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A Study of Passive Earth Pressure in Anisotropic Sand with Various Wall Movement Modes

Achmad Bakri Muhiddin

Old Dominion University

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A STUDY OF PASSIVE EARTH PRESSURE IN ANISOTROPIC SAND WITH VARIOUS WALL MOVEMENT MODES

by

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A Dissertation Submitted to the Faculty of Old Dominion University in Partial Fulfillment of the Requirement for the Degree of

DOCTOR OF PHILOSOPHY
CIVIL ENGINEERING
OLD DOMINION UNIVERSITY
August 2010

Approved by:

Isao Ishibashi (Director)

Duc T. Nguyen (Member)

Gene Hou (Member)
This study investigated the effect of anisotropy on passive pressure in sands by developing computer simulation utilizing FLAC code for plane strain condition. A series of wall movement modes was applied namely translation, rotation about a point below the wall, RBT, and rotation about a point above the wall, RTT.

From comparisons with other FLAC model in translation mode with isotropic material, the coefficients of passive pressure $K_p$ were similar to each other except for some combinations of zero dilation, low wall friction, and high angle of internal friction $\phi$. Dilation angle has less effect on $K_p$ than the effect of $\phi$. Dilation angle of a half of $\phi$ could be used without significant effects on $K_p$.

When comparing simulations with anisotropic material properties and model wall experiment in translation mode, the values of peak $K_{px}$ ($K_p$ in x direction) from simulations were higher for loose sand, close for medium dense, and about the same for dense sand. Strains to reach the maximum $K_{px}$ were less for loose sand, close for medium sand, and higher for dense sand. In RBT modes, $K_{px}$ values were higher for low “n”, and close for high “n” values”, where “n” is the ratio of distance of center of rotation to the wall height. In RTT mode, $K_{px}$ values were higher from simulation with low “n”, and
close for high “n”. For all modes, points of application of resultant of lateral earth pressure “a” at large wall displacement were practically similar. However, in the early stage of wall movement, there exist some differences.

From simulations with increasing “n” with various relative densities, $K_{px}$ values for RBT and RTT modes reached similar maximum at “n” about 2 and 15 respectively. For simulations with various $\phi$ angles in translation, RBT ($n=0$), and RTT ($n=0$) modes, $K_p$ values of anisotropic simulations were significantly smaller than the isotropic simulations. Increasing wall high from 0.5 m to 4.0 m resulted in lower $K_{px}$ values in anisotropic simulations with an average reduction of 13%.
This dissertation is dedicated to my parents and family
ACKNOWLEDGMENTS

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# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>LIST OF TABLES</td>
<td>ix</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>x</td>
</tr>
<tr>
<td>LIST OF SYMBOLS</td>
<td>xvi</td>
</tr>
<tr>
<td>1 INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>2 OBJECTIVE AND SCOPE OF STUDY</td>
<td>5</td>
</tr>
<tr>
<td>3 LITERATURE SURVEY</td>
<td>6</td>
</tr>
<tr>
<td>3.1 Introduction</td>
<td>6</td>
</tr>
<tr>
<td>3.2 Analytical Method in Passive Pressure</td>
<td>7</td>
</tr>
<tr>
<td>3.3 Laboratory Research on Passive Pressure</td>
<td>13</td>
</tr>
<tr>
<td>3.3.1 Translational Wall Movement Mode</td>
<td>14</td>
</tr>
<tr>
<td>3.3.2 Rotational Wall Movement Mode</td>
<td>18</td>
</tr>
<tr>
<td>4 NUMERICAL SIMULATION TECHNIQUE</td>
<td>21</td>
</tr>
<tr>
<td>4.1 FLAC (Fast Lagrangian Analysis of Continua)</td>
<td>21</td>
</tr>
<tr>
<td>4.2 Plane Strain Model Input</td>
<td>22</td>
</tr>
<tr>
<td>4.3 Soil’s Angle of Internal Friction and Soil-Wall Interface Parameters</td>
<td>45</td>
</tr>
<tr>
<td>5 EFFECT OF VARIOUS PARAMETERS AND VERIFICATION OF MODEL WALL TESTS BY FLAC SIMULATION</td>
<td>50</td>
</tr>
<tr>
<td>5.1 Effect of Mesh Configuration, Dilation Angle, and Isotropy versus Anisotropy on Passive Pressure</td>
<td>52</td>
</tr>
<tr>
<td>5.1.1 Effect of Mesh Configuration and Dilation Angle on Isotropic Materials</td>
<td>52</td>
</tr>
<tr>
<td>5.1.2 Effect of Dilation Angle in Isotropic Soils</td>
<td>56</td>
</tr>
<tr>
<td>5.1.3 Effect of Anisotropy</td>
<td>60</td>
</tr>
<tr>
<td>5.2 Simulation of Model Retaining Wall Experiments</td>
<td>61</td>
</tr>
<tr>
<td>5.2.1 Translation Mode</td>
<td>63</td>
</tr>
<tr>
<td>5.2.2 Rotation About A Point Below The Wall Base (RBT) Mode</td>
<td>76</td>
</tr>
<tr>
<td>5.2.3 Rotation About A Point Above The Top (RTT) Mode</td>
<td>86</td>
</tr>
<tr>
<td>5.2.4 Summary of Comparison between Simulations and Experiments</td>
<td>98</td>
</tr>
<tr>
<td>6 LATERAL EARTH PRESSURE SIMULATIONS FOR DIFFERENT WALL MOVEMENTS WITH VARIOUS DENSITIES</td>
<td>100</td>
</tr>
<tr>
<td>6.1 Passive Cases in Translation, RBT, and RTT Modes with Relative Density of 60%, 70%, and 80%</td>
<td>100</td>
</tr>
<tr>
<td>6.2 Effect of Model Scale on Maximum $K_{p_x}$ Values</td>
<td>103</td>
</tr>
</tbody>
</table>
6.3 Effects of Anisotropy in RBT and RTT Modes ........................................... 106

7 SUMMARY AND CONCLUSIONS ......................................................................... 114

BIBLIOGRAPHY ................................................................................................. 117

APPENDIX ........................................................................................................ 120

VITA ................................................................................................................ 194
LIST OF FIGURES

Fig. 3.1 Two types of passive wall movement: (a) RTT mode, (b) RBT mode (Fang et al. (1994)) .................................................. 20

Fig. 4.1 Principal stress directions at failure of soil under a foundation footing .......... 22

Fig. 4.2 Typical stress-strain relations for tests at $\sigma'_3=0.05\text{kgf/cm}^2$ for dense samples, after Tatsuoka et al. (1986) .................................................. 24

Fig. 4.3 Typical stress-strain relations for tests at $\sigma'_3=0.05\text{kgf/cm}^2$ for loose samples, after Tatsuoka et al. (1986) .................................................. 25

Fig. 4.4 Typical stress-strain relations for tests at $\sigma'_3=4.0\text{ kgf/cm}^2$ for dense samples, after Tatsuoka et al. (1986) .................................................. 26

Fig. 4.5 Typical stress-strain relations for tests at $\sigma'_3=4.0\text{ kgf/cm}^2$ for loose samples, after Tatsuoka et al. (1986) .................................................. 27

Fig. 4.6 Angle of internal friction $\phi$ vs. principal strain $\varepsilon_1$ for particular relative density, confining pressure, and principal stress direction ............. 28

Fig. 4.7 $\tan\phi(\alpha_f)/\tan(\alpha_t=90^\circ,\text{PSC})$ versus $\alpha_f = 90^\circ - \delta$ of Toyoura sand in Tatsuoka et al. (1990) .................................................. 29

Fig. 4.8 $R(\delta) = \phi(\delta)/\phi$ ($\delta=90^\circ$) versus $\delta$ for different relative density ................. 30

Fig. 4.9 Minimum of $(\phi_{\text{peak}}@\delta)/\phi(\delta=90^\circ)$ vs. Dr ................................................. 32

Fig. 4.10 $\delta$ value at Minimum$(\phi_{\text{peak}}@\delta)/\phi(\delta=90^\circ)$ vs. Dr ................................................. 33

Fig. 4.11 Effect of low $\sigma_3$ on $\phi_{\text{peak}}$ for $\delta=90^\circ$ ................................................. 34

Fig. 4.12 Effect of low $\sigma_3$ on $\phi_{\text{res}}$ for any $\delta$ ................................................. 34

Fig. 4.13 Effect of Dr on $\varepsilon_{\text{peak}}$ and $\varepsilon_{\text{res}}$ for $\delta = 90^\circ$ data read from Alshibli and Sture (2000), Tatsuoka et al. (1986) ................................................. 35

Fig. 4.14 $\phi/\phi_{\text{peak}}$ vs. $\varepsilon_1/\varepsilon_{\text{peak}}$ (0 to peak) ................................................. 38

Fig. 4.15 $(\phi - \phi_{\text{res}})/(\phi_{\text{peak}} - \phi_{\text{res}})$ vs. $(\varepsilon_1 - \varepsilon_{1\text{peak}})/(\varepsilon_{1\text{res}} - \varepsilon_{1\text{peak}})$, (peak to residual) ................................................. 39

Fig. 4.16 Dilation angle $\psi$ for $\sigma_3 = 0.05\text{ kgf/cm}^2$, Dense ................................................. 40
Fig. 4.17 Dilation angle $\psi$ for $\sigma_3 = 0.05 \text{ kg/cm}^2$, Loose ........................................ 41

Fig. 4.18 Dilation angle $\psi$ for $\sigma_3 = 4.0 \text{ kg/cm}^2$, Dense ........................................ 42

Fig. 4.19 Dilation angle $\psi$ for $\sigma_3 = 4.0 \text{ kg/cm}^2$, Loose ........................................ 43

Fig. 4.20 Variation of dilation angle ($\psi$) to principal strain ........................................ 44

Fig. 4.21 Effect of Dr on direct shear angle of internal friction of Ottawa sand, $\varphi_{DS}$; data from Fang et al. (2002) ................................................................. 45

Fig. 4.22 Effect of Dr on ratio of steel wall-sand friction of Ottawa sand (Fang et al., 2002) .......... 49

Fig. 5.1 Model of sandbox and soil elements used in FLAC ..................................................... 51

Fig. 5.2 Mesh used in FLAC simulation by Bennebake et al. (2006) ........................................ 53

Fig. 5.3 Isotropic solution of $K_p$ with $\delta/\varphi=0$ by current solution and Bennebake et al. (2006) ................................................................. 54

Fig. 5.4 Isotropic solution of $K_p$ with $\delta/\varphi=1/3$ by current solution and Bennebake et al. (2006) ................................................................. 55

Fig. 5.5 Isotropic solution of $K_p$ with $\delta/\varphi=2/3$ by current solution and Bennebake et al. (2006) ................................................................. 56

Fig. 5.6 Isotropic solution of $K_p$ for $\delta/\varphi = 0, 1/3, 2/3$ and $\psi/\varphi = 0, 1/2, 1$ ................. 57

Fig. 5.7 Anisotropic solution of $K_p$ with fixed $\psi/\varphi = 1/2$ and varying $\psi$, for $\delta/\varphi = 0, 1/3, 2/3$ ................................................................. 59

Fig. 5.8 Translation Mode: $K_p$ of Anisotropic and isotropic simulations .................................. 61

Fig. 5.9 Typical horizontal stress ($\sigma_h$) by soil gravity ......................................................... 62

Fig. 5.10 Typical vertical stress ($\sigma_v$) by soil gravity ............................................................. 63

Fig. 5.11 Translation mode: contour of accumulated plastic shear strain .................................. 64

Fig. 5.12 Translation mode: grid distortion and principal stress direction ............................... 65

Fig. 5.13 Translation mode: elements indicated as yield in shear ............................................ 66
Fig. 5.14 Stress strain relation (φ vs. ε₁) of elements adjacent to moving wall ............... 67

Fig. 5.15 Comparison of horizontal stress from Fang’s experiment and FLAC simulation at different stages of wall translation for dense soil of Dᵣ = 80% (file: atn0580m08c.dat) ................................................................. 68

Fig. 5.16 Development of Kₓ during translation for soil of Dᵣ = 32% from experiment by Fang et al. (1994) and FLAC simulation of different wall heights ......................... 70

Fig. 5.17 Points of application of horizontal stress of loose soil with Dᵣ = 32%, experiment by Fang et al. (1994) and FLAC simulation of different wall heights ................................................................. 71

Fig. 5.18 Development of Kₓ during translation for soil of Dᵣ = 38% from experiment by Fang et al. (2002) and FLAC simulation of different wall heights ......................... 73

Fig. 5.19 Points of application of horizontal stress of loose soil with Dᵣ = 38% experiment by Fang et al. (2002) and FLAC simulation of different wall heights ......................... 73

Fig. 5.20 Development of Kₓ during translation for soil of Dᵣ = 63% from experiment by Fang et al. (2002) and FLAC simulation of different wall heights ......................... 74

Fig. 5.21 Points of application of horizontal stress of medium soil with Dᵣ = 63% experiment by Fang et al. (2002) and FLAC simulation of different wall heights ................................................................. 74

Fig. 5.22 Development of Kₓ during translation for soil of Dᵣ = 80% from experiment by Fang et al. (2002) and FLAC simulation of different wall heights ......................... 75

Fig. 5.23 Points of application of horizontal stress of dense soil with Dᵣ = 80% experiment by Fang et al. (2002) and FLAC simulation of different wall heights ................................................................. 75

Fig. 5.24 RBT mode: contour of accumulated plastic shear strain............................ 77

Fig. 5.25 RBT mode: grid distortion and principal stress direction............................ 78

Fig. 5.26 RBT mode: elements indicated as yield in shear .................................... 79

Fig. 5.27 RBT mode: Horizontal pressure for Dᵣ = 32%, “n” = 0.5 from Fang et al. (1994) and FLAC simulations ................................................................. 80

Fig. 5.28 RBT mode: Kₓ for Dᵣ = 32%, n=0, from Fang et al. (1994) and simulation with H = 0.5m, 1m, 1.5m, 2m, 4m ................................................................. 81
Fig. 5.29 RBT mode: point of application "a" for $D_r = 32\%$, $n=0$, from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$ ..................... 82

Fig. 5.30 RBT mode: $K_{px}$ for $D_r = 32\%$, $n=0.21$, from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$ ..................... 82

Fig. 5.31 RBT mode: point of application "a" for $D_r = 32\%$, $n=0.21$, from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$ ..................... 83

Fig. 5.32 RBT mode: $K_{px}$ for $D_r = 32\%$, $n=0.5$, from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$ ..................... 83

Fig. 5.33 RBT mode: point of application "a" for $D_r = 32\%$, $n=0.5$, from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$ ..................... 84

Fig. 5.34 RBT mode: $K_{px}$ for $D_r = 32\%$, $n=13.78$ from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$ ..................... 85

Fig. 5.35 RBT mode: point of application a for $D_r = 32\%$, $n=13.78$ from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$ ..................... 85

Fig. 5.36 RTT mode: contour of accumulated plastic shear strain ................................. 87

Fig. 5.37 RTT mode: grid distortion and principal stress direction ................................. 88

Fig. 5.38 RTT mode: elements indicated as yield in shear ......................................... 89

Fig. 5.39 RTT mode: $D_r = 32\%$, "n" = 0 from Fang et al. (1994) and simulation ............ 90

Fig. 5.40 RTT mode: $K_{px}$ for $D_r = 32\%$, $n=0$ from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$ ............................... 91

Fig. 5.41 RTT mode: point of application "a" for $D_r = 32\%$, $n=0$ from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$ ............................... 92

Fig. 5.42 RTT mode: $K_{px}$ for $D_r = 32\%$, $n=0.5$ from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$ ............................... 92

Fig. 5.43 RTT mode: point of application "a" for $D_r = 32\%$, $n=0.5$ from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$ ............................... 93

Fig. 5.44 RTT mode: $K_{px}$ for $D_r = 32\%$, $n=1.81$ from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$ ............................... 94
Fig. 5.45 RTT mode: point of application “a” for $D_r = 32\%$, $n=1.81$ from Fang et al. (1994) and simulation with $H = 0.5m$, $1m$, $1.5m$, $2m$, $4m$ ................................. 95

Fig. 5.46 RTT mode: $K_{px}$ for $D_r = 32\%$, $n=7.43$ from Fang et al. (1994) and simulation with $H = 0.5m$, $1m$, $1.5m$, $2m$, $4m$ ................................. 95

Fig. 5.47 RTT mode: point of application “a” for $D_r = 32\%$, $n=7.43$ from Fang et al. (1994) and simulation with $H = 0.5m$, $1m$, $1.5m$, $2m$, $4m$ ................................. 96

Fig. 5.48 $K_{px}$ values for RBT and RTT modes with increasing “n” values from Fang et al. (1994) and FLAC simulation for $D_r = 32\%$ at $\text{Smax}/H=0.1$ ................................. 97

Fig. 5.49 Point of application “a” for RBT and RTT modes with increasing “n” values from Fang et al. (1994) and FLAC simulation for $D_r = 32\%$ at $\text{Smax}/H=0.1$ ................................. 98

Fig. 6.1 Max $K_{px}$ (up to $S\text{max}/H=0.1$) with increasing “n” for RBT, RTT, translation modes for $H=4m$ and $D_r$: 60%, 70%, and 80% ................................. 101

Fig. 6.2 Varying point of application “a” with $S\text{max}/H$ for RBT, RTT (n: 0, 0.5, 2, 7, 15), and Translation modes for $H = 4m$, $D_r = 80\%$ ................................. 102

Fig. 6.3 Translation mode: $K_{px}$ for soil depth: 0.5m, 4m, and $D_r$: 60%, 70%, and 80% ................................. 103

Fig. 6.4 RBT mode (n=0): $K_{px}$ for soil depth: 0.5m, 2m, 4m, and $D_r$: 60%, 70%, and 80% ................................. 104

Fig. 6.5 RBT mode (n=15): $K_{px}$ for soil depth: 0.5m, 2m, 4m, and $D_r$: 60%, 70%, and 80% ................................. 104

Fig. 6.6 RTT mode (n=0): $K_{px}$ for soil depth: 0.5m, 2m, 4m, and $D_r$: 60%, 70%, and 80% ................................. 105

Fig. 6.7 RTT mode (n=15): $K_{px}$ for soil depth: 0.5m, 4m, and $D_r$: 60%, 70%, and 80% ................................. 105

Fig. 6.8 RBT mode: $K_p$ for Isotropic and Anisotropic simulations with $\phi = 30^\circ, 35^\circ, 40^\circ$ ................................. 107

Fig. 6.9 RBT mode: point of application “a” for Isotropic and Anisotropic simulations with $\phi = 30^\circ, 35^\circ, 40^\circ$ ................................. 108

Fig. 6.10 RBT mode: Max of $K_p$ up to $S\text{max}/H=0.1$ vs. $\phi_p$ for Isotropic and Anisotropic simulations ................................. 109

Fig. 6.11 RTT mode: $K_p$ for Isotropic and Anisotropic simulations with $\phi = 30^\circ, 35^\circ, 40^\circ$ ................................. 111
Fig. 6.12 RTT mode: point of application “a” for Isotropic and Anisotropic simulations with $\phi = 30^\circ, 35^\circ, 40^\circ$ ................................................................. 112

Fig. 6.13 RTT mode: Max of $K_p$ up to $S_{max}/H=0.1$ vs. $\varphi_{PS}$ for Isotropic and Anisotropic simulations................................................................. 113
### LIST OF SYMBOLS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>Point of application of resultant as a ratio of soil height</td>
</tr>
<tr>
<td>a\text{1}, a\text{2}, a\text{3}</td>
<td>Coefficients used in regression equation in Table 4.1</td>
</tr>
<tr>
<td>dat\text{min}</td>
<td>Value of angle $\delta$ at minimum of $(\varphi_{\text{peak}} @ \delta / \varphi_{\text{peak}} @ 90^\circ)$ as shown in Fig. 4.10</td>
</tr>
<tr>
<td>dila</td>
<td>Maximum dilation angle as defined in Fig. 4.20</td>
</tr>
<tr>
<td>dil\text{res}</td>
<td>Residual dilation angle as defined in Fig. 4.20</td>
</tr>
<tr>
<td>dil\text{x2}</td>
<td>Major principal strain at which dilation = 0 as defined in Fig. 4.20</td>
</tr>
<tr>
<td>dilyl</td>
<td>Dilation angle at major principal strain = 0 as defined in Fig. 4.20</td>
</tr>
<tr>
<td>Dr</td>
<td>Relative density</td>
</tr>
<tr>
<td>e</td>
<td>Void ratio</td>
</tr>
<tr>
<td>E</td>
<td>Elastic modulus</td>
</tr>
<tr>
<td>e_{0.05}</td>
<td>Void ratio at confined pressure 0.05 kgf/cm$^2$</td>
</tr>
<tr>
<td>e_{\text{max}}</td>
<td>Maximum void ratio</td>
</tr>
<tr>
<td>e_{\text{min}}</td>
<td>Minimum void ratio</td>
</tr>
<tr>
<td>FLAC</td>
<td>Fast Lagrangiang Analysis of Continua, a finite difference program</td>
</tr>
<tr>
<td>FLAC3D</td>
<td>FLAC for 3 dimension analysis</td>
</tr>
<tr>
<td>G</td>
<td>Shear modulus</td>
</tr>
<tr>
<td>GLE</td>
<td>General limit equilibrium</td>
</tr>
<tr>
<td>G_{\text{max}}</td>
<td>Maximum shear modulus</td>
</tr>
<tr>
<td>G_{s}</td>
<td>Specific gravity</td>
</tr>
<tr>
<td>H</td>
<td>Wall height</td>
</tr>
</tbody>
</table>
K: Bulk modulus

$k_n$: Normal stiffness of contact plane (in FLAC program)

$K_o$: Ratio of horizontal stress to vertical stress in at-rest condition

$K_p$: Coefficient of passive earth pressure

$K_{px}$: Coefficient of passive earth pressure in horizontal direction

$k_s$: Shear stiffness of contact plane (in FLAC program)

M: The power to OCR used in Eq. (4.7)

$\text{minrat}$: Minimum value of ratio $R$ as shown in Fig. 4.9

n: Ratio of the distant of point of rotation to wall height as defined in Fig. 3.1

OCR: Over-compaction ratio

$p_a$: Atmospheric pressure

$P_p$: Passive earth force

$P_{px}$: Component of passive earth force in x direction

PSC: Plane strain compression

R: Ratio of $\phi$ at $\delta$ to the $\phi$ at $\delta = 90^\circ$ as shown in Fig. 4.8

RBT: Rotation about a point below the wall base as defined in Fig. 3.1

RTT: Rotation about a point above the wall top as defined in Fig. 3.1

S: Horizontal wall movement in translation mode.

Coefficient used in Eq. (4.7)

S_{max}: Maximum horizontal wall movement in RTT or RBT modes as defined in Fig. 3.1

TC: Triaxial compression
TSS  Torsional simple shear
z  Depth of soil element
α  Slope of volumetric strain curve with major principal strain
α_r  90-δ
γ  Unit weight
γ_w  Unit weight of water
δ  Angle between bedding plane and major principal stress direction
Δz_{min}  The smallest width of an adjoining zone in the normal direction to the interface of wall and soils (in FLAC program)
ε_1  Major principal strain
ε_{1peak}  Major principal strain at the peak value of angle of internal friction as defined in Fig. 4.6
ε_{1res}  Major principal strain at the residual value of angle of internal friction as defined in Fig. 4.6
ε_v  Volumetric strain
ν  Poisson ratio
ξ  Dilation angle from ξ log-spiral technique by Chen and Liu (1990)
\bar{σ}_o  Average principal stress
σ_1  Major principal stress
σ'_1  Effective major principal stress
σ_3  Minor principal stress
σ'_3  Effective minor principal stress
σ_h  Horizontal stress
σ_v  Vertical stress
<table>
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<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_x$</td>
<td>Horizontal stress</td>
</tr>
<tr>
<td>$\varphi$</td>
<td>Angle of internal friction</td>
</tr>
<tr>
<td>$\varphi'$</td>
<td>Effective angle of internal friction</td>
</tr>
<tr>
<td>$\varphi_{DS}$</td>
<td>Angle of internal friction from direct shear test</td>
</tr>
<tr>
<td>$\varphi_{peak}$</td>
<td>Peak angle of internal friction as defined in Eq. (4.1) and Fig. 4.6</td>
</tr>
<tr>
<td>$\varphi_{ps}$</td>
<td>Angle of internal friction from plane strain test</td>
</tr>
<tr>
<td>$\varphi_{res}$</td>
<td>Residual angle of internal friction as defined in Eq. (4.1) and Fig. 4.6</td>
</tr>
<tr>
<td>$\varphi_{tx}$</td>
<td>Angle of internal friction from triaxial test</td>
</tr>
<tr>
<td>$\psi$</td>
<td>Dilation angle</td>
</tr>
</tbody>
</table>
1 INTRODUCTION

Varieties of structures are subject to lateral earth pressures from backfill soils, including such structures as bridge abutment, anchored bulkhead, quay wall, basement wall, wall around cut and fill along highway, and others. To properly design such structures, the understanding of interaction between soil and structure movement is essential. In the case of wall movement relative toward the backfill soil, the lateral pressure built up against the wall is termed passive earth pressure.

Coulomb (1776) and Rankine (1857) formulated passive pressure theories, as part of lateral earth pressure theories, which are still widely used in practice. The theories are based on condition of isotropic and homogeneous soil. Rankine assumed a frictionless contact between the wall and backfill soil. While Coulomb’s theory, using the limit equilibrium method, allows analysis for rough wall to soil contact and assumes a planar failure surface. A different failure surface (i.e. log spiral) was introduced by Terzaghi (1943). Comparisons between these methods were presented in Duncan and Mokwa (2001). The comparisons between those theoretical methods and laboratory experimental results were made by Fang et al. (2002).

Assumption of isotropy is not in agreement with the anisotropic nature of soil fabric and its strength. Anisotropy of sands has been reported by Oda (1972), Oda et al. (1978), Oda (1981), Ochiai and Lade (1983), Tatsuoka et al. (1986), Lam and Tatsuoka (1988), Tatsuoka et al. (1990), Park and Tatsuoka (1994), and Abelev and Lade (2003),

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1 Journal of Geotechnical and Geoenvironmental Engineering, ASCE, is used as the format model for reference cited.
etc. These laboratory investigations on sands showed that stress-strain responses were appreciably affected by the principal stress direction relative to the bedding plane or fabric orientation. This characteristic could be applied into a context of wall movement toward backfill. Soils at different points adjacent to the wall could have different orientations of the principal stresses and those orientations may also change as the wall movement proceeds. The differences in principal stress orientation among soil elements are even more prominent in the case of wall rotation rather than in the case of simple translational wall movement. Therefore, in anisotropic soil condition, stress distribution, resultant, and its point of application could be different from those of traditional analysis with isotropy assumption.

This study focuses on determination of passive earth pressure with various modes of wall movement. Recently Fang and others (Fang et al. (1994), Fang et al. (1997), Fang et al. (2002)) have conducted experiments on passive earth pressure on a model using dry sand on static conditions. Various static wall movement modes were investigated; translation, rotation about a point above the top of the wall, and rotation about a point below the base of the wall. Experimental data obtained by Fang and others were utilized to validate a computer code which is used in this research. A commercially available computer code specifically developed for geotechnical engineering applications, FLAC, a 2-D explicit finite difference program, was utilized. After validating of the code with experimental results, the code was used to solve many different conditions of passive earth pressure case. In order to utilize the code to simulate experimental model tests, realistic stress-strain relations of the soil should first be modeled.
System of soil and wall movement in the two-dimensional experiment can be best regarded as a plane strain condition. Therefore, soil parameters for this condition should be best represented by the results of plane strain compression tests. However, most of the available soil's stress-strain data are the results of triaxial compression tests. There are very few available experimental data on stress-strain relation of sand performed in plane strain condition. Based on these few available data, a series of rather simplified stress-strain models was developed. Observing the results of experimental data on sand, it was shown that the stress strain-relation is of strain-hardening/softening type. The first portion is an increase in stress to a maximum value, then a decrease to a lower residual stress, beyond which the stress is more or less constant. Linear portion of the stress-strain relation can only be identified at the onset within a very small strain relative to the strain at the maximum stress value. Stress-strain relations are significantly affected by the confined pressure, relative density, and anisotropic characteristics of soils. The stress-strain relations are obtained for low and high confined stress conditions and for loose and dense conditions of the sand. Soil's anisotropy was treated as a function of the inclination angle of principal stress application relative to the bedding plane direction of sand. To simulate experimental model tests, a procedure was incorporated in the program to allow soil's parameters at different locations be governed by an appropriate stress-strain relation.

Based on the developed stress-strain relations, verification of the computer code was performed by comparing the computed results with the results of laboratory static model retaining wall tests conducted by Fang and others. The distribution of soil pressure behind the wall in terms of the resultant and the point of application were used as
variables in the verification. Then a larger prototype model of retaining wall, which is subjected to passive pressure in a static condition, was further developed. In the prototype model, a more common wall size with a more common stress magnitude was simulated with various soil parameters and conditions of wall movements. Parametric studies on this study resulted in an added understanding of passive earth pressure during static wall movement cases.
2 OBJECTIVE AND SCOPE OF STUDY

The objective of this study is to investigate the effect of anisotropy of backfill sand to passive pressure and further to develop a set of design guideline. The anisotropy of sand will be simulated by applying different sets of stress-strain relation to every point in the soil mass. A series of possible static wall movement is applied to soils of different relative density in order to learn passive pressure built-up on the wall. The static wall movements consist of translation, rotation about a point below the wall, and rotation about a point above the wall.

The scope of this study will be confined to backfill of dry homogenous but anisotropic sand. Backfill soil structure is confined to horizontal and without surface load. Problems will be considered plane strain, which is applicable for most actual passive pressure condition.
3 LITERATURE SURVEY

3.1 Introduction

Coulomb (1776) and Rankine (1857) proposed theories that are still widely used in the geotechnical engineering profession. The theories are based on the assumptions of isotropic and homogeneous soil. The rupture surface and backfill surface are assumed to be planar and failure is regarded as a plane strain problem. The difference between the two theories is that Rankine assumes a frictionless contact between the wall and backfill soil, and no soil cohesion, while Coulomb’s theory, which uses a limit equilibrium method, allows analysis for rough wall soil contact. Failure wedge is considered a rigid body undergone translation.

According to Duncan and Mokwa (2001), Morgenstern and Eisenstein (1970), and Narain et al. (1969), Coulomb’s theory overestimated passive pressure. Meanwhile the assumption used in Rankine theory limited its applicability due to the lack of wall friction angle. Due to doubt on the validity of the previous theories, early investigators developed different techniques to estimate passive earth pressure. Terzaghi (1943) or later Terzaghi et al. (1996) used a combination of a logarithmic-spiral curve and a straight line for failure surface. Caquot and Kérisel (1948) and Kerisel and Absi (1990) produced tables of passive earth pressure based on the arc of an ellipse for failure surfaces with a limit equilibrium method. Application of the limit equilibrium method gave non-conservative results. Information concerning the critical failure surface and its kinematic admissibility for a specified movement of the wall is still lacking. Sokolovski (1960) developed a technique called method of characteristics using finite-difference
solutions. The method was based on the assumptions that sand was everywhere in equilibrium, and that sand was everywhere yielding according to the Mohr-Coulomb criteria. All these methods are basically theoretical solutions that lack experimental validations.

The following section will present subsequent investigations on analytical and experimental methods in determining passive earth pressure. Some of the experimental investigations were performed in laboratory settings and the other in field experiments.

3.2 Analytical Method in Passive Pressure

Shields and Tolunay (1973) computed the coefficients of passive earth pressure using the method of slices similar to that employed in slope stability analysis. Failure zone, which is a combination of logarithmic spiral and straight line, was divided into several vertical slices. Calculations were conducted for horizontal sand backfill with vertical wall. The resulted coefficient of passive earth pressure compared favorably with experimental findings for dense sand. However, those were a little lower for loose sand, and, therefore the method was considered conservative. It was then proposed to use a reduced value of angle of internal friction for dense sand. The method is simple and may be extended to problems involving sloping backfill and surcharge loading.

Chen (1975) applied an upper-bound technique of limit analysis to obtain a solution to lateral earth pressure problems on rigid retaining wall. The coefficient of passive pressure was obtained by equating the rate of external work to the rate of internal energy dissipation. He introduced a log-sandwich failure surface where a logarithmic spiral sandwiched between two rigid blocks. It was assumed that the spiral function was
defined by the angle of internal friction $\phi$, and that the vector velocity of soil adjacent to the wall was parallel to the wall surface. The resulted solutions of this method were in good agreement with stress characteristics from Sokolovski. Later Chen and Liu (1990) modified the log-sandwich mechanism of upper-bound limit analysis by earlier Chen (1975). They defined logarithmic spiral surface by $\xi$ angle, with $\xi \leq \phi$, and termed the technique $\xi$-log-sandwich mechanism. They also adopted a non-associated flow rule (or partial friction – partial dilatation model) where velocity vector of soil adjacent to the wall is not parallel to the wall. Comparison between previous and modified versions of limit analysis showed that the modified technique gave somewhat lower $K_p$ values.

Martin and Yan (1995) presented results of a numerical study modeling the passive earth pressure characteristics of a bridge abutment. The numerical modeling is carried out using the computer program FLAC, a two-dimensional explicit finite difference code for geotechnical engineering applications. In the modeling, numerical analyses are performed for a series of typical abutment wall heights, soil types and soil properties of the abutment backfill. The investigation showed different results when compared with other analytical methods if the friction angle of wall and backfill are high or low. The model, however, did not handle the anisotropy of soil properties.

Kumar and Subba Rao (1997) used a combination of a logarithmic spiral and a straight line as a failure surface. Comprehensive charts were developed to determine the passive earth pressure coefficients and the positions of the critical failure surface for positive as well as negative wall friction. Translational movements of the wall were examined, considering the soil as either an associated flow dilatant material or non-
dilatant material, to determine the kinematic admissibility of the limit equilibrium solutions.

Zhang et al. (1998) developed a methodology for solving earth pressure problems under any boundary strain constraint. The method was based on the strong dependence of an earth pressure coefficient on strain increment ratio that was revealed based on triaxial loading tests along different constant strain paths. Earth pressure equations were obtained by extending the formula of Rankine and Coulomb theories. The proposed equations can be used to determine lateral earth pressure for normally consolidated cohesionless soil for any lateral deformation between the active and passive states of stress, including at rest condition. Charts corresponding to several simple cases were provided for actual design. Simplified methods were also suggested to determine the parameters in the proposed equations and to evaluate the earth pressure for different types of lateral deformation. Further theoretical and experimental investigation is needed to confirm effectiveness of the proposed method.

Zakerzadeh et al. (1999) calculated the lateral earth force on a retaining wall by using the method of slices and limit equilibrium concepts. Important steps in formulating the solution are assuming circular slip surface and selecting appropriate inter-slice force function (i.e., the ratio of the shear force to the normal force of vertical slices along the slip surface). Interslice force functions were used to compute the active and passive earth forces. An example of a problem involving a vertical wall with a horizontal backfill surface was analyzed using the general limit equilibrium (GLE) method and the proposed inter-slice force functions. Lateral earth force and the point of application were determined and compared with classical solutions. For the passive case, reasonable
results were obtained when using an inter-slice force function that remains at zero from the starting point of the slip surface (at some distance from the wall) to the midpoint of the slip surface and then varies linearly from the midpoint of the slip surface to the end point of the slip surface (adjacent to the wall). Based from the fact that the example given is of simple geometry, and that the shape of sliding surface is close to logarithmic, the application of this method for a more complex geometry will need further study.

Zhu and Qian (2000) proposed a procedure for determining passive earth pressure coefficients using triangular slices within the framework of the limit equilibrium method. The potential sliding mass was subdivided into a series of triangular slices, rather than vertical slices in previous methods, with inclination angles of the slice bases to be determined. The forces between two adjacent slices (inter-slice forces) were expressed in terms of inter-slice force coefficients, and recursive equations for solving inter-slice coefficients were derived. By using the principle of optimality, the critical inclination angles of slice bases, minimum inter-slice force coefficients, and passive earth pressure coefficients were determined. A form of function for describing the distribution of inter-slice force inclination (inter-slice force function) was suggested and the scaling parameter contained in the function was determined by satisfying the moment equilibrium condition for the final sliding mass. Comparisons were made with other accepted methods and tables for passive earth pressure coefficients were presented for practical use.

Soubra (2000) investigated passive earth pressure problems by means of the kinematical method of the limit analysis theory. A translational kinematically admissible failure mechanism is composed of a sequence of rigid triangles. This mechanism allows
the calculation of the passive earth pressure coefficients in both the static and seismic cases. Quasi-static representation of earthquake effects using the seismic coefficient concept was adopted. Rigorous upper-bound solutions were obtained in a framework of the limit analysis theory. The numerical results of the static and seismic passive earth pressure coefficients were presented and compared with the results of other methods. From comparison of static case, the results were almost identical to those given by Kerisel and Absi (1990) using a slip line method and those given by Chen and Liu (1990) using the upper-bound method in limit analysis with a log-sandwich mechanism.

Kumar (2001) compared limit equilibrium method by Kumar and Subba Rao (1997) and upper bound limit analysis by Soubra (2000) for static and pseudo static earthquake forces. The result of the comparisons showed that the limit analysis to be either almost the same or marginally greater than the limit equilibrium method.

Lancellotta (2002) proposed analytical solution for earth pressure coefficient by using lower bound theorem of plasticity, which is a conservative estimate of the exact solution. The equation was developed for calculating passive coefficients for vertical wall with friction and with horizontal backfill surface.

Maciejewski and Jarzebowski (2004) applied kinematically admissible mechanisms to passive pressure soil mechanics boundary value problems. The method considered basic relations of material behavior along velocity discontinuity lines and block equilibrium method. The solution for a linear Mohr-Coulomb material was compared with the solution for a nonlinear material. The sensitivity of the soil failure mechanisms to material parameters was discussed. A numerical example based on the
method was presented for oscillatory loads with advanced displacement beyond initial failure.

Shamsabadi et al. (2005) developed formulation for mobilized force-displacement-capacity in seismic design for a bridge abutment-embankment system. The formulation was based on logarithmic spiral surface, method of slices, and stress-strain behavior of the soil. The stress-strain behavior of soil in conjunction with mobilized abutment-soil resistance surface was evaluated to assess the corresponding displacement. The Mohr-Coulomb strength criteria were used to develop shape function for distribution of the inter-slice forces. Therefore, abutment-embankment lateral force and interslice forces and their directions with ever changing (mobilized) soil mass were computed explicitly, without trial and error procedure. The nonlinear force-displacement-capacity prediction was in very good agreement with the results obtained from small- and full-scale experimental static tests in cohesionless and cohesive backfill.

Benmebarek et al. (2006) studied the effect of seepage flow on the passive and active earth pressures on a vertical wall in cohesionless soil using FLAC code. Effective passive earth pressure coefficients in the presence of upward seepage forces were calculated for associative and nonassociative materials. It showed that the dilation angle influenced the effective passive earth pressures for a large angle of internal friction. It was also shown that the dilation angle influenced the effective active earth pressures for a large angle of internal friction. The passive pressures decreased when upward seepage pressure increased. The results were in good agreement with those using an upper-bound approach in limit analysis for an associative material. Investigation on the effect of downward seepage forces on the active earth pressures showed a significant increase in
the effective active earth pressures. Further Benmebarek et al. (2008) studied 3D passive earth pressures for associative soils using FLAC3D. It was shown that passive earth pressures coefficients increased due to the decrease of the wall breadth. The results were compared with other investigations with limit equilibrium method, upper-bound method in limit analysis, as well as experimental measures. Results were presented in a form of design tables relating the geometrical parameters, soil properties, and 3D passive earth pressure coefficients. This FLAC analysis used isotropic materials.

Ming et al. (2007) considered anisotropy of undrained sand on seismic performance of retaining structures subjected to active and passive earth pressures. Analyses were conducted using a set of fully coupled finite-element analyses. The analyses revealed that the impact of fabric anisotropy could be significant when the retaining structure is under passive earth pressure conditions, but the effect was practically inconsequential for retaining wall under active pressure condition.

3.3 Laboratory Research on Passive Pressure

There have been few laboratory investigations on passive earth pressure and even fewer in field experiments. The experiments can be categorized based on the application of wall movement modes as either translation or rotation. Experiments with the translation mode were performed by Rowe and Peaker (1965), Mackey and Kirk (1967), Matsuo et al. (1978), Fang et al. (1997), Kobayashi (1998), Duncan and Mokwa (2001), Fang et al. (2002), and Hanna and Khoury (2005) and others. While experiments with rotation mode of the wall were conducted by Schofield (1961), Narain et al. (1969),
James and Bransby (1970), and Fang et al. (1994) and others. The experimental works are described in the following section.

3.3.1 **Translational Wall Movement Mode**

Rowe and Peaker (1965) used an apparatus for measuring passive earth pressure of dry sand that allowed the control of wall movement direction in space and consequent rate of mobilization of wall friction. It was found that for loose sand a good agreement between theory and observations was obtained after large wall displacements, which are not acceptable in practice. Meanwhile, for dense sand, progressive failure of elements in the backfill led to smaller average maximum Coulomb values of $\varphi'$ than those predicted by plane strain compression test. It was demonstrated that the peak values of angles of internal friction and friction angles between wall and sand should not be necessarily used in theoretical computations of passive earth pressure due to the nature of progressive failure. It was suggested that the correct solutions should utilize stress-strain-dilatancy laws for soils subjected to any stress path.

Mackey and Kirk (1967) studied at-rest, active and passive pressures acting on a rigid steel wall using earth pressure cells. Three different types of sands, each in loose and dense states, were used in the investigation. Pressure distributions and failure surfaces were obtained for various amounts of wall movement. Active pressures measured in dense sand were greater than those obtained in loose sand were; this was completely contrary to the theories. At-rest pressure with the sands in the dense condition approached those of the simple Rankine passive state. It was suggested that a part of kinetic energy from compaction was attributed to develop higher residual lateral stress.
In case of passive pressures, the results were compared well with those obtained by other researchers except by Coulomb.

Matsuo et al. (1978) conducted a field investigation on a 10 m high concrete retaining wall with backfill materials of silty sand and slags. Measurements were taken during translational wall movement from at rest to active state and then to a passive state. After construction of backfill, resultant lateral earth pressure gradually increased, then stabilized to signify at-rest state. Then, using oil jack, the wall was moved to active state with a rapid decrease in pressure and remained at that position for 20 days. During this time, pressure was slightly and gradually recovered. Later the wall was pushed back toward passive state with a large increase in force relative to at-rest state. After one or two days, pressure in oil jack dropped, and it could not maintain such a high passive force in the field conditions. Experiment was stopped without obtaining passive earth pressure as initially planned.

Fang et al. (1997) conducted an experimental investigation on earth pressure acting against a vertical rigid wall, which translated away from or toward dry loose backfill sand with an inclined backfill surface. The facility that was used consisted of four components, namely, the model retaining wall, soil bin, driving system, and data acquisition system. The instrumented retaining wall was used to investigate the variation of earth pressure induced by the translational wall movement. It was found that the earth pressure distributions were essentially linear at each stage of wall movement. Wall movement required for the backfill to reach an active or a passive state increased with an increasing backfill inclination. The experimental active and passive earth pressure coefficients for various backfill sloping angles were in good agreement with the values
calculated by Coulomb's theory. They also cited that Rankine's theoretical equations to compute passive or active earth pressure coefficient with a sloping backfill gave identical results for either soil surface sloping up or down. It was, therefore, concluded that it might not be appropriate to adopt the Rankine theory to determine either passive or active earth pressure against a rigid wall with sloping backfills.

By experiments Kobayashi (1998) tried to verify theoretical predictions for passive earth pressure based on the rigid plasticity theory particularly for the case of a large wall slanted angle and large wall friction angle. The results demonstrated that the observed failure zone was similar to that predicted by the characteristics method based on the rigid plasticity theory. Except for the lowest part of the passive wall, the earth pressure increased linearly with depth. For a small wall friction angle, passive earth pressure coefficients $K_p$ were nearly equal to the theoretical predictions, whereas in the case of a large wall friction angle, the coefficients $K_p$ were smaller than the theoretical values. This difference suggested that the effect of progressive failure as observed in strain distribution and displacement contours plays important role in case of the passive case.

Duncan and Mokwa (2001) performed a passive pressure field test on anchor blocks and compared the result with the proposed method utilizing load deflection behavior, and also with other theories, namely, Rankine, Coulomb, and log-spiral methods. The proposed method considered the amount and direction of structure movement, strength and stiffness of the soil, friction or adhesion between the structure and soil, and the shape of structure. A hyperbolic expression, together with estimated values of soil modulus and ultimate resistance, gave the relationship between structural
movement and passive resistance. Passive load test were conducted in undrained stiff sandy silt and drained well-graded gravel. The comparison between measured and calculation showed that the log spiral theory, corrected for 3D effects, and the hyperbolic load-deflection relationship provided an adequate mean of estimating passive resistance for a wide range of conditions.

Fang et al. (2002), using the same retaining wall facility as described before in this section (Fang et al., 1997), investigated the effects of soil density on the development of passive earth pressure. Three different relative densities, 38, 63, and 80%, were used in the experiment. For dense sand, Coulomb and Terzaghi' log spiral solutions with the peak internal friction angle of the soil were found to significantly overestimate the ultimate passive thrust. As the wall movement exceeded 12% of the wall height, the passive earth thrust reached a constant value, regardless of the initial backfill density. Under such a large wall movement, soils along the rupture surface had reached the critical state, and the shearing strength on the surface could be properly represented with the residual angle of internal-friction. The ultimate passive earth pressure was successfully estimated by adopting the critical state concept to either Terzaghi or Coulomb theory. In the closure, investigators suggested to consider the use of dilation and a displacement-based approach involving both peak and residual strength.

Hanna and Khoury (2005) investigated the effect of over-compaction ratio on the passive earth pressure of cohesionless soil. A model of a vertical rough wall with horizontal backfill was instrumented to measure the total passive earth pressure, the passive earth pressure on selected locations on the wall, and the over-compaction ratio (OCR) of the sand. Over-compacted sand was produced by placing the sand in thin layers
and compacted mechanically for a period of time. The wall was pushed with translational mode toward the backfill. For comparison, the analytical method of slices developed for predicting the passive earth pressure for normally compacted soil was adopted for the conditions stated above. The theoretical values compared well with the experimental results. OCR and the condition of underlying soil layer significantly affect the value of passive earth pressure. Design charts were developed for passive earth pressure for several OCR values.

3.3.2 Rotational Wall Movement Mode

Schofield (1961) conducted laboratory experiments on passive earth pressure using a rotating wall model with only force measurement. A sharp-edged, rough-faced, flat model wall was embedded vertically in a body of homogeneous sand with horizontal ground surface. The result showed that the magnitude of the force increased when the wall movement increased, and the inclination of the force decreased in a certain definite relationship. This relationship was confirmed by theoretical calculations of soil pressure made both by the friction circle method and by the method of characteristics. It was suggested that an additional pressure due to soil dilation would have been developed within a portion of sand sample that was failed.

Narain et al. (1969) conducted earth pressure model experiments with in glass plates on both sides to observe rupture surfaces and distribution of pressures on rigid retaining wall. Emphasis was laid on the effect of rotation of wall on magnitude and distribution of pressure and the shape and the size of rupture wedges. It was concluded that the mode of wall displacement was one of important factors affecting pressures and
rupture wedges. It was found that common earth pressure theories were inadequate to assess passive pressures correctly.

James and Bransby (1970) investigated passive failure by rotating an initially vertical rough wall about its toe into a dry backfill sand with horizontal backfill surface. Normal and shear stresses on the wall were measured, and the strains in the soil mass were determined by X-ray of the position of buried lead shots. The strain data were used to investigate the mobilized $\phi$ constant assumption of the Sokolovski method in the entire section of failed soil mass and the solutions were compared with the experiments. The assumption of mobilized value of $\phi$ constant was satisfied over a large region of deforming mass of dense sand, but not in loose sand. There was an excellent agreement between the Sokolovski prediction of principal compressive stress directions and the observed principal compressive strain increment directions in dense sand, while there was only moderate agreement for loose sand.

Fang et al. (1994) studied earth pressure acting against a vertical rigid wall, which moved into a mass of dry loose sand with a horizontal ground surface under various wall-movement modes. Using the same retaining wall facility of the same investigator as described before, wall movements were rotation about a point above the top (RTT) and rotation about a point below the wall base (RBT) as seen in Fig. 3.1. In RTT mode, parameter “$n$” in the figure is the ratio of the distance from center of rotation to the wall top, and the wall height. In RBT mode, “$n$” is the ratio of the distance from center of rotation to wall bottom, to wall height. It was found that, for a wall under translational movement, the passive pressure distribution is linear and in good agreement with Terzaghi's prediction based on the general wedge theory. For a wall under either RTT or
RBT mode, the magnitude of passive thrust and its point of application were significantly affected by the mode of wall displacement. However, if the parameter “n” is greater than 2.0, the influence of movement mode on passive earth pressure becomes less important and those values become to that of translational move.

Fig. 3.1 Two types of passive wall movement: (a) RTT mode, (b) RBT mode (Fang et al. (1994))
4 NUMERICAL SIMULATION TECHNIQUE

4.1 FLAC (Fast Lagrangian Analysis of Continua)

FLAC is an explicit finite difference program for engineering mechanics computation (Itasca Consulting Group, 2002). The FLAC used in this research is a 2D program with its basic formulation for a plane-strain condition. Soil mass analyzed was represented by elements that form grids. The elements can follow linear or nonlinear stress-strain relationship caused by external loads or boundary restraints. The technique used in this program enables a large strain element deformation. Since using finite difference technique, there is no large matrix developed during calculation, and therefore, the program is able to accommodate calculation of large 2D grid without too much memory requirement.

There are ten built-in material's constitutive models in FLAC namely: (1) null model, (2) isotropic elastic model, (3) elastic transversely isotropic model, (4) Drucker-Prager model, (5) Mohr-Coulomb model, (6) ubiquitous-joint model, (7) strain-hardening/softening model, (8) bilinear strain-hardening/softening ubiquitous-joint model, (9) double-yield model, and (10) modified Cam-clay model. FLAC has a built-in programming language FISH that allows users to write their own functions or even create other constitutive models. FLAC also has an interface model to represent distinct interface between elements. The interface is used in analyzing slip and separation between planes.

Other features of FLAC will be discussed in the following section along with discussion of model input and verification of experimental test.
4.2 Plane Strain Model Input

From laboratory experiment on plane-strain compression tests by Tatsuoka et al. (1986), it was found that sands show strong strength anisotropy; that is, the angle of internal friction $\phi$ reached its minimum value at a certain range of angle $\delta$ which is defined as the angle between bedding plane and the major principal stress direction. This finding could affect the result of an analysis on varieties of soil problems. For example, Fig. 4.1 describes changing of $\delta$ angle along a sliding plane of failure under a footing foundation. The same condition may also apply to problems in analyzing stress around moving wall as in the case of passive. Therefore as input in FLAC program this strength anisotropic should be incorporated.

![Diagram](image)

**Fig. 4.1** Principal stress directions at failure of soil under a foundation footing

In order to obtain stress-strain relations as input to the program, Tatsuoka et al. (1986)'s experimental data as shown in Fig. 4.2, Fig. 4.3, Fig. 4.4, and Fig. 4.5 were used
as a basic reference. The four figures present test results with low and high confining pressure applied to sands for loose and dense conditions. Plane strain compression experiments were performed with different values of $\delta$ from zero to 90 degrees. To incorporate common characteristics of stress-strain relations shown in those figures, a simplified stress-strain relationship was modeled in Fig. 4.6.
Fig. 4.2 Typical stress-strain relations for tests at $\sigma'_3 = 0.05$ kgf/cm$^2$ for dense samples, after Tatsuoka et al. (1986)
Fig. 4.3 Typical stress-strain relations for tests at $\sigma'_3=0.05\text{ kgf/cm}^2$ for loose samples, after Tatsuoka et al. (1986)
Fig. 4.4 Typical stress-strain relations for tests at $\sigma'_3 = 4.0$ kgf/cm$^2$ for dense samples, after Tatsuoka et al. (1986)
Fig. 4.5 Typical stress-strain relations for tests at $\sigma_3' = 4.0$ kgf/cm² for loose samples, after Tatsuoka et al. (1986)
Stress development during the experiment is presented as $\varphi$, which is defined in Eq. (4.1). Stress-strain relation, in terms of $\varphi$ vs. $\varepsilon_1$ curve, as described in Fig. 4.6 shows an increasing plane-strain $\varphi$ values from $(0, 0)$ to a maximum value at point $(\varepsilon_{1\text{peak}}, \varphi_{\text{peak}})$. After reaching the maximum, the $\varphi$ value decreases to a residual point $(\varepsilon_{1\text{res}}, \varphi_{\text{res}})$. Beyond the residual point the curve then levels off. The extent of the differences between peak and residual values depend on soil relative density, confining pressure, and the orientation of principal stress. Based on the shape of stress-strain relation, the model that is used in this research is a strain-hardening/softening model.

$$\varphi = \sin^{-1}\left(\frac{\sigma_1-\sigma_2}{\sigma_1+\sigma_2}\right) \quad (4.1)$$

Fig. 4.6 Angle of internal friction $\varphi$ vs. principal strain $\varepsilon_1$ for particular relative density, confining pressure, and principal stress direction
Based on the data in Fig. 4.2, Fig. 4.3, Fig. 4.4, and Fig. 4.5, Tatsuoka et al. (1990) normalized the angle of internal friction $\phi$ values by the $\phi$ at $\alpha_f = 0$ for various conditions of confining pressure and relative density as function of $\alpha_f$ value in Fig. 4.7, where $\alpha_f$ value is defined as $90-\delta$. The normalized ratio is then designated as $R(\delta) = \phi(\delta)/\phi(\delta = 90^\circ)$. Using known values of $e_{\text{max}} = 0.99$ and $e_{\text{min}} = 0.63$ of the same Toyoura sand as reported by Oda (1981), the values of relative density $D_r$ in Fig. 4.7 were calculated. For $e = 0.7$ and $e = 0.8$, those $D_r$ values were 80.56% and 52.78%, respectively. The average curves of these ratios for PSC (Plain Strain Compression) test data were redrawn in Fig. 4.8. The circular data points in Fig. 4.8 indicate the minimum points of the curves for all relative density data. To obtain the ratio for other relative densities, interpolation between those known curves was performed. Equations for curves in Fig. 4.8 are presented in Table 4.1. Curve 1 is from $\delta = 0^\circ$ to the minimum values of the ratio, and curve 2 is from the minimum value of the ratio to $\delta = 90^\circ$.

**Fig. 4.7** $R = \phi(\alpha_f)/\phi(\alpha_f = 0^\circ, \text{PSC})$ versus $\alpha_f = 90^\circ - \delta$ of Toyoura sand in Tatsuoka et al. (1990)
Fig. 4.8 $R(\delta) = \phi(\delta)/\phi(\delta=90^\circ)$ versus $\delta$ for different relative density.
Table 4.1 Equations associated with curves on Fig. 4.8

<table>
<thead>
<tr>
<th>Dr</th>
<th>Curve</th>
<th>C</th>
<th>a1</th>
<th>a2</th>
<th>a3</th>
</tr>
</thead>
<tbody>
<tr>
<td>38.00%</td>
<td>1</td>
<td>0.90600</td>
<td>-0.0004116</td>
<td>2.75237E-05</td>
<td>-7.96501E-07</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.57738</td>
<td>0.0113531</td>
<td>-9.86773E-05</td>
<td>2.76431E-07</td>
</tr>
<tr>
<td>52.78%</td>
<td>1</td>
<td>0.89994</td>
<td>-0.0001663</td>
<td>-1.84038E-05</td>
<td>-2.92135E-07</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.60664</td>
<td>0.0101827</td>
<td>-8.33269E-05</td>
<td>2.08477E-07</td>
</tr>
<tr>
<td>60.00%</td>
<td>1</td>
<td>0.89706</td>
<td>-0.0001907</td>
<td>-2.52990E-05</td>
<td>-4.33252E-07</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.62638</td>
<td>0.0093072</td>
<td>-7.06343E-05</td>
<td>1.48363E-07</td>
</tr>
<tr>
<td>63.00%</td>
<td>1</td>
<td>0.89584</td>
<td>-0.0001605</td>
<td>-3.25021E-05</td>
<td>-3.83718E-07</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.63457</td>
<td>0.0089435</td>
<td>-6.35603E-05</td>
<td>1.23385E-07</td>
</tr>
<tr>
<td>70.00%</td>
<td>1</td>
<td>0.89299</td>
<td>-0.0000902</td>
<td>-4.93093E-05</td>
<td>-2.68138E-07</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.65370</td>
<td>0.0080947</td>
<td>-5.30545E-05</td>
<td>6.51036E-08</td>
</tr>
<tr>
<td>80.00%</td>
<td>1</td>
<td>0.88893</td>
<td>0.0000102</td>
<td>-7.33196E-05</td>
<td>-1.03025E-07</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.68103</td>
<td>0.0068822</td>
<td>-3.54747E-05</td>
<td>-1.81561E-08</td>
</tr>
<tr>
<td>80.56%</td>
<td>1</td>
<td>0.88870</td>
<td>0.0000158</td>
<td>-7.46641E-05</td>
<td>-9.37784E-08</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.66991</td>
<td>0.0074706</td>
<td>-4.52399E-05</td>
<td>3.31965E-08</td>
</tr>
</tbody>
</table>

*Curve 1 is for $\delta = 0$ to the minimum point in Fig. 4.8
Curve 2 is for $\delta$ at the minimum to $90^\circ$

From Fig. 4.8 the minimum values of the ratio $R(\delta) = \phi(\delta)/\phi(\delta = 90^\circ)$ with their associated Dr values were plotted in Fig. 4.9. This value is inputted as minrat in the program. When the orientation of the major principal stress coincides with the $\delta$ angle to give the minimum ratio, the angle of internal friction at failure is actually the same as $\phi$ value from direct shear test ($\phi_{DS}$). Since the failure direction in the direct shear device is horizontal, which coincides with its bedding direction of soils, it provides the lowest shear resistance and the direct shear test is considered as a plane strain test. By knowing values from direct shear test and minrat from Fig. 4.9, the values of plane strain $\phi$ for $\delta =$
90° can be calculated. This technique will be used in modeling Fang et. al’s experiments since they reported only direct-shear test φ values.

From Fig. 4.8 the value of angle δ at minrat is plotted in Fig. 4.10 with function of Dr and those δ values are defined as datmin. Thus for a known relative density, datmin is obtained. It is the boundary point between left curve and right curve of Fig. 4.8 or between curve 1 and curve 2 in Table 4.1.

Based on the data from Fig. 4.2, Fig. 4.3, Fig. 4.4, and Fig. 4.5, all φpeak values were normalized by φpeak value of δ = 90° of a high confining pressure (σ₃ = 4.0 kg/cm²). Fig. 4.11 shows normalized data points at σ₃ = 0.05 kg/cm² and σ₃ = 4.0 kg/cm² for each Dr data. The two data points are connected by assuming a logarithmic curve. Similar curves were used to obtain other relative density values by interpolation. These curves are
used to correct \( \phi \) values of low confining pressure for a small-scale laboratory model test in comparison with high confining pressure for prototype walls.

A similar procedure was performed to obtain corrections for a residual angle of internal friction, \( \varphi_{\text{res}} \), of low confining pressure in the model to high confining pressure in prototype walls. Averaging on \( \varphi_{\text{res}} \) was made for all \( \delta \) values for the same confining pressure, since \( \varphi_{\text{res}} \) values converge to a certain value at large strain levels regardless of its original density for a given confining pressure. This correction is presented in Fig. 4.12.

\[
\delta_{\text{at Minimum}(\varphi_{\text{peak}}(\delta) / \varphi_{\text{peak}}(\delta=90^\circ))} (\text{degree})
\]

\[
datmin = 0.000030133D^3 - 0.003303401D^2 - 0.231960511D + 50.674121885
\]

**Fig. 4.10** \( \delta \) value at Minimum(\( \varphi_{\text{peak}}@\delta / \varphi_{\text{peak}}(\delta=90^\circ) \)) vs. Dr
Fig. 4.11 Effect of low $\sigma_3$ on $\varphi_{\text{peak}}$ for $\delta=90^\circ$

Fig. 4.12 Effect of low $\sigma_3$ on $\varphi_{\text{res}}$ for any $\delta$
Fig. 4.13 presents relationships between relative density $D_r$ and $\varepsilon_{\text{peak}}$ as well as $\varepsilon_{\text{res}}$ as defined in Fig. 4.6 for $\delta = 90^\circ$. The two equations in Fig. 4.13 were developed based on plane strain data from Tatsuoka et al. (1986), and Alshibli and Sture (2000). Since $\varepsilon_{\text{peak}}$ data of $\sigma_3 = 4$ kg/cm$^2$ were not closely scattered with other data to form a straight line, and also since this confining stress was too high for typical wall high, therefore, data of $\sigma_3 = 4$ kg/cm$^2$ were excluded in forming the equation for $\varepsilon_{\text{peak}}$. For the directions of other principal stresses than $\delta = 90^\circ$, data from Tatsuoka et al. (1986) of peak and residual strains were normalized to those of $\delta = 90^\circ$ values in Table 4.2 and Table 4.3, respectively. For other values of $D_r$, $\sigma_3$, and $\delta$ than those shown in the tables, linear interpolations were made. FLAC provides a simple linear interpolation procedure by using function “table” which is operated within FISH function. In the last row in these tables “tab” numbers are used in the input program of FLAC.

![Graph showing relationships between $D_r$ and $\varepsilon_{\text{peak}}, \varepsilon_{\text{res}}$]

$\varepsilon_{\text{peak}} = -0.015207D_r + 3.403184$

$\varepsilon_{\text{res}} = -0.081055D_r + 11.346580$

- $\Delta$ Toyoura, 0.05 kg/cm$^2$, $\varepsilon_{\text{peak}}$
- $\circ$ Ottawa, 0.15 kg/cm$^2$, $\varepsilon_{\text{peak}}$
- $\bigtriangleup$ Ottawa, 1.02 kg/cm$^2$, $\varepsilon_{\text{peak}}$
- $\Box$ Toyoura, 4 kg/cm$^2$, $\varepsilon_{\text{peak}}$
- $\Delta$ Toyoura, 0.05 kg/cm$^2$, $\varepsilon_{\text{res}}$
- $\circ$ Ottawa, 0.15 kg/cm$^2$, $\varepsilon_{\text{res}}$
- $\bigtriangleup$ Ottawa, 1.02 kg/cm$^2$, $\varepsilon_{\text{res}}$
- $\Box$ Toyoura, 4 kg/cm$^2$, $\varepsilon_{\text{res}}$

Fig. 4.13 Effect of $D_r$ on $\varepsilon_{\text{peak}}$ and $\varepsilon_{\text{res}}$ for $\delta = 90^\circ$ data read from Alshibli and Sture (2000), Tatsuoka et al. (1986)
### Table 4.2 Ratio $\varepsilon_{\text{peak}}(\delta)/\varepsilon_{\text{peak}}(\delta=90^\circ)$ for low and high $\sigma_s$ and dense and loose condition

<table>
<thead>
<tr>
<th>$\delta$</th>
<th>$\varepsilon_{\text{peak}}(\delta)/\varepsilon_{\text{peak}}(\delta=90^\circ)$</th>
<th>$\delta$</th>
<th>$\varepsilon_{\text{peak}}(\delta)/\varepsilon_{\text{peak}}(\delta=90^\circ)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.647059</td>
<td>0</td>
<td>2.111111</td>
</tr>
<tr>
<td>23</td>
<td>1.441176</td>
<td>23</td>
<td>1.355556</td>
</tr>
<tr>
<td>34</td>
<td>0.882353</td>
<td>34</td>
<td>1.511111</td>
</tr>
<tr>
<td>45</td>
<td>1.058824</td>
<td>45</td>
<td>1.410714</td>
</tr>
<tr>
<td>90</td>
<td>1.000000</td>
<td>90</td>
<td>1.000000</td>
</tr>
</tbody>
</table>

**low $\sigma_s = 4903.325$ Pa**

**high $\sigma_s = 392266$ Pa**

### Table 4.3 Ratio $\varepsilon_{\text{res}}(\delta)/\varepsilon_{\text{res}}(\delta=90^\circ)$ for low and high $\sigma_s$ and dense and loose condition

<table>
<thead>
<tr>
<th>$\delta$</th>
<th>$\varepsilon_{\text{res}}(\delta)/\varepsilon_{\text{res}}(\delta=90^\circ)$</th>
<th>$\delta$</th>
<th>$\varepsilon_{\text{res}}(\delta)/\varepsilon_{\text{res}}(\delta=90^\circ)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.295082</td>
<td>0</td>
<td>1.380952</td>
</tr>
<tr>
<td>23</td>
<td>1.065574</td>
<td>23</td>
<td>0.928571</td>
</tr>
<tr>
<td>34</td>
<td>0.918033</td>
<td>34</td>
<td>1.119048</td>
</tr>
<tr>
<td>45</td>
<td>1.131148</td>
<td>45</td>
<td>1.238095</td>
</tr>
<tr>
<td>90</td>
<td>1.000000</td>
<td>90</td>
<td>1.000000</td>
</tr>
</tbody>
</table>

**low $\sigma_s = 4903.325$ Pa**

**high $\sigma_s = 392266$ Pa**
Table 4.4 Normalized plane strain angle of internal friction with normalized principal strain.

<table>
<thead>
<tr>
<th>Curvature from 0 to peak stress</th>
<th>Curvature from peak to residual stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon_1/\varepsilon_{1\text{peak}}$</td>
<td>$\varphi/\varphi_{\text{peak}}$</td>
</tr>
<tr>
<td>0.00</td>
<td>0.00000</td>
</tr>
<tr>
<td>0.05</td>
<td>0.43300</td>
</tr>
<tr>
<td>0.10</td>
<td>0.59900</td>
</tr>
<tr>
<td>0.20</td>
<td>0.74000</td>
</tr>
<tr>
<td>0.30</td>
<td>0.81800</td>
</tr>
<tr>
<td>0.40</td>
<td>0.87080</td>
</tr>
<tr>
<td>0.50</td>
<td>0.91450</td>
</tr>
<tr>
<td>0.60</td>
<td>0.94260</td>
</tr>
<tr>
<td>0.70</td>
<td>0.96330</td>
</tr>
<tr>
<td>0.80</td>
<td>0.98000</td>
</tr>
<tr>
<td>0.90</td>
<td>0.99310</td>
</tr>
<tr>
<td>1.00</td>
<td>1.00000</td>
</tr>
</tbody>
</table>

In Fig. 4.14 and Fig. 4.15, the generated normalized curves are drawn with data points by Tatsuoka et al. (1986) for zero to peak stress, and for peak to residual stress, respectively. Coordinates of the average points of normalized angle of internal friction and the strain on those curves are shown in Table 4.4.

After obtaining values of $\varphi_{\text{peak}}$, $\varphi_{\text{res}}$, $\varepsilon_{1\text{peak}}$, and $\varepsilon_{1\text{res}}$ as defined in Fig. 4.6, complete coordinates of stress strain relations were then calculated by using normalized values from Table 4.4.
Fig. 4.14 $\varphi/\varphi_{\text{peak}}$ vs. $\varepsilon_1/\varepsilon_{1\text{peak}}$ (0 to peak)
Fig. 4.15 \((\phi - \phi_{res})/(\phi_{peak} - \phi_{res})\) vs. \((\epsilon_1 - \epsilon_{1peak})/(\epsilon_{1res} - \epsilon_{1peak})\), (peak to residual)
The last variable needed in FLAC is dilation angle (= \( \psi \)). Dilation angle accounts for shear dilatancy, which is the change in volume that occurs with shear distortion. This angle is related to the ratio of plastic volume change to plastic shear strain. Based on volumetric strain in Fig. 4.2, Fig. 4.3, Fig. 4.4, and Fig. 4.5 the slopes of volumetric strain curve (\( \alpha \)) were determined. Then using Eq. (4.2) by Vermeer and de Borst (1984), the dilation angles \( \psi \) were obtained.

\[
\tan \alpha = \left( \frac{2 \sin \psi}{1 - \sin \psi} \right)
\]  

(4.2)

Results of calculation of dilation angles are shown in Fig. 4.16, Fig. 4.17, Fig. 4.18, and Fig. 4.19 for different \( \sigma_3 \) values and soil densities.
Fig. 4.17 Dilation angle $\psi$ for $\sigma_3 = 0.05$ kg/cm$^2$, Loose
Fig. 4.18 Dilation angle $\psi$ for $\sigma_3 = 4.0 \text{ kg/cm}^2$, Dense
Based on those data for dilation angle measurement, the variation of the dilation angle ($\psi$) was modeled by three straight lines as shown in Fig. 4.20. The three lines were defined by three coordinates namely ($dilx_2$, 0), ($dilx_2$+0.02, $dila$), and ($dilx_2$+0.06, $dilres$) as seen in the figure. Variable $dila$ is a function of relative density $Dr$, while $dilres$ is a function of confining pressure $\sigma_3$. Variable $dilx_2$ is a function of both $Dr$ and $\sigma_3$. Table 4.5 provides values of these variables. Interpolation is used for values of $\sigma_3$ and $Dr$ that are not in the table. Value of $dilyl$ in Fig. 4.20 is $y$-intercept of the first straight line and automatically determined from $dilx_2$ and $dila$. 

**Fig. 4.19** Dilation angle $\psi$ for $\sigma_3$ = 4.0 kg/cm$^2$, Loose
Fig. 4.20 Variation of dilation angle ($\psi$) to principal strain

Table 4.5 Values of $dila$, $dilres$, and $dilx2$

<table>
<thead>
<tr>
<th>$Dr$ (%)</th>
<th>$dila$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>79.42</td>
<td>14</td>
</tr>
<tr>
<td>54.91</td>
<td>7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$\sigma_3$ (Pa)</th>
<th>$dilres$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4903.3</td>
<td>4</td>
</tr>
<tr>
<td>392266</td>
<td>1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$dilx2$ (in/in)</th>
<th>$Dr$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_3$ (Pa)</td>
<td></td>
</tr>
<tr>
<td>79.42</td>
<td>54.91</td>
</tr>
<tr>
<td>4903.3</td>
<td>0.006</td>
</tr>
<tr>
<td>392266</td>
<td>0.020</td>
</tr>
</tbody>
</table>
4.3 Soil’s Angle of Internal Friction and Soil-Wall Interface Parameters

To use computer code FLAC in investigating passive earth pressure problems, the code is first verified by simulating the model retaining wall experiments that were done by Fang et al. (1994) and Fang et al. (2002). In this section input of soil characteristic for the FLAC simulation is described. Fang and others provided direct shear tests (\( \phi_{DS} \)) data for the soil they utilized as shown in Fig. 4.21. This \( \phi_{DS} \) is used as the minimum \( \phi \) value (minrat) in the simulation.

A computation task was developed to obtain the direction of principal stress \( \delta \) and confining pressure \( \sigma_3 \) for every element in the model. This function can be called on regularly after a certain number of iterations/steps of the computation process. With new \( \delta \) and \( \sigma_3 \) user can update other variables affected by those values such as the angle of internal friction and the dilation angle.

![Graph of Dr vs \( \phi_{DS} \)](image)

**Fig. 4.21** Effect of Dr on direct shear angle of internal friction of Ottawa sand, \( \phi_{DS} \); data from Fang et al. (2002)
The followings equations are used to determine $K_o$, ratio of horizontal stress to vertical stress in at-rest condition. $v$ is Poisson ratio, $G$ is shear modulus, and $K$ is bulk modulus. Jaky’s equation (4.3) is used to compute $K_o$ value. $\varphi'$ in Eq. (4.3) is the one from triaxial experiment and Lade and Lee’s equations (Eq.(4.4)), in Holtz and Kovacs (1981), are used to convert the angle $\varphi_{ps}$ in plane strain to $\varphi_{tx}$ in triaxial test and vice versa.

\[
K_o = 1 - \sin \varphi'_{tx} \tag{4.3}
\]

\[
\varphi_{ps} = 1.5 \varphi_{tx} - 17^\circ \quad (\varphi_{tx} > 34^\circ)
\]
\[
\varphi_{ps} = \varphi_{tx} \quad (\varphi_{tx} \leq 34^\circ) \tag{4.4}
\]

Combining Eq.(4.3) and Eq. (4.5) and by the elastic theory, value of Poisson ration can be obtain in Eq. (4.6).

\[
K_o = \frac{v}{1 - v} \tag{4.5}
\]

\[
v = \frac{1 - \sin \varphi'_{tx}}{2 - \sin \varphi'_{tx}} \tag{4.6}
\]

Stresses caused by gravitation are calculated before any wall movement relative to the backfill soil occurs. At this condition, the soil is at rest. Shear modulus at that stage
uses the maximum shear modulus obtained by Eq. (4.7) by Hardin (1978) with modified unit in Kulhawy and Mayne (1990).

\[ G_{\text{max}}/p_a = \frac{S \cdot OCR^M (\bar{\sigma}_o/p_a)^{0.5}}{2(1 + \nu)(0.3 + 0.7e^2)} \]  

(4.7)

Where:

- \( S = \) coefficient between 1200 and 1500. Average value of 1350 is used.
- \( p_a = \) atmospheric pressure.
- \( OCR^M = \) taken = 1.
- \( \bar{\sigma}_o = \) average principal stress, calculated by Eq. (4.8).

\[ \bar{\sigma}_o = \gamma \cdot z (1 + 2K_o)/3 \]  

(4.8)

Using elastic theory relationships in Eq. (4.9) and Eq. (4.10), bulk modulus is obtained in Eq. (4.11). With known specific gravity \( G_s, e_{\text{max}}, \) and \( e_{\text{min}} \), the void ratio, \( e \) in Eq. (4.12) and thus the soil density \( \gamma \) is obtained by Eq.(4.13).

\[ K = \frac{E}{3(1 - 2\nu)} \]  

(4.9)

\[ G = \frac{E}{2(1 + \nu)} \]  

(4.10)

\[ K = \frac{2G(1 + \nu)}{3(1 - 2\nu)} \]  

(4.11)

\[ e = e_{\text{max}} - Dr(e_{\text{max}} - e_{\text{min}}) \]  

(4.12)

\[ \gamma = \frac{G_s \gamma_w}{1 + e} \]  

(4.13)
To facilitate interaction between the moving wall and adjacent soil, an interface function was utilized. Interface represents discontinuity or contact planes. It requires shear and normal stiffness of contact planes, $k_s$ and $k_n$, and friction angle between the wall and soils. According to Itasca Consulting Group (2002), the values of $k_s$ and $k_n$ should be set to ten times the equivalent stiffness of the stiffest neighboring zone as given by Eq. (4.14), where $\Delta z_{\text{min}}$ is the smallest width of an adjoining zone in the normal direction to the interface of wall and soils.

$$k_s \text{ or } k_n = 10 \times \max \left[ \frac{K + \frac{4}{3}G}{\Delta z_{\text{min}}} \right]$$

(4.14)

In the model there are two types of friction occur; i.e. between soil and vertical wall, and between soil and soil base. Friction between wall and soil used in the model was affected by relative density as shown in Fig. 4.22 according to Fang et al. (2002). The vertical axis in the figure is the ratio of wall friction angle to angle of internal friction of the soil from direct shear test.

For the soil base, the experimental model used safety walk, an antislip frictional material to provide adequate friction. Therefore, the value used for the base friction in FLAC simulation is equal to the full angle of internal friction of soil from direct shear test.
Fig. 4.22 Effect of Dr on ratio of steel wall-sand friction of Ottawa sand (Fang et al., 2002)

\[ y = -0.000025502D_r^2 + 0.003856170D_r + 0.187259946 \]
5 EFFECT OF VARIOUS PARAMETERS AND VERIFICATION OF MODEL WALL TESTS by FLAC SIMULATION

First, a numerical code FLAC was used to evaluate the effect of various simulation parameters such as mesh configuration, dilation angle, and anisotropic characteristics of soil. Earlier Benmebarek et al. (2006)'s work was evaluated to compare the results with this simulation.

Then FLAC has been utilized to simulate a laboratory retaining wall experiments at National Chiao Tung University. The experiments were conducted by Fang et al. (1994), Fang et al. (1997), and Fang et al. (2002) extensively for passive earth pressure investigations. The simulated sandbox and soil element mesh is shown in Fig. 5.1. The depth of sand is 0.613m and the length is 2.0 m. The left wall is moveable up to a depth of 0.5 m from sand surface. The soil is divided into a mesh of 22 x 72 elements for finite difference application.
In Fang’s experiments, the moveable wall was instrumented with transducers, which measured horizontal earth pressure. To simulate the horizontal pressure reading, the program records the normal stress in x-direction ($\sigma_x$) of every element adjacent to the left wall in the computer model. Horizontal passive force ($P_{px}$) is determined by integrating the stress of all elements adjacent to the moveable wall and the coefficient of horizontal passive pressure ($K_{px}$) is obtained by Eq. (5.1).

$$K_{px} = \frac{P_{px}}{\frac{1}{2} \gamma h^2}$$

Fig. 5.1 Model of sandbox and soil elements used in FLAC
With a fully mobilized wall friction angle \( \delta \), Eq. (5.2) and (5.3) are used to obtain passive earth force \( (P_p) \) and passive earth pressure coefficient \( (K_p) \), respectively, from computed horizontal stress \( P_{px} \).

\[
P_p = \frac{P_{px}}{\cos \delta}
\]

\[
K_p = \frac{P_p}{\frac{1}{2} \gamma h^2}
\]

### 5.1 Effect of Mesh Configuration, Dilation Angle, and Isotropy versus Anisotropy on Passive Pressure

#### 5.1.1 Effect of Mesh Configuration and Dilation Angle on Isotropic Materials

In order to investigate the effect of dimensional ratio of the sand box and the soil mesh used in the current research, comparisons were performed with the work by Benmebarek et al. (2006). Benmebarek et al. conducted an investigation using FLAC on passive and active pressure in the presence of groundwater flow with a translational rigid wall. Soil mesh selected in their model is shown in Fig. 5.2. Configuration of wall and soil shows higher ratios of soil depth to wall height, and of soil width to the wall height, in comparison with the soil mesh utilized in the current investigation as seen in Fig. 5.1. Simulation programs were performed for the same isotropic soil parameters as those used in Benmebarek et al. for a particular case where there was no presence of seepage flow.
In Benmebarek et al.'s investigation, sand is assumed as an elastic-perfectly plastic, non-associative Mohr-Coulomb model with elastic bulk modulus $K = 60$ MPa and shear modulus $G = 22.5$ MPa. This analysis used combinations of four values of the angle of internal friction ($\varphi = 20^\circ, 30^\circ, 35^\circ, 40^\circ$), three values of friction angle at the soil/wall interface ($\delta/\varphi = 0, 1/3, 2/3$) and three values of dilation angle ($\psi/\varphi = 0, 1/2, 2/3$ or 1). The results of calculations are compared and presented in the following figures.

Fig. 5.3, Fig. 5.4, and Fig. 5.5 show the effects of increasing values of dilation angle and angle of internal friction for a given wall friction angle, on the coefficient of passive earth pressure. The current computation results and Benmebarek et al.'s show a close values of $K_P$ except for the case of $\varphi = 40^\circ$ with $\psi/\varphi = 0$ in Fig. 5.3 and Fig. 5.4 where the current results are higher than those of Benmebarek et al.'s. These three figures also show that for $\varphi = 20^\circ, 30^\circ, \text{and} 35^\circ$, increasing dilation angle $\psi$ has very little effects on $K_P$ values. However, for $\varphi = 40^\circ$ $K_P$ increases as the dilation angle increases from 0 to $2/3$ of $\varphi$. The values of dilation angle had more effect on dense sand, but a little effect on
loose sand. Small differences observed between Benmebarek et al. and this simulation might be associated with differences in mesh configurations as shown in Fig. 5.1 and Fig. 5.2.

**Fig. 5.3** Isotropic solution of $K_p$ with $\delta/\varphi=0$ by current solution and Benmebarek et al. (2006)
Fig. 5.4 Isotropic solution of $K_p$ with $\delta/\varphi=1/3$ by current solution and Benmebarek et al. (2006)
Fig. 5.5 Isotropic solution of $K_p$ with $\delta/\varphi = 2/3$ by current solution and Benmebarek et al. (2006)

5.1.2 Effect of Dilation Angle in Isotropic Soils

Fig. 5.6 was developed from combining data from the simulations of isotropic sand of different wall friction angle and dilation angle. The figure shows that an increase in wall friction angle has more effect on the values of $K_p$ than an increase in dilation angle.
Fig. 5.6 Isotropic solution of $K_p$ for $\delta/\varphi = 0, 1/3, 2/3$ and $\psi/\varphi = 0, 1/2, 1$
The dilation angle in the current solution for anisotropic condition follows a more rigorous method as described in Fig. 4.20. Based on the results shown in the investigation of isotropic conditions and comparison with Benmebarek et al.'s results, the effect of dilation angle is not as sensitive as other parameters in the case of low to medium angle of internal friction (20° to 35°). However, a dilation angle that is below half of the angle of internal friction has some effect in the case of high angle of internal friction (40°). In order to select a suitable simple value of the angle of internal friction, Fig. 5.7 was drawn to compare the $K_p$ which resulted from anisotropic simulations with the rigorous dilation angle and from a fixed dilation angle of half the angle of internal friction. The comparison shows relatively small differences on $K_p$ values for both dilation angles used. Thus, it is concluded that a variation of dilation angles with anisotropic materials is not important in the final results of $K_p$ computations.

In summary, the dilation angle has little effect on the coefficient of passive earth pressure particularly for backfill sand with a low angle of internal friction. For the high angle of internal friction and high wall friction, the dilation angle has higher effect. Varying the dilation angle in the current model could be substituted with a fixed value of half angle of internal friction of soil. However, for all subsequent anisotropic simulations the varying dilation angle was used.
Fig. 5.7 Anisotropic solution of $K_p$ with fixed $\psi/\varphi = \frac{1}{2}$ and varying $\psi$, for $\delta/\varphi = 0, \frac{1}{3}, \frac{2}{3}$
5.1.3 Effect of Anisotropy

The next step is to investigate the effect of isotropic and anisotropic conditions on $K_p$ caused by translational wall movement. Calculation in isotropic condition was performed with an average ratio of wall friction to plane strain angle of internal friction of about 0.28, and with ratio of dilation angle to plane strain angle of internal friction of 0.5. Calculation in anisotropic condition was performed using the procedures as in section 4.2 with a given peak value of plane strain angle of internal friction. Comparison of the two conditions is presented in Fig. 5.8. In the figure, data from Fang et al. (2002)’s experiment, Coulomb’s estimation, and anisotropic simulations for $H=0.5\text{m}$ and $H=4.0\text{m}$ were plotted. Before plotting, data $\varphi_{DS}$ from Fang et al.’s result were converted first to $\varphi_{PS}$, and $K_p$ values were obtained from $K_{px}$ values and wall friction angles. The figure shows that anisotropic conditions give lower $K_p$ values compared to isotropic conditions. The differences between the two conditions are higher as the angle of internal friction increases. Coulomb’s prediction appeared to be closer to isotropic simulations than to anisotropic simulations. Meanwhile Fang et al.’s results were closer to the results from anisotropic simulations, in particular to the one with $H=4.0\text{m}$. 
5.2 Simulation of Model Retaining Wall Experiments

In every run of the program, two stages are performed. The first stage is the application of gravitational force, which the backfill soil is subjected to by its own
weight. In this stage horizontal and vertical stresses are uniformly distributed in the entire elements at the same depth. A typical result for horizontal stress due to the soil’s weight is shown in Fig. 5.9, and for vertical stress is shown in Fig. 5.10. All stress units resulted from the program is in N/m² (=Pa). The sign convention used for stress is positive for tension and negative for compression.

Fig. 5.9 Typical horizontal stress ($\sigma_x$) by soil gravity
The second stage is to move the wall to cause passive condition in the backfill soil. As the wall moves, the displacement and the horizontal stresses developed in the elements next to the wall are recorded. Similar to the experimental investigation, three movement modes were simulated, namely Translation, Rotation about a point below the wall base (RBT), and Rotation about a point above the top (RTT).

5.2.1 Translation Mode

After initial gravitational force application, the wall was moved horizontally until the ratio of horizontal displacement to the wall height reaches 20%, or until the program stops when some elements undergo severe deformation, whichever comes first. At the
end of the wall movement, pictures are drawn depicting the latest condition of elements such as those shown in the following figures.

Fig. 5.11 Translation mode: contour of accumulated plastic shear strain

Fig. 5.11 shows an example of the end condition of accumulated plastic shear strain for sand with 80% relative density. The translation movement was stopped after 388132 steps of calculation for each element. The picture clearly shows a sliding zone marked by high shear strain. The higher values occurred around the bottom of the moving wall and lower values occurred at farther elements. This transition is expected since elements close to the wall are the first to deform. The high values of shear strain of
elements around the corner of the wall are caused by the effect of a sharp corner in the geometry.

Fig. 5.12 shows a part of the sandbox near the wall to focus on the elements near the wall. The vectors in the figure indicate the directions of the major principal stresses and their relative values. Directions of the major principal stress on the middle to upper elements near the wall are more or less horizontal, while for those of elements below the moving wall appears to be sloping downward. This picture also shows the shape of shear deformation of elements around the sliding surface.
Fig. 5.13 Translation mode: elements indicated as yield in shear

At the end of the program run, some elements are in yielding conditions as indicated in Fig. 5.13. All elements along the failure line are in yield condition and so are several elements inside the failed zone. The shape of the failure line is similar to that of Terzaghi's log-spiral failure line. One element that is located right below the wall has undergone failure in tension, which could have caused the termination of the program before the targeted wall movement was reached. Since the simulation was based on plane strain compression data, the occurrence of the element that failed in tension could be considered a drawback of the simulation. However, this tension failure does not affect the final calculation of $K_{px}$ since the location is below the wall.
Fig. 5.14 Stress strain relation ($\varphi$ vs. $\varepsilon_1$) of elements adjacent to moving wall

During the movement of the wall each element is governed by its own stress strain relation that is updated as the principal stress direction of the element and the confining pressure change. Information of coordinates of points in the stress strain relation is stored in a function called “Table.” Fig. 5.14 shows some of the stress strain relations of the elements with the moving wall up to the termination of the program.
Fig. 5.15 Comparison of horizontal stress from Fang’s experiment and FLAC simulation at different stages of wall translation for dense soil of $D_r = 80\%$ (file: atn0580m08c.dat)

Fig. 5.15 shows an example of comparisons between horizontal stresses measured in the experiment by Fang et al. (2002), and by this simulation for the sand with relative density of 80%. $S$ in the graph is the amount of wall translation and $H (=0.5m)$ is the height of the moving wall. Soil elements in the simulation located near the bottom of the moving wall showed a relatively large increase in the horizontal pressure compared to the
pressure increment of soil elements on the upper portion. The graph does not show the result of simulation for S/H ratio of 0.2 since the program was terminated before movement reached that high due to severe deformation of some elements near the wall base.

The resultant of the horizontal force was obtained by integrating the distribution of horizontal pressure along elements adjacent to the moving wall. The coefficient of horizontal passive pressure, $K_{px}$, is obtained by applying Eq. (5.1). To obtain the point of application of the horizontal force (= $a$), moment calculation was made for each segment of pressure distribution relative to the bottom of the moving wall. Both calculations were performed for four types of relative density, 32%, 38%, 63%, and 80%, and, in each relative density, with five different soil depths, 0.5m, 1m, 1.5m, 2m, and 4m. The results of these calculations are presented in Fig. 5.16 through Fig. 5.23.
Fig. 5.16 shows development of $K_{px}$ for $D_r$ of 32%. For lower S/H values, computed $K_{px}$ fluctuates and is slightly higher than the experimental result. The fluctuation of value of $K_{px}$ decreases as S/H increases. The difference on $K_{px}$ values between the simulation and the experiment also lessens as S/H increases. In all simulations, the maximum values of $K_{px}$ were reached at an earlier stage than that of the experiment. In the case of sand with $D_r$ of 38% as shown in Fig. 5.18 the fluctuations of the data are also observed, with a lesser degree. For relative density of 63% shown in Fig. 5.20, $K_{px}$ values reached the maximum at a later stage compared to the experiment, and then decreased with a low rate. The maximum value was a little higher than the
experiment for simulation with soil depth of 0.5m; however, the values were closer as the depth increased. For dense sand with relative density of 80% in Fig. 5.22, both experiment and simulation with a depth of 0.5m resulted in close maximum $K_{px}$ values, although simulation reached that value at a later stage of the wall movement. Simulations show higher residual values of $K_{px}$ than experiment does.

![Graph showing points of application of horizontal stress of loose soil with $D_r = 32\%$](image)

Fig. 5.17 Points of application of horizontal stress of loose soil with $D_r = 32\%$, experiment by Fang et al. (1994) and FLAC simulation of different wall heights

The points of application "a" of horizontal passive pressure resultant are shown in Fig. 5.17, Fig. 5.19, Fig. 5.21, and Fig. 5.23 for both the simulations and the experiment for different soil depth, and relative densities. There were no obvious differences on the points of application, "a", of horizontal passive pressure resultant. However, all figures showed a larger experimental "a" values compared to those of simulation results at the
initial movement of the wall. The larger experimental “a” values indicate a non-linear horizontal pressure distribution with a higher stress on the upper parts. These higher values are possibly caused by the effect of compaction during soil preparation in the experiments, which resulted in different distribution as compared to that of a natural deposition process.

At a later stage of wall movements, the points of applications of horizontal passive pressure become similar between the experiment and the simulation. However, when the wall moved further the simulations showed somewhat larger values than the experiment. Similar phenomena are observed in other relative densities. The later increase in the point of application values in the simulation indicates that there is changing in pressure distribution from linear to higher values on the upper elements.
Fig. 5.18 Development of $K_{px}$ during translation for soil of $D_r = 38\%$ from experiment by Fang et al. (2002) and FLAC simulation of different wall heights.

Fig. 5.19 Points of application of horizontal stress of loose soil with $D_r = 38\%$ experiment by Fang et al. (2002) and FLAC simulation of different wall heights.
Fig. 5.20 Development of $K_{p\theta}$ during translation for soil of $D_r = 63\%$ from experiment by Fang et al. (2002) and FLAC simulation of different wall heights

Fig. 5.21 Points of application of horizontal stress of medium soil with $D_r = 63\%$ experiment by Fang et al. (2002) and FLAC simulation of different wall heights
Fig. 5.22 Development of Kpx during translation for soil of $D_r = 80\%$ from experiment by Fang et al. (2002) and FLAC simulation of different wall heights.

Fig. 5.23 Points of application of horizontal stress of dense soil with $D_r = 80\%$ experiment by Fang et al. (2002) and FLAC simulation of different wall heights.
5.2.2 Rotation About A Point Below The Wall Base (RBT) Mode

RBT mode is the where the wall at the top point is moved at a faster rate than at the bottom point. With this movement the wall is rotated around a point at or below the base of the wall. The symbol “n” is equal to the distance from that said point to initial location of wall bottom divided by the wall height as defined in Fig. 3.1. The program simulated the experiments by Fang et al. (1994) which used sand with a low relative density of 32% with “n” values of 0, 0.21, 0.5, and 13.75.

The coefficients of horizontal passive pressure, $K_{px}$, and the point of application, “a”, are computed for various “n” values in simulations. As an example, simulation with “n” = 0.5 is presented in Fig. 5.24 where the contour of accumulated plastic shear strain is shown. Corresponding grid distortion and principal stress directions are shown in Fig. 5.25, elements that yielded in Fig. 5.26, and horizontal pressure distributions are presented in Fig. 5.27.
Fig. 5.24 RBT mode: contour of accumulated plastic shear strain

Fig. 5.24 shows accumulated plastic shear strain after the program performed 339,568 computation steps. Elements near the backfill surface near the wall and the wall bottom underwent relatively higher shear strain compared to the rest of the elements. The distortions of surface elements around the wall were caused by high wall translation with small confinement due to the lack of vertical stress. At the bottom, element distortion was caused by sharp corner below the moving wall as shown in Fig. 5.25. Orientations and relative amounts of principal stresses are also presented in the figure.
Fig. 5.26 presents elements that have yielded which spreads across the backfill at the time when the program was terminated. Near the moving wall a chain of elements clearly shows yielding elements from the wall bottom slowly sloping upward toward the backfill surface. Another chain of elements show yielding elements from soil surface near the wall down to the middle of the previous chain. Obviously, the failure pattern is quite different from the case of translation wall movement as seen in Fig. 5.13.
Fig. 5.26 RBT mode: elements indicated as yield in shear

Fig. 5.27 shows the horizontal stress distribution of elements adjacent to the moving wall and experimental results as the wall movement progressed. In the figure, Smax is the horizontal wall movement at the backfill surface as defined in Fig. 3.1. The distribution shows a higher pressure on the upper elements compared to a triangular pressure distribution. At the bottom of the wall, the pressure drops even lower than the pressure before the wall moved.
Fig. 5.27 RBT mode: Horizontal pressure for Dr = 32%, “n” = 0.5 from Fang et al. (1994) and FLAC simulations.

Fig. 5.28 presents $K_{px}$ values for both experiment and simulations for “n” value of 0. Simulations were conducted for five different soil depths and corresponding wall heights. The figure shows that all the simulations results are higher than experimental
results; however, the differences became less as the wall movement progressed. Points of application of the resultant of pressure “a” are shown in Fig. 5.29. The values from simulations are less than those of experiment even though they both higher than triangular pressure distribution. Larger simulation model size shows lower “a” values. The similar trends on $K_{px}$ and “a” values are also observed in Fig. 5.30 and Fig. 5.31 for “n” of 0.21, and in Fig. 5.32 and Fig. 5.33 for “n” of 0.5, respectively.

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**Fig. 5.28** RBT mode: $K_{px}$ for $D_r = 32\%$, $n=0$, from Fang et al. (1994) and simulation with $H = 0.5m$, 1m, 1.5m, 2m, 4m
Fig. 5.29 RBT mode: $K_{px}$ for $D_r = 32\%$, $n=0.21$, from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$.

Fig. 5.30 RBT mode: $K_{px}$ for $D_r = 32\%$, $n=0.21$, from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$.
Fig. 5.31 RBT mode: point of application “a” for $D_r = 32\%$, $n=0.21$, from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$

Fig. 5.32 RBT mode: $K_{px}$ for $D_r = 32\%$, $n=0.5$, from Fang et al. (1994) and simulation with $H = 0.5m, 1m, 1.5m, 2m, 4m$
Fig. 5.33 RBT mode: point of application “a” for $D_r = 32\%$, $n=0.5$, from Fang et al. (1994) and simulation with $H = 0.5m$, 1m, 1.5m, 2m, 4m

Fig. 5.34 and Fig. 5.35 show a development of values of $K_{px}$ and “a”, respectively, from simulations and experiment with “n” value of 13.78. Simulations for all five model sizes did not survive the same maximum wall movement as in the experiment. Both simulation and experiment have reached the similar $K_{px}$ and “a” value at the movement reached the ratio of around 10 percent of $S_{max}/H$.

For all “n” conditions of passive RBT simulations, $K_{px}$ results are higher than those of experiments. $K_{px}$ values of both simulation and experiments increase with a similar rate except for “n” equal 13.78, which is closer to a translation mode such as seen in Fig. 5.16 and Fig. 5.18.
**Fig. 5.34** RBT mode: $K_{px}$ for $D_r = 32\%$, $n=13.78$ from Fang et al. (1994) and simulation with $H = 0.5\text{m}, 1\text{m}, 1.5\text{m}, 2\text{m}, 4\text{m}$

**Fig. 5.35** RBT mode: point of application $a$ for $D_r = 32\%$, $n=13.78$ from Fang et al. (1994) and simulation with $H = 0.5\text{m}, 1\text{m}, 1.5\text{m}, 2\text{m}, 4\text{m}$
5.2.3 Rotation About A Point Above The Top (RTT) Mode

RTT mode is where the wall at the level of the sand surface is moved at a slower rate than at the wall bottom. With this movement, the wall is rotated around a point at or above the wall top. The symbol “n” is equal to the distance from that said point to the wall top divided by the wall height as defined in Fig. 3.1. The program simulated the experiments by Fang et al. (1994) which used sand with a low relative density of 32% with “n” values of 0, 0.5, 1.81, and 7.43.

Coefficients of horizontal passive pressure, $K_{px}$, and the point of application, “a”, are computed for various “n” values. As examples, simulation with “n” = 0 is presented in Fig. 5.36 for the contour of accumulated plastic shear strain, Fig. 5.37 for grid distortions and principal stress directions, Fig. 5.38 for elements that yielded, and Fig. 5.39 for horizontal pressure distributions.
Fig. 5.36 RTT mode: contour of accumulated plastic shear strain

Fig. 5.36 shows the contour of shear strain after the program performed 632,737 computation steps. Elements around the wall bottom underwent relatively higher shear strain compared to the rest of the elements. The figure also shows a distinct failure surface emanating from the wall bottom to the backfill soil surface. The distortions of elements near the wall as shown in Fig. 5.37 were caused by high wall translation and sharp corner at the wall bottom. Orientations and relative amounts of principal stresses are also presented in the figure.
Fig. 5.37 RTT mode: grid distortion and principal stress direction

Fig. 5.38 presents elements that have yielded when the program was terminated. The figure clearly shows a chain of yielding elements curving upward from the wall bottom to backfill soil surface. The observed failure surface is clearer than the case of RBT (Fig. 5.26), but it is smaller than translational case (Fig. 5.13).
Fig. 5.38 RTT mode: elements indicated as yield in shear

Fig. 5.39 shows the stress distribution of elements adjacent to the moving wall of the simulation and the experiment. In the figure, Smax is the horizontal movement of the wall bottom as seen in Fig. 3.1. The distribution shows a higher pressure on the lower elements compared to a triangular pressure distribution. At the bottom of the wall the pressure drops even lower than the pressure before the wall moved.
Fig. 5.39 RTT mode: $D_r = 32\%$, "n" = 0 from Fang et al. (1994) and simulation

Fig. 5.40 presents $K_{px}$ values for both experiment and simulations for “n” value of 0. Simulations were conducted for five different model sizes. The figure shows that all the simulations results are higher than the experimental results. Simulations with soil depth of 1m, 1.5m, 2m, and 4m show $K_{px}$ values are close to each other and lower than
the simulation result with the model depth of 0.5 m. Developments of $K_{px}$ for simulations and experiment show similar curvatures but only the magnitude are different. Points of application, "a", of the resultant of the pressure are shown in Fig. 5.41. The values from simulations are slightly less than those of the experiment at the initial stage and became closer at a later stage. Values of "a" for both simulations and experiment are smaller than triangular pressure distribution. The similar trends for $K_{px}$ and "a" values are also observed in Fig. 5.42 and Fig. 5.43 for "n" = 0.5. For "n" = 0.5, $K_{px}$ values from simulation of different model sizes are closer to each other compared to the $K_{px}$ values for "n" = 0.

Fig. 5.40 RTT mode: $K_{px}$ for $D_t = 32\%$, $n=0$ from Fang et al. (1994) and simulation with $H = 0.5 m, 1 m, 1.5 m, 2 m, 4 m$
Fig. 5.41 RTT mode: point of application “a” for $D_r = 32\%$, $n=0$ from Fang et al. (1994) and simulation with $H = 0.5\text{m}, 1\text{m}, 1.5\text{m}, 2\text{m}, 4\text{m}$

Fig. 5.42 RTT mode: $K_px$ for $D_r = 32\%$, $n=0.5$ from Fang et al. (1994) and simulation with $H = 0.5\text{m}, 1\text{m}, 1.5\text{m}, 2\text{m}, 4\text{m}$
Fig. 5.43 RTT mode: point of application “a” for $D_r = 32\%$, $n=0.5$ from Fang et al. (1994) and simulation with $H = 0.5\text{m}$, 1m, 1.5m, 2m, 4m

Fig. 5.44 presents $K_{px}$ values for both experiment and simulations for “$n$” value = 1.81. Simulations did not survive the movements as the experiment. Initially the simulation results are higher than those of the experiment. However, both final $K_{px}$ values are about the same, although simulation reached the final value earlier than did the experiment. Fig. 5.45 shows points of application “a” of the resultant. The values from simulations are less than those of the experiment at the initial stage, and those are smaller than triangular pressure distributions. At a later stage both values converged to a similar value, which was less than the triangular distribution. The same trends for $K_{px}$ and “a” values are also observed in Fig. 5.46 and Fig. 5.47, respectively, for “$n$” = 7.43. As the
“n” value increases, the points of applications of the resultant in the experiments increase at small Smax/H, which could be caused by the arching effect at the upper section of the elements. Initial increase in “a” value was not observed in all simulations.

Fig. 5.44 RTT mode: Kpx for D_r = 32%, n=1.81 from Fang et al. (1994) and simulation with H = 0.5m, 1m, 1.5m, 2m, 4m
**Fig. 5.45** RTT mode: point of application “a” for $D_r = 32\%$, $n=1.81$ from Fang et al. (1994) and simulation with $H = 0.5\text{ m}, 1\text{ m}, 1.5\text{ m}, 2\text{ m}, 4\text{ m}$

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**Fig. 5.46** RTT mode: $K_{px}$ for $D_r = 32\%$, $n=7.43$ from Fang et al. (1994) and simulation with $H = 0.5\text{ m}, 1\text{ m}, 1.5\text{ m}, 2\text{ m}, 4\text{ m}$
Fig. 5.47 RTT mode: point of application “a” for $D_t = 32\%$, $n=7.43$ from Fang et al. (1994) and simulation with $H = 0.5m$, 1m, 1.5m, 2m, 4m

Values of $K_{px}$ and “a” associated with wall movement at $S_{max}/H = 0.1$ were selected in order to compare the experimental results and the simulations for models 0.5 m of wall height for both RBT and RTT modes. Fig. 5.48 shows relation between “n” and values of $K_{px}$. In RBT mode, the values of $K_{px}$ in simulation and experiment eventually became a close value at higher “n” value. Both $K_{px}$ values for RTT modes had some different at $S_{max}/H = 0.1$ level. However, those values in RTT modes converged to similar values at larger $S_{max}/H$ level.
Fig. 5.48 $K_{px}$ values for RBT and RTT modes with increasing “n” values from Fang et al. (1994) and FLAC simulation for $D_r = 32\%$ at $S_{max}/H=0.1$

Fig. 5.49 shows the relation between “n” and “a” values for both RBT and RTT modes from both simulations and experiments. Close agreements on “a” values in the same wall movement mode indicate that the shape of earth pressure distribution of the simulation and the experiment are in good agreements. At “n” = 0, the “a” values of RBT are larger (about 0.5) than that of triangular distribution, while the “a” values of RTT is smaller (about 0.2) than that of triangular distribution. As “n” increases toward
translation mode, values of “a” from RBT and RTT modes converge to the value of triangular stress distribution, and to the value (0.33) of the translational mode.

![Diagram](image)

**Fig. 5.49** Point of application “a” for RBT and RTT modes with increasing “n” values from Fang et al. (1994) and FLAC simulation for $D_r = 32\%$ at $S_{max}/H=0.1$

5.2.4 **Summary of Comparison between Simulations and Experiments**

In summary, by comparing simulations and Fang et al.’s experiments the following can be concluded. For translation modes, simulations with low $D_r$ resulted in
the maximum \( K_{px} \) values higher than those of the experiment. However, for larger relative densities the maximum \( K_{px} \) values for both experiments and simulations are similar.

Simulations and comparisons with RBT and RTT modes were performed for low relative density. In RBT mode with all \( n \) values, maximum \( K_{px} \) are similar for both experiments and simulations. In RTT mode with low \( n \) values, the maximum \( K_{px} \) in simulations are larger than those of the experiments. However, \( K_{px} \) values with high \( n \) values are similar for the experiment and the simulation.

For all the modes, the values of point of application of resultant "a" are more or less similar for both experiments and simulations with large wall displacements.

However, in an earlier stage of wall movement, there exist some differences between the simulations and the experiments. Those differences might be attributed to the initial compaction of the backfill soils. In the experiment, some degree of initial compaction exists. Those differences appeared in many cases in an early stage of wall movement level. For example, initial high "a" values were observed in all the cases of the experiments. When wall movement increased, the initial compaction effect might disappear and reach to similar values at high wall movement level.

In addition, other possible causes of the differences could be due to the fact that low relative densities of 32% and 38% were below the range of relative density of plane strain experimental data which was at 52.78% of Dr as the lowest as shown in Fig. 4.7. Stress-strain relations of the simulations with low relative densities were determined by extrapolation.
6 LATERAL EARTH PRESSURE SIMULATIONS FOR DIFFERENT WALL MOVEMENTS WITH VARIOUS DENSITIES

6.1 Passive Cases in Translation, RBT, and RTT Modes with Relative Density of 60%, 70%, and 80%

Simulations by FLAC were conducted on 4.0 m deep anisotropic backfill sand with three different relative densities; namely 60%, 70%, and 80%. Those were conducted in translation, RBT, and RTT modes. RBT and RTT modes were performed with “n” values of 0, 0.5, 2, 7, 15, and 50. The results of simulations were summarized in Fig. 6.1. The vertical axis on the graph is the maximum values of $K_{px}$ obtained during the period between the start of the wall movement and $S_{max}/H$ equal to 0.1. The results from translation modes were plotted as horizontal lines at higher “n” range ($n > 30$). The curves show that RTT and RBT converge at high “n” and it becomes the value of the translation mode. Maximum $K_{px}$ values for RBT and RTT increase as “n” increases, reach the highest points at certain “n” values, and then decrease beyond that points. The values of “n” at which $K_{px}$ reaches the maximum are about 2.0 for RBT and about 15 for RTT. For various configurations of wall movements, RBT and RTT modes gave higher $K_{px}$ values than translation mode.
Fig. 6.1 Max $K_{px}$ (up to $S_{max}/H=0.1$) with increasing “n” for RBT, RTT, translation modes for $H=4m$ and $D_r$: 60%, 70%, and 80%

Fig. 6.2 describes the development of the point of application of passive earth pressures against anisotropic sand backfill with a particular relative density of 80%. As “n” increases from 0 to 15, curves of “a” for both RBT and RTT approach the curve of the translation mode. The changes in values of “a” with increasing $S_{max}/H$ were attributed to various factors such as: anisotropic nature of sand at certain relative density
of soil, movement modes, and values of “n”. For example in RBT modes for low “n” value, the point of application moves upward due to increased stress in upper elements at small Smax/H stage. As the wall movement progresses the point shifts downward because the upper elements enter to residual stress stage while the stresses in the lower elements still increase due to their small shear strain there.

![Graph](image)

**Fig. 6.2** Varying point of application “a” with Smax/H for RBT, RTT (n: 0, 0.5, 2, 7, 15), and Translation modes for H = 4m, Dr = 80%
6.2 Effect of Model Scale on Maximum $K_{px}$ Values

Fig. 6.3 through Fig. 6.7 describes the effect of model scale and thus confining pressure on the coefficients of horizontal passive earth pressure. Fig. 6.3 demonstrates the effect of wall height in translation mode in different relative densities. The height effects on RBT modes are displayed in Fig. 6.4 for “n” = 0, and in Fig. 6.5 for “n” = 15. For RTT modes the effect of wall height are shown in Fig. 6.6 for “n” = 0, and in Fig. 6.7 for “n” = 15. RTT mode with “n” value of 0 shown in Fig. 6.6 shows minor variations in $K_{px}$ with different relative densities of sand for all the wall height. The results for all wall movement modes consistently show decreases in coefficients of horizontal passive earth pressure as the wall height increases. In an average, the reduction on the maximum of $K_{px}$ value from 0.5 m wall to 4.0 m wall was about 13% with a range of 8.9% to 19.2%.

![Translation](image)

**Fig. 6.3** Translation mode: $K_{px}$ for soil depth: 0.5m, 4m, and Dr: 60%, 70%, and 80%
Fig. 6.4 RBT mode (n=0): $K_{px}$ for soil depth: 0.5m, 2m, 4m, and Dr: 60%, 70%, and 80%

Fig. 6.5 RBT mode (n=15): $K_{px}$ for soil depth: 0.5m, 2m, 4m, and Dr: 60%, 70%, and 80%
Fig. 6.6 RTT mode (n=0): $K_{px}$ for soil depth: 0.5m, 2m, 4m, and Dr: 60%, 70%, and 80%

Fig. 6.7 RTT mode (n=15): $K_{px}$ for soil depth: 0.5m, 4m, and Dr: 60%, 70%, and 80%
6.3 Effects of Anisotropy in RBT and RTT Modes

Simulations were performed in both RBT and RTT wall movement modes in order to investigate the effects of backfill sand anisotropy on the coefficients of passive earth pressure. Only RBT and RTT modes were performed for the case of “n” = 0, and the same soil parameters as shown in Fig. 5.8 were utilized.

Fig. 6.8 presents the development of $K_p$ values in RBT mode. $K_p$ values of isotropic sand were much larger than those of anisotropic sands for corresponding densities. Both sands showed sharp increase at the initial stage and keep increasing with a slower rate without peak values up to $S_{max}/H$ equal to 0.1. Significant differences on the point of application of the resultants are shown in Fig. 6.9. The use of elastic-perfectly plastic model for isotropic sand in RBT mode causes immediate jump in pressure at upper elements that shift the point of application upward. Subsequent movement does not increase pressure at the upper elements. Instead, this movement starts increasing the pressure at the lower part, and thus it lowers the point of application thereafter. Stress-strain relation of anisotropic materials had smooth transitions as seen in Fig. 4.6 and thus “a” values showed smooth increases. Fig. 6.10 presents comparisons of $K_p$ values up to $S_{max}/H = 0.1$ with varying $\phi_{ps}$ values. In the figure, Coulomb’s theoretical predictions, Fang et al. (1994)’s data, and anisotropic simulations for $H=0.5m$ and $H=4.0m$ were plotted. Anisotropic solution showed much smaller $K_p$ values than that of isotropic materials. Coulomb’s predictions were close to the results from isotropic simulations. Fang et al.’s $K_p$ was smaller than the anisotropic simulations at the level of wall movement up to $S_{max}/H = 0.1$. 
Fig. 6.8 RBT mode: $K_p$ for Isotropic and Anisotropic simulations with $\phi = 30^\circ$, $35^\circ$, $40^\circ$
Fig. 6.9 RBT mode: point of application “a” for Isotropic and Anisotropic simulations with $\varphi = 30^\circ, 35^\circ, 40^\circ$
Fig. 6.10 RBT mode: Max of $K_p$ up to $S_{max}/H=0.1$ vs. $\varphi_{ps}$ for Isotropic and Anisotropic simulations

Fig. 6.11 shows $K_p$ values in RTT mode. The $K_p$ values of isotropic sand were much higher than anisotropic materials. Both sands showed sharp increases at the initial stage and then increased with lower rates to peak values. The $K_p$ values then decreased or leveled off. Values of the point of application of resultant “a” are shown in Fig. 6.12. Soil
model in isotropic sand in RTT mode caused lower values of “a” at the initial stage. In subsequent movement, both sands did not show any significant differences. Fig. 6.13 presents comparisons of $K_p$ values for isotropic and anisotropic sands with changing $\varphi_{PS}$. In the figure, Coulomb’s theoretical predictions, Fang et al. (1994)’s experimental result, and anisotropic simulations for $H=0.5m$ and $H=4.0m$ were plotted. Anisotropic sand had much smaller $K_p$ values for all $\varphi_{PS}$ values than isotropic sand. Coulomb’s predictions appeared closer to isotropic simulation. However, Coulomb’s prediction in RTT mode was not as close as those in translation and RBT modes. Meanwhile, Fang et al.’s result was closer to anisotropic simulations than either to isotropic simulations or to Coulomb’s predictions.
Fig. 6.11 RTT mode: $K_p$ for Isotropic and Anisotropic simulations with $\phi = 30^\circ, 35^\circ, 40^\circ$
Fig. 6.12 RTT mode: point of application “a” for Isotropic and Anisotropic simulations with $\varphi = 30^\circ, 35^\circ, 40^\circ$
Fig. 6.13 RTT mode: Max of $K_p$ up to $S_{max}/H=0.1$ vs. $\varphi_{ps}$ for Isotropic and Anisotropic simulations
7 SUMMARY AND CONCLUSIONS

Computer simulation utilizing FLAC code has been utilized to simulate passive earth pressure in plane strain condition. The simulation adopts a model of stress strain relation of strain-hardening/softening model, and considers the anisotropic nature of dry sands. The stress strain relation and anisotropy characteristics were both built from laboratory experimental results on sands. The following conclusions can be made through this research.

(1) Comparisons in isotropic simulation were performed between the current system and other FLAC model with different structure and element mesh. The resulted $K_p$ values were practically the same for various combinations of the angle of internal friction, dilation angle, and wall friction, with the exception for combination of zero dilation, low wall friction, and high angle of internal friction. The small differences were caused probably by the differences in wall and element mesh configuration.

(2) For both isotropic and anisotropic simulations, the dilation angle appears to have less effect than the angle of internal friction of soils on $K_p$ values. The dilation angle as a half of the angle of internal friction could be used without significant effects on $K_p$ values instead of the rigorous determination of the dilation angle.

(3) Compared to model wall experimental results in translation mode the anisotropic simulations yielded higher coefficients of passive earth pressure, $K_{px}$, for loose sand. However, the coefficients were close for medium sand, and about the same for dense sand. $K_{px}$ values fluctuate for small wall high model. However, the fluctuation decreases as model scale or relative density increases. The simulation strain to reach the maximum
lateral pressure values is less for loose sand, close for medium sand, and higher for dense sand.

(4) From the comparisons between anisotropic simulations and experiments in RBT modes, $K_{px}$ values from simulations were higher for low "n", and close for high "n" values. From the comparison in RTT, with low "n" values, $K_{px}$ values from simulation were higher. Meanwhile, for high "n" values, the results were close.

(5) For all modes, points of application of resultant of lateral earth pressure “a” are practically similar in both anisotropic simulation and experimental results with large wall displacement. Some differences, however, are observed, in particular, at the earlier stage of wall movements. Those differences might be attributed to the initial compaction of the backfill soils in the experiment.

(6) From RBT and RTT anisotropic simulations with increasing “n” values in soils with various relative densities, $K_{px}$ values of the same relative density reached similar maximum at “n” about 2 and 15 for RBT and RTT modes respectively.

(7) From anisotropic simulations with different wall heights, in all movement modes, increasing wall height resulted in lower $K_{px}$ values with an average reduction of 13% from 0.5 m wall to 4.0 m wall.

(8) For passive state simulations in translation, RBT (n=0), and RTT (n=0) modes, with various angle of internal friction, $K_p$ values of anisotropic simulations are significantly smaller than those of isotropic simulations. Design practice with assumption of isotropic conditions result in higher $K_p$ values than they may actually exist.
For all movement modes, Coulomb's theoretical predictions were similar to the results from isotropic simulations. Results from Fang et al.'s experiments were closer to anisotropic simulations than to isotropic simulations.

Further study on the anisotropic soil model could be continued for improvement as more experimental data becomes available.
BIBLIOGRAPHY


APPENDIX

INPUT FILE FOR FLAC ANALYSIS

Example for Passive Translation in Isotropic Soil

title
z32isg22k60di05f35w13a2.dat:H=4.0m,tran,G=22.5Mpa,K60Mpa,dil=0.5f,f=35,d:/f=1/3
set cust1
Dept.of Civil and Env. Eng.
set cust2
Old Dominion U.,Norfolk,Virginia
config extra 2
set echo off
grid 72 22
mod ss
set grav=9.81 ;m/s2
def dimension
;soil------
hsoil = 4.0 ; INPUT, depth of soil adjacent to moving wall (m)
n = 7e12 ; INPUT, rotation about point at nH (m) below wall base
dsoil = 0.613*hsoil/0.5 ; soil depth (m), the same proportion with Fang's experiment
lsoil = 2.0*hsoil/0.5 ; soil length (m)
bed = 0.113*hsoil/0.5 ; bed depth (m)
bed1 = 0.99999*bed
nh = n*hsoil ; nH (m)

; hwall = 0.8*hsoil/0.5 ; wall top to soil bottom (m)
hwall = hwall-bed ; height of movable wall
twall = 0.12*hsoil/0.5 ; wall thickness (m)
awall = twall*1. ; area of movable wall in (m2)
iwall = 1*(twall^3)/12. ; moment area of the wall (m4)
ewall = 200e9 ; assuming steel (Pa)
bwall = -0.133*hsoil/0.5 ; coordinate of front edge of wall base
; final wall movement
maxshr = 0.15 ; maximum ratio of horizontal movement to soil depth
end
dimension
def backfill
; Ottawa sand used in Fang test
gs = 2.65 ; INPUT, specific gravity
eemax = 0.76 ; INPUT, maximum void ratio
eemin = 0.50 ; INPUT, minimum void ratio
gerldens = 32. ; INPUT, relative density (%)
; coefficients for variation ratio R(d) for all angle for a given Dr,
ca = 0.90843629742579 ; INPUT for Dr = 32
ax1 = -4.719059E-04
ax2 = 4.192983E-05
ax3 = -8.95695E-07
cb = 0.54985314357006
bx1 = 1.270233E-02
bx2 = -1.198577E-04
bx3 = 3.814908E-07

; coefficient due to small sig3 effect (model scale effect) --
; for fdirsh, cfdirsh = afdirsh ln(sigma3) + bfdirsh ; function of Dr
afdirsh = -0.006043 ; for DR=32
bfdirsh = 1.077827

; for fres, cfres = afres ln(sigma3) + bfres; the same for all Dr
afres = -0.0121220 ; for all DR
bfres = 1.1561240

walsanrat=-0.000025502*(reldens)^2+0.00385617*reldens+0.187259946 ; ratio

; wall/sand friction to direct shear

gamwater = 9.81*1000 ; unitweight of water N/m3
ee=eemax-reldens*(eemax-eemin)/100. ; void ratio
fdirsh = 0.000275*(reldens)^2+0.184275*reldens+25.601176 ; direct shear friction angle
fres=31.5 ; residual angle of friction from DS, independent of test type

; Minimum ratio R(d) for a given Dr

minrat = cr + cdl*reldens + cd2*(reldens)^2 + cd3*(reldens)^3 ; minimum fpeak@d

fwall = walsanrat*fdirsh ; wall friction

; scaling to the value used in Benmebarek 2008
angle_used = 35. ; fplst
fwall_soi= 1./3. ; ratio of d wall to soil friction
unitwei = 20000 ; N/m3
scale4fres = fres/fplst
fplst = angle_used
fwall = fwall_soil*fplst
fres = angle_used
fdirsh = angle_used
end
backfill
; grid generation
gen 0 0 0 bed lsoil bed lsoil 0 i=1,73 j=1,5
gen 0 bed 0 dsoil lsoil dsoil lsoil bed i=1,73 j=5,23
def setprop
float zz sigm gg kk pois yc
dila=(((14.-7.)/(79.4167-54.9075))*(reldens-54.9075)+7. ; for maximum dilation angle
dilaat0=-dila/2. ; negative start at zero
pa_psf=0.020885 ; conversion pascal to psf
psi_pa=6894.75728 ; conversion psi to pascal
pa = 101300 ; atmospheric pressure in N/m2 (=Pa)
sss = 1350 ; stiffness coefficient Hardin 1978 Earthquake eng and soil dynamics p3-90
loop i (1,izones)
  loop j (1, jzones)
    yc = (y(i,j)+y(i+1,j)+y(i,j+1)+y(i+1,j+1))/4.
    zz = y(l,jgp) - yc
  ; plain strain to triaxial -to find f triax for obtaining poisson ratio..Holtz n Kovacs p517
    if fplst > 34 then
      ftriax = (fplst + 17.)/1.5
    else
      ftriax = fplst
    endif
    ko = 1 -sin(ftriax*degrad) ; to be used in calculating average sigma
    pois=(1-sin(ftriax*degrad))/(2-sin(ftriax*degrad)) ; Ko=1-sin f = pois/(1-pois)
    sigm=(unitwei*zz*(1.+2.*ko)/3.) ; average stress (Pascal)
    gg=(sss*1*(sigm*pa)^0.5)/2*(1+pois)*(0.3+0.7*ee^2)) ; in N/m2
    kk=2.*gg*(1.+pois)/(3.*(1.-2.*pois)) ; initial K
    shear_mod(i,j)=gg
    bulk_mod(i,j)=kk
    density(i,j)=unitwei/9.81 ; kg/m3
    cohesion(i,j)=0.
    tension(i,j)=0.
    ; modification for Benmebarek 2006
    shear_mod(i,j)=22.5e6
    bulk_mod(i,j)=60.0e6
    density(i,j)=20000./9.81 ; kg/m3
  endloop
endloop
dila= 0.5*angle_used
dilaat0= 0.5*angle_used

end

setprop
tab 15 0.,dilaat0 0.03,dila 0.07,dila ; constant
tab 16 0.,fplst 0.2,fplst ; initial value of friction
tab 20 hsoil,n rel dens,unitwei fplst,pois fdirsh,fwall
prop dtab 15
prop ftab 16
fix y j=1 ;bottom
fix x i=73 ;right wall
fix x i=1 ;left wall
set plot pcx
def angle_pq ;angle and pq for all elements
loop i (1,izones)
  loop j (1,jzones)
    aaa=-(sxx(i,j)+syy(i,j))/2. ; change sign to positive
    ccc=(sxx(i,j)-syy(i,j))/2.
    bbb=((ccc)^2 + (sxy(i,j))^2)^0.5 ; always positive
    sigma1=aaa + bbb ; sig1 since this is larger , always positive
    sigma3=aaa - bbb ; sig3 since this is lower
    ex_2(i,j)=sigma3 ; will be used for evaluating confined pressure
    if sigma1 # abs(sxx(i,j)) then
      chi=atan(sxy(i,j)/(sigma1-abs(sxx(i,j))))
    else
      chi=90.*degrad
    endif
    ex_1(i,j)=90-abs(chi/degrad)
  end_loop
end_loop
end

; solve for gravity
solve
plot grid esyy fill
copy 0gravsyy.pcx
plot grid esxx fill
copy 0gravsxx.pcx

; average depth of zone attached to the wall = dc..
def dc1_22
  dc22 = (dsoil - (y(1,22)+y(2,22)+y(1,23)+y(2,23))/4.)
  dc21 = (dsoil - (y(1,21)+y(2,21)+y(1,22)+y(2,22))/4.)
  dc20 = (dsoil - (y(1,20)+y(2,20)+y(1,21)+y(2,21))/4.)
  dc19 = (dsoil - (y(1,19)+y(2,19)+y(1,20)+y(2,20))/4.)
  dc18 = (dsoil - (y(1,18)+y(2,18)+y(1,19)+y(2,19))/4.)
  dc17 = (dsoil - (y(1,17)+y(2,17)+y(1,18)+y(2,18))/4.)
\[
\begin{align*}
dc16 &= (dsoil - (y(1,16)+y(2,16)+y(1,17)+y(2,17))/4.) \\
dc15 &= (dsoil - (y(1,15)+y(2,15)+y(1,16)+y(2,16))/4.) \\
dc14 &= (dsoil - (y(1,14)+y(2,14)+y(1,15)+y(2,15))/4.) \\
dc13 &= (dsoil - (y(1,13)+y(2,13)+y(1,14)+y(2,14))/4.) \\
dc12 &= (dsoil - (y(1,12)+y(2,12)+y(1,13)+y(2,13))/4.) \\
dc11 &= (dsoil - (y(1,11)+y(2,11)+y(1,12)+y(2,12))/4.) \\
dc10 &= (dsoil - (y(1,10)+y(2,10)+y(1,11)+y(2,11))/4.) \\
dc9 &= (dsoil - (y(1,9)+y(2,9)+y(1,10)+y(2,10))/4.) \\
dc8 &= (dsoil - (y(1,8)+y(2,8)+y(1,9)+y(2,9))/4.) \\
dc7 &= (dsoil - (y(1,7)+y(2,7)+y(1,8)+y(2,8))/4.) \\
dc6 &= (dsoil - (y(1,6)+y(2,6)+y(1,7)+y(2,7))/4.) \\
dc5 &= (dsoil - (y(1,5)+y(2,5)+y(1,6)+y(2,6))/4.) \\
dc4 &= (dsoil - (y(1,4)+y(2,4)+y(1,5)+y(2,5))/4.) \\
dc3 &= (dsoil - (y(1,3)+y(2,3)+y(1,4)+y(2,4))/4.) \\
dc2 &= (dsoil - (y(1,2)+y(2,2)+y(1,3)+y(2,3))/4.) \\
dc1 &= (dsoil - (y(1,1)+y(2,1)+y(1,2)+y(2,2))/4.) \\
end
\]

ini xdis=0. ydis=0.; reset displacement after gravity
prop e_plastic = 0.; reset plastic strain
def tiltangle
  ytop = y(l,23)
  ymid = y(l,14)
  ybot = y(l,5)
  smaxrat = (nh+hsoil)/(ymid-ybot+nh); max displ at soil surface if displ at (1,11)=1
  xvbotrat = nh/(hsoil+nh) ; velocity at the wall bottom if velocity at soil surface =1
  xvtoprat = (hmwall+nh)/(hsoil+nh) ; velocity at the wall top if vel at soil surface =1
end
tiltangle
; ratio of horizontal displacement to backfill height
def shrat
  float shrat
  while_stepping
    shrat = xdisp(l,14)/hsoil; displ at soil surface/(height of backfill above wall base)
  end
hist 1 nstep= 2000 shrat ; ratio of horizontal displacement to backfill height
hist 2 unbalance ; unbalance force
hist 3 e_plastic i=1 j=14
hist 41 sxx i=1 j=1
hist 42 sxx i=1 j=2
hist 43 sxx i=1 j=3
hist 44 sxx i=1 j=4
hist 45 sxx i=1 j=5
hist 46 sxx i=1 j=6
hist 47 sxx i=1 j=7
hist 48 sxx i=1 j=8
hist 49 sxx i=1 j=9
hist 50 sxx i=1 j=10
hist 51 sxx i=1 j=11
hist 52 sxx i=1 j=12
hist 53 sxx i=1 j=13
hist 54 sxx i=1 j=14
hist 55 sxx i=1 j=15
hist 56 sxx i=1 j=16
hist 57 sxx i=1 j=17
hist 58 sxx i=1 j=18
hist 59 sxx i=1 j=19
hist 60 sxx i=1 j=20
hist 61 sxx i=1 j=21
hist 62 sxx i=1 j=22
def fildata
  array adatshrat(17) ; datshrat(7)
anoshrat=17
  nns=20+2*izones*jzones ; after tatsuoka,friction,dilation
  adatshrat(1) = 0.0  ;
  adatshrat(2) = 0.001  ;
  adatshrat(3) = 0.005  ;
  adatshrat(4) = 0.010  ;
  adatshrat(5) = 0.020  ;
  adatshrat(6) = 0.030  ;
  adatshrat(7) = 0.040  ;
  adatshrat(8) = 0.050  ;
  adatshrat(9) = 0.060  ;
  adatshrat(10) = 0.070  ;
  adatshrat(11) = 0.080  ;
  adatshrat(12) = 0.090  ;
  adatshrat(13) = 0.100  ;
  adatshrat(14) = 0.110  ;
  adatshrat(15) = 0.120  ;
  adatshrat(16) = 0.130  ;
  adatshrat(17) = maxshrat  ;
end
fildata
def scanstress
  int count
  while stepping
    nst=20+2*izones*jzones ; +noshrat ; after tatsuoka,friction,dilation,fangstress
    loop i (1,noshrat)
      if i > count then
        if shrat >= adatshrat(i)
          del 22
nst = nst + i

ystate(nst, 22) = dc1
ystate(nst, 21) = dc2
ystate(nst, 20) = dc3
ystate(nst, 19) = dc4
ystate(nst, 18) = dc5
ystate(nst, 17) = dc6
ystate(nst, 16) = dc7
ystate(nst, 15) = dc8
ystate(nst, 14) = dc9
ystate(nst, 13) = dc10
ystate(nst, 12) = dc11
ystate(nst, 11) = dc12
ystate(nst, 10) = dc13
ystate(nst, 9) = dc14
ystate(nst, 8) = dc15
ystate(nst, 7) = dc16
ystate(nst, 6) = dc17
ystate(nst, 5) = dc18
ystate(nst, 4) = dc19
ystate(nst, 3) = dc20
ystate(nst, 2) = dc21
ystate(nst, 1) = dc22
ystate(nst, 23) = shrat
xstate(nst, 22) = -sxx(1,1)
xstate(nst, 21) = -sxx(1,2)
xstate(nst, 20) = -sxx(1,3)
xstate(nst, 19) = -sxx(1,4)
xstate(nst, 18) = -sxx(1,5)
xstate(nst, 17) = -sxx(1,6)
xstate(nst, 16) = -sxx(1,7)
xstate(nst, 15) = -sxx(1,8)
xstate(nst, 14) = -sxx(1,9)
xstate(nst, 13) = -sxx(1,10)
xstate(nst, 12) = -sxx(1,11)
xstate(nst, 11) = -sxx(1,12)
xstate(nst, 10) = -sxx(1,13)
xstate(nst, 9) = -sxx(1,14)
xstate(nst, 8) = -sxx(1,15)
xstate(nst, 7) = -sxx(1,16)
xstate(nst, 6) = -sxx(1,17)
xstate(nst, 5) = -sxx(1,18)
xstate(nst, 4) = -sxx(1,19)
xstate(nst, 3) = -sxx(1,20)
xstate(nst, 2) = -sxx(1,21)
xtable(nst,1) = -sxx(1,22)
xtable(nst,23) = shrat

count=i
exit
endif
endif
end_loop
end

set large
struct prop=l E=ewall I=iwall area=awall
def nnodelem
no_a = 1
elemab = 50
no_b = no_a + elemab
elembc = 5
no_c = no_b + elembc
gabcd = 10
no_d = no_c + gabcd
elemde = 4
no_e = no_d + elemde
elemef = 85
no_f = no_e + elemef
elemfg = 28
no_g = no_f + elemfg
end

nnodelem
struc node no_a 0., hwall
struc node no_b 0., bed pin fix y
struc node no_c bwall, bed
struc node no_d 0, bed1 fix x y
struc node no_e 0, 0 fix x y
struc node no_f lsoil, 0 fix x y
struc node no_g lsoil, hwall fix x y
struc beam beg node no_a end node no_b seg=elemab pr=l
struc beam beg node no_b end node no_c seg=elembc pr=l
struc beam beg node no_d end node no_e seg=elemde pr=l
struc beam beg node no_e end node no_f seg=elemef pr=l
struc beam beg node no_f end node no_g seg=elemfg pr=l
struc node 1 30 fix y
free x i=1
free x i=73
free y j=1

; INTERFACE
interface 1 aside from node no_a to node no_b bside from 3,23 to 1,3
interface 2 aside from node no_b to node no_c bside from 1,11 to 1,2
interface 3 aside from node no d to node no e bside from 1,11 to 1,1
interface 4 aside from node no e to node no f bside from 1,1 to 73,1
interface 5 aside from node no f to node no g bside from 73,1 to 73,23
def knksinter
dzmin1235=lsoil/izones
dzmin4=bed/4. ; height of lower portion
knks1=10*(bulk_mod(1,3)+4*shear_mod(1,3)/3)/dzmin1235
knks2=10*(bulk_mod(1,2)+4*shear_mod(1,2)/3)/dzmin1235
knks3=10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin1235
knks4=10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin4
knks5=10*(bulk_mod(izones,1)+4*shear_mod(izones,1)/3)/dzmin1235 ; fbase=fdirsh
command
interface 1 friction=fwall kn=knks1 ks=knks1 tbond=0 sbr=0 bslip on
interface 2 friction=0. kn=knks2 ks=knks2 tbond=0 sbr=0 bslip on
interface 3 friction=fwall kn=knks3 ks=knks3 tbond=0 sbr=0 bslip on
interface 4 friction=fbase kn=knks4 ks=knks4 tbond=0 sbr=0 bslip on
interface 5 friction=fwall kn=knks5 ks=knks5 tbond=0 sbr=0 bslip on
end_command
end
knksinter
window
def tab dila ;correcting value of dilation angle as function of relative density
lowsig3=4903. ; lowest sig3 (Pa) data =0.05 kg/cm2
higsig3=392266. ; highest sig3 (Pa) data =4.0 kg/cm2
loodr=54.9075 ; average lowest data of Dr of Tatsuoka
dendr=79.4167 ; average highest data of Dr
nnd=20+izones*jzones
dila=((14.-7.)/(dendr-loodr))*(reldens-loodr)+7. ; for maximum dilation angle
loop m (1,izones)
loop n (1,jzones)
sigma3=abs(ex_2(m,n))
dendilx2=((0.020 - 0.006)/(higsig3-lowsig3))*(sigma3-lowsig3)+0.006
loodilx2=((0.031 - 0.010)/(higsig3-lowsig3))*(sigma3-lowsig3)+0.010
dilx2=((dendilx2-loodilx2)/(dendr-loodr))*(reldens-loodr)+loodilx2
dilres=((l.- 4.)/(higsig3-lowsig3))*(sigma3-lowsig3)+4.
dilyl=((dila)/(0.02))*(-dilx2)+0.
nnd=nnd+1
xtable(nnd,1)=0.
ytable(nnd,1)=dily1
xtable(nnd,2)=0.02+dilx2
ytable(nnd,2)=dila
xtable(nnd,3)=0.06+dilx2
ytable(nnd,3)=dilres
command
prop dtab nnd i=m j=n
end_command
end_loop ;n
end_loop ;m
end
def tab friction ; creating friction table for each zone=
nnn=20
td_dr=80.56 ;Dr from tatsuoka, average dense
tl_dr=52.78 ;Dr from tatsuoka, average loose
minsig3=9.8 ; the lowest allowed in the log equation
lowsig3=4903.325 ;lowest sig3 (Pa),tatsuoka
higsig3=392266. ;highest sig3 (Pa),tatsuoka
ld_dr=79.278 ; DR dense at low sig3
ll_dr=55.278 ; DR loose at low sig3,
hd_dr=79.556 ; DR dense at high sig3,
hl_dr=54.611 ; DR loose at high sig3,
loop m (l,izones) ; apply the operations to all elements
loop n (ljzones)
nnn=nnn+l
  dangle=ex_l (m,n)
sigma3=abs(ex_2(m,n)) ; obtain the current sigma 3
  ^epres^tableO 1, dangle) ; low sig3 dense at delta
  ll_epres=table(12, dangle) ; low sig3 loose at delta
  lo_epres=((ll_epres-ld_epres)/(ll_dr-ld_dr))*(reldens-ld_dr)+ld_epres
  hd_epres=table(13, dangle) ; high sig3 dense at delta
  hl_epres=table(14, dangle) ; high sig3 loose at delta
  hi_epres=((hl_epres-hd_epres)/(hl_dr-hd_dr))*(reldens-hd_dr)+hd_epres
  sigma3=abs(ex_2(m,n)) ; obtain the current sigma 3
epsres1=((hi_epres-lo_epres)/(higsig3 -lowsig3)) * (sigma3 -lowsig3 )+lo_epres ;
epsres=epsres1*e1resatd90 ; ratio of peak friction at dangle to peak of dangle=90 deg
if dangle <= datmin then
  fpeakrat = ca + ax1*dangle + ax2*(dangle)^2 + ax3*(dangle)^3
else
  fpeakrat = cb + bx1*dangle + bx2*(dangle)^2 + bx3*(dangle)^3
endif ; model effect-------
if sigma3 <= minsig3 then
  sigma3=minsig3
endif
cfdirsh = afdirsh*ln(sigma3) + bfdirsh ; correction to fdirsh function of Dr and sigma3
fpeak=fpeakrat*(fplst)*cfdirsh ; fpeak of current dangle; fplst = angle - plane strain
ld_epeak=table(7,dangle) ; low sig3 dense at delta change to decimal
ll_epeak=table(8,dangle) ; low sig3 loose at delta
lo_epeak=((ll_epeak-ld_epeak)/(ll_dr-ld_dr))+ld_epeak
hd_epeak=table(9,dangle) ; high sig3 dense at delta
hl_epeak=table(10,dangle) ; high sig3 loose at delta
hi_epeak=((hl_epeak-hd_epeak)/(hl_dr-hd_dr))*(reldens-hd_dr)+hd_epeak
epspeak1=((hi_epeak-lo_epeak)/(higsig3-lowsig3))*(sigma3-lowsig3)+lo_epeak
epspeak=epspeak1*e1peakatd90

; model effect-
cfres = afres*ln(sigma3) + bfres ; the same for all Dr
fresl=fres*cfres
if fpeak <= fresl then
  fpeak=fresl
  epspeak=epsres
endif
if epsres <= epspeak then
  epsres=epspeak
  fpeak=fresl
endif

itab1=table_size(1)
itab2=table_size(2)
loop i (1,itab1) ; before peak
  xtable(nnn,i)=xtable(1,i)*epspeak
  ytable(nnn,i)=ytable(1,i)*fpeak
end_loop
loop i (1,itab2) ; after peak
  ccc=itab1+i
  xtable(nnn,ccc)=xtable(2,i)*(epsres-epspeak)+epspeak
  ytable(nnn,ccc)=ytable(2,i)*(fpeak-fresl)+fresl
end_loop
command
  prop ftab nnn i=m j=n
end_command
end_loop ;n
end_loop ;m
end

; curvature========================================
tab 1 0.0,0.05,0.433 0.1,0.599 0.2,0.74 0.3,0.818 0.4,0.8708 ; curvature
  tab 1 0.5,0.9145 0.6,0.9426 0.7,0.9633 0.8,0.98 0.9,0.9931 1,1.  ; 0 to peak ;
tab 2 0.1,0.9689 0.2,0.8782 0.4,0.581 0.6,0.3132 0.8,0.101 0.9,0.041 1,0. ; peak -residu
  ; ----------- e1peakatd/elpeakatd90
  tab 7 0.,1.647059 23.,1.441176 34.,0.882353 45.,1.058824 90.,1. ;dense Dr=0.7927778
  ;low sig3=4903.325 Pa
  tab 8 0.,2.111111 23.,1.355556 34.,1.511111 45.,1.410714 90.,1. ; loose Dr=0.55278056
  ;low sig3=4903.325 Pa
  tab 9 0.,1.295082 23.,1.065574 34.,0.918033 45.,1.131148 90.,1. ; dense Dr=0.7955556
  ; high sig3=3392266 Pa
def movethewall
    shrat = 0.0
    xvtop = 2.5e-7*hsoil/0.5
    loop n (1,1500) ;*****
        loop m (1,400) ;==
            command
                struc node range no_a no b initial xvel xvtop ;;m/time step
                step 1
                end_command
            endloop ;== loop m
            createmovie1
            createmovie2
            if shrat>=maxshrat
                exit
            endif
        endloop ;***** loop n
    end

def createmovie1 ; shear strain fill structure
    command
        window
        movie on file 9maxssi.dcx size 1080,670
        plot grid mag=1 green beam mag=1 yellow ssi fill interval 0.1
        movie off
        window
    end_command
end

def createmovie2 ; principle stress
    xlow = -0.05
    xupp = hsoil/0.5*1.
    ylow = -0.05
    yupp = hsoil/0.5*0.9171
    command
        wind xlow,xupp ylow,yupp

movie on file 9prinstr.dcx size 1080,670
plot grid mag=1 iw beam mag=1 red stress
movie off
window
end_command
end
movethewall
plot hist 2 vs 1 ; unbalance vs s/h ratio
copy 3unbalnc.pcx
pl hist 62 61 60 59 58 57 56 55 54 53 52 51 50 49 48 47 46 45 44 43 42 41 vs 1;
;horizontal stress
copy 1horstrs.pcx ; spt0
plot table 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42
-copy 4tabfric.pcx
plot hist 3 vs 1
copy 5eplast_shrat.pcx
plot hist 41 42 43 44 45 46 vs 1
copy 6botstress.pcx
plot grid mag=1 iw beam mag=1 yellow ssi fill interval 0.1
-copy 2prnstrs.pcx
plot gridmag=l iwbeammag=l yellow pi
-copy 3plastic.pcx
wind xlow,xupp ylow,yupp
plot grid mag=1 iw beam mag=1 red stress
-copy 2prnstrs.pcx
window
; FILE 8fang_result.txt
set log 8fang_result.txt ; printing properties
set log on
print table 20
def cetak
nst=20+2*izones*jzones ;+noshrat ; stress result
loop j (l,anoshrat)
   nst=nst+1
   command
      print table nst
   end_command
end_loop :j
end
cetak
set log off
return
Example for Passive Translation in Anisotropic Soil

title
z08tn4mand80.dat:H=4.0m,trans,Dr=80%
set cust1
Dept.of Civil and Env. Eng.
set cust2
Old Dominion U.,Norfolk, Virginia
config extra 2
set echo off
grid 72 22
mod ss
set grav=9.81 ;m/s2
def dimension
;soil------
hsoil = 4.0 ; INPUT, depth of soil adjacent to moving wall (m)
n = 7e12 ; INPUT, rotation about point at nH (m) below wall base
dsoil = 0.613*hsoil/0.5 ; soil depth (m), the same proportion with Fang's experiment
lsoil = 2.0*hsoil/0.5 ; soil length (m)
bed = 0.113*hsoil/0.5 ; bed depth (m)
bed1 = 0.99999*bed
nh = n*hsoil ; nH (m)
;walls------
hwall = 0.8*hsoil/0.5 ; wall top to soil bottom (m)
hmwall = hwall-bed ; height of movable wall
twall = 0.12*hsoil/0.5 ; wall thickness (m)
awall = twall*1. ; area of movable wall in (m2)
iwall = 1*(twall^3)/12. ; moment area of the wall (m4)
ewall = 200e9 ; assuming steel (Pa)
bwall = -0.133*hsoil/0.5 ; coordinate of front edge of wall base
maxshrat = 0.15 ; maximum ratio of horizontal movement to soil depth
end
dimension
def backfill
; Ottawa sand used in Fang test
gs = 2.65 ; INPUT, specific gravity
eemax = 0.76 ; INPUT, maximum void ratio
eemin = 0.50 ; INPUT, minimum void ratio
treldens = 80. ; INPUT, relative density (%)
; coefficients for variation ratio R(d)
ca = 0.888930072 ; INPUT for Dr = 80
ax1 = 1.02003E-05
ax2 = -7.33196E-05
ax3 = -1.03025E-07
cb = 0.681033805
bx1 = 0.006882165
bx2 = -3.54747E-05
bx3 = -1.81561E-08

; for fdirsh, cfdirsh = afdirsh ln(sigma3) + bfdirsh ; function of Dr
afdirsh = -0.016789 ; for DR=80
bfdirsh = 1.216241

; for fres, cfres = afres ln(sigma3) + bfres; the same for all Dr
afres = -0.0121220 ; for all DR
bfres = 1.1561240

; walsanrat=-0.000025502*(reldens)^2+0.00385617*reldens+0.187259946
gamwater = 9.81*1000 ; unitweight of water N/m3
ee=eemax-relens*(eemax-eemin)/100. ; void ratio
fdirsh = 0.000275*(relens)^2+0.184275*relens+25.601176 ; direct shear
fres=31.5 ; residual DS, independent of test type

; Minimum ratio R(d) for a given Dr
cr = 0.96975756535439
cd1 = -2.687556E-03
cd2 = 1.147410E-05
cd3 = 1.618853E-08

minrat = cr + cd1*relens + cd2*(relens)^2 + cd3*(relens)^3 ;
fplst = fdirsh/minrat ; fpeak at d=90

; delta at Minimum ratio R(d) for a given Dr
cdr = 50.6741218848
cdd1 = -2.319605E-01
cdd2 = -3.303401E-03
cdd3 = 3.013308E-05
datmin = cdr + cdd1*relens + cdd2*(relens)^2 + cdd3*(relens)^3 ;
elpeakatd90=(-0.015207*relens+3.403184)/100. ; e1peak for given density at d=90;
elresatd90=(-0.081055*relens+11.346580)/100. ; e1res for given density at d=90;

unitwei = gs*gamwater/(ee+1) ; unitweight (N/m3)

fwall = walsanrat*fdirsh ; wall friction

end

backfill

gen 0 0 0 bed lsoil bed lsoil 0 i=1,73 j=1,5

gen 0 bed 0 dsoil lsoil dsoil lsoil bed i=1,73 j=5,23

def setprop

float zz sigm gg kk pois yc

float ko
dila=((-14.-7.)/(79.4167-54.9075))*(relens-54.9075)+7. ; for maximum dilation angle
dilaat0=-dila/2. ; negative start at zero
pa_psf=0.020885 ; conversion pascal to psf
psi_pa=6894.75728 ; conversion psi to pascal
pa = 101300 ; atmospheric pressure in N/m2 (=Pa)

sss = 1350 ;
loop i (1,izones)
  loop j (1, jzones)
    yc = (y(i,j)+y(i+1,j)+y(i,j+1)+y(i+1,j+1))/4.
    zz = y(i,jgp) - yc
    if fplst > 34 then
      triax = (fplst + 17.)/1.5
    else
      triax = fplst
    endif
    ko = 1-sin(triax*degrad) ; to be used in calculating average sigma
    pois=(1-sin(triax*degrad))/(2-sin(triax*degrad)) ;
    sigm=(unitwei*zz*(1.+2.*ko)/3.) ; average stress (Pascal)
    gg=-(sxx(i,j)+syy(i,j))/2. ; change sign to positive
    ccc=(sxx(i,j)-syy(i,j))/2. ; always positive
    bbb=((ccc)^2 + (sxy(i,j))^2)^0.5 ; always positive
    sigma1=aaa + bbb ; sig1 since this is larger, always positive
    sigma3=aaa - bbb ; sig3 since this is lower
    ex_2(i,j)=sigma3 ; will be used for evaluating confined pressure
    if sigma1 # abs(sxx(i,j)) then
      chi=atan(sxy(i,j)/(sigma1-abs(sxx(i,j))))
    else
      chi=90.*degrad
endif
ex_l(i,j)=90-abs(chi/degrad)
end_loop
end_loop
end
solve
plot grid esyy fill
copy 0gravsyy.pcx
plot grid esxx fill
copy 0gravsxx.pcx
def dcl_22
dc22 = (dsoil - (y(1,22)+y(2,22)+y(1,23)+y(2,23))/4.)
dc21 = (dsoil - (y(1,21)+y(2,21)+y(1,22)+y(2,22))/4.)
dc20 = (dsoil - (y(1,20)+y(2,20)+y(1,21)+y(2,21))/4.)
dc19 = (dsoil - (y(1,19)+y(2,19)+y(1,20)+y(2,20))/4.)
dc18 = (dsoil - (y(1,18)+y(2,18)+y(1,19)+y(2,19))/4.)
dc17 = (dsoil - (y(1,17)+y(2,17)+y(1,18)+y(2,18))/4.)
dc16 = (dsoil - (y(1,16)+y(2,16)+y(1,17)+y(2,17))/4.)
dc15 = (dsoil - (y(1,15)+y(2,15)+y(1,16)+y(2,16))/4.)
dc14 = (dsoil - (y(1,14)+y(2,14)+y(1,15)+y(2,15))/4.)
dc13 = (dsoil - (y(1,13)+y(2,13)+y(1,14)+y(2,14))/4.)
dc12 = (dsoil - (y(1,12)+y(2,12)+y(1,13)+y(2,13))/4.)
dc11 = (dsoil - (y(1,11)+y(2,11)+y(1,12)+y(2,12))/4.)
dc10 = (dsoil - (y(1,10)+y(2,10)+y(1,11)+y(2,11))/4.)
dc9 = (dsoil - (y(1,9)+y(2,9)+y(1,10)+y(2,10))/4.)
dc8 = (dsoil - (y(1,8)+y(2,8)+y(1,9)+y(2,9))/4.)
dc7 = (dsoil - (y(1,7)+y(2,7)+y(1,8)+y(2,8))/4.)
dc6 = (dsoil - (y(1,6)+y(2,6)+y(1,7)+y(2,7))/4.)
dc5 = (dsoil - (y(1,5)+y(2,5)+y(1,6)+y(2,6))/4.)
dc4 = (dsoil - (y(1,4)+y(2,4)+y(1,5)+y(2,5))/4.)
dc3 = (dsoil - (y(1,3)+y(2,3)+y(1,4)+y(2,4))/4.)
dc2 = (dsoil - (y(1,2)+y(2,2)+y(1,3)+y(2,3))/4.)
dc1 = (dsoil - (y(1,1)+y(2,1)+y(1,2)+y(2,2))/4.)
end
ini xdis=0. ydis=0. ; reset displacement after gravity
prope_plastic = 0. ; reset plastic strain
def tiltangle
ytop = y(1,23)
ymid = y(1,14)
ybot = y(1,5)
smaxrat = (nh+hsoil)/(ymid-ybot+nh) ; max disp at soil surface if disp at (1,11)=1
xvbotrat = nh/(hsoil+nh) ; velocity at the wall bottom if velocity at soil surface =1
xvtoprat = (hmwall+nh)/(hsoil+nh) ; velo at the wall top if vel at soil surface =1
end
tiltangle
def shrat
    float shrat
    while_stepping
        shrat = xdisp(1,14)/hsoil ; displ at soil surface/(height of backfill above wall base)
    end
    hist 1 nstep= 2000 shrat
    hist 2 unbalance
    hist 3 e_plastic i=1 j=14
    hist 41 sxx i=1 j=1 ; sept 16,
    hist 42 sxx i=1 j=2
    hist 43 sxx i=1 j=3
    hist 44 sxx i=1 j=4
    hist 45 sxx i=1 j=5
    hist 46 sxx i=1 j=6
    hist 47 sxx i=1 j=7
    hist 48 sxx i=1 j=8
    hist 49 sxx i=1 j=9
    hist 50 sxx i=1 j=10
    hist 51 sxx i=1 j=11
    hist 52 sxx i=1 j=12
    hist 53 sxx i=1 j=13
    hist 54 sxx i=1 j=14
    hist 55 sxx i=1 j=15
    hist 56 sxx i=1 j=16
    hist 57 sxx i=1 j=17
    hist 58 sxx i=1 j=18
    hist 59 sxx i=1 j=19
    hist 60 sxx i=1 j=20
    hist 61 sxx i=1 j=21
    hist 62 sxx i=1 j=22
    def fildata
        array adatshrat(17) ;datshrat(7)
        anoshrat=17
        nns=20+2*izones*jzones ;
        adatshrat(1) = 0.0 ;
        adatshrat(2) = 0.001 ;
        adatshrat(3) = 0.005 ;
        adatshrat(4) = 0.010 ;
        adatshrat(5) = 0.020 ;
        adatshrat(6) = 0.030 ;
        adatshrat(7) = 0.040 ;
        adatshrat(8) = 0.050 ;
        adatshrat(9) = 0.060 ;
        adatshrat(10) = 0.070 ;
        adatshrat(11) = 0.080 ;
adatshr(12) = 0.090 ;
adatshr(13) = 0.100 ;
adatshr(14) = 0.110 ;
adatshr(15) = 0.120 ;
adatshr(16) = 0.130 ;
adatshr(17) = maxshr ;
end

fildata
def scanstress
    int count
    while stepping
        nst=20+2*izones*jzones ;+noshrat
    loop i (1,noshrat)
        if i > count then
            if shrat >= adatshr(i)
                dc1_22
                nst=nst+i
                ytable(nst,22) = dc1
                ytable(nst,21) = dc2
                ytable(nst,20) = dc3
                ytable(nst,19) = dc4
                ytable(nst,18) = dc5
                ytable(nst,17) = dc6
                ytable(nst,16) = dc7
                ytable(nst,15) = dc8
                ytable(nst,14) = dc9
                ytable(nst,13) = dc10
                ytable(nst,12) = dc11
                ytable(nst,11) = dc12
                ytable(nst,10) = dc13
                ytable(nst,9) = dc14
                ytable(nst,8) = dc15
                ytable(nst,7) = dc16
                ytable(nst,6) = dc17
                ytable(nst,5) = dc18
                ytable(nst,4) = dc19
                ytable(nst,3) = dc20
                ytable(nst,2) = dc21
                ytable(nst,1) = dc22
                ytable(nst,23) = shrat
                xtable(nst,22) = -sxx(1,1)
                xtable(nst,21) = -sxx(1,2)
                xtable(nst,20) = -sxx(1,3)
                xtable(nst,19) = -sxx(1,4)
                xtable(nst,18) = -sxx(1,5)
xtable(nst, 17) = -sxx(1,6)
xtable(nst, 16) = -sxx(1,7)
xtable(nst, 15) = -sxx(1,8)
xtable(nst, 14) = -sxx(1,9)
xtable(nst, 13) = -sxx(1,10)
xtable(nst, 12) = -sxx(1,11)
xtable(nst, 11) = -sxx(1,12)
xtable(nst, 10) = -sxx(1,13)
xtable(nst, 9) = -sxx(1,14)
xtable(nst, 8) = -sxx(1,15)
xtable(nst, 7) = -sxx(1,16)
xtable(nst, 6) = -sxx(1,17)
xtable(nst, 5) = -sxx(1,18)
xtable(nst, 4) = -sxx(1,19)
xtable(nst, 3) = -sxx(1,20)
xtable(nst, 2) = -sxx(1,21)
xtable(nst, 1) = -sxx(1,22)
xtable(nst, 23) = shrat

count = i

exit
endif
endif

end_loop
end

set large

struct prop = 1 E = ewall I = iwall area = awall
def nnod elem

no_a = 1
elemab = 50
no_b = no_a + elemab
elembc = 5
no_c = no_b + elembc
gabcd = 10
no_d = no_c + gabcd
elemde = 4
no_e = no_d + elemde
elemef = 85
no_f = no_e + elemef
elemfg = 28
no_g = no_f + elemfg
end

nnod elem
struc node no_a 0., hwall
struc node no_b 0., bed pin fix y
struc node no_c bwall, bed
struc node no_d   0,  bed1  fix x y
struc node no_e   0,  0  fix x y
struc node no_f | lsoil,  0  fix x y
struc node no_g | lsoil,  hwall  fix x y
struc beam beg node no_a end node no_b seg=elemab pr=l
struc beam beg node no_b end node no_c seg=elembc pr=l
struc beam beg node no_d end node no_e seg=elemdc pr=l
struc beam beg node no_e end node no_f seg=elemed pr=l
struc beam beg node no_f end node no_g seg=elemef pr=l
struc node 1 30 fix y
free x i=1
free x i=73
free y j=1
interface 1 aside from node no_a to node no_b bside from 3,23 to 1,3
interface 2 aside from node no_b to node no_c bside from 1,11 to 1,2
interface 3 aside from node no_d to node no_e bside from 1,11 to 1,1
interface 4 aside from node no_e to node no_f bside from 1,1 to 73,1
interface 5 aside from node no_f to node no_g bside from 73,1 to 73,23

def knksinter
dzmin1235=lsoil/izones
dzmin4=bed/4. ; height of lower portion
knks1=10*(bulk_mod(1,3)+4*shear_mod(1,3)/3)/dzmin1235
knks2=10*(bulk_mod(1,2)+4*shear_mod(1,2)/3)/dzmin1235
knks3=10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin1235
knks4=10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin4
knks5=10*(bulk_mod(izones,1)+4*shear_mod(izones,1)/3)/dzmin1235 ;
fbase=fdirsh

command
interface 1 friction=fwall kn=knks1 ks=knks1 tbond=0 sbr=0 bslip on
interface 2 friction=0.  kn=knks2 ks=knks2 tbond=0 sbr=0 bslip on
interface 3 friction=fwall kn=knks3 ks=knks3 tbond=0 sbr=0 bslip on
interface 4 friction=fbase kn=knks4 ks=knks4 tbond=0 sbr=0 bslip on
interface 5 friction=fwall kn=knks5 ks=knks5 tbond=0 sbr=0 bslip on
end_command

end

knksinter

window

def tab_dila  ;correcting value of dilation angle as function of relative density
lowsig3=4903. ;lowest sig3 (Pa) data =0.05 kg/cm2
higsig3=392266. ;highest sig3 (Pa) data =4.0 kg/cm2
loodr=54.9075 ;average lowest data of Dr of Tatsuoka
dendr=79.4167  ;average highest data of Dr
nnnd=20+izones*jzones
dila=((14.-7.)/(dendr-loodr))*(reldens-loodr)+7. ; maximum dilation angle
loop m (1,izones)
loop n (1,jzones)
sigma3=abs(ex_2(m,n))
dendilx2=((0.020 - 0.006)/(higsig3-lowsig3))*(sigma3-lowsig3)+0.006
loodilx2=((0.031 - 0.010)/(higsig3-lowsig3))*(sigma3-lowsig3)+0.010
dilx2=((dendilx2-loodilx2)/(dendr-loodr))*(reldens-loodr)+loodilx2
dilres=((l.- 4.)/(higsig3-lowsig3))*(sigma3-lowsig3)+4.
dily1=((dila)/(0.02))*(-dilx2)+0.
nnd=nnd+1
xtable(nnd,1)=0.
ytable(nnd,1)=dily1
xtable(nnd,2)=dila
xtable(nnd,3)=dilres
command
 prop dtab nnd i=m j=n
end_command
end_loop ;n
end_loop ;m
def tabfriction ; !creating friction table for each zone==
nnn=20
td_dr=80.56 ;Dr from tatsuoka, average dense (3)tatsuoka_Fig3_1990
tl_dr=52.78 ;Dr from tatsuoka, average loose (3)tatsuoka_Fig3_1990
minsig3=9.8 ; the lowest allowed in the log equation
lowsig3=4903.325 ;lowest sig3 (Pa) ,tatsuoka
higsig3=392266. ;highest sig3 (Pa) ,tatsuoka
ld_dr=79.278 ; DR dense at low sig3,(1)epeak_res_july23
ll_dr=55.278 ; DR loose at low sig3,
hd_dr=79.556 ; DR dense at high sig3,
hl_dr=54.611 ; DR loose at high sig3,
loop m (1,izones) ; apply the operations to all elements
loop n (1,jzones)
nnn=nnn+1
dangle=ex_1(m,n)
sigma3=abs(ex_2(m,n)) ; obtain the current sigma 3
ld_epres=table(11,dangle) ; low sig3 dense at delta
ll_epres=table(12,dangle) ; low sig3 loose at delta
lo_epres=(((ll_epres-ld_epres)/(ll_dr-ld_dr))*(reldens-ld_dr)+ld_epres
hd_epres=table(13,dangle) ; high sig3 dense at delta
hl_epres=table(14,dangle) ; high sig3 loose at delta
hi_epres=(((hi_epres-lo_epres)/(hl_dr-hl_dr))*(reldens-hl_dr)+hl_epres
sigma3=abs(ex_2(m,n)) ; obtain the current sigma 3
epsres1=(((hi_epres-lo_epres)/(higsig3-lowsig3))*(sigma3-lowsig3)+lo_epres
epsres=epsres1*e1resatd90
if dangle <= datmin then
    fpeakrat = ca + ax1*dangle + ax2*(dangle)^2 + ax3*(dangle)^3
else
    fpeakrat = cb + bx1*dangle + bx2*(dangle)^2 + bx3*(dangle)^3
endif
if sigma3 <= minsig3 then
    sigma3=minsig3
endif
cfdirsh = afdirsh*ln(sigma3) + bfdirsh ; correction to fdirsh function of Dr and sigma3;
fpeak=fpeakrat*(fplst)*cfdirsh  ; fpeak of current dangle; fplst = plane strain
ld_epeak=table(7,dangle) ; low sig3 dense at delta change to decimal
ll_epeak=table(8,dangle) ; low sig3 loose at delta
lo_epeak=((ll_epeak-ld_epeak)/(ll_dr-ld_dr))*(reldens-ld_dr)+ld_epeak
hd_epeak=table(9,dangle) ; high sig3 dense at delta
hl_epeak=table(10,dangle) ; high sig3 loose at delta
hi_epeak=((hl_epeak-hd_epeak)/(hl_dr-hd_dr))*(reldens-hd_dr)+hd_epeak
epspeak1=((hi_epeak-lo_epeak)/(higsig3 -lo wsig3 ))* (sigma3 -lo wsig3 )+lo_epeak
epspeak=epspeak1*e1peakatd90
cfres = afres*ln(sigma3) + bfres ; the same for all Dr
fres1=fres*cfres
if fpeak <= fres1 then
    fpeak=fres1
    epspeak=epsres
endif
if epsres <= epspeak then
    epsres=epspeak
    fpeak=fres1
endif
itab1=table_size(1)
itab2=table_size(2)
loop i (1,itab1) ; before peak
    xtable(nnn,i)=xtable(l ,i)*epspeak
    ytable(nnn,i)=ytable( 1 ,i)*fpeak
end_loop
loop i (l,itab2)  ; after peak
    ccc=itabl+i
    xtable(nnn,ccc)=xtable(2,i)*(epsres-epspeak)+epspeak
    ytable(nnn,ccc)=ytable(2,i)* (fpeak-fres1 )+fres1
endloop
command
    prop ftab nnn i=m j=n
end_command
end loop ;n
end_loop ;m
end
Example for Passive RBT in Isotropic Soil

title
rb02isf35di05w028n00m18.dat: Passive H=4.0m,Dr=60%,n=0,G=22.5Mpa, K=60Mpa, ;dil=0.5f,f=35,d/f=0.28, RBT
set cust1
Dept.of Civil and Env. Eng.
set cust2
Old Dominion U.,Norfolk,Virginia
config extra 2
set echo off
grid 72 22
mod ss
set grav=9.81 ;m/s2
def dimension
  hsoil = 4.0 ; INPUT, depth of soil adjacent to moving wall (m)
  n = 0.0 ; INPUT, rotation about point at nH (m) below wall base
  dsoil = 0.613*hsoil/0.5 ; soil depth (m), the same proportion with Fang's experiment
  lsoil = 2.0*hsoil/0.5 ; soil length (m)
  bed = 0.113*hsoil/0.5 ; bed depth (m)
  bedl = 0.999*bed
  nh = n*hsoil ; nH (m)
  hwall = 0.8*hsoil/0.5 ; wall top to soil bottom (m)
  hmwall = hwall-bed ; height of movable wall
  twall = 0.12*hsoil/0.5 ; wall thickness (m)
  awall = twall^1. ; area of movable wall in (m2)
  iwall = 1*(twall^3)/12. ; moment area of the wall (m4)
  ewall = 200e9 ; assuming steel (Pa)
  bwall = -0.133*hsoil/0.5 ; coordinate of front edge of wall base
end
def backfill
gs = 2.65 ; INPUT, specific gravity
eemax = 0.76 ; INPUT, maximum void ratio
eemin = 0.50 ; INPUT, minimum void ratio
reldens = 70. ; INPUT, relative density (%)
ca = 0.892993869350 ; INPUT for Dr = 70
ax1 = -9.023847E-05
ax2 = -4.930927E-05
ax3 = -2.681383E-07
cb = 0.65370450092769
bx1 = 8.094700E-03
bx2 = -5.305446E-05
bx3 = 6.510364E-08
afdirsh = -0.014550 ; for DR=70
bfdirsh = 1.187405
afres = -0.0121220 ; for all DR
bfres = 1.1561240
walsanrat=-0.000025502*(reldens)^2+0.00385617*reldens+0.187259946 ;
gamwater = 9.81*1000 ; unit weight of water N/m3
ee=eemax-reldens*(eemax-eemin)/100. ; void ratio
fdirsh = 0.000275*(reldens)^2+0.184275*reldens+25.601176; direct shear angle -
fres=31.5 ; residual angle of friction from DS,
cr = 0.96975756535439
cd1 = -2.687556E-03
cd2 = 1.147410E-05
cd3 = 1.618853E-08
minrat = cr + cd1*reldens + cd2*(reldens)^2 + cd3*(reldens)^3 ; minimum
fpeak@d/fpeak@d=90,
    fplst = fdirsh/minrat ; fpeak at d=90
    cdr = 50.6741218848
    cdd1 = -2.319605E-01
    cdd2 = -3.303401E-03
    cdd3 = 3.013308E-05
    datmin = cdr + cdd1*reldens + cdd2*(reldens)^2 + cdd3*(reldens)^3 ; minimum
    fpeak@d/fpeak@d=90,
    elpeakatd90=(-0.015207*reldens+3.403184)/100. ; elpeak for given density at d=90;
    elresatd90=(-0.081055*reldens+11.346580)/100. ; elres for given density at d=90;
    unitwei = gs*gamwater/(ee+1) ; unitweight (N/m3)
fwall = walsanrat*fdirsh ; wall friction
    angle_used = 35. ; fplst
    fwall_soil= 0.28 ; ratio of d wall to soil friction
    unitwei = 20000 ; N/m3
    scale4fres=fres/fplst
    fplst = angle_used
    fwall = fwall_soil*fplst
    fres = angle_used
    fdirsh = angle_used
end
backfill
gen 0 0 0 bed lsoil bed lsoil 0 i=1,73 j=1,5
gen 0 bed 0 dsoil lsoil dsoil lsoil bed i=1,73 j=5,23
def setprop
    float zz sigm gg kk pois yc
    float ko
    dila=((14.-7.)/(79.4167-54.9075))*(reldens-54.9075)+7. ; maximum dilation angle
    dilaat0=-dila/2. ; negative start at zero
    pa_psf=0.020885 ; conversion pascal to psf
    psi_pa=6894.75728 ; conversion psi to pascal
    pa = 101300 ; atmospheric pressure in N/m2 (=Pa)
    sss = 1350 ; stiffness coefficient Hardin 1978 Earthquake eng n soil dynamics p3-90
loop i (1,izones)
    loop j (1,jzones)
        yc = (y(i,j)+y(i+1,j)+y(i,j+1)+y(i+1,j+1))/4.
        zz = y(1,jgp) - yc
        if fplst > 34 then
            ftriax = (fplst + 17.)/1.5
        else
            ftriax = fplst
        endif
        ko = 1-sin(ftriax*degrad) ; to be used in calculating average sigma
pois=(1-sin(ftriax*degrad))/(2-sin(ftriax*degrad)) ; Ko=1-sin f = pois/(1-poiss)

$\text{sigm}=(\text{unitwei} \cdot \text{zz} \cdot (1.+2. \cdot \text{ko})/3.)$ ; average stress (Pascal)

$\text{gg}=((\text{sss} \cdot 1. \cdot (\text{sigm} \cdot \text{pa})^0.5)/(2*(1+\text{pois})*(0.3+0.7*\text{ee}^2)) \};$ in N/m²

$\text{kk}=2.*\text{gg}*(1.+\text{pois})/(3.*(1.-2.*\text{pois}))$ ; initial K

$\text{shear\_mod}(i,j)=\text{gg}$
$\text{bulk\_mod}(i,j)=\text{kk}$
$\text{density}(i,j)=\text{unitwei}/9.81$ ; kg/m³
$\text{cohesion}(i,j)=0.$
$\text{tension}(i,j)=0.$
$\text{shear\_mod}(i,j)=22.5e6$
$\text{bulk\_mod}(i,j)=60.0e6$
$\text{density}(i,j)=20000./9.81$ ; kg/m³

endloop
endloop

$\text{dila}=0.5*\text{angle\_used}$
$\text{dila0}=0.5*\text{angle\_used}$
end

setprop

$\text{tab} 15 0.,\text{dila}0 0.03,\text{dila} 0.07,\text{dila}$ ; constant
$\text{tab} 16 0.,\text{fplst} 0.2,\text{fplst}$ ; initial value of friction
$\text{tab} 20 \text{hsoil},\text{n reldens},\text{unitwei} \text{fplst},\text{pois} \text{fdirsh},\text{fwall}$

$\text{prop} \text{ dtab} 15$
$\text{prop} \text{ ftab} 16$

$\text{fixyj}=\text{l}$ ;bottom
$\text{fixx i}=73$ ;right wall
$\text{fixx i}=\text{l}$ ;left wall

set plot pex

def angle_pq ; angle and pq for all elements

loop i (1,izones)
  loop j (1,jzones)
    $\text{aaa}=-(\text{sxx}(i,j)+\text{syy}(i,j))/2.$ ; change sign to positive
    $\text{ccc}=(\text{sxx}(i,j)-\text{syy}(i,j))/2.$
    $\text{bbb}=((\text{ccc})^2 + (\text{sxy}(i,j))^2)^0.5$ ; always positive
    $\text{sigma1}=\text{aaa} + \text{bbb}$ ; sig1 since this is larger, always positive
    $\text{sigma3}=\text{aaa} - \text{bbb}$ ; sig3 since this is lower
    $\text{ex}_2(i,j)=\text{sigma3}$ ; will be used for evaluating confined pressure
    if $\text{sigma1} \# \text{abs(sxx}(i,j))$ then
      $\text{chi}=\text{atan(sxy}(i,j)/(\text{sigma1}-\text{abs(sxx}(i,j))))$
    else
      $\text{chi}=90.*\text{degrad}$
    endif
    $\text{ex}_1(i,j)=90-\text{abs(chi/degrad}$
  end_loop
end_loop

de
solve
plot grid esyy fill
copy Ogravsyy.pcx
plot grid esxx fill
copy Ogravsxx.pcx
def dc1_22
  dc22 = (dsoil - (y(1,22)+y(2,22)+y(1,23)+y(2,23))/4.)
dc21 = (dsoil - (y(1,21)+y(2,21)+y(1,22)+y(2,22))/4.)
dc20 = (dsoil - (y(1,20)+y(2,20)+y(1,21)+y(2,21))/4.)
dc19 = (dsoil - (y(1,19)+y(2,19)+y(1,20)+y(2,20))/4.)
dc18 = (dsoil - (y(1,18)+y(2,18)+y(1,19)+y(2,19))/4.)
dc17 = (dsoil - (y(1,17)+y(2,17)+y(1,18)+y(2,18))/4.)
dc16 = (dsoil - (y(1,16)+y(2,16)+y(1,17)+y(2,17))/4.)
dc15 = (dsoil - (y(1,15)+y(2,15)+y(1,16)+y(2,16))/4.)
dc14 = (dsoil - (y(1,14)+y(2,14)+y(1,15)+y(2,15))/4.)
dc13 = (dsoil - (y(1,13)+y(2,13)+y(1,14)+y(2,14))/4.)
dc12 = (dsoil - (y(1,12)+y(2,12)+y(1,13)+y(2,13))/4.)
dc11 = (dsoil - (y(1,11)+y(2,11)+y(1,12)+y(2,12))/4.)
dc10 = (dsoil - (y(1,10)+y(2,10)+y(1,11)+y(2,11))/4.)
dc9  = (dsoil - (y(1,9)+y(2,9)+y(1,10)+y(2,10))/4.)
dc8  = (dsoil - (y(1,8)+y(2,8)+y(1,9)+y(2,9))/4.)
dc7  = (dsoil - (y(1,7)+y(2,7)+y(1,8)+y(2,8))/4.)
dc6  = (dsoil - (y(1,6)+y(2,6)+y(1,7)+y(2,7))/4.)
dc5  = (dsoil - (y(1,5)+y(2,5)+y(1,6)+y(2,6))/4.)
dc4  = (dsoil - (y(1,4)+y(2,4)+y(1,5)+y(2,5))/4.)
dc3  = (dsoil - (y(1,3)+y(2,3)+y(1,4)+y(2,4))/4.)
dc2  = (dsoil - (y(1,2)+y(2,2)+y(1,3)+y(2,3))/4.)
dc1  = (dsoil - (y(1,1)+y(2,1)+y(1,2)+y(2,2))/4.)
end
ini xdis=0. ydis=0. ; reset displacement after gravity
prop e_plastic = 0. ; reset plastic strain
def tiltangle
  ytop  = y(1,23)
ymid  = y(1,14)
ybot  = y(1,5)
smaxrat = (nh+hsoil)/(ymid-ybot+nh) ; max displ at soil surface if displ at (1,14)=1
xvbotrat = nh/(hsoil+nh) ; velocity at the wall bottom if velocity at soil surface =1
xvtoprat = (hmwall+nh)/(hsoil+nh) ; velocity at the wall top if vel at soil surface =1
end
tiltangle
def shrat
  float shrat
  while_stepping
    shrat = abs(smaxrat*xdisp(1,14)/hsoil) ; displ at soil surface/(height of above wall base)
end
hist 1 nstep= 2000 shrat ; ratio of horizontal displacement to backfill height
hist 2 unbalance ; unbalance force
hist 3 e_plastic i=1 j=14
hist 41 sxx i=1 j=1
hist 42 sxx i=1 j=2
hist 43 sxx i=1 j=3
hist 44 sxx i=1 j=4
hist 45 sxx i=1 j=5
hist 46 sxx i=1 j=6
hist 47 sxx i=1 j=7
hist 48 sxx i=1 j=8
hist 49 sxx i=1 j=9
hist 50 sxx i=1 j=10
hist 51 sxx i=1 j=11
hist 52 sxx i=1 j=12
hist 53 sxx i=1 j=13
hist 54 sxx i=1 j=14
hist 55 sxx i=1 j=15
hist 56 sxx i=1 j=16
hist 57 sxx i=1 j=17
hist 58 sxx i=1 j=18
hist 59 sxx i=1 j=19
hist 60 sxx i=1 j=20
hist 61 sxx i=1 j=21
hist 62 sxx i=1 j=22
def fldata
    array adatshrat(17);datshrat(7)
anoshrat=17
nns=20+2*izones*jzones ; after tatsuoka,friction,dilation
adatshrat(1) = 0.0
adatshrat(2) = 0.0001
adatshrat(3) = 0.0002
adatshrat(4) = 0.0004
adatshrat(5) = 0.0006
adatshrat(6) = 0.0008
adatshrat(7) = 0.001
adatshrat(8) = 0.002
adatshrat(9) = 0.004
adatshrat(10) = 0.006
adatshrat(11) = 0.008
adatshrat(12) = 0.010
adatshrat(13) = 0.020
adatshrat(14) = 0.040
adatshrat(15) = 0.060
adatshrat(16) = 0.080
adatshrat(17) = 0.100 ;
end
findata
def scanstress
int count
while stepping
nst=20+2*izones*jzones ;+noshrat ;after tatsuoka,friction,dilation,fangstress
loop i (1,anoshrat)
  if i > count then
    if shrat >= adatshrat(i)
dc1_22
    nst=nst+i
    ytable(nst,22) = dc1
    ytable(nst,21) = dc2
    ytable(nst,20) = dc3
    ytable(nst,19) = dc4
    ytable(nst,18) = dc5
    ytable(nst,17) = dc6
    ytable(nst,16) = dc7
    ytable(nst,15) = dc8
    ytable(nst,14) = dc9
    ytable(nst,13) = dc10
    ytable(nst,12) = dc11
    ytable(nst,11) = dc12
    ytable(nst,10) = dc13
    ytable(nst,9) = dc14
    ytable(nst,8) = dc15
    ytable(nst,7) = dc16
    ytable(nst,6) = dc17
    ytable(nst,5) = dc18
    ytable(nst,4) = dc19
    ytable(nst,3) = dc20
    ytable(nst,2) = dc21
    ytable(nst,1) = dc22
    ytable(nst,23) = shrat
    xtable(nst,22) = -sxx(1,1)
    xtable(nst,21) = -sxx(1,2)
    xtable(nst,20) = -sxx(1,3)
    xtable(nst,19) = -sxx(1,4)
    xtable(nst,18) = -sxx(1,5)
    xtable(nst,17) = -sxx(1,6)
    xtable(nst,16) = -sxx(1,7)
    xtable(nst,15) = -sxx(1,8)
    xtable(nst,14) = -sxx(1,9)
    xtable(nst,13) = -sxx(1,10)
xtable(nst,12) = -sxx(1,11)
xtable(nst,11) = -sxx(1,12)
xtable(nst,10) = -sxx(1,13)
xtable(nst,9) = -sxx(1,14)
xtable(nst,8) = -sxx(1,15)
xtable(nst,7) = -sxx(1,16)
xtable(nst,6) = -sxx(1,17)
xtable(nst,5) = -sxx(1,18)
xtable(nst,4) = -sxx(1,19)
xtable(nst,3) = -sxx(1,20)
xtable(nst,2) = -sxx(1,21)
xtable(nst,1) = -sxx(1,22)
xtable(nst,23) = shrat

count=i
exit
endif
endif
end_loop
end
set large
struct prop = 1 E=ewall I=iwall area=awall
def nnodelem
no_a = 1
elemab = 50
no_b = no_a + elemab
elembc = 5
no_c = no_b + elembc
gabcd = 10
no_d = no_c + gabcd
elemde = 4
no_e = no_d + elemde
elemef = 85
no_f = no_e + elemef
elemfg = 28
no_g = no_f + elemfg
elemhd = 5
no_h = no_d - elemhd
end
nnodelem
struc node no_a 0., hwall
struc node no_b 0., bed pin fix y
struc node no_c bwall, bed
struc node no_d 0, bed l fix x y
struc node no_e 0, 0 fix x y
struc node no_f lsoil, 0 fix x y
struc node no_g lsoil, hwall fix x y
struc node no_h bwall, bed1 fix x y ; for active
struc beam beg node no_a end node no_b seg=elemab pr=l
struc beam beg node no_b end node no_c seg=elembc pr=l
struc beam beg node no_d end node no_e seg=elemde pr=l
struc beam beg node no_e end node no_f seg=elemef pr=l
struc beam beg node no_f end node no_g seg=elemfg pr=l
struc beam beg node no_h end node no_d seg=elemfg pr=l ; for
struc node no_a no_b fix y
free x i=1
free x i=73
free y j=1
interface 1 aside from node no_a to node no_b bside from 3,23 to 1,3
interface 3 aside from node no_d to node no_e bside from 1,11 to 1,1
interface 4 aside from node no_e to node no_f bside from 1,1 to 73,1
interface 5 aside from node no_f to node no_g bside from 73,1 to 73,23
interface 6 aside from node no_h to node no_d bside from 1,11 to 1,1 ;
def knksinter
dzmin1235=lsoil/izones
dzmin4=bed/4. ; height of lower portion
knks1=10*(bulk_mod(1,3)+4*shear_mod(1,3)/3)/dzmin1235
knks2=10*(bulk_mod(1,2)+4*shear_mod(1,2)/3)/dzmin1235
knks3=10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin1235
knks4=10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin4
knks5=10*(bulk_mod(izones,1)+4*shear_mod(izones,1)/3)/dzmin1235 ;
kns6=10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin1235 ;
fbase=fdirsh
command
interface 1 friction=fwall kn=knks1 ks=knks1 tbond=0 sbr=0 bslip on
interface 2 friction=0. kn=knks2 ks=knks2 tbond=0 sbr=0 bslip on
interface 3 friction=fwall kn=knks3 ks=knks3 tbond=0 sbr=0 bslip on
interface 4 friction=fbase kn=knks4 ks=knks4 tbond=0 sbr=0 bslip on
interface 5 friction=fwall kn=knks5 ks=knks5 tbond=0 sbr=0 bslip on
interface 6 friction=fwall kn=knks6 ks=knks6 tbond=0 sbr=0 bslip on ;
end command
end
knksinter
window
def tab dila ;correcting value of dilation angle as function of relative density
lowsig3=4903. ;lowest sig3 (Pa) data =0.05 kg/cm2
higsig3=392266. ;highest sig3 (Pa) data =4.0 kg/cm2
loodr=54.9075 ;average lowest data of Dr of Tatsuoka
dendr=79.4167 ;average highest data of Dr
nnd=20+izones*jzones
dila=((14.-7.)/(dendr-loodr))*(reldens-loodr)+7. ; maximum dilation angle
loop m (1,izones)
    loop n (1,jzones)
        sigma3=abs(ex_2(m,n))
        dendilx2=((0.020 - 0.006)/(higsig3-lowsig3))*(sigma3-lowsig3)+0.006
        loodilx2=((0.031 - 0.010)/(higsig3-lowsig3))*(sigma3-lowsig3)+0.010
        dilx2=((dendilx2-loodilx2)/(dendr-loodr))*(reldens-loodr)+loodilx2
        dilres=((1.- 4.)/(higsig3-lowsig3))*(sigma3-lowsig3)+4.
        dily1=((dila)/(0.02))*(-dilx2)+0.
        nnd=nnd+1
        xtable(nnd,1)=0.
        ytable(nnd,1)=dily1
        xtable(nnd,2)=0.02+dilx2
        ytable(nnd,2)=dila
        xtable(nnd,3)=0.06+dilx2
        ytable(nnd,3)=dilres
        command
            prop dtab nnd i=m j=n
        end_command
    end_loop ;n
end_loop ;m
end

def tab_friction ; creating friction table for each zone=
    nnn=20
    td_dr=80.56 ;Dr from tatsuoka, average dense
    tl_dr=52.78 ;Dr from tatsuoka, average loose
    minsig3=9.8 ; the lowest allowed in the log equation
    lows sig3=4903.325 ;lowest sig3 (Pa), tatsuoka
    higa sig3=392266. ;highest sig3 (Pa), tatsuoka
    ld_dr=79.278 ; DR dense at low sig3
    ll_dr=55.278 ; DR loose at low sig3,
    hd_dr=79.556 ; DR dense at high sig3,
    hl_dr=54.611 ; DR loose at high sig3,
loop m (1,izones) ; apply the operations to all elements
    loop n (1,jzones)
        nnn=nnn+1
        dangle=ex_1(m,n)
        sigma3=abs(ex_2(m,n)) ; obtain the current sigma 3
        ld_epres=table(11,dangle) ; low sig3 dense at delta
        ll_epres=table(12,dangle) ; low sig3 loose at delta
        lo_epres=((ll_epres-ld_epres)/(ll_dr-ld_dr))*(reldens-ld_dr)+ld_epres
        hd_epres=table(13,dangle) ; high sig3 dense at delta
        hl_epres=table(14,dangle) ; high sig3 loose at delta
        hi_epres=((hl_epres-hd_epres)/(hl_dr-hd_dr))*(reldens-hd_dr)+hd_epres
        sigma3=abs(ex_2(m,n)) ; obtain the current sigma 3
epsres1 = ((hi_epres-lo_epres)/(higsig3-lowsig3))*(sigma3-lowsig3)+lo_epres ;
epsres = epsres1*e1resatd90
if dangle <= datmin then
  fpeakrat = ca + ax1*dangle + ax2*(dangle)^2 + ax3*(dangle)^3
else
  fpeakrat = cb + bx1*dangle + bx2*(dangle)^2 + bx3*(dangle)^3
endif
if sigma3 <= minsig3 then
  sigma3 = minsig3
endif
cfdirsh = afdirsh*ln(sigma3) + bfdirsh ; correction to fdirsh function of Dr and sigma3
fpeak = fpeakrat*(fplst)*cfdirsh ; fpeak of current dangle; fplst = fric angle plane strain
ld_epeak = table(7,dangle) ; low sig3 dense at delta change to decimal
ll_epeak = table(8,dangle) ; low sig3 loose at delta
lo_epeak = ((ll_epeak-ld_epeak)/(ll_dr-ld_dr))*(reldens-ld_dr)+ld_epeak
hd_epeak = table(9,dangle) ; high sig3 dense at delta
hl_epeak = table(10,dangle) ; high sig3 loose at delta
hi_epeak = ((hi_epeak-hd_epeak)/(hi_dr-hd_dr))*(reldens-hd_dr)+hd_epeak
epspeak1 = ((hi_epeak-lo_epeak)/(higsig3-lowsig3))*(sigma3-lowsig3)+lo_epeak
epspeak = epspeak1*e1peakatd90
cfres = afres*ln(sigma3) + bfres ; the same for all Dr
fres1 = fres*cfres
if fpeak <= fres1 then
  fpeak = fres1
  epspeak = epsres
endif
if epsres <= epspeak then
  epsres = epspeak
  fpeak = fres1
endif
itab1 = table_size(1)
itab2 = table_size(2)
loop i (1,itab1) ; before peak
  xtable(nnn,i) = xtable(1,i)*epspeak
  ytable(nnn,i) = ytable(1,i)*fpeak
end_loop
loop i (1,itab2) ; after peak
  ccc = itab1+i
  xtable(nnn,ccc) = xtable(2,i)*(epsres-epspeak)+epspeak
  ytable(nnn,ccc) = ytable(2,i)*(fpeak-fres1)+fres1
end_loop
command
  prop ftab nnn i=m j=n
end_command
end_loop ;n
def movethewall
    shrat = 0.0
    xvtop = xvtoprat*(2.5e-7)*hsoil/0.5
    xvbot = xvbotrat*(2.5e-7)*hsoil/0.5
    delxv = (xvbot-xvtop)/(no_b-no_a)
    loop n (1,1500) ;******
        loop m (1,400) ;===
            loop i (no_a,no_b)
                xvtopi = xvtop + (i-no_a)*delxv
                command
                    struc node i initial xvel xvtopi ;;m/time step
                end_command
            endloop
            command
                step 1
            end_command
        endloop ;== loop m
        if shrat>=0.1001 loop n
            exit
        endif
    endloop ;****** loop n
end

window
movie on file 9maxssi.dcx size 1080,670
plot grid mag=1 green beam mag=1 yellow ssi fill interval 0.1
movie off
window
end_command
end
def framemovie2
xlow = -0.05
xupp = hsoil/0.5*1.
ylow = -0.05
yupp = hsoil/0.5*0.9171
end
framemovie2
def createmovie2 ; principle stress
command
    wind xlow, xupp ylow, yupp
movie on file 9prinstr.dcx size 1080,670
plot grid mag=1 iw beam mag=1 red stress
movie off
window
end_command
end
movethewall
plot hist 2 vs 1 ; unbalance vs s/h ratio
copy 3unbalanc.pcx
plot hist 62 61 60 59 58 57 56 55 54 53 52 51 50 49 48 47 46 45 44 43 42 41 vs 1 ;
copy 1horstrs.pcx ; spt0
plot table 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42
copy 4tabfric.pcx
plot hist 3 vs 1
copy 5eplast_shrat.pcx
plot hist 41 42 43 44 45 46 vs 1
copy 6botstress.pcx
plot grid mag=1 iw beam mag=1 yellow ssi fill interval 0.1
copy 1maxssi.pcx
plot grid mag=1 iw beam mag=1 yellow pl
copy 3plastic.pcx
wind xlow, xupp ylow, yupp
plot grid mag=1 iw beam mag=1 red stress
copy 2prnstrs.pcx
window
set log 8fang_result.txt ; printing properties
set log on
print table 20
def cetak
nst=20+2*izones*jzones ;+noshrat ; stress result
loop j (1,anoshrat)
nst=nst+1
    command
    print table nst
    end_command
end_loop :j
def cetak
set log off
return

Example for Passive RBT in Anisotropic Soil

title
z12rb4mand32.dat: Passive H=4.0m,Dr=32%,n=0,RBT
set cust1
Dept.of Civil and Env. Eng.
set cust2
Old Dominion U.,Norfolk, Virginia
config extra 2
set echo off
grid 72 22
mod ss
set grav=9.81 ;m/s2
def dimension
;soil------
hsoil = 4.0 ; INPUT, depth of soil adjacent to moving wall (m)
n = 0.0 ; INPUT, rotation about point at nH (m) below wall base
dsoil = 0.613*hsoil/0.5 ; soil depth (m), the same proportion with Fang's experiment
lsoil = 2.0*hsoil/0.5 ; soil length (m)
bed = 0.113*hsoil/0.5 ; bed depth (m)
b1d = 0.999*bed
nh = n*hsoil ; nH (m)
;walls------
hwall = 0.8*hsoil/0.5 ; wall top to soil bottom (m)
hmwall = hwall-bed ; height of movable wall
twall = 0.12*hsoil/0.5 ; wall thickness (m)
awall = twall*1. ; area of movable wall in (m2)
iwall = twall*twall^3/12. ; moment area of the wall (m4)
ewall = 200e9 ; assuming steel (Pa)
bwall = -0.133*hsoil/0.5 ; coordiate of front edge of wall base
end
dimension
def backfill

; Ottawa sand used in Fang test

gs = 2.65 ; INPUT, specific gravity
eemax = 0.76 ; INPUT, maximum void ratio
eemin = 0.50 ; INPUT, minimum void ratio
reldens = 32. ; INPUT, relative density (%)
ca = 0.90843629742579 ; INPUT for Dr = 32
ax1 = -4.719059E-04
ax2 = 4.192983E-05
ax3 = -8.955695E-07
cb = 0.54985314357006
bx1 = 1.270233E-02
bx2 = -1.198577E-04
bx3 = 3.814908E-07
afdirsh = -0.006043 ; for DR=32
bfdirsh = 1.077827
afres = -0.0121220 ; for all DR
bfres = 1.1561240
walsanrat = -0.000025502*(reldens)^2+0.00385617*reldens+0.187259946
gamwater = 9.81*1000 ; unit weight of water N/m^3
ee = eemax-reldens*(eemax-eemin)/100. ; void ratio
fdirsh = 0.000275*(reldens)^2+0.184275*reldens+25.601176
fres = 31.5

\[
\text{cr} = 0.96975756535439
\]
\[
\text{cdl} = -2.687556E-03
\]
\[
\text{cd2} = 1.147410E-05
\]
\[
\text{cd3} = 1.618853E-08
\]
\[
\text{minrat} = \text{cr} + \text{cdl}*(\text{reldens}) + \text{cd2}*(\text{reldens})^2 + \text{cd3}*(\text{reldens})^3
\]
\[
\text{fplst} = \text{fdirsh}/\text{minrat} ; \text{peak at d=90}
\]
\[
; \text{delta at Minimum ratio R(d) for a given Dr}
\]
\[
\text{cdr} = 50.6741218848
\]
\[
\text{cdd1} = -2.319605E-01
\]
\[
\text{cdd2} = -3.303401E-03
\]
\[
\text{cdd3} = 3.013308E-05
\]
\[
\text{datmin} = \text{cdr} + \text{cdd1}*(\text{reldens}) + \text{cdd2}*(\text{reldens})^2 + \text{cdd3}*(\text{reldens})^3
\]
\[
\text{e1peakatd90} = (-0.015207*\text{reldens}+3.403184)/100. ;
\]
\[
\text{e1resatd90} = (-0.081055*\text{reldens}+11.346580)/100. ;
\]
\[
\text{unitwei} = \text{gs}^*\text{gamwater}/(\text{ee}+1) ; \text{unit weight (N/m^3)}
\]
\[
\text{fwall} = \text{walsanrat}^*\text{fdirsh} ; \text{wall friction}
\]
end

backfill

\[
\text{gen 0 0 bed lsoil bed lsoil 0 i=1,73 j=1,5}
\]
\[
\text{gen 0 bed 0 dsoil lsoil dsoil lsoil bed i=1,73 j=5,23}
\]
def setprop

float zz sigm gg kk pois yc
float ko
dila=((14.-7.)/(79.4167-54.9075))*(reldens-54.9075)+7. ; maximum dilation angle
dilaat0=-dila/2. ; negative start at zero May 19, 2007
pa_psf=0.020885 ; conversion pascal to psf
psi_pa=6894.75728 ; conversion psi to pascal
pa = 101300 ; atmospheric pressure in N/m² (=Pa)
sss = 1350 ;
loop i (1,izones)
   loop j (1,jzones)
      yc = (y(i,j)+y(i+1,j)+y(i,j+1)+y(i+1,j+1))/4.
      zz = y(j,gp)-yc
      if fplst > 34 then
         ftriax = (fplst + 17.)/1.5
      else
         ftriax = fplst
      endif
      ko = 1-sin(ftriax*deggrad) ; to be used in calculating average sigma
      pois=(1-sin(ftriax*deggrad))/(2-sin(ftriax*deggrad)) ;
      sigm=(unitwei*zz*(1.+2.*ko)/3.) ; average stress (Pascal)
      gg=(sss*1.*(sigm*pa)*0.5)/(2.*(1+pois)*(0.3+0.7*ee^2)) ; in N/m²
      kk=2.*gg*(1.+pois)/(3.*(1.-2.*pois)) ; initial K
      shear_mod(i,j)=gg
      bulk_mod(i,j)=kk
      density(i,j)=unitwei/9.81 ; kg/m³
      cohesion(i,j)=0.
      tension(i,j)=0.
    endloop
endloop
end
setprop
tab 15 0., dilaat0 0.03, dila 0.07, 4. ; initial value of dilation: loose and 0.05 of Tatsuoka
tab 16 0., fplst 0.2, fplst ; initial value of friction
tab 20 hsoil, n reldens, unitwei fplst, pois fdirsh, fwall
prop dtab 15
prop ftab 16
fix y j=1 ; bottom
fix x i=73 ; right wall
fix x i=1 ; left wall
set plot pce bw
def angle_pq ; angle and pq for all elements
loop i (1, izones)
   loop j (1, jzones)
      aaa=-(sxx(i,j)+syy(i,j))/2. ; change sign to positive
      ccc=(sxx(i,j)-syy(i,j))/2.
      bbb=((ccc)^2 + (sxy(i,j))^2)^0.5 ; always positive
   endloop
endloop
sigma1 = aaa + bbb ; sig1 since this is larger, always positive
sigma3 = aaa - bbb ; sig3 since this is lower
ex_2(i,j) = sigma3 ; will be used for evaluating confined pressure
if sigma1 ≥ abs(sxx(i,j)) then
  chi = atan(sxy(i,j)/(sigma1 - abs(sxx(i,j))))
else
  chi = 90.*degrad
endif
ex_1(i,j) = 90 - abs(chi/degrad)
end_loop
end_loop
end
solve
plot grid esyy fill
copy 0gravsyy.pcx
plot grid esxx fill
copy 0gravsxx.pcx
def del 22
  dc22 = (dsoil - (y(1,22)+y(2,22)+y(1,23)+y(2,23))/4.)
dc21 = (dsoil - (y(1,21)+y(2,21)+y(1,22)+y(2,22))/4.)
dc20 = (dsoil - (y(1,20)+y(2,20)+y(1,21)+y(2,21))/4.)
dc19 = (dsoil - (y(1,19)+y(2,19)+y(1,20)+y(2,20))/4.)
dc18 = (dsoil - (y(1,18)+y(2,18)+y(1,19)+y(2,19))/4.)
dc17 = (dsoil - (y(1,17)+y(2,17)+y(1,18)+y(2,18))/4.)
dc16 = (dsoil - (y(1,16)+y(2,16)+y(1,17)+y(2,17))/4.)
dc15 = (dsoil - (y(1,15)+y(2,15)+y(1,16)+y(2,16))/4.)
dc14 = (dsoil - (y(1,14)+y(2,14)+y(1,15)+y(2,15))/4.)
dc13 = (dsoil - (y(1,13)+y(2,13)+y(1,14)+y(2,14))/4.)
dc12 = (dsoil - (y(1,12)+y(2,12)+y(1,13)+y(2,13))/4.)
dc11 = (dsoil - (y(1,11)+y(2,11)+y(1,12)+y(2,12))/4.)
dc10 = (dsoil - (y(1,10)+y(2,10)+y(1,11)+y(2,11))/4.)
dc9 = (dsoil - (y(1,9)+y(2,9)+y(1,10)+y(2,10))/4.)
dc8 = (dsoil - (y(1,8)+y(2,8)+y(1,9)+y(2,9))/4.)
dc7 = (dsoil - (y(1,7)+y(2,7)+y(1,8)+y(2,8))/4.)
dc6 = (dsoil - (y(1,6)+y(2,6)+y(1,7)+y(2,7))/4.)
dc5 = (dsoil - (y(1,5)+y(2,5)+y(1,6)+y(2,6))/4.)
dc4 = (dsoil - (y(1,4)+y(2,4)+y(1,5)+y(2,5))/4.)
dc3 = (dsoil - (y(1,3)+y(2,3)+y(1,4)+y(2,4))/4.)
dc2 = (dsoil - (y(1,2)+y(2,2)+y(1,3)+y(2,3))/4.)
dc1 = (dsoil - (y(1,1)+y(2,1)+y(1,2)+y(2,2))/4.)
end
ini xdis=0. ydis=0. ; reset displacement after gravity
prop e_plastic = 0. ; reset plastic strain
def tiltangle
  ytop = y(1,23)
ymid = y(1,14)  
ymbot = y(1,5)  
smaxrat = (nh+hsoil)/(ymid-ymbot+nh) ; max displ at soil surface if disp 1(14)=1  
xvbotrat = nh/(hsoil+nh) ; velocity at the wall bottom if velocity at soil surface =1  
xvtoprat = (hmwall+nh)/(hsoil+nh) ; vel at the wall top if vel at soil surface =1
end

tiltangle_def shrat
float shrat
while _stepping
  shrat = abs(smaxrat*disp(1,14)/hsoil)  ;
end
hist 1 nstep= 2000 shrat ; ratio of horizontal displacement to backfill height
hist 2 unbalance ; unbalance force
hist 3 e_plastic i=1 j=14
hist 41 sxx i=1 j=1
hist 42 sxx i=1 j=2
hist 43 sxx i=1 j=3
hist 44 sxx i=1 j=4
hist 45 sxx i=1 j=5
hist 46 sxx i=1 j=6
hist 47 sxx i=1 j=7
hist 48 sxx i=1 j=8
hist 49 sxx i=1 j=9
hist 50 sxx i=1 j=10
hist 51 sxx i=1 j=11
hist 52 sxx i=1 j=12
hist 53 sxx i=1 j=13
hist 54 sxx i=1 j=14
hist 55 sxx i=1 j=15
hist 56 sxx i=1 j=16
hist 57 sxx i=1 j=17
hist 58 sxx i=1 j=18
hist 59 sxx i=1 j=19
hist 60 sxx i=1 j=20
hist 61 sxx i=1 j=21
hist 62 sxx i=1 j=22
def fildata
  array adatshrat(17) ;datshrat(7)
anoshrat=17
  nns=20+2*izones*jzones ; after tatsuoka,friction,dilation
adatshrat(1) = 0.0  ;
adatshrat(2) = 0.0001  ;
adatshrat(3) = 0.0002  ;
adatshrat(4) = 0.0004  ;
adatshrat(5) = 0.0006 
adatshrat(6) = 0.0008 
adatshrat(7) = 0.001 
adatshrat(8) = 0.002 
adatshrat(9) = 0.004 
adatshrat(10) = 0.006 
adatshrat(11) = 0.008 
adatshrat(12) = 0.010 
adatshrat(13) = 0.020 
adatshrat(14) = 0.040 
adatshrat(15) = 0.060 
adatshrat(16) = 0.080 
adatshrat(17) = 0.100 
end 
fildata 
def scanstress
  int count
  while stepping
    nst = 20 + 2 * izones * jzones ; + noshrat
    loop i (1, anoshrat)
      if i > count then
        if shrat >= adatshrat(i) then
          dcl_22
          nst = nst + i
          ytable(nst, 22) = dc1
          ytable(nst, 21) = dc2
          ytable(nst, 20) = dc3
          ytable(nst, 19) = dc4
          ytable(nst, 18) = dc5
          ytable(nst, 17) = dc6
          ytable(nst, 16) = dc7
          ytable(nst, 15) = dc8
          ytable(nst, 14) = dc9
          ytable(nst, 13) = dc10
          ytable(nst, 12) = dc11
          ytable(nst, 11) = dc12
          ytable(nst, 10) = dc13
          ytable(nst, 9) = dc14
          ytable(nst, 8) = dc15
          ytable(nst, 7) = dc16
          ytable(nst, 6) = dc17
          ytable(nst, 5) = dc18
          ytable(nst, 4) = dc19
          ytable(nst, 3) = dc20
          ytable(nst, 2) = dc21
ytable(nst, 1) = dc22
ytable(nst, 23) = shrat
xtable(nst, 22) = -sxx(1, 1)
xtable(nst, 21) = -sxx(1, 2)
xtable(nst, 20) = -sxx(1, 3)
xtable(nst, 19) = -sxx(1, 4)
xtable(nst, 18) = -sxx(1, 5)
xtable(nst, 17) = -sxx(1, 6)
xtable(nst, 16) = -sxx(1, 7)
xtable(nst, 15) = -sxx(1, 8)
xtable(nst, 14) = -sxx(1, 9)
xtable(nst, 13) = -sxx(1, 10)
xtable(nst, 12) = -sxx(1, 11)
xtable(nst, 11) = -sxx(1, 12)
xtable(nst, 10) = -sxx(1, 13)
xtable(nst, 9) = -sxx(1, 14)
xtable(nst, 8) = -sxx(1, 15)
xtable(nst, 7) = -sxx(1, 16)
xtable(nst, 6) = -sxx(1, 17)
xtable(nst, 5) = -sxx(1, 18)
xtable(nst, 4) = -sxx(1, 19)
xtable(nst, 3) = -sxx(1, 20)
xtable(nst, 2) = -sxx(1, 21)
xtable(nst, 1) = -sxx(1, 22)
xtable(nst, 23) = shrat

count = i
exit
endif
endif
end_loop
end
set large
struct prop = 1 E=ewall l=iwall area=awall
def nnodelem
no_a = 1
elemab = 50
no_b = no_a + elemab
elembc = 5	no_c = no_b + elembc
gabcd = 10	no_d = no_c + gabcd
elemde = 4	no_e = no_d + elemde
eleme = 85	no_e = no_e + eleme
elemfg = 28
no_g = no_f + elemfg ; additional for active
elemhd = 5
no_h = no_d - elemhd
end
nnodelem
struc node no_a 0., hwall
struc node no_b 0., bed pin fix y
struc node no_c bwall, bed
struc node no_d 0, bedl fix x y
struc node no_e 0, 0 fix x y
struc node no_f lsoil, 0 fix x y
struc node no_g lsoil, hwall fix x y
struc node no_h bwall, bedl fix x y ; for active
struc beam beg node no_a end node no_b seg=elemab pr=1
struc beam beg node no_b end node no_c seg=elembc pr=1
struc beam beg node no_d end node no_e seg=elemde pr=1
struc beam beg node no_e end node no_f seg=elemef pr=1
struc beam beg node no_f end node no_g seg=elemfg pr=1
struc beam beg node no_h end node no_d seg=elemfg pr=1 ; for active
struc node no_a no_b fix y
free x i=1
free x i=73
free y j=1
interface 1 aside from node no_a to node no_b bside from 3,23 to 1,3
interface 2 aside from node no_b to node no_c bside from 1,11 to 1,2
interface 3 aside from node no_d to node no_e bside from 1,11 to 1,1
interface 4 aside from node no_e to node no_f bside from 1,1 to 73,1
interface 5 aside from node no_f to node no_g bside from 73,1 to 73,23
interface 6 aside from node no_h to node no_d bside from 1,11 to 1,1
def knksinter
dzmin1235 = lsoil/izones
dzmin4 = bed/4. ; height of lower portion
knks1 = 10*(bulk_mod(1,3)+4*shear_mod(1,3)/3)/dzmin1235
knks2 = 10*(bulk_mod(1,2)+4*shear_mod(1,2)/3)/dzmin1235
knks3 = 10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin1235
knks4 = 10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin4
knks5 = 10*(bulk_mod(izones, 1)+4*shear_mod(izones, 1)/3)/dzmin1235 ;
knks6 = 10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin1235
fbase = fdirsh
command
interface 1 friction=fwall kn=knks1 ks=knks1 tbond=0 sbr=0 bslip on
interface 2 friction=0. kn=knks2 ks=knks2 tbond=0 sbr=0 bslip on
interface 3 friction=fwall kn=knks3 ks=knks3 tbond=0 sbr=0 bslip on
def tab_dila ;correcting value of dilation angle as function of relative density
lowsig3=4903. ;lowest sig3 (Pa) data =0.05 kg/cm2
higsig3=392266. ;highest sig3 (Pa) data =4.0 kg/cm2
loodr=54.9075 ;average lowest data of Dr of Tatsuoka
dendr=79.4167 ;average highest data of Dr
nnd=20+izones*jzones

dila=((14.-7.)/(dendr-loodr))*(reldens-loodr)+7. ; for maximum dilation angle
loop m (1,izones)
  loop n (1,jzones)
    sigma3=abs(ex_2(m,n))
    dendilx2=((0.020 - 0.006)/(higsig3-lowsig3))*(sigma3-lowsig3)+0.006
    loodilx2=((0.031 - 0.010)/(higsig3-lowsig3))*(sigma3-lowsig3)+0.010
    dilx2=((dendilx2-loodilx2)/(dendr-loodr))*(reldens-loodr)+loodilx2
    dilres=((l.- 4.)/(higsig3-lowsig3))*(sigma3-lowsig3)+4.
    dilyl=((dila)/(0.02))*(-dilx2)+0.
    nnd=nnd+l
    xtable(nnd,l)=0.
    ytable(nnd,1)=dily1
    xtable(nnd,2)=0.02+dilx2
    ytable(nnd,2)=dila
    xtable(nnd,3)=0.06+dilx2
    ytable(nnd,3)=dilres
  command
    prop dtab rind i=m j=n
  end_command
end_loop ;n
end_loop ;m
end

def tab_friction ; creating friction table for each zone;
nnn=20

td_dr=80.56 ;
tl_dr=52.78 ;

minsig3=9.8 ; the lowest allowed in the log equation
lowsig3=4903.325 ;
higsig3=392266. ;

ld_dr=79.278 ; DR dense at low sig3
ll_dr=55.278 ; DR loose at low sig3,
hd_dr=79.556 ; DR dense at high sig3,
hl_dr=54.611; DR loose at high sig3,
loop m (1,izones); apply the operations to all elements
loop n (1,jzones)
nnn=nnn+1
dangle=ex_1(m,n)
sigma3=abs(ex_2(m,n)); obtain the current sigma 3
ld_epres=table(11,dangle); low sig3 dense at delta
ll_epres=table(12,dangle); low sig3 loose at delta
lo_epres=((ll_epres-ld_epres)/(ll_dr-ld_dr))*(reldens-ld_dr)+ld_epres
hd_epres=table(13,dangle); high sig3 dense at delta
hl_epres=table(14,dangle); high sig3 loose at delta
hi_epres=((hl_epres-hd_epres)/(hl_dr-hd_dr))*(reldens-hd_dr)+hd_epres
sigma3=abs(ex_2(m,n)); obtain the current sigma 3
epsres1=(((hi_epres-lo_epres)/(higsig3-lowsig3))* (sigma3 -lowsig3)+lo_epres ;
epsres=epsres1 *e1resatd90
if dangle <= datmin then
  fpeakrat = ca + ax1*dangle + ax2*(dangle)^2 + ax3*(dangle)^3
else
  fpeakrat = cb + bx1*dangle + bx2*(dangle)^2 + bx3*(dangle)^3
endif
if sigma3 <= minsig3 then
  sigma3=minsig3
endif
cfdirs = afdirs *ln(sigma3) + bfdirs; correction to fdirs function of Dr and sigma3;
fpeak=fpeakrat*(fplst)*cfdirs; fpeak of current dangle; fplst = angle from plane strain
ld_epeak=table(7,dangle); low sig3 dense at delta change to decimal
ll_epeak=table(8,dangle); low sig3 loose at delta
lo_epeak=(((ll_epeak-ld_epeak)/(ll_dr-ld_dr))*(reldens-ld_dr)+ld_epeak
hd_epeak=table(9,dangle); high sig3 dense at delta
hl_epeak=table(10,dangle); high sig3 loose at delta
hi_epeak=(((hl_epeak-hd_epeak)/(hl_dr-hd_dr))*(reldens-hd_dr)+hd_epeak
epspeak1=(((hi_epeak-lo_epeak)/(higsig3-lowsig3))* (sigma3 -lowsig3)+lo_epeak
epspeak=epspeak1 *e1peakatd90
cfres = ares *ln(sigma3) + bres; the same for all Dr
fres1=fres1* cfres
if fpeak <= fres1 then
  fpeak=fres1
  epspeak=epsres
endif
if epsres <= epspeak then
  epsres=epspeak
  fpeak=fres1
endif
itab1=table_size(1)
itab2=table_size(2)
loop i (1, itabl) ; before peak
    xtable(nnn,i)=xtable(1,i)*epspeak
    ytable(nnn,i)=ytable(1,i)*fpeak
end_loop
loop i (1, itab2) ; after peak
    ccc=itabl+i
    xtable(nnn,ccc)=xtable(2,i)*(epsres-epspeak)+epspeak ;
    ytable(nnn,ccc)=ytable(2,i)*(fpeak-fresl)+fresl
end_loop
command
    prop ftab nnn i=m j=n
end_command
end loop ;n
end loop ;m
end
tab 1  0.,0.05,0.433 0.1,0.599 0.2,0.74 0.3,0.818 0.4,0.8708
   0.5,0.9145 0.6,0.9426 0.7,0.9633 0.8,0.98 0.9,0.9931 1.,1.
tab 2  0.1,0.9689 0.2,0.8782 0.4,0.581 0.6,0.3132 0.8,0.101 0.9,0.041 1.,0.
tab 7 0.,1.647059 23.,1.441176 34.,0.882353 45.,1.058824 90.,1.
tab 8 0.,2.111111 23.,1.355556 34.,1.511111 45.,1.410714 90.,1.
tab 9 0.,1.295082 23.,1.065574 34.,0.918033 45.,1.131148 90.,1.
tab 10 0.,1.380952 23.,0.928571 34.,1.119048 45.,1.238095 90.,1
   0.1,1.455882 23.,1.250000 34.,0.823529 45.,1.308824 90.,1.
tab 12 0.,1.232759 23.,0.732759 34.,0.663793 45.,0.729834 90.,1.
tab 13 0.,1.252874 23.,1.321839 34.,0.850575 45.,1.172414 90.,1.
tab 14 0.,1.108527 23.,0.806202 34.,0.899225 45.,1.356589 90.,1
def movethewall
    shrat = 0.0
    xvtop = xvtoprat*(2.5e-7)*hsoil/0.5
    xvbot = xvbotrat*(2.5e-7)*hsoil/0.5
    delxv = (xvbot-xvtop)/(no_b-no_a)
loop n (1, 1500) ;*****
    angle_pq
    tab_dila
    tab_friction
    loop m (1, 400) ;=
        loop i (no_a, no_b)
            xvtopi = xvtop + (i-no_a)*delxv
            command
                struc node i initial xvel xvtopi ;;m/time step
            end_command
        endloop
    command
    step 1
end_command
endloop ;==== loop m
if shrat>=0.1001
exit
endif
endloop ;****** loop n
end
def framemovie2
xlow = -0.05
xupp = hsoil/0.5*1.
ylow = -0.05
yupp = hsoil/0.5*0.9171
end
framemovie2
movethewall
plot hist 2 vs 1
copy 3unbalnc.pcx
plot hist 62 61 60 59 58 57 56 55 54 53 52 51 50 49 48 47 46 45 44 43 42 41 vs 1
copy 1horstrs.pcx
plot table 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42
copy 4tabfric.pcx
plot hist 3 vs 1
copy 5eplast_shrat.pcx
plot hist 41 42 43 44 45 46 vs 1
copy 6botstress.pcx
plot grid mag=l iw beam mag=l yellow ssi fill interval 0.1
copy 1maxssi.pcx
plot boundary blue beam mag=1 Imagenta pl blue
copy 3plastic.pcx
wind xlow,xupp ylow,yupp
plot grid mag=l iw beam mag=l red stress
copy 2prnstrs.pcx
window
set log 8fang_result.txt ; printing properties
set log on
print table 20
def cetak
nst=20+2*izones*jzones ;+noshrat ; stress result
loop j (l,anoshrat)
nst=nst+1
command
print table nst
end_command
end_loop :j
def cetak
Example for Passive RTT in Isotropic Soil

title
rt03isf40di05w028n00m21.dat: passive H=4.0m,n=0,G=22.5Mpa,K=60Mpa,dil=0.5f,;f=40,d/f=0.28, RTT
set cust1
Dept.of Civil and Env. Eng.
set cust2
Old Dominion U.,Norfolk,Virginia
config extra 2
set echo off
grid 72 22
mod ss
set grav=9.81 ;m/s2
def dimension
hsoil = 4.0 ; INPUT, depth of soil adjacent to moving wall (m)
n = 0.0 ; INPUT, rotation about point at nH (m) below wall base
dsoil = 0.613*hsoil/0.5 ; soil depth (m),
lsoil = 2.0*hsoil/0.5 ; soil length (m)
bed = 0.113*hsoil/0.5 ; bed depth (m)
bed1 = 0.999*bed
nh = n*hsoil ; nH (m)
hwall = 0.8*hsoil/0.5 ; wall top to soil bottom (m)
hmwall = hwall-bed ; height of movable wall
twall = 0.12*hsoil/0.5 ; wall thickness (m)
awall = twall*1. ; area of movable wall in (m2)
iwall = 1*(twall^3)/12. ; moment area of the wall (m4)
ewall = 200e9 ; assuming steel (Pa)
bwall = -0.133*hsoil/0.5 ; coordinate of front edge of wall base
end
dimension
def backfill
gs = 2.65 ; INPUT, specific gravity
eemax = 0.76 ; INPUT, maximum void ratio
eemin = 0.50 ; INPUT, minimum void ratio
creldens = 80. ; INPUT, relative density (%) 
ca = 0.888930072 ; INPUT for Dr = 80
ax1 = 1.02003E-05
ax2 = -7.33196E-05
ax3 = -1.03025E-07
cb = 0.681033805
bx1 = 0.006882165
bx2 = 3.54747E-05
bx3 = 1.81561E-08
afdirsh = 0.016789 ; for DR=80
bdirsh = 1.216241
afres = 0.0121220 ; for all DR
bfres = 1.1561240
walsanrat=-0.000025502*(reldens)^2+0.00385617*reldens+0.187259946 ;
gamwater = 9.81*1000 ;unitweight of water N/m3
ee=eeemax-reldens*(eeemax-eemin)/100. ;void ratio
fdirsh = 0.000275*(reldens)^2+0.184275*reldens+25.601176 ;direct shear frict angle
fres=31.5 ;residual angle of friction from DS,
cr = 0.96975756535439
cd1 = -2.687556E-03
cd2 = 1.147410E-05
cd3 = 1.618853E-08
minrat = cr + cd1*reldens + cd2*(reldens)^2 + cd3*(reldens)^3 ;
fplst = fdirsh/minrat
cdr = 50.6741218848
ccd1 = -2.319605E-01
ccd2 = -3.303401E-03
ccd3 = 3.013308E-05
datmin = cdr + ccd1*reldens + ccd2*(reldens)^2 + ccd3*(reldens)^3 ;
e1peakatd90=(-0.015207*reldens+3.403184)/100. ;e1peak for given density at d=90;
e1resatd90=(-0.081055*reldens+11.346580)/100. ;e1res for given density at d=90;
unitwei = gs*gamwater/(ee+1) ;unitweight (N/m3)
fwall = walsanrat*fdirsh ;wall friction
  angle_used = 40. ; fplst
  fwall_soil = 0.28 ; ratio of d wall to soil friction
  unitwei = 20000 N/m3
  scale4fres=fres/fplst
  fplst = angle_used
  fwall = fwall_soil*fplst
  fres = angle_used
  fdirsh = angle_used
end
backfill
gen 0 0 0 bed lsoil bed lsoil 0 i=1,73 j=1,5
gen 0 bed 0 dsoil lsoil dsoil lsoil bed i=1,73 j=5,23
def setprop
  float zz sigm gg kk pois yc
  float ko
dila=((14.-7.)/(79.4167-54.9075))*(reldens-54.9075)+7.
dilaat0=-dila/2. ; negative start at zero
pa_psf=0.020885 ; conversion pascal to psf
psi_pa=6894.75728 ; conversion psi to pascal
pa = 101300  ; atmospheric pressure in N/m2 (=Pa)
sss = 1350
loop i (1, izones)
  loop j (1, jzones)
    yc = (y(i,j)+y(i+1,j)+y(i,j+1)+y(i+1,j+1))/4.
    zz = y(jgp)-yc
    if fplst > 34 then
      ftriax = (fplst + 17.)/1.5
    else
      ftriax = fplst
    endif
    ko = 1-sin(ftriax*degrad)  ; to be used in calculating average sigma
    pois=(1-sin(ftriax*degrad))/(2-sin(ftriax*degrad)) ; Ko=1-sin f = pois/(1-pois)
    sigm=(unitwei*zz*(1.+.2.*ko)/3.) ; average stress (Pascal)
    gg=(sss*1*(sigm*pa)\(^0.5)/(2*(1+pois)*(0.3+0.7*ee^2)) ; in N/m2
    kk=2.*gg*(1.+pois)/(3.*(1.-2.*pois)) ; initial K
    shear_mod(i,j)=gg
    bulk_mod(i,j)=kk
    density(i,j)=unitwei/9.81  ; kg/m3
    cohesion(i,j)=0.
    tension(i,j)=0.
      shear_mod(i,j)=22.5e6
      bulk_mod(i,j)=60.0e6
      density(i,j)=20000./9.81  ; kg/m3
  endloop
endloop

dila= 0.5*angle_used
dilat0= 0.5*angle_used

setprop
  tab 15 0.,dilaat0 0.03,dila 0.07,dila ; constant
  tab 16 0.,fplst 0.2,fplst ; initial value of friction
  tab 20 hsoil,n reldens,unitwei fplst,pois fdirsh,fwall
prop  dtab 15
prop  ftab 16
fix y j=1 ;bottom
fix x i=73 ;right wall
fix x i=1  ;left wall
set plot pcx
def angle_pq ;angle and pq for all elements
loop i (1,izones)
  loop j (1, jzones)
    aaa=-(sxx(i,j)+syy(i,j))/2.  ; change sign to positive
    ccc=(sxx(i,j)-syy(i,j))/2.
    bbb=((ccc)^2 + (sxy(i,j))^2)\(^0.5  ; always positive
sigmal = aaa + bbb ; sigl since this is larger, always positive
sigma3 = aaa - bbb ; sig3 since this is lower
ex_2(i,j) = sigma3 ; will be used for evaluating confined pressure
if sigma1 # abs(sxx(i,j)) then
  chi = atan(sxy(i,j)/(sigma1-abs(sxx(i,j))))
else
  chi = 90.*degrad
endif
ex_1(i,j) = 90-abs(chi/degrad)
end_loop
end_loop
end
solve
plot grid esyy fill
copy 0gravsyy.pcx
plot grid esxx fill
copy 0gravsxx.pcx
def dc1_22
  dc22 = (dsoil - (y(1,22)+y(2,22)+y(1,23)+y(2,23))/4.)
dc21 = (dsoil - (y(1,21)+y(2,21)+y(1,22)+y(2,22))/4.)
dc20 = (dsoil - (y(1,20)+y(2,20)+y(1,21)+y(2,21))/4.)
dc19 = (dsoil - (y(1,19)+y(2,19)+y(1,20)+y(2,20))/4.)
dc18 = (dsoil - (y(1,18)+y(2,18)+y(1,19)+y(2,19))/4.)
dc17 = (dsoil - (y(1,17)+y(2,17)+y(1,18)+y(2,18))/4.)
dc16 = (dsoil - (y(1,16)+y(2,16)+y(1,17)+y(2,17))/4.)
dc15 = (dsoil - (y(1,15)+y(2,15)+y(1,16)+y(2,16))/4.)
dc14 = (dsoil - (y(1,14)+y(2,14)+y(1,15)+y(2,15))/4.)
dc13 = (dsoil - (y(1,13)+y(2,13)+y(1,14)+y(2,14))/4.)
dc12 = (dsoil - (y(1,12)+y(2,12)+y(1,13)+y(2,13))/4.)
dc11 = (dsoil - (y(1,11)+y(2,11)+y(1,12)+y(2,12))/4.)
dc10 = (dsoil - (y(1,10)+y(2,10)+y(1,11)+y(2,11))/4.)
dc9 = (dsoil - (y(1,9)+y(2,9)+y(1,10)+y(2,10))/4.)
dc8 = (dsoil - (y(1,8)+y(2,8)+y(1,9)+y(2,9))/4.)
dc7 = (dsoil - (y(1,7)+y(2,7)+y(1,8)+y(2,8))/4.)
dc6 = (dsoil - (y(1,6)+y(2,6)+y(1,7)+y(2,7))/4.)
dc5 = (dsoil - (y(1,5)+y(2,5)+y(1,6)+y(2,6))/4.)
dc4 = (dsoil - (y(1,4)+y(2,4)+y(1,5)+y(2,5))/4.)
dc3 = (dsoil - (y(1,3)+y(2,3)+y(1,4)+y(2,4))/4.)
dc2 = (dsoil - (y(1,2)+y(2,2)+y(1,3)+y(2,3))/4.)
dc1 = (dsoil - (y(1,1)+y(2,1)+y(1,2)+y(2,2))/4.)
end
ini xdis=0. ydis=0. ; reset displacement after gravity
prop e_plastic = 0. ; reset plastic strain
def tiltangle
  ytop = y(l,23)
ymid = y(1,14)
ybot = y(1,5)
smaxrat = (nh+hsoil)/(ytop-ymid+nh) ; max displ at wall bottom if displt at (1,11)=1
xvtoprat = (nh+hsoil-hmwall)/(nh+hsoil); vel at the wall top if vel at bottom of wall=1
xvbotrat = 1.
end
tiltangle
def shrat
  float shrat
  while_stepping
    shrat = abs(smaxrat*xdisp(l,14)/hsoil) ;
  end
hist 1 nstep= 2000 shrat
hist 2 unbalance
hist 3 e_plastic i=1 j=14
hist 41 sxx i=1 j=1
hist 42 sxx i=1 j=2
hist 43 sxx i=1 j=3
hist 44 sxx i=1 j=4
hist 45 sxx i=1 j=5
hist 46 sxx i=1 j=6
hist 47 sxx i=1 j=7
hist 48 sxx i=1 j=8
hist 49 sxx i=1 j=9
hist 50 sxx i=1 j=10
hist 51 sxx i=1 j=11
hist 52 sxx i=1 j=12
hist 53 sxx i=1 j=13
hist 54 sxx i=1 j=14
hist 55 sxx i=1 j=15
hist 56 sxx i=1 j=16
hist 57 sxx i=1 j=17
hist 58 sxx i=1 j=18
hist 59 sxx i=1 j=19
hist 60 sxx i=1 j=20
hist 61 sxx i=1 j=21
hist 62 sxx i=1 j=22
def fildata
  array adatshrat(17) ;datshrat(7)
anoshrat=17
nns=20+2*izones*jzones
adatshrat(1) = 0.0 ;
adatshrat(2) = 0.0001 ;
adatshrat(3) = 0.0002 ;
adatshrat(4) = 0.0004 ;
adatshrat(5) = 0.0006 ;
adatshrat(6) = 0.0008 ;
adatshrat(7) = 0.001 ;
adatshrat(8) = 0.002 ;
adatshrat(9) = 0.004 ;
adatshrat(10) = 0.006 ;
adatshrat(11) = 0.008 ;
adatshrat(12) = 0.010 ;
adatshrat(13) = 0.020 ;
adatshrat(14) = 0.040 ;
adatshrat(15) = 0.060 ;
adatshrat(16) = 0.080 ;
adatshrat(17) = 0.100 ;
end
fildata
def scanstress
  int count
  while stepping
    nst = 20 + 2 * izones * jzones
    loop i (1, anoshrat)
      if i > count then
        if shrat >= adatshrat(i)
          dcl_22
          nst = nst + i
          ytable(nst, 22) = dc1
          ytable(nst, 21) = dc2
          ytable(nst, 20) = dc3
          ytable(nst, 19) = dc4
          ytable(nst, 18) = dc5
          ytable(nst, 17) = dc6
          ytable(nst, 16) = dc7
          ytable(nst, 15) = dc8
          ytable(nst, 14) = dc9
          ytable(nst, 13) = dc10
          ytable(nst, 12) = dc11
          ytable(nst, 11) = dc12
          ytable(nst, 10) = dc13
          ytable(nst, 9) = dc14
          ytable(nst, 8) = dc15
          ytable(nst, 7) = dc16
          ytable(nst, 6) = dc17
          ytable(nst, 5) = dc18
          ytable(nst, 4) = dc19
          ytable(nst, 3) = dc20
          ytable(nst, 2) = dc21
ytable(nst,1) = dc22
ytable(nst,23) = shrat
xtable(nst,22) = -sxx(1,1)
xtable(nst,21) = -sxx(1,2)
xtable(nst,20) = -sxx(1,3)
xtable(nst,19) = -sxx(1,4)
xtable(nst,18) = -sxx(1,5)
xtable(nst,17) = -sxx(1,6)
xtable(nst,16) = -sxx(1,7)
xtable(nst,15) = -sxx(1,8)
xtable(nst,14) = -sxx(1,9)
xtable(nst,13) = -sxx(1,10)
xtable(nst,12) = -sxx(1,11)
xtable(nst,11) = -sxx(1,12)
xtable(nst,10) = -sxx(1,13)
xtable(nst,9) = -sxx(1,14)
xtable(nst,8) = -sxx(1,15)
xtable(nst,7) = -sxx(1,16)
xtable(nst,6) = -sxx(1,17)
xtable(nst,5) = -sxx(1,18)
xtable(nst,4) = -sxx(1,19)
xtable(nst,3) = -sxx(1,20)
xtable(nst,2) = -sxx(1,21)
xtable(nst,1) = -sxx(1,22)
xtable(nst,23) = shrat

count=i
exit
endif
endif
end_loop
end

set large

struct prop=l E=ewall l=iwall area=awall
def nnodelem
no_a = 1
elemab = 50
no_b = no_a + elemab
elembc = 5
no_c = no_b + elembc
gabcd = 10
no_d = no_c + gabcd
elemde = 4
no_e = no_d + elemde
elemef = 85
no_f = no_e + elemef
elemfg = 28
no_g = no_f + elemfg
elemhd=5
no_h = no_d - elemhd
end
nnodelem
struc node no_a 0., hwall
struc node no_b 0., bed pin fix y
struc node no_c bwall, bed
struc node no_d 0, bed1 fix x y
struc node no_e 0, 0 fix x y
struc node no_f lsoil, 0 fix x y
struc node no_g lsoil, hwall fix x y
struc node no_h bwall, bed1 fix x y ; for
struc beam beg node no_a end node no_b seg=elemab pr=1
struc beam beg node no_b end node no_c seg=elembc pr=1
struc beam beg node no_d end node no_e seg=elemde pr=1
struc beam beg node no_e end node no_f seg=elemef pr=1
struc beam beg node no_f end node no_g seg=elemfg pr=1
struc beam beg node no_h end node no_d seg=elemfg pr=1 ; for active
struc node no_a no_b fix y
free x i=1
free x i=73
free y j=1
interface 1 aside from node no_a to node no_b bside from 3,23 to 1,3
interface 2 aside from node no_b to node no_c bside from 1,11 to 1,2
interface 3 aside from node no_d to node no_e bside from 1,11 to 1,1
interface 4 aside from node no_e to node no_f bside from 1,1 to 73,1
interface 5 aside from node no_f to node no_g bside from 73,1 to 73,23
interface 6 aside from node no_h to node no_d bside from 1,11 to 1,1 ; active
def knksinter
dzmin1235=lsoil/izones
dzmin4=bed/4. ; height of lower portion
knks1=10*(bulk_mod(1,3)+4*shear_mod(1,3)/3)/dzmin1235
knks2=10*(bulk_mod(1,2)+4*shear_mod(1,2)/3)/dzmin1235
knks3=10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin1235
knks4=10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin4
knks5=10*(bulk_mod(izones,1)+4*shear_mod(izones,1)/3)/dzmin1235 ;
knks6=10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin1235 ; active
fbase=fdirsh
command
interface 1 friction=fwall kn=knks1 ks=knks1 tbond=0 sbr=0 bslip on
interface 2 friction=0. kn=knks2 ks=knks2 tbond=0 sbr=0 bslip on
interface 3 friction=fwall kn=knks3 ks=knks3 tbond=0 sbr=0 bslip on
interface 4 friction=fbase kn=knks4 ks=knks4 tbond=0 sbr=0 bslip on
interface 5 friction=fwall kn=knks5 ks=knks5 tbond=0 sbr=0 bslip on
interface 6 friction=fwall kn=knks6 ks=knks6 tbond=0 sbr=0 bslip on ; active
end_command
end
knksinter
window
def tab dila  ;correcting value of dilation angle as function of relative density
lowsig3=4903. ;lowest sig3 (Pa) data =0.05 kg/cm2
higsig3=392266. ;highest sig3 (Pa) data =4.0 kg/cm2
loodr=54.9075 ;average lowest data of Dr of Tatsuoka
dendr=79.4167 ;average highest data of Dr
nnd=20+izones*j zones
dila=((14.-7.)/(dendr-loodr))*(reldens-loodr)+7. ; for maximum dilation angle
loop m (l,izones)
loop n (l,j zones)
sigma3=abs(ex_2(m,n))
dendilx2=((0.020 - 0.006)/(higsig3-lowsig3))*(sigma3-lowsig3)+0.006
loodilx2=((0.031 - 0.010)/(higsig3-lowsig3))*(sigma3-lowsig3)+0.010
dilx2=((dendilx2-loodilx2)/(dendr-loodr))*(reldens-loodr)+loodilx2
dilres=((l.- 4.)/(higsig3-lowsig3))*(sigma3-lowsig3)+4.
dily1=((dila)/(0.02))*(-dilx2)+0.
nnd=nnd+1
xtable(nnd,1)=0.
ytable(nnd,1)=dily1
xtable(nnd,2)=0.02+dilx2
ytable(nnd,2)=dila
xtable(nnd,3)=0.06+dilx2
ytable(nnd,3)=dilres
command
prop dtab nnd i=m j=n
end_command
end_loop ;n
end_loop ; ;m
end
def tab_friction ;
nnn=20
td_dr=80.56 ;Dr from tatsuoka, average dense
tl_dr=52.78 ;Dr from tatsuoka, average loose
minsig3=9.8 ; the lowest allowed in the log equation
lowsig3=4903.325 ;lowest sig3 (Pa)
higsig3=392266. ;highest sig3 (Pa)
ld_dr=79.278 ; DR dense at low sig3
ll_dr=55.278 ; DR dense at low sig3,
hd_dr=79.556 ; DR dense at high sig3,
hl_dr=54.611 ; DR loose at high sig3,
loop m (1,izones) ; apply the operations to all elements
loop n (1,jzones)
nnn=nnn+1
dangle=ex_1(m,n)
sigma3=abs(ex_2(m,n)) ; obtain the current sigma 3
ld_epres=table(11,dangle) ; low sig3 dense at delta
ll_epres=table(12,dangle) ; low sig3 loose at delta
lo_epres=((ll_epres-ld_epres)/(ll_dr-ld_dr))*(reldens-ld_dr)+ld_epres
hd_epres=table(13,dangle) ; high sig3 dense at delta
hl_epres=table(14,dangle) ; high sig3 loose at delta
hi_epres=((hl_epres-hd_epres)/(hl_dr-hd_dr))*(reldens-hd_dr)+hd_epres
sigma3=abs(ex_2(m,n)) ; obtain the current sigma 3
epsres1=(((hi_epres-lo_epres)/(higsig3-lowsig3))*(sigma3-lowsig3)+lo_epres ;
epsres=epsres1*e1resatd90
if dangle <= datmin then
  fpeakrat = ca + ax1*dangle + ax2*(dangle)^2 + ax3*(dangle)^3
else
  fpeakrat = cb + bx1*dangle + bx2*(dangle)^2 + bx3*(dangle)^3
endif
if sigma3 <= minsig3 then
  sigma3=minsig3
endif
cfdirsh = afdirsh*ln(sigma3) + bfdirsh ;
fpeak=fpeakrat*(fplst)*cfdirsh ;
ld_epeak=table(7,dangle) ; low sig3 dense at delta change to decimal
ll_epeak=table(8,dangle) ; low sig3 loose at delta
lo_epeak=((ll_epeak-ld_epeak)/(ll_dr-ld_dr))*(reldens-ld_dr)+ld_epeak
hd_epeak=table(9,dangle) ; high sig3 dense at delta
hl_epeak=table(10,dangle) ; high sig3 loose at delta
hi_epeak=((hl_epeak-hd_epeak)/(hl_dr-hd_dr))*(reldens-hd_dr)+hd_epeak
epspeak1=(((hi_epeak-lo_epeak)/(higsig3-lowsig3))*(sigma3-lowsig3)+lo_epeak
epspeak=epspeak1*e1peakatd90
cfres = afres*ln(sigma3) + bfres ; the same for all Dr
fres1=fres*cfres
if fpeak <= fres1 then
  fpeak=fres1
  epspeak=epsres
endif
if epsres <= epspeak then
  epsres=epspeak
  fpeak=fres1
endif
itab1=table_size(1)
itab2=table_size(2)
loop i (1,itab1) ; before peak
\[\text{xtable}(nnn,i) = \text{xtable}(1,i) \times \text{epspeak} \]
\[\text{ytable}(nnn,i) = \text{ytable}(1,i) \times f\text{peak} \]
\text{end}\text{\_loop}
\text{loop } i (1, itab2) ; after peak
\text{ccc=itabl+i}
\text{\hspace{1cm} xtable}(nnn,ccc) = \text{xtable}(2,i) \times (\text{epsres-epspeak}) + \text{epspeak} ;
\text{\hspace{1cm} ytable}(nnn,ccc) = \text{ytable}(2,i) \times (f\text{peak-fres}1) + f\text{res}1
\text{end}\text{\_loop }
\text{command}
\text{\hspace{1cm} prop ftab nnn i=m j=n}
\text{end\_command }
\text{end\_loop ;n}
\text{end\_loop ;m}
\text{end}

\text{tab 1} 0.,0. 0.05,0.433 0.1,0.599 0.2,0.74 0.3,0.818 0.4,0.8708 ;
\text{tab 1} 0.5,0.9145 0.6,0.9426 0.7,0.9633 0.8,0.98 0.9,0.9931 1.,1. ;
\text{tab 2} 0.1,0.9689 0.2,0.8782 0.4,0.581 0.6,0.3132 0.8,0.101 0.9,0.041 1.,0. ;
\text{tab 7} 0.,1.647059 23.,1.441176 34.,0.882353 45.,1.058824 90.,1.;
\text{tab 8} 0.,2.111111 23.,1.355556 34.,1.511111 45.,1.410714 90.,1.;
\text{tab 9} 0.,1.295082 23.,1.065574 34.,0.918033 45.,1.131148 90.,1.;
\text{tab 10} 0.,1.380952 23.,0.928571 34.,1.119048 45.,1.238095 90.,1.;
\text{tab 11} 0.,1.455882 23.,1.250000 34.,0.823529 45.,1.308824 90.,1.;
\text{tab 12} 0.,1.232759 23.,0.732759 34.,0.663793 45.,0.729834 90.,1.;
\text{tab 13} 0.,1.252874 23.,1.321839 34.,0.850575 45.,1.172414 90.,1.;
\text{tab 14} 0.,1.108527 23.,0.806202 34.,0.899225 45.,1.356589 90.,1.;
\text{def movethewall}
\text{shrat = 0.0}
\text{xvtop = xvtoprat*(2.5e-7)*hsoil/0.5 }
\text{xvbot = xvbotrat*(2.5e-7)*hsoil/0.5 }
\text{delxv = (xvbot-xvtop)/(no_b-no_a)}
\text{loop n (1,1500) ;*****}
\text{loop m (1,400) ;==}
\text{loop i (no_a,no_b)}
\text{xvtopi = xvtop + (i-no_a)*delxv}
\text{command}
\text{\hspace{1cm} struc node i initial xvel xvtopi ;;m/time step}
\text{end\_command}
\text{end\_loop }
\text{command}
\text{\hspace{1cm} step 1}
\text{end\_command}
\text{end\_loop ;== loop m}
\text{if shrat}>=0.1001
\text{exit}
\text{endif}
endloop ;****** loop n
end
def createmovie1 ; shear strain fill structure
command
  window
  movie on file 9maxssi.dcx size 1080,670
  plot grid mag=1 green beam mag=1 yellow ssi fill interval 0.1
  movie off
  window
end_command
end
def framemovie2
xlow = -0.05
xupp = hsoil/0.5*1.
ylow = -0.05
yupp = hsoil/0.5*0.9171
end
framemovie2
def createmovie2 ; principle stress
command
  wind xlow,xupp ylow,yupp
  movie on file 9prinstr.dcx size 1080,670
  plot grid mag=1 iw beam mag=1 red stress
  movie off
  window
end_command
end
movethewall
plot hist 2 vs 1
copy 3unbalnc.pcx
plot hist 62 61 59 58 57 56 55 54 53 52 51 50 49 48 47 46 45 44 43 42 41 vs 1 ;
copy 1horstrs.pcx
plot table 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42
copy 4tabfric.pcx
plot hist 3 vs 1
copy 5eplast_shrat.pcx
plot hist 41 42 43 44 45 46 vs 1
copy 6botstress.pcx
plot grid mag=1 iw beam mag=1 yellow ssi fill interval 0.1
copy 1maxssi.pcx
plot grid mag=1 iw beam mag=1 yellow pl
copy 3plastic.pcx
wind xlow,xupp ylow,yupp
plot grid mag=1 iw beam mag=1 red stress
copy 2prnstrs.pcx
Example for Passive RTT in Anisotropic Soil

title
z1&r4mand32.dat: passive H=4.0m,n=0,Dr=32%,RTT
set cust1
Dept.of Civil and Env. Eng.
set cust2
Old Dominion U., Norfolk, Virginia
config extra 2
set echo off
grid 72 22
mod ss
set grav=9.81 ; m/s2
def dimension
hsoil = 4.0 ; INPUT, depth of soil adjacent to moving wall (m)
n = 0.0 ; INPUT, rotation about point at nH (m) below wall base
dsoil = 0.613*hsoil/0.5 ; soil depth (m), the same proportion with Fang's experiment
lsoil = 2.0*hsoil/0.5 ; soil length (m)
bed = 0.113*hsoil/0.5 ; bed depth (m)
bedl = 0.999*bed
nh = n*hsoil ; nH (m)
hwall = 0.8*hsoil/0.5 ; wall top to soil bottom (m)
hmwall = hwall-bed ; height of movable wall
twall = 0.12*hsoil/0.5 ; wall thickness (m)
awall = twall*1. ; area of movable wall in (m2)
iwall = 1*(twall^3)/12. ; moment area of the wall (m4)
evwall = 200e9 ; assuming steel (Pa)
bwall = -0.133*hsoil/0.5 ; coordinate of front edge of wall base
end
dimension
def backfill
  gs = 2.65 ; INPUT, specific gravity
  eemax = 0.76 ; INPUT, maximum void ratio
  eemin = 0.50 ; INPUT, minimum void ratio
  reldens = 32. ; INPUT, relative density (%)
  ca = 0.90843629742579 ; INPUT for Dr = 32
  ax1 = -4.719059E-04
  ax2 = 4.192983E-05
  ax3 = -8.95695E-07
  cb = 0.54985314357006
  bx1 = 1.270233E-02
  bx2 = -1.198577E-04
  bx3 = 3.814908E-07
  afdirsh = -0.006043 ; for DR=32
  bfdirsh = 1.077827
  afres = -0.0121220 ; for all DR
  bfres = 1.1561240
  walsanrat = -0.000025502*(reldens)^2 + 0.00385617*reldens + 0.
  gamwater = 9.81*1000
  ee = eemax-reldens*(eemax-eemin)/100.
  fdirsh = 0.000275*(reldens)^2 + 0.184275*reldens + 25.601176
  fres = 31.5
  cr = 0.96975756535439
  cd1 = -2.687556E-03
  cd2 = 1.147410E-05
  cd3 = 1.618853E-08
  minrat = cr + cd1*reldens + cd2*(reldens)^2 + cd3*(reldens)^3 ;
  fplst = fdirsh/minrat
  cdr = 50.6741218848
  cddl = -2.319605E-01
  cdd2 = -3.303401E-03
  cdd3 = 3.013308E-05
  datmin = cdr + cddl*reldens + cdd2*(reldens)^2 + cd3*(reldens)^3 ;
  e1peakat90 = (-0.015207*reldens + 3.403184)/100. ;
  e1resat90 = (-0.081055*reldens + 11.346580)/100. ;
  unitwei = gs*gamwater/(ee+1); unitweight (N/m3)
  fwall = walsanrat*fdirsh; wall friction
end
backfill
gen 0 0 0 bed lsoil bed lsoil 0 i=1,73 j=1,5
gen 0 bed 0 dsoil lsoil dsoil lsoil bed i=1,73 j=5,23
def setprop
  float zz sigm gg kk pois yc
float ko
dila=((14.-7.)/(79.4167-54.9075))*(reldens-54.9075)+7.;
dilaat0=-dila/2.

pa_psf=0.020885;  conversion pascal to psf
psi_pa=6894.75728;  conversion psi to pascal
pa = 101300;  atmospheric pressure in N/m2 (=Pa)
sss=1350;

loop i (1,izones)
  loop j (1,jzones)
    yc=(y(i,j)+y(i+1,j)+y(i,j+1)+y(i+1,j+1))/4.
    zz=y(jgp)-yc
    if fplst > 34 then
      ftriax=(fplst+17.)/1.5
    else
      ftriax=fplst
    endif
    ko=1-sin(ftriax*degrad)
    pois=(1-sin(ftriax*degrad))/(2-sin(ftriax*degrad))
    sigm=(unitwei*zz*(1.+2.*ko)/3.); average stress (Pascal)
    gg=(sss*1.*(sigm*pa)^0.5)/(2*(1+pois)*(0.3+0.7*ee^2)); in N/m2
    kk=2.*gg*(1.+pois)/(3.*(1.-2.*pois)); initial K
    shear_mod(i,j)=gg
    bulk_mod(i,j)=kk
    density(i,j)=unitwei/9.81; kg/m3
    cohesion(i,j)=0.
    tension(i,j)=0.
  endloop
endloop

setprop
tab 15 0.,dilaat0 0.03,dila 0.07,4.; initial value of dilation: loose and 0.05 of Tatsuoka
tab 16 0.,fplst 0.2,fplst; initial value of friction
tab 20 hsoil,n reldens,unitwei fplst,pois fdirsh,fwall
prop dtab 15
prop ftab 16
fix y j=1;bottom
fix x i=73;right wall
fix x i=1;left wall
set plot pcx bw
def angle_pq; angle and pq for all elements
loop i (1,izones)
  loop j (1,jzones)
    aaa=-(sxx(i,j)+syy(i,j))/2.; change sign to positive
    ccc=(sxx(i,j)-syy(i,j))/2.
    bbb=((ccc)^2+(sxy(i,j))^2)^0.5; always positive
\[ \sigma_1 = \text{aaa} + \text{bbb} \]
\[ \sigma_3 = \text{aaa} - \text{bbb} \]
\[ \text{ex}_2(i,j) = \sigma_3 \]

if \( \sigma_1 \neq \text{abs}(\text{sxx}(i,j)) \) then

\[ \chi = \text{atan} \left( \text{sxy}(i,j) / (\sigma_1 - \text{abs}(\text{sxx}(i,j))) \right) \]

else

\[ \chi = 90. \times \text{degrad} \]

endif

\[ \text{ex}_l(i,j) = 90 - \text{abs}(\chi / \text{degrad}) \]

end_loop

end_loop

def tiltangle

\[ y_{\text{top}} = y(l,23) \]
ymid = y(l,14)
ybot = y(l,5)
smaxrat = (nh+hsoil)/(ytop-ymid+nh); max disp at wall bottom if disp at (1,11)=1
xvtoprat = (nh+hsoil-hmwall)/(nh+hsoil); vel at the wall top if vel at bottom of wall=1
xvbotrat = 1.
end
tiltangle
def shrat
  float shrat
  while stepping
    shrat = abs(smaxrat*xdisp(l,14)/hsoil);
  end
hist 1 nstep= 2000 shrat
hist 2 unbalance
hist 3 e_plastic i=1 j=14
hist 41 sxx i=1 j=1 ; sept 16,
hist 42 sxx i=1 j=2
hist 43 sxx i=1 j=3
hist 44 sxx i=1 j=4
hist 45 sxx i=1 j=5
hist 46 sxx i=1 j=6
hist 47 sxx i=1 j=7
hist 48 sxx i=1 j=8
hist 49 sxx i=1 j=9
hist 50 sxx i=1 j=10
hist 51 sxx i=1 j=11
hist 52 sxx i=1 j=12
hist 53 sxx i=1 j=13
hist 54 sxx i=1 j=14
hist 55 sxx i=1 j=15
hist 56 sxx i=1 j=16
hist 57 sxx i=1 j=17
hist 58 sxx i=1 j=18
hist 59 sxx i=1 j=19
hist 60 sxx i=1 j=20
hist 61 sxx i=1 j=21
hist 62 sxx i=1 j=22
def fildata
  array adatshrat(17);datshrat(7)
anoshrat=17
  nns=20+2*izones*jzones ; after tatsuoka,friction,dilation
  adatshrat(1) = 0.0     ;
  adatshrat(2) = 0.0001  ;
  adatshrat(3) = 0.0002  ;
  adatshrat(4) = 0.0004  ;
def scanstress
    int count
    whilestepping
        nst=20+2*izones*jzones
        loop i (1,anoshrat)
            if i > count then
                if shrat >= adatshrat(i)
                    nst=nst+i
                    ytable(nst,22) = dc1
                    ytable(nst,21) = dc2
                    ytable(nst,20) = dc3
                    ytable(nst,19) = dc4
                    ytable(nst,18) = dc5
                    ytable(nst,17) = dc6
                    ytable(nst,16) = dc7
                    ytable(nst,15) = dc8
                    ytable(nst,14) = dc9
                    ytable(nst,13) = dc10
                    ytable(nst,12) = dc11
                    ytable(nst,11) = dc12
                    ytable(nst,10) = dc13
                    ytable(nst,9) = dc14
                    ytable(nst,8) = dc15
                    ytable(nst,7) = dc16
                    ytable(nst,6) = dc17
                    ytable(nst,5) = dc18
                    ytable(nst,4) = dc19
                    ytable(nst,3) = dc20
                    ytable(nst,2) = dc21
end
ytable(nst,1) = dc22
ytable(nst,23) = shrat
xtable(nst,22) = -sxx(1,1)
xtable(nst,21) = -sxx(1,2)
xtable(nst,20) = -sxx(1,3)
xtable(nst,19) = -sxx(1,4)
xtable(nst,18) = -sxx(1,5)
xtable(nst,17) = -sxx(1,6)
xtable(nst,16) = -sxx(1,7)
xtable(nst,15) = -sxx(1,8)
xtable(nst,14) = -sxx(1,9)
xtable(nst,13) = -sxx(1,10)
xtable(nst,12) = -sxx(1,11)
xtable(nst,11) = -sxx(1,12)
xtable(nst,10) = -sxx(1,13)
xtable(nst,9) = -sxx(1,14)
xtable(nst,8) = -sxx(1,15)
xtable(nst,7) = -sxx(1,16)
xtable(nst,6) = -sxx(1,17)
xtable(nst,5) = -sxx(1,18)
xtable(nst,4) = -sxx(1,19)
xtable(nst,3) = -sxx(1,20)
xtable(nst,2) = -sxx(1,21)
xtable(nst,1) = -sxx(1,22)
xtable(nst,23) = shrat
count=i
exit
endif
endif
end_loop
end
set large
struct prop=1 E=ewall I=iwall area=awall
def nnodelem
no_a = 1
elemab = 50
no_b = no_a + elemab
elembc = 5
no_c = no_b + elembc
gabcd = 10
no_d = no_c + gabcd
elemde = 4
no_e = no_d + elemde
elemef = 85
no_f = no_e + elemef
elemfg = 28
no_g = no_f + elemfg
elemhd=5
no_h = no_d - elemhd
end
nnodelem
struc node no_a 0., hwall
struc node no_b 0., bed pin fix y
struc node no_c bwall, bed
struc node no_d 0, bed1 fix x y
struc node no_e 0, 0 fix x y
struc node no_f lsoil, 0 fix x y
struc node no_g lsoil, hwall fix x y
struc node no_h bwall, bed1 fix x y ; for active
struc beam beg node no_a end node no_b seg=elemab pr=1
struc beam beg node no_b end node no_c seg=elembc pr=1
struc beam beg node no_d end node no_e seg=elemde pr=1
struc beam beg node no_e end node no_f seg=elemef pr=1
struc beam beg node no_f end node no_g seg=elemfg pr=1
struc beam beg node no_h end node no_d seg=elemfg pr=1 ; for active
struc node no_a no_b fix y
free x i=1
free x i=73
free y j=1
interface 1 aside from node no_a to node no_b bside from 3,23 to 1,3
interface 2 aside from node no_b to node no_c bside from 1,11 to 1,2
interface 3 aside from node no_d to node no_e bside from 1,11 to 1,1
interface 4 aside from node no_e to node no_f bside from 1,1 to 73,1
interface 5 aside from node no_f to node no_g bside from 73,1 to 73,23
interface 6 aside from node no_h to node no_d bside from 1,11 to 1,1 ; active
def knksinter
dzmin1235=lsoil/izones
dzmin4=bed/4. ; height of lower portion
knks1=10*(bulk_mod(1,3)+4*shear_mod(1,3)/3)/dzmin1235
knks2=10*(bulk_mod(1,2)+4*shear_mod(1,2)/3)/dzmin1235
knks3=10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin1235
knks4=10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin4
knks5=10*(bulk_mod(izones,1)+4*shear_mod(izones,1)/3)/dzmin1235
knks6=10*(bulk_mod(1,1)+4*shear_mod(1,1)/3)/dzmin1235 ; active
fbase=fdirsh
command
interface 1 friction=fwall kn=knks1 ks=knks1 tbond=0 sbr=0 bslip on
interface 2 friction=0. kn=knks2 ks=knks2 tbond=0 sbr=0 bslip on
interface 3 friction=fwall kn=knks3 ks=knks3 tbond=0 sbr=0 bslip on
interface 4 friction=fbase kn=knks4 ks=knks4 tbond=0 sbr=0 bslip on
interface 5 friction=fwall kn=knks5 ks=knks5 tbond=0 sbr=0 bslip on
interface 6 friction=fwall kn=knks6 ks=knks6 tbond=0 sbr=0 bslip on ; active
end_command
end
knksinter
window
def tab dila  ;correcting value of dilation angle as function of relative density
lowsig3=4903. ; lowest sig3 (Pa) data = 0.05 kg/cm2
higsig3=392266. ; highest sig3 (Pa) data = 4.0 kg/cm2
loodr=54.9075 ; average lowest data of Dr of Tatsuoka
dendr=79.4167 ; average highest data of Dr
nnd=20+izones*jzones
dila=((14.-7.)/(dendr-loodr))*(reldens-loodr)+7. ; for maximum dilation angle
loop m (1,izones)
loop n (1,jzones)
sigma3=abs(ex_2(m,n))
dendilx2=((0.020 - 0.006)/(higsig3-lowsig3))*(sigma3-lowsig3)+0.006
loodilx2=((0.031 - 0.010)/(higsig3-lowsig3))*(sigma3-lowsig3)+0.010
dilx2=((dendilx2-loodilx2)/(dendr-loodr))*(reldens-loodr)+loodilx2
dilres=((l .- 4.)/(higsig3-lowsig3))*(sigma3-lowsig3)+4.
dily1=((dila)/(0.02))*(-dilx2)+0.
nnd=nnd+1
xtable(nnd,1)=0.
ytable(nnd,1)=dily1
xtable(nnd,2)=0.02+dilx2
ytable(nnd,2)=dila
xtable(nnd,3)=0.06+dilx2
ytable(nnd,3)=dilres
command
  prop dtab nnd i=m j=n
end_command
end_loop ;n
end_loop ;m
def tab friction ; creating friction table for each zone==
nnn=20
td_dr=80.56 ; Dr from tatsuoka, average dense
tl_dr=52.78 ; Dr from tatsuoka, average loose
minsig3=9.8 ; the lowest allowed in the log equation
lowsig3=4903.325 ; lowest sig3 (Pa)
higsig3=392266. ; highest sig3 (Pa)
l_d_dr=79.278 ; DR dense at low sig3
ll_dr=55.278 ; DR loose at low sig3,
h_d_dr=79.556 ; DR dense at high sig3,
hl_dr=54.611 ; DR loose at high sig3,
loop m (1,izones) ; apply the operations to all elements
loop n (1,jzones)
nnn=nnn+1

dangle=ex_1(m,n)
sigma3=abs(ex_2(m,n)) ; obtain the current sigma 3
ld_epres=table(11,dangle) ; low sig3 dense at delta
ll_epres=table(12,dangle) ; low sig3 loose at delta
lo_epres=((ll_epres-ld_epres)/(ll_dr-ld_dr))*(reldens-ld_dr)+ld_epres
hd_epres=table(13,dangle) ; high sig3 dense at delta
hl_epres=table(14,dangle) ; high sig3 loose at delta
hi_epres=((hl_epres-hd_epres)/(hl_dr-hd_dr))*(reldens-hd_dr)+hd_epres
sigma3=abs(ex_2(m,n)) ; obtain the current sigma 3
epsres1=((hi_epres-lo_epres)/(higsig3-lowsig3))*(sigma3-lowsig3)+lo_epres ;
epsres=epsres1*e1resatd90
if dangle <= datmin then
    fpeakrat = ca + ax1*dangle + ax2*(dangle)^2 + ax3*(dangle)^3
else
    fpeakrat = cb + bx1*dangle + bx2*(dangle)^2 + bx3*(dangle)^3
endif
if sigma3 <= minsig3 then
    sigma3=minsig3
endif

cfdirsh = afdirsh*ln(sigma3) + bfdirsh ; correction to fdirsh
fpeak=fpeakrat*(fplst)*cfdirsh; fpeak of current dangle; fplst = from plane strain
ld_epeak=table(7,dangle) ; low sig3 dense at delta change to decimal
ll_epeak=table(8,dangle) ; low sig3 loose at delta
lo_epeak=((ll_epeak-ld_epeak)/(ll_dr-ld_dr))*(reldens-ld_dr)+ld_epeak
hd_epeak=table(9,dangle) ; high sig3 dense at delta
hl_epeak=table(10,dangle) ; high sig3 loose at delta
hi_epeak=((hl_epeak-hd_epeak)/(hl_dr-hd_dr))*(reldens-hd_dr)+hd_epeak
epspeak1=((hi_epeak-lo_epeak)/(higsig3-lowsig3))*(sigma3-lowsig3)+lo_epeak
epspeak=epspeak1*e1peakatd90
cfres = afres*ln(sigma3) + bfres ; the same for all Dr
fres1=fres*cfres
if fpeak <= fres1 then
    fpeak=fres1
    epspeak=epsres
endif
if epsres <= epspeak then
    epsres=epspeak
    fpeak=fres1
endif

itab1=table_size(1)
itab2=table_size(2)
loop i (1,itab1) ; before peak
xtable(nnn,i) = xtable(1,i)*epspeak
ytable(nnn,i) = ytable(1,i)*fpeak
end_loop
loop i (1, itab2) ; after peak
    ccc = itab1 + i
    xtable(nnn,ccc) = xtable(2,i)*(epsres-epspeak)+epspeak ;
    ytable(nnn,ccc) = ytable(2,i)*(fpeak-fresl)+fresl
end_loop
command
    prop ftab nnn i=m j=n
end_command
end_loop ; n
end_loop ; m
end

def movethewall
    shrat = 0.0
    xvtop = xvtoprat*(2.5e-7)*hsoil/0.5
    xvbot = xvbotrat*(2.5e-7)*hsoil/0.5
    delxv = (xvbot-xvtop)/(no_b-no_a)
loop n (1,1500) ;*****
    angle_pq
    tab_dila
    tab_friction
loop m (1,400) ;===
    loop i (no_a,no_b)
        xvtopi = xvtop + (i-no_a)*delxv
        command
            struc node i initial xvel xvtopi ;; m/time step
        end_command
    endloop
    command
    endloop
    step 1
    end_command
endloop ;=== loop m
; createmovie1
; createmovie2
  if shrat>=0.1001
    exit
  endif
endloop  ;****** loop n
end

def createmovie1  ; shear strain fill structure
  command
    window
      movie on file 9maxssi.dcx size 1080,670
      plot grid mag=1 green beam mag=1 yellow ssi fill interval 0.1
      movie off
    window
  end_command
end

def framemovie2
  xlow = -0.05
  xupp = hsoil/0.5*1.
  ylow = -0.05
  yupp = hsoil/0.5*0.9171
end
framemovie2

def createmovie2  ; principle stress
  command
    wind xlow,xupp ylow,yupp
    movie on file 9prinstr.dcx size 1080,670
    plot grid mag=1 iw beam mag=1 red stress
    movie off
    window
  end_command
end
movethewall
plot grid mag=1 blue beam mag=1 lmagenta ssi fill interval 0.1
copy 1maxssi.pcx
plot grid mag=1 blue beam mag=1 lmagenta pl blue
copy 3plastic01.pcx
plot grid mag=1 blue beam mag=1 lmagenta pl green
copy 3plastic02.pcx
plot grid mag=1 blue beam mag=1 lmagenta pl cyan
copy 3plastic03.pcx
plot grid mag=l blue beam mag=l lmagenta pl red
copy 3plastic04.pcx
plot grid mag=l blue beam mag=l lmagenta pl magenta
copy 3plastic05.pcx
plot grid mag=l blue beam mag=l lmagenta pl brown
copy 3plastic06.pcx
plot grid mag=l blue beam mag=l lmagenta pl white
copy 3plastic07.pcx
plot grid mag=l blue beam mag=l lmagenta pl gray
copy 3plastic08.pcx
plot grid mag=l blue beam mag=l lmagenta pl lblue
copy 3plastic09.pcx
plot grid mag=l blue beam mag=l lmagenta pl lgreen
copy 3plastic10.pcx
plot grid mag=l blue beam mag=l lmagenta pl lcyan
copy 3plastic11.pcx
plot grid mag=l blue beam mag=l lmagenta pl lred
copy 3plastic12.pcx
plot grid mag=l blue beam mag=l lmagenta pl limage

copy 3plastic13.pcx
plot grid mag=l blue beam mag=l lmagenta pl yellow
copy 3plastic14.pcx
plot grid mag=l blue beam mag=l lmagenta pl lwhite
copy 3plastic15.pcx
wind xlow,xupp ylow,yupp
plot grid mag=l blue beam mag=l lmagenta stress green
copy 2prnstrs.pcx
window
set log 8fang_result.txt ; printing properties
set log on
print table 20
def cetak
nst=20+2*izones*jzones ;+noshrat ; stress result
loop j (1,anoshrat)
    nst=nst+1
    command
    print table nst  
    end_command
end_loop : j
end
cetak
set log off
return
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