 2** a 50m thick soil deposit is analyzed under 45^{o} dip angle reverse fault motion with different volumetric behavior scenarios. Using Generalized plasticity (Pastor, Zienkowicz)model with ability to simulate the variable dilatancy and volumetric compaction features of soil, almost the induced dislocation is absorbed locally and replace sheared zone, a wide compacted zone are formed.

In the case of associate flow rule with Mohr-Coulomb model due to dilatancy two conjugate shear zone are formed, while with zero dilatancy assumption, narrow single sheared zone is appeared.

On associated flow rule case, if the gravity acceleration is multiplied by ten the narrowest sheared zone is created (with elasticity,… parameters); because due to increase in pressure, soil strengthened and the plastic deformation magnitude (so dilatancy) is smaller. In this case some how result is similar to zero dilatancy ones; which means variation in soil strength doesn’t have direct effect on displacement field and it’s effect is rather by changing dilatancy.

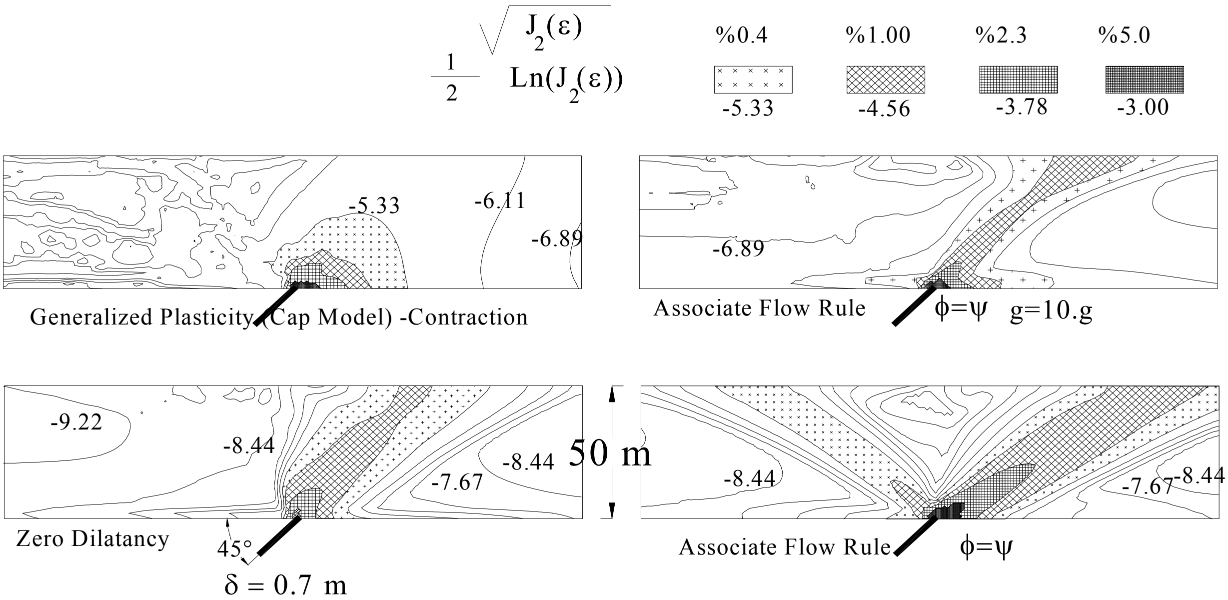

Figure 2. The Effect of Dilatancy and gravitational acceleration on the shear strain field

Soils in nature are often rich-graded granular assemblages. When a granular soil is sheared, it keeps dilating without showing any clear sign of contraction (**Figure 3**), 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.
Because of lower pressure and strength of the upper soil layers shear zone appears on the surface with wedge shape (flower structure Bray[3]) which based on the dip angle it will spread more toward to hanging or foot wall side. Here as an extreme case the shallow soil deposit over stiff layer is considered where the induced deformations on the surface wedge has its extreme value.

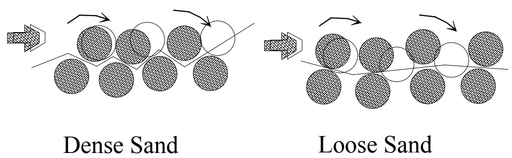

Figure 3. Dilating and contracting behavior of granular assemblage

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.[5]), 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).

This single-beam analogy has been proven to provide close approximations of both axial and flexial motions of a pile group, and therefore allows a crosswise interaction between these two components will be rationally described.

In the following discussion, it is necessary to introduce the idea of active pile length for describing lateral response of a pile group. Under lateral loading at the pile cap over practical range, 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,[2], 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_{pile} / E_{soil} = 1, 4 and 10, respectively.

For keeping pile group stresses below allowable range and avoiding overestimation of axial interaction effect, a thin layer of soil is put between pile head and bedrock

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.

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 6-a 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 shaded 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.

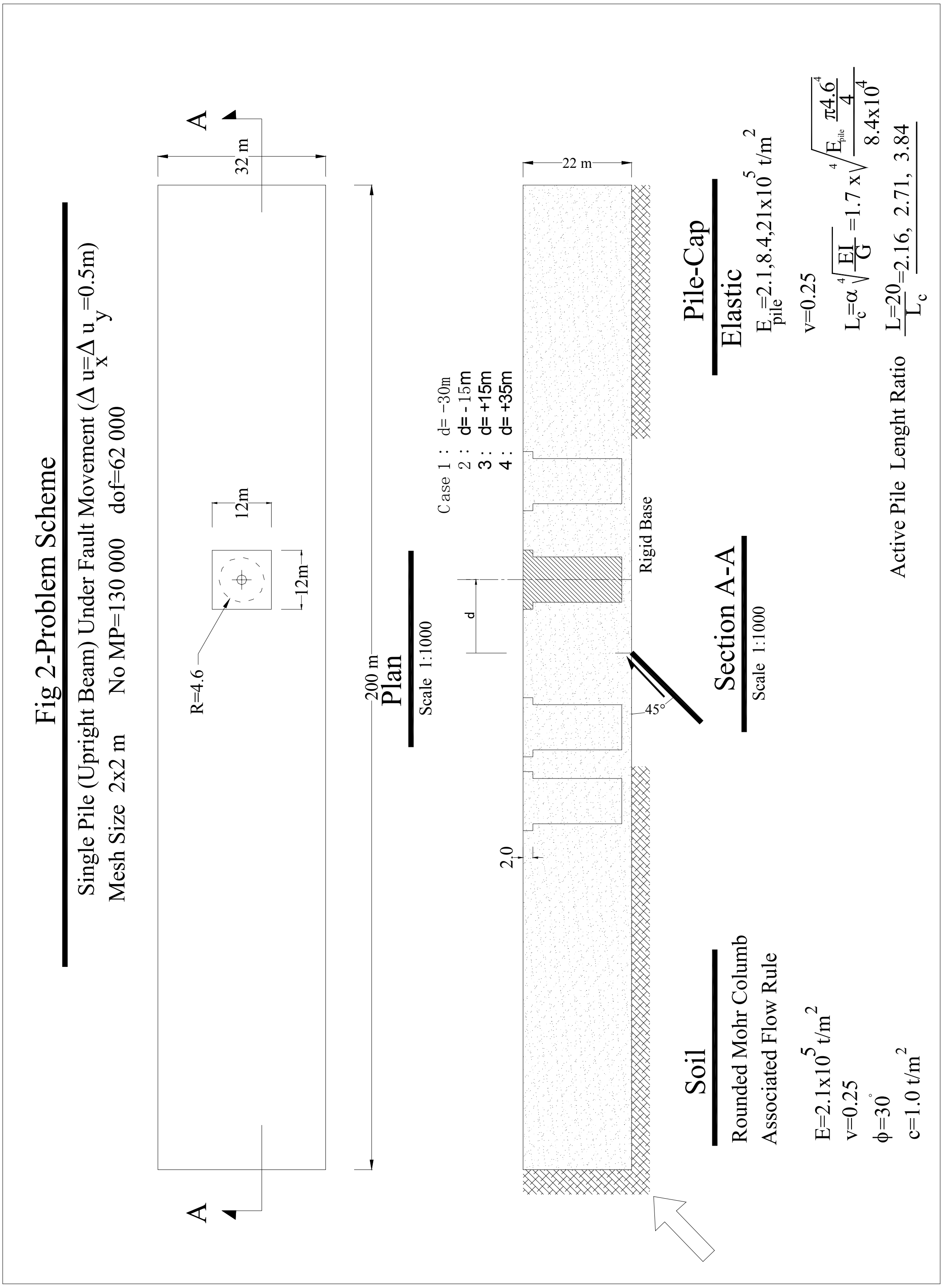

Figure 4. Fault geometry

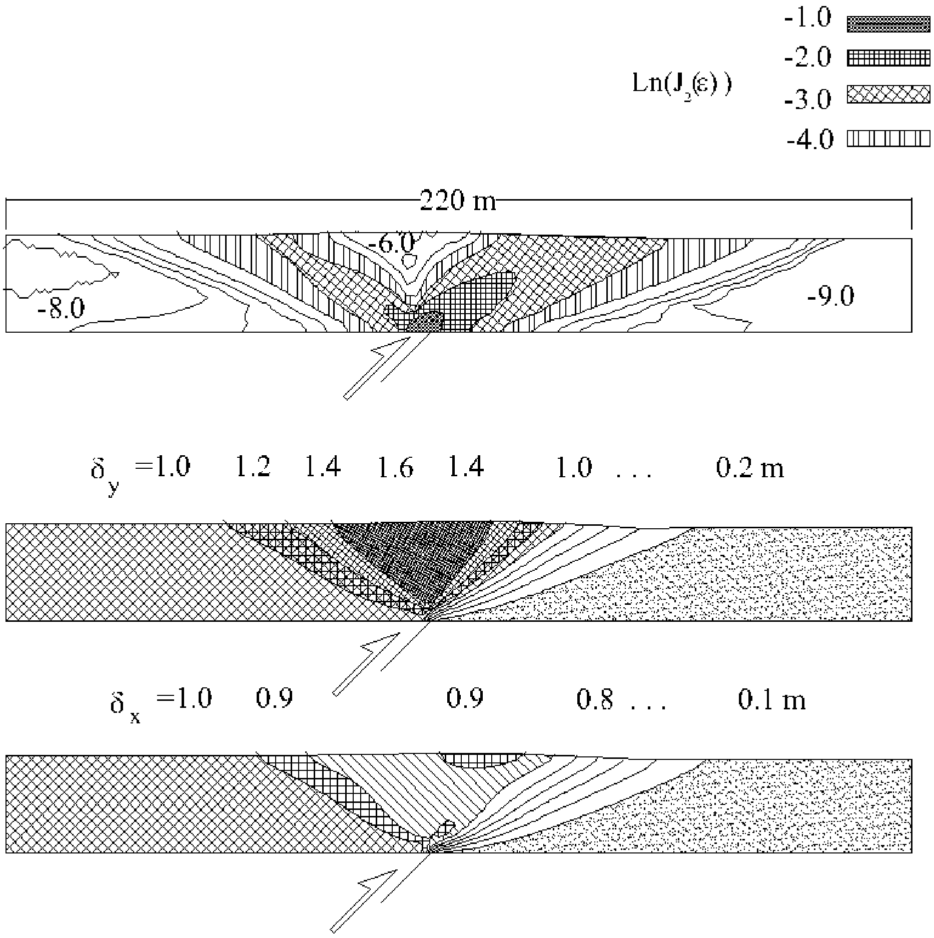

Figure 5. Horizontal and Vertical Displacement Plot After Fault 1 m 45 offset

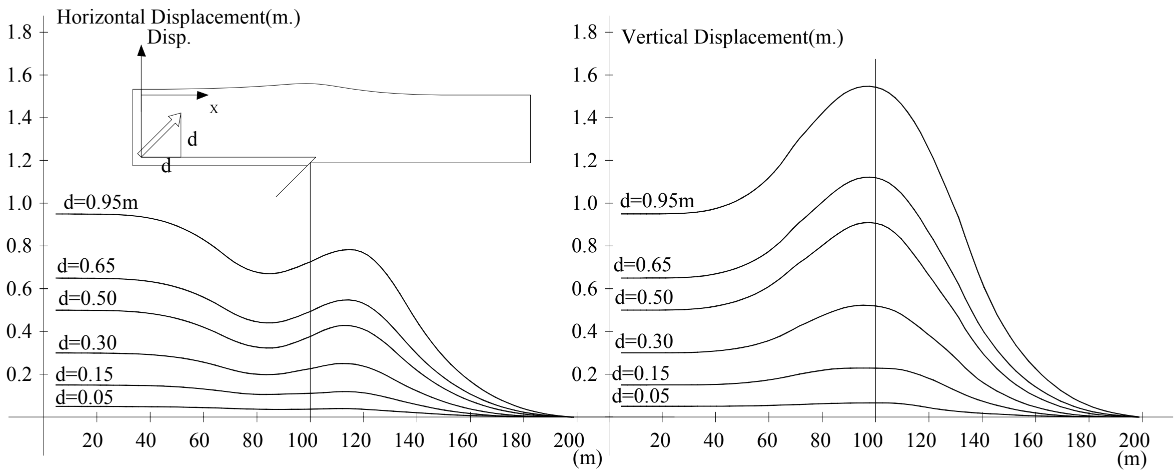

Figure 6. Horizontal and vertical displacements after a 45^{o} fault offset of 1 m.

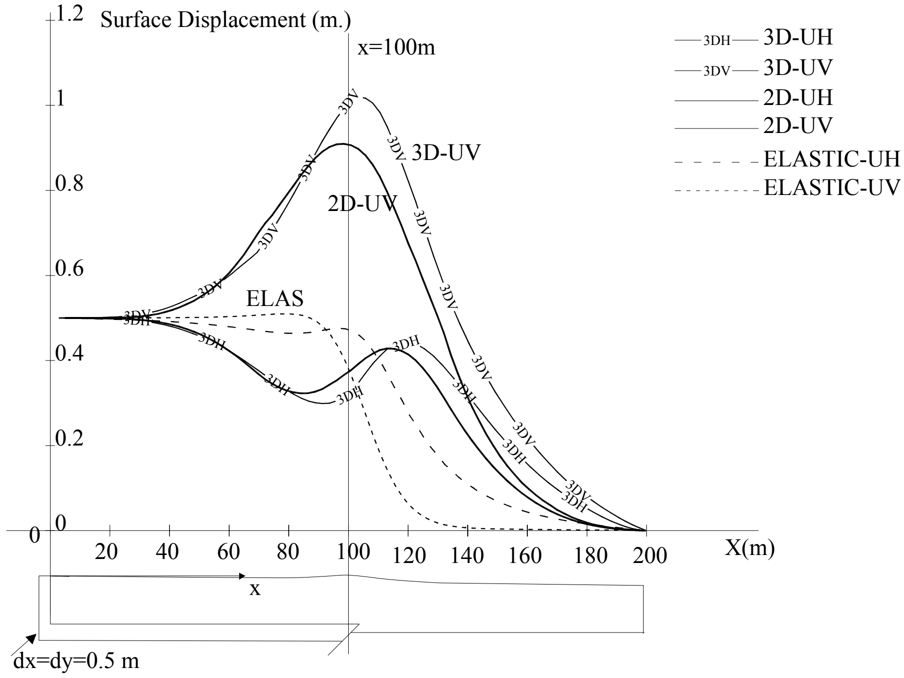

Figure 7. Results from 2D and 3D MPM analyses

For a thorough discussion, variations of ground surface displacements are shown in **Figure 6** 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 6**) 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 7 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 7** 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 and dilatancy.

On **Figure 8** shear strain distribution with single pile group located on footwall side, 15 m ahead of dislocation point is presented. From conjugate sheared zone, the forward one is dispersed by elastic pile group presence and a uniformly sheared zone around pile group is formed by its rigid motion. On Section A-A(soil surface) %1.3 shear strain contour is obviously diverted toward pile group or shear zone is narrowed on the trace of pile group while on section B-B(mid height) it is not so affected. The reason for this lays in on weaker soil near the surface so pile group can affect the dimension of sheared zone.

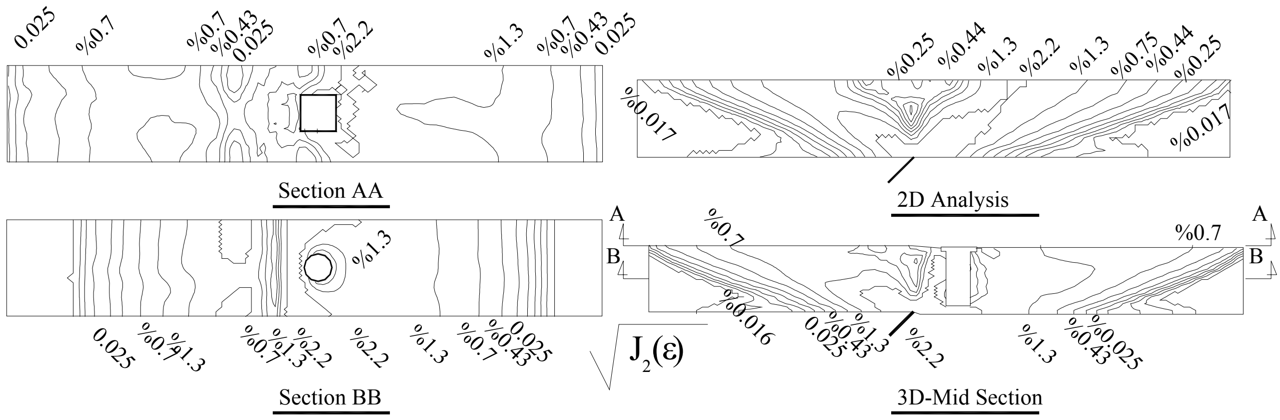

Figure 8. Shear strain when the Pile Group is 15m ahead on footwall side with 0.5m Fault offset

On the next part the surface soil deformation is calculated by putting a pile group on different locations (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. As for horizontal displacements, 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_{pile} / E_{soil} = 1, 4 and 10), and the flexural pile group followed closely the motion of the surrounding soil. On the other hand, vertical displacements were obviously changed by the presence of the pile group. This indicates that the pile group, though flexible in its lateral direction, is stiff enough to pull down the heaving soil. This effect is clearer on the hanging wall side where the thrusting movement of the fault pushes up the soil block in between the two conjugate shear band

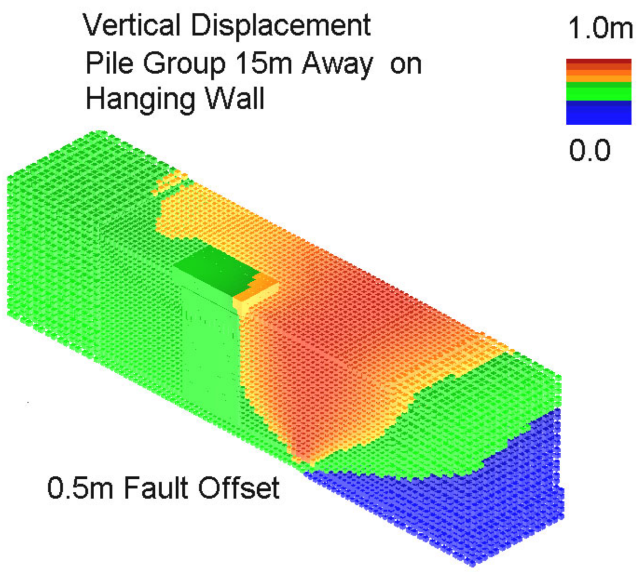

Figure 9. Spatial distribution of vertical soil displacement

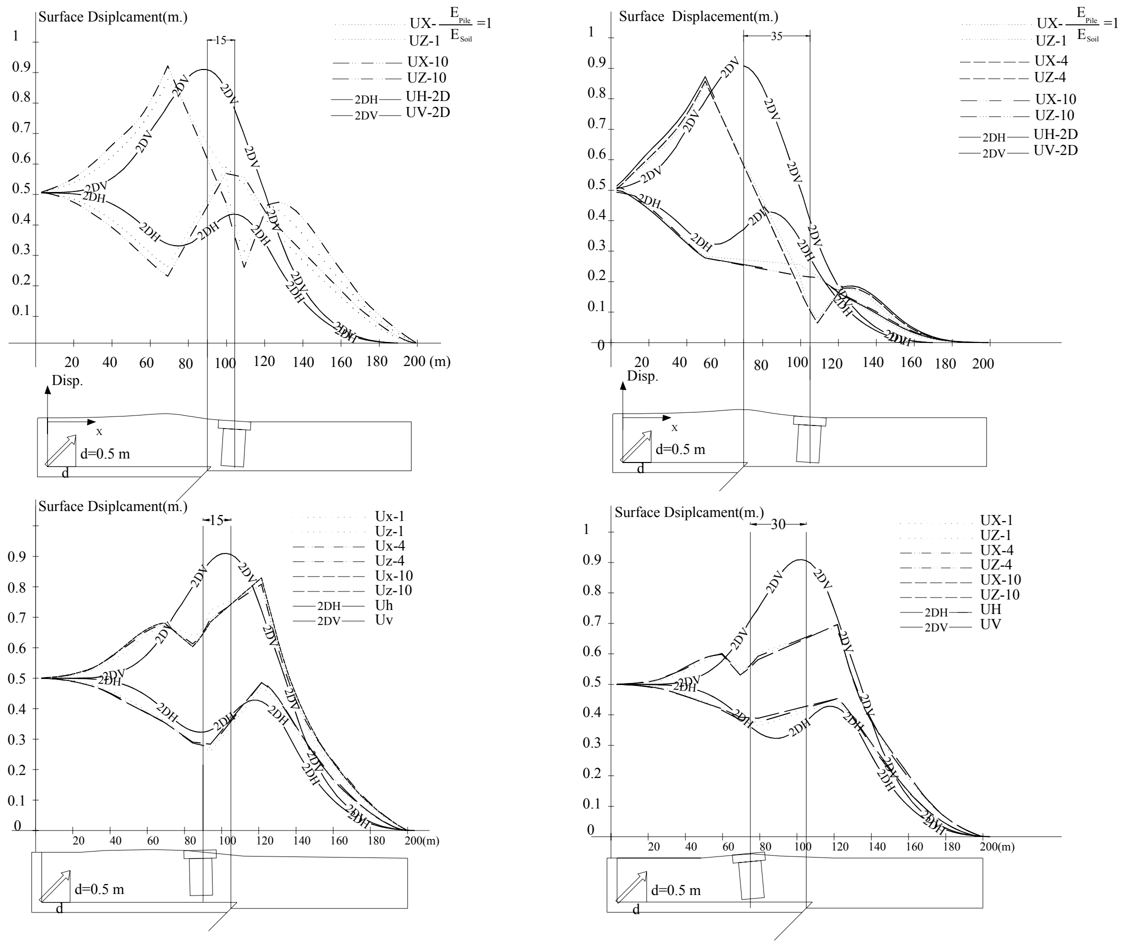

Figure 10. Surface soil displacements calculated for different locations of pile group, +15m and +35m (on the footwall side) and -15m, -30m (on the hanging wall side) off the point of bedrock fault Rupture

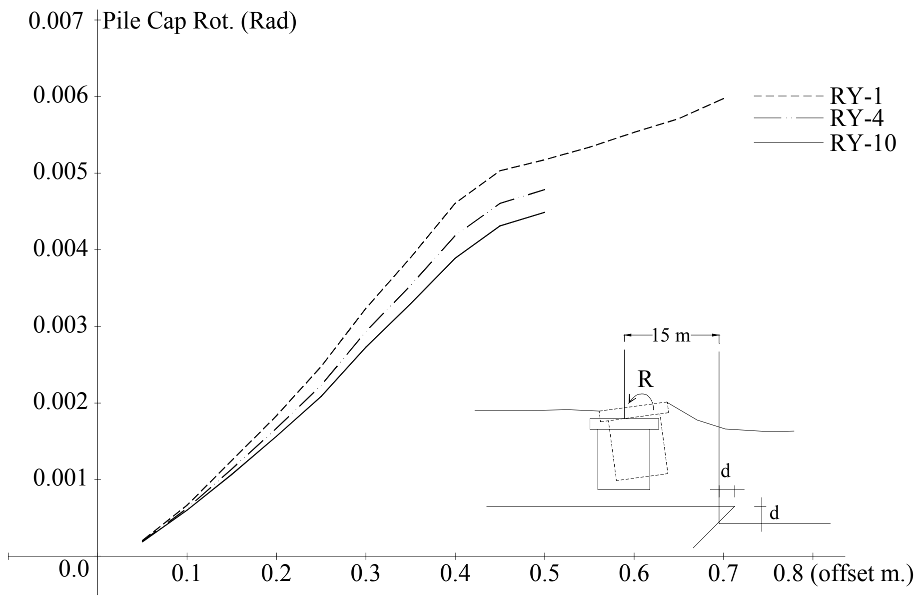

Figure 11. Pile Cap rotation history with increasing fault offset

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 offset exceeded one cell size.

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. Surface deposit rupturing(shearing) mechanism is mainly affected by soil dilatancy feature. Selecting realistic parameters for dilatancy based on factors like deposit grain structure requires careful Engineering judgment.

2. Deformations of surface soil deposit is first Analyzed by excluding the Pile group. Two conjugate shear bands (sheared zone) 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.

3. The presence of the pile group certainly caused the displacement distribution to change in the vicinity. As for horizontal displacements, the flexural effect of pile group followed rather closely the motion of surrounding soil, while clear changes in vertical displacement indicate the pile group axial interaction importance even in the case of piles head are not fixed on bedrock.

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