EXPERIMENTAL VALIDATION OF A NUMERICAL MODEL: REVERSE FAULT RUPTURE PROPAGATION THROUGH SAND

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In this study, a sophisticated numerical model incorporating a hardening-softening constitutive model with shear band is calibrated from the direct shear test results, and validated for the prediction of the behaviour of medium dense Fotainebleau sand bed for quasi-static displacement induced by reverse fault’s base rock with dip angle of 60°. The vertical displacements profile of the ground surface, minimum vertical base displacement for the rupture to reach the ground, the average dip angle propagated into the soil as well as the horizontal extent of the deformed surface ground due to different stress fields (centrifuge and 1g tests) are evaluated by having close agreement between experiments and numerical analyses.

Key Words: sand deposit, reverse fault, elasto-plastic, progressive failure, shear band, finite element, mesh size, scale effect

1. INTRODUCTION

In recent years, the localization of deformation into shear bands or shear zones has received much attention. The proper understanding of shear band mechanism provides a useful basis for soil-structure interaction problems. The failure of sand mass is, usually, progressive in nature. It is related to the development of a shear band of localized deformation. It is very important for the geotechnical engineers to understand the fault rupture propagation through overlaying sand mass to site and design structure near or across fault plane.

Cole & Lade¹ performed extensive sandbox tests using dry loose and dense sands. They predicted the shape of the failure surface over dip-slip fault as a function of the depth of the soil, the angle of dilation, and the dip angles. The downsides to this approach are the introduction of adhoc assumptions for the shape of failure surface, overlooking the progressive failure behavior of soil. Scott & Schoustra² performed numerical simulation of 800 m-deep soil mass over vertical fault by two-dimensional finite element method (FEM), assuming a linear-perfectly plastic relation. They overlooked the hardening softening nature of the soil. Consequently, their results showed the rupture zone bending over to the up-thrown side, which was not consistent with ex-
per experimental results. Roth et al.\textsuperscript{3} compared the centrifuge tests and the shear rupture in 6 m deposits with their finite difference simulation. They concluded that the simulation could duplicate the experiments only qualitatively. Walters & Thomas\textsuperscript{4} performed sandbox experiment and numerical simulation of their experiment by FEM. They found that non-associated flow rule and strain softening were essential in modeling the location, development, and propagation of localized failure surfaces in the granular material. But in their FE analysis, failure surfaces propagated through the sand and broke the ground surface with only a fraction of the displacement observed in experiments. Bray et al.\textsuperscript{5} performed FE analyses and compared the results with the clay-box experiments and anchor pullout experiments. The analogous trapdoor and anchor problems with sand deposits were modeled using FEM by Sakai & Tanaka\textsuperscript{6} and Tanaka & Sakai\textsuperscript{7}. Where, they incorporated the strain hardening-softening behavior of sand with shear band effect.

In this study, a sophisticated numerical modeling procedure is discussed and verified for its usefulness in direct shear tests with medium dense Fontainebleau sand. Then, the objective failure mechanisms and soil deformation patterns of the overlying medium dense Fontainebleau sand deposit over a 60\textdegree dip angled reverse fault are evaluated by comparing the results of a conventional 1g-model test with those of numerical analyses. Afterward, the scale effect is also evaluated between the 1g and 115g centrifuge tests using the results obtained by this numerical model due to the difference in stress level.

2. TESTING PROCEDURES

For the experiment of fault rupture propagation through Fontainebleau sand ($D_{50}=0.244\text{mm}$, $U_0=1.33$, $G_v=2.59$, $e_{max}=0.833$, $e_{min}=0.55$, fines content=0\%) deposit, University of Dundee’s beam centrifuge was used. The strong box internal model dimensions were 800 mm $\times$ 500 mm $\times$ 500 mm (Fig. 1), with front and back transparent Perspex plates, through which the models were monitored during the tests. Two hydraulic cylinders were used to push the hanging or right part up or down to simulate reverse and normal faulting. A central guidance (G) and three wedges ($A_1$-$A_3$) were used to guide the imposed displacement at the desired dip angle (60\textdegree, Fig. 1). Sand was pluviated in the strong box on 20-30 mm thick layers to fill up to desired depth. On top of each layer, a line of dyed sand was laid behind each Perspex wall to clearly visualize the shear bands. The corner and internal cans were placed to verify the sand unit weights inside the strong box and near the edges at the bottom and in the middle of the model ground. A series of digital images were taken for the displaced model ground after each stepwise fault dip slip of about 0.5 to 1.5 mm (time: 10 to 30 seconds) till to the total machine-allowable dip slip or maximum vertical base dislocation ($h_{max}$, in Table 1). The displacement vectors and shear strains in the model ground were analyzed using the deformation measurement system (Geo-PIV program of White et al.\textsuperscript{8}). In addition, linearly variable differential transformers (LVDTs) were used to monitor the vertical settlements of the model ground surface, and the vertical component of the base dislocation ($h$, shown in Fig. 2). All the other definitions of the physical model used in this work are shown in Fig. 2. The strong box was mounted on the centrifuge, and spun to the predetermined g-levels. The prototype dimensions and parameters used in the experiments are given in Table 1. The detailed technical description of this facility and the testing procedures can be found in El Nahas et al.\textsuperscript{9, 10}.

3. NUMERICAL MODELING

This FE model uses an elasto-plastic framework with non-associated flow rule and strain hardening/softening law. An explicit dynamic relaxation method\textsuperscript{11} is used for the solution of the nonlinear equations.

The modeling of the materials having softening properties is full of serious difficulties both in modeling strain localization and from the view point of numerical analysis. The straightforward use of the material softening model in a classical continuum, generally, does not result in a well-posed problem. The standard finite element solution of strain localization in a rate-dependent material results in solutions that is strongly mesh-sensitive. Higher order constitutive models can solve this problem: viscoplastic model\textsuperscript{12}, non local theory\textsuperscript{13}, gradient elasto-plastic model\textsuperscript{14}, otherwise, Gudehus & Nubel\textsuperscript{15}, showed the size of elements has to be in the order of $3D_{50}$. Such fine mesh size prohibits the rigorous application of FE method to real-scaled problems. So, objectively, the shear band effect is introduced into the constitutive equation. The shear band effect

\begin{table}[h]
\centering
\begin{tabular}{|c|c|c|c|c|c|c|}
\hline
Test name & Centrif. & $D_1$ & $H$ & $L$ & $W$ & $h_{max}$ \\
& accel. & (%) & (m) & (m) & (m) & (m) \\
\hline
Test 8 1g R & 1g & 60 & 0.22 & 0.66 & 0.21 & 0.03 \\
Test 8 & 115g & 60.9 & 25.3 & 75.9 & 24.2 & 2.56 \\
\hline
\end{tabular}
\caption{Prototype dimensions and basic parameters for the centrifuge experiments.}
\end{table}
is introduced in the form of a parameter $S$, which is the ratio of shear band area to finite element area and is introduced into the elasto-plastic constitutive model as a characteristic length. This method is similar to the one proposed by Pietruszczak & Mroz\textsuperscript{16}). Unlike their method, however, no direction of shear banding is specified in the present study. Rather, it is implicitly assumed that the direction of shear band coincides in a broad sense with the direction of the maximum shear strain.

\[
e_b = \frac{b_F}{F_e}
\]

where, $b_F$ is the area of shear band in one element and $F_e$ is the total area of an element. An approximated form of $SB$ used in the present study can be expressed as

\[
S = \frac{SB}{\sqrt{F_e}}
\]  

(2)

The return mapping algorithm\textsuperscript{17,18} is used, where the elastically predicted stresses ($\sigma_a$) are relaxed onto a suitably updated yield surface ($\sigma_B$). A change in stresses can cause an associated change in the elastic strain given by

\[
de^e = D^{-1}(\sigma_B - \sigma_a)
\]  

(3)

where, $D$ is the elasticity matrix.

In the present study, it is also assumed that the deformation of a given sand element under uniform boundary stress conditions is homogeneous in the pre-peak regime, and the strain localization in a shear band starts suddenly at the peak stress state. The rate of post-peak strain softening associated with shear banding depends on the strain localization parameter $S$ which is a function of the shear band width. The shear band thickness is known to be about 16-30 times the mean particle diameter ($D_{50}$)\textsuperscript{19,20}. As the total strain does not change during the relaxation process, and, thus, the plastic strain change is balanced by an equal and opposite change in the elastic strain:

\[
Sde^p = -de^e = -D^{-1}(\sigma_B - \sigma_a)
\]  

(4)

The plastic strain increments can be obtained by the following equation:

\[
de^p = \lambda \frac{\partial \Psi}{\partial \sigma}
\]  

(5)

where, $\lambda$ is a positive scalar multiplier to be determined with the aid of the loading-unloading criterion.

Combining equations (4) and (5) $\sigma_B$ can be solved for to obtain:

\[
\sigma_B = \sigma_a - SLD \frac{\partial \Psi}{\partial \sigma}
\]  

(6)

The plastic strain and internal variable (“kappa”) are given by the following equation, respectively:

\[
\varepsilon_B = \varepsilon_a + \lambda \frac{\partial \Psi}{\partial \sigma}
\]  

(7)

\[
\kappa_B = \kappa_a + d\kappa
\]  

(8)

The value of $\lambda$ is

\[
\lambda = \frac{f(\sigma_a, \kappa_a)}{S \frac{\partial f}{\partial \sigma} - \frac{\partial \Psi}{\partial \kappa}}
\]  

(9)

A yield function ($f$) corresponding to the Mohr-Coulomb model and a plastic potential function ($\Psi$), geometrically, represented by the Drucker-Prager model, are employed:

\[
f = -3\alpha(\kappa)\sigma_m + \sqrt{J_2} = 0
\]  

(10)

\[
\Psi = -3\alpha'\kappa\sigma_m + J_2 = 0
\]  

(11)

\[
\kappa = \frac{d\tilde{e}_p}{d\tilde{e}_m}
\]  

(12a)

\[
(d\tilde{e}_m)^2 = 2(d\tilde{e}_p)^2 + (d\tilde{e}_q)^2 + (d\tilde{e}_r)^2 + (d\tilde{c}_{ij})^2 = \lambda^2
\]  

(12b)

where, $\sigma_m$ is the mean stress (positive in compression), $J_2$ is the second invariant of deviatoric...
stresses, $\theta$ is the Lode angle and $de_{\theta}$, $de_y$, $de_{e_y}$, $d\gamma_{e_y}$ are incremental deviatoric plastic strains in the coordinate axes $x$, $y$, and $z$.

In the case of the Mohr-Coulomb model, the Lode angle function, $g(\theta)$ and $\theta$ are given by following equations:

$$g(\theta) = \frac{3 - \sin \phi_{mob}}{2\sqrt{3} \cos \theta - 2 \sin \theta \sin \phi_{mob}}$$  \hspace{1cm} (13a)$$

$$\theta = \frac{1}{3} \cos^{-1} \left[\frac{3\sqrt{3} J_3}{2 J_2^{3/2}}\right]$$  \hspace{1cm} (13b)$$

where, $J_3$ is the third invariant of deviatoric stresses. The mobilized friction angle of $\phi_{mob}$ is given by the equation:

$$\phi_{mob} = \sin^{-1} \left[\frac{3\sqrt{3} \alpha(k)}{2 + 3\sqrt{3} \alpha(k)}\right]$$  \hspace{1cm} (14)$$

The frictional hardening (when $\kappa \leq \varepsilon_f$) and softening (when $\kappa > \varepsilon_f$) functions $\alpha(\kappa)$ are expressed as

$$\alpha(\kappa) = \left[\frac{2 \sqrt{\kappa \varepsilon_f}}{\kappa + \varepsilon_f}\right]^m \alpha_p \text{ (hardening-regime)}$$  \hspace{1cm} (15a)$$

$$\alpha(\kappa) = \alpha_r + (\alpha_r - \alpha_p) \exp \left[-\frac{(\kappa - \varepsilon_f)}{\varepsilon_r}\right] \text{ (softening-regime)}$$  \hspace{1cm} (15b)$$

where, $\alpha_p$, $\varepsilon_f$ and $\varepsilon_r$ are the hardening/softening material parameters and the parameters of $\alpha_p$ and $\alpha_r$ are estimated using the following equations:

$$\alpha_p = \frac{2 \sin \phi_p}{\sqrt{3}(3 - \sin \phi_p)}$$  \hspace{1cm} (16a)$$

$$\alpha_r = \frac{2 \sin \phi_r}{\sqrt{3}(3 - \sin \phi_r)}$$  \hspace{1cm} (16b)$$

where, $\phi_p$, the peak friction angle, is estimated from the empirical relations\(^{[21]}\) to consider the stress level effect, obtaining $\phi_r$, the residual friction angle, from DS tests:

$$I_r = D_r \left[10 - \ln(\sigma_u)\right] - 1$$  \hspace{1cm} (17a)$$

$$\phi_p = 3 I_r + \phi_f$$  \hspace{1cm} (17b)$$

The plastic potential function $\alpha'(\kappa)$ is defined for plane strain conditions as,

$$\alpha'(\kappa) = \tan \psi \sqrt{9 + 12 \tan^2 \psi}$$  \hspace{1cm} (18)$$

The dilatancy angle of $\psi$ is estimated from modified Rowe’s stress-dilatancy relationship,

$$\sin \psi = \frac{\sin \phi_{mob} - \sin \phi_r}{1 - \sin \phi_{mob} \sin \phi_r}$$  \hspace{1cm} (19a)$$

The elastic moduli are estimated from modified equation proposed by Hardin & Black\(^{[22]}\) and are given by the following equations in the case of clean sand:

$$G_{max} = G_e \left(\frac{2(1 - \nu)\varepsilon_0^3}{1 + \varepsilon_0^2} \left(P_a - P_s\right)\right)^{0.5}$$  \hspace{1cm} (20a)$$

$$K = \frac{2(1 + \nu)}{3(1 - 2\nu)} G_{max}$$  \hspace{1cm} (20b)$$

where, $\nu$ is Poisson’s ratio, $\varepsilon_0$ is the initial void ratio, $G_e$ is the initial-shear-modulus constant and $P_a$ is atmospheric pressure.

### 4. VALIDATION OF THE NUMERICAL MODEL

**(I) Direct shear test box**

To justify the usefulness of the constitutive model incorporated in the numerical model discussed earlier, the experimental global response of the direct shear (DS) test box (shear stress-vertical displacement-horizontal displacement) with medium dense Fontainebleu sand is compared with predicted results by the numerical model. The DS tests closely mimics the shearing from the faults, though it has been severely criticized for the nonuniformity of stresses and strains inside the soil sample\(^{[23]}\). The sand was pluviated in the shear box and four Aluminum cans around the box, to check the average unit weight of the pluviated sand ($D=63\pm4\%$). Then, direct shear (DS) tests were

| Table 2 Material parameters of the numerical model. |
|-----------------|-----------------|
| Density (kN/m$^3$) | 15.57 |
| Initial void ratio ($\varepsilon_0$) | 0.64 |
| Initial earth pressure coefficient ($K_0$) | 0.5 |
| Coefficient of shear modulus ($G_e$, kPa) | 50 |
| Residual friction angle ($\phi_r$,˚) | 30.2 |
| Poisson’s ratio (ν) | 0.3 |
| Shear band thickness | 3.84 |
| (SB : mm, model scale) | |
| $\varepsilon_f$ | 0.2 |
| $\varepsilon_r$ | 0.6 |
| $\varepsilon_d$ | 0.3 |
| $m$ | 0.8 |
| $\beta$ | 0.2 |
conducted at a quasistatic displacement rate of 0.01 mm/sec, to determine the sand’s shear strength parameters. The specimen’s length and thickness were 60 and 30.4 mm, respectively. The test procedures followed the test standards BS 1377-7(20). The details can be found in El Nahas et al.9). The finite element mesh used for analyzing the direct shear box test is shown Fig. 3. It contains 360 elements. The analysis is performed for plane strain, with the following boundary conditions. The nodes along the bottom of “A” were fixed. Nodes along the upper box sides were given a prescribed displacement in the horizontal direction. The center of an element row “C” in the sample represents the gap between the upper box and the lower box. The upper box was free to move relative to the lower box in the vertical direction. The solid top element row “B” was assumed to be a cap for transmitting the normal stress. The solid top cap is free to rotate and to move in the vertical direction. The top row and side box of elements model a solid material, with linear elastic properties, Young’s modulus of 2.1×10^4 MPa and Poisson’s ratio of 0.3. Interfaces were assumed to be a soil sample attached to all steel walls with angle of wall friction of a maximum of 6 degrees (determined from DS tests).

To understand the effect of the parameters of such a sophisticated constitutive model on the relationship between average shear stress of elements along the prescribed horizontal shear plane (Fig. 3) and average vertical displacements of the loading plate with respect to relative prescribed horizontal displacements in the DS tests, a detailed parametric study was conducted and some are shown in Fig. 4(a). It can be observed that the hardening/softening material parameters \( \varepsilon_f, \varepsilon_r, \) and \( m \) influence the pre and post-peak of that relationship. The parameter \( \varepsilon_f \) is related to \( \varepsilon_r \). The parameter \( m \) influences the hardening-regime and \( \varepsilon_r \) influences the softening-regime. Also, the stress-dilatancy material parameters \( \beta \) and \( m \) control the mobilization of dilatancy and shear stress in the DS test box. Fig. 4(b) shows close agreement between numerical and experimental results of some DS tests with effective normal stress=200 and 385 kPa, using the calibrated material parameters given in Table 2. So, the calibrated numerical model has derived it’s validation for stress-level effect, strain hardening-softening nature, and dilation in the DS tests. Thus, the material parameters of the numerical model (summarized in Table 2) have been calibrated for medium dense Fontainebleau sand and will be used for the fault analyses in the ensuing sections.

(2) Mesh size sensitivity

The problems of fault rupture propagation are analyzed in this paper, using the quadrilateral isoparametric finite elements. The finite element dis-

cretisation is displayed in Fig. 5. It refers to a uniform soil deposit of thickness, H, at the base of which a reverse fault, dipping at angle of 60° (measured from the horizontal), ruptures and

\[ D_{\text{shear stress}}(\text{MPa}) \]

\[ D_{\text{vertical displacement}}(\text{mm}) \]

\[ D_{\text{horizontal displacement}}(\text{mm}) \]

Fig. 3 Finite element mesh and constituents for direct shear test box analysis (A=soil, B=loading plate, C=gap, not to scale).

Fig. 4 Parametric study on direct shear test box with normal effective stress of 100 kPa (a), and comparison of experimental and numerical results (using calibrated material parameters) with normal effective stress of 200 and 385 kPa (b).

Fig. 5 Finite element mesh and boundary conditions.
produces upward displacement, with a vertical component, \( h \). Following the recommendation of Bray\(^2\) and to minimize undesired boundary effects, the width, \( L \), of the FE model was set equal to \( 4H \). The discretisation is finer in the central part of the model with the quadrilateral elements than those at the two edges where limited deformation is expected. The differential quasielastic displacement is applied to the right part of the model (hanging-wall) in small consecutive increments as shown in Fig. 5. Such a numerical model incorporating hardening-softening model must be verified before to apply to the real-world fault problems, as strain softening makes the analysis sensitive to mesh size. For this purpose, finite elements of sizes: \( 1\text{m}\times1\text{m} \) (width\times height), \( 1.5 \text{m}\times1.5 \text{m} \), \( 2 \text{m}\times2 \text{m} \), respectively, are used in the central part. At the two edges, \( 2\text{m}\times1\text{m} \), \( 3\text{m}\times1.5 \text{m} \), \( 4\text{m}\times2 \text{m} \), respectively, are used (Fig. 5). For all the numerical analyses in this study, the used model parameters are shown in Table 2. In this study, the deformation field is normalized by the soil thickness, \( H \), as suggested by Cole and Lade\(^1\), Bray\(^2\). The results are compared with an arbitrary set experimental result (Test 8). Fig. 6 shows that the normalized vertical displacements (\( \delta y/H \)) are mesh size independent (where, \( d \) is measured from the point of application of the base dislocation, Fig. 2, and normalized by soil thickness, \( H \)). Fig. 7 shows the plots for apparent maximum shear strain averaged at element level for different mesh sizes, and it is observed that the widening of the shear zone is dependent on mesh size, but the orientation of the progressive path is less affected by the mesh size: reducing the size of mesh leads slight shifting of out cropping location towards the foot wall. So, the inclusion of shear band effect into the constitutive relation makes the numerical model insensitive to mesh size.

5. RESULTS AND DISCUSSION

These two reverse faults were conducted at \( 1g \) (Test8_1g_R) and \( 115g \) (Test 8) centrifuge acceleration. However, the sand (\( D_{50}=0.24 \) mm), modeled in the centrifuge sand box, corresponds to a prototype material with mean particle size diameter equals to \( nD_{50} \) (where, \( n \) is scale factor). So, the shear band thicknesses used for \( 1g \) \((n=1) \) and \( 115g \) \((n=115) \) model test analyses are \( 16D_{50} \) (\( \approx 3.84 \) mm) and \( 115\times16D_{50} \) (\( \approx 441.5 \) mm), respectively. Mesh with \( FE=2 \text{m} \) and \( 2/115(\approx0.0174) \text{m} \) are used for \( 115g \) and band thicknesses used for \( 1g \) \((n=1) \) and \( 115g \) \((n=115) \) model test analyses are \( 16D_{50} \) (\( \approx 3.84 \) mm) and \( 115\times16D_{50} \) (\( \approx 441.5 \) mm), respectively. Mesh with \( FE=2 \text{m} \) and \( 2/115(\approx0.0174) \text{m} \) are used for \( 115g \) and \( 1g \) tests, respectively. Numerical predictions
are compared with the experiments in terms of normalized vertical displacements on the ground surface. Additionally, the photographs of the deformed ground and a characteristic shear strain contour are compared with the numerical results.

Figs. 8 and 9 show that the numerically predicted normalized vertical displacements on the ground surface for 1g (for relative base dislocations: \(h/H=2.1\) to 10.3%) and 115g (\(h/H=2\) to 10.1%), respectively, are, satisfactorily, closed to the experimental data points. The development of failure mechanism in the model ground during reverse fault rupture in free field is shown in the images in Figs. 10(a) to 10(d) and 12(a) to 12(d) for Test 8_1g_R and Tests 8, respectively, indicating with the sequence number of shear band formation. The shear bands are drawn by naked eye observation from the deformed colored sand layers, while the average shear strain on the strain plot was about 20% or more. The strain localization in narrow shear bands starts at relative vertical base displacement, \(h/H=4.1\%\) (experiment: \(h/H=3.9\%\)) and \(h/H=5\%\) (experiment: \(h/H=5.1\%\)) at 1 and 115g level, respectively. Increasing the g-level causes late start of shear bands. First, a shear band propagated in the upward direction from the fault point, and later became inactive. Thereafter, the soil shear strains were localized in one or more shear bands which had a steeper inclination to the horizontal than the first shear band and represented the active fault rupture shear bands. This type of progressive nature of shear band formation is predicted also by the the proposed numerical model to a satisfactory extent, shown in Figs. 10(e) to 10(h) and 12(e) to 12(h) for 1g and centrifuge, respectively. The fault rupture lines gradually decreased their inclinations to the horizontal, as they propagated upward. At 1g level, the fault reached the ground surface and developed a scarp after \(h/H=7.3\%\) (experiment: \(h/H=7.4\%\)). In the centrifuge tests, the fault rupture didn’t reach the ground surface, even at the test’s maximum vertical base displacement, \(h_{\text{max}}\) (22 mm, in model scale), whereas the analysis predicts the outermost shear band outcrops at \(h/H=11.2\%\), in model scale. Now, for the calculation of the inclination of the fault main slip surface in the model ground to the horizontal (\(\Delta\), Fig. 2), an operative definition\(^26\) for the location of the fault rupture at the ground surface is used to identify it by the point with the maximum absolute value of the second derivative of the vertical displacement along the horizontal direction. Fig. 11(a) shows such a point on the characteristic normalized vertical displacement plot of the ground surface and that lies within the corresponding shear bands (Fig. 11(b)). Thus, the average dip angle (\(\Delta\)) at maximum \(h\), is 48° (experiment: 50°) and 55° (experiment: 60°) at 1g and 115g levels, respectively. Additionally, Figs. 13 and 14 show the satisfactory comparison between the experimentally and numerically obtained characteristic average shear strain contours in the 1g and centrifuge, respectively. The inclination of the line between the faults rupture point and the ends of the soil deformations at ground surface at the end of each test (\(\theta_{\text{max}}\), Fig. 2) is 33° (experiment: 40.3°) and 35° (experiment: 44°) at 1g and 115g levels, respectively. Here, the numerical model satisfactorily predicts the extension of the ground surface deformation. So, the increasing of the centrifuge acceleration to produce the prototype stress fields in the model ground results increased \(\Delta\) and \(\theta_{\text{max}}\).

6. CONCLUSIONS

The emphasis of this study is on the verification of a sophisticated numerical model for prediction of progressive failure in dry cohesionless material above reverse fault with dip angle of 60° in
Fig. 10 Comparison of experimental results with numerical prediction for Test8_1g_R: (a) to (d) are ground images and (e) to (h) are numerical average shear strain plot on deformed FE mesh (the darkest color indicates shear strain equal or more than 20%).

Fig. 11 Identification of fault outcropping location: (a) a characteristic vertical displacement on the ground surface and the outcropping location (b) corresponding average shear strain plot.
Fig. 12 Comparison of experimental results with numerical predictions for Test 8: (a) to (d) are ground images and (e) to (h) are numerical average shear strain plot on deformed FE mesh (the darkest color indicates shear strain equal or more than 20%).

Fig. 13 Characteristic experimental (a) and numerical (b) average shear strain contour for Test8_1g_R.
relation to soil mechanics and displacement based finite element method. In this regards, the analytical results and extensive experimental results at 1g and 115g level in terms of some important design factors, generally considered for the design of structures near or above the active faults: the normalized vertical displacements profile of the ground surface, minimum normalized vertical base displacement for the rupture to reach the ground, the average dip angle propagated into the soil, inclination of the line joining the faults rupture point and the ends of the soil deformations at ground surface at the end of each test to measure the horizontal extent of the deformed surface ground, are compared. From the results, the scale effects are found as: in the 1g tests, the shear bands start and reach the ground faster, the average dip angle is shallower, as well as, the horizontal extent of soil displacement on the ground surface is larger than that in the centrifuge. So, such a numerical model, including the factors: effect of confining pressure, non-linear strain hardening and softening relationship, stress-dilatancy relationship and shear band effect, can take care when projecting the model test results to prototype scale, like the centrifuge does.

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