Remediation of liquefaction effects for embankment dams using soil-cement walls

[20-25 min]
Soil-cement walls/grids in dam foundations

- Examples of US projects using soil-cement walls/grids
  - Clemson Diversion Dams, SC (Wooten & Foreman 2005)
  - Sunset North Basin Dam, CA (Barron et al. 2006)
  - San Pablo Dam, CA (Kirby et al. 2010)
  - Perris Dam, CA (Friesen & Balakrishnan 2012)
  - Chabot Dam, CA (EBMUD 2016)

- Common analysis approach is a 2D section with composite (area-weighted) properties for the treatment zone

Clemson Diversion Dam (Wooten & Foreman 2005)
Motivation for present study

Questions:
- Will soil-cement walls or grids be effective in reducing deformations if they crack extensively during shaking?
- Do our 2D analysis methods adequately predict the benefits of soil-cement wall or grid reinforcements?

Approach:
- Dynamic centrifuge model test with soil-cement walls designed to
  - not shear or crack (extensively) in a smaller shaking event
  - shear/crack extensively in a larger shaking event
- 2D nonlinear dynamic analyses (NDAs) using composite properties for the treatment zone
Outline of presentation

- **Dynamic centrifuge model test**
  - Configuration and a few observations

- **Nonlinear dynamic analyses**
  - Model development and parameter calibration/selections
  - Results for the two large shaking events

- **Discussion & concluding remarks**
Acknowledgments

- Collaborators & coauthors

Mohammad Khosravi  
Post-doc UC Davis

Ali Khosravi  
Sharif Tech U

Dan Wilson  
Assoc. Director CGM
Centrifuge test
9-m radius centrifuge at UC Davis

- A national shared-use NHERI Equipment Facility sponsored by NSF
Model configuration

- Test at 65g; Results in prototype units unless specified otherwise
- 28-m tall embankment over 9-m-thick loose sand layer
- Pore fluid was methylcellulose mixture with viscosity of about 15 cP
Model configuration

- 9 walls in longitudinal direction (also shaking direction)
- Flexible shear beam container
Model configuration

- Model was shaken 3 times with a modified recording from Port Island, 1995 Kobe earthquake
  - Scaled to peak base accelerations (PBA) of 0.05, 0.26, & 0.54 g
Soil-cement walls

- Sand-cement mixture formed & cured in molds.
- Wall thickness = 1.4 m; Spacing = 5.8 m; Area ratio = 24%
- $q_{ucs} = 2.0$ MPa at time of centrifuge testing
- Crack detectors embedded in walls
- Post-test excavation photos (toe of berm is to the left side of photos)
Soil-cement walls

- Toe of berm is to the bottom of these photos
- Liquefied sand extruded up to 0.8 m relative to the walls

Construction  Dissection
NDA model
Mesh and materials

- 2D coupled simulations using FLAC 8.0 (Itasca 2016)
  - Foundation & embankment sands: PM4Sand (version 3)
  - Soil cement: Mohr Coulomb with area-weighted properties
  - Container: Elastic with equivalent density & stiffness per unit width
  - "Bedrock": Elastic

![Diagram showing mesh and materials](image-url)

- Monterey sand, dense
- Soil-cement panel zone
- Ottawa sand, loose
- "Bedrock" (concrete) base
Plasticity model for sand – PM4Sand

- PM4Sand version 3 by Boulanger & Ziotopoulou (2015) and Ziotopoulou & Boulanger (2016)
  - Critical state & stress-ratio based, bounding surface plasticity model developed to approximate stress-strain responses important to earthquake engineering practice
- Generalized calibration for liquefaction problems
  - Three primary parameters for most applications \((D_R, G_o, h_{po})\)
  - Default values for secondary parameters
- Implemented as a user-defined dynamic link library (dll) for use with FLAC 7.0 (Itasca 2011) and FLAC 8.0 (Itasca 2016)
  - Manual, dll, and example files are at: https://pm4sand.engr.ucdavis.edu/
Calibration of PM4Sand for Ottawa Sand

- Secondary parameters: all default values except $e_{\text{max}}=0.83$, $e_{\text{min}}=0.51$
- Cyclic strengths from DSS tests by Parra Bastidas et al. (2017)
- 1st & 2nd events
  - $D_R=42\%$
  - $G_0=770$ for $V_{s1}=185$ m/s
  - $h_{po}=0.54$ for
    - $CRR_{15\text{cycles}}=0.093$ at $\sigma'_{vc}=400$ kPa

![Diagram showing cyclic stress ratio vs. number of cycles to 3% peak shear strain]
Calibration of PM4Sand for Ottawa Sand

- Secondary parameters: all default values except $e_{\text{max}} = 0.83$, $e_{\text{min}} = 0.51$
- Cyclic strengths from DSS tests by Parra Bastidas et al. (2017)
- 1st & 2nd events
  \[ D_R = 42\% \]
  \[ G_o = 770 \text{ for } V_{s1} = 185 \text{ m/s} \]
  \[ h_{po} = 0.54 \text{ for } \]
  \[ \text{CRR}_{15\text{cycles}} = 0.093 \text{ at } \sigma'_{vc} = 400 \text{ kPa} \]
- 3rd event
  \[ D_R = 45\% \]
  \[ G_o = 788 \text{ for } V_{s1} = 187 \text{ m/s} \]
  \[ h_{po} = 1.20 \text{ for } \]
  \[ \text{CRR}_{15\text{cycles}} = 0.120 \text{ at } \sigma'_{vc} = 400 \text{ kPa} \]
Calibration of PM4Sand for Ottawa Sand

- Secondary parameters: all default values except $e_{\text{max}}=0.83$, $e_{\text{min}}=0.51$

- Cyclic strengths from DSS tests by Parra Bastidas et al. (2017)

1\textsuperscript{st} & 2\textsuperscript{nd} events

\[ D_r = 42\% \]

$G_0 = 770$ for $V_{s1} = 185$ m/s

\[ h_{po} = 0.54 \]

\[ CRR_{15\text{cycles}} = 0.093 \text{ at } \sigma_{vc}' = 400 \text{ kPa} \]

3\textsuperscript{nd} event

\[ D_r = 45\% \]

$G_0 = 788$ for $V_{s1} = 187$ m/s

\[ h_{po} = 1.20 \]

\[ CRR_{15\text{cycles}} = 0.120 \text{ at } \sigma_{vc}' = 400 \text{ kPa} \]
Calibration of PM4Sand for Ottawa Sand

- Simulated response for an initial static shear bias representative of conditions beneath the embankment:
  a) With uniform CSR loading
  b) With stepped CSR loading

- Other key behaviors for the model are illustrated in Boulanger & Ziotopoulou (2015)
Soil-cement stress-strain responses

- Tatsuoka & Kobayashi (1983) showed relatively modest strain-softening for cement stabilized Tokyo Bay Clay (LL=100, PI=46) with w=120% and binder factor of 14% in ICU triaxial compression tests.

- Filz (personal communication) observed a greater rate of strain-softening in ICU triaxial tests for cemented treated loamy soil.

- The average resistance during shaking will be intermediate to the peak and final strengths and depend on the degree of strain softening.
  - Used 80% of peak strength for our baseline model.
Composite properties for treatment zone

- **Soil-cement shear strength:**
  - Mohr Coulomb failure envelope with $\phi = 0$
  - Used 80% of peak strength to allow for progressive reduction in shear resistance with increasing strain
  - Baseline case is therefore $(S_u)_{\text{panel}} = 0.8 (q_{\text{ucs}}/2)$

- **Composite strength in saturated loose sand layer**
  - Assumed zero shear resistance in the loose sand between the panels because it liquefies and the panels are much stiffer
  - Area-weighted strength for Mohr Coulomb $(c-\phi)$ model is
    \[ c = A_r (S_u)_{\text{panel}} \text{ with } \phi = 0 \]

- **Composite strength in dense embankment sand**
  - Assumed drained strength for the dense Monterey sand
  - Area-weighted strength for Mohr Coulomb $(c-\phi)$ model is
    \[ c = A_r (S_u)_{\text{panel}} \\
    \phi = \tan^{-1}[(1-A_r)\tan(\phi)_{\text{sand}}] \]
NDA results for baseline model:

Kobe 0.26 g event
Baseline model – Kobe 0.26 g event

Measured

Computed

Crest (AH-E18)

Emb. face (AH-E16)

Top of berm (AH-E6)

Mid emb. (AH-E10)

Mid emb. (AH-E11)

Top of SC wall (AH-W11)

Upper fdtn. (AH-S11)

Upper fdtn. (AH-S12)

Upper fdtn. (AH-S15)

Base

Base

Time (s)
Baseline model – Kobe 0.26 g event

- **P11** ($\sigma'_{vo} = 451$ kPa)
- **P12** ($\sigma'_{vo} = 313$ kPa)
- **P22** ($\sigma'_{vo} = 18$ kPa)
- **P1** ($\sigma'_{vo} = 504$ kPa)
- **P7** ($\sigma'_{vo} = 358$ kPa)
- **P10** ($\sigma'_{vo} = 56$ kPa)
Baseline model – Kobe 0.26 g event

Embarkment crest

Measured (E5)

Computed

Berm over the soil-cement section

Measured (E2)

Computed

Time (s)

Vertical displacement (m)

Horizontal displacement relative to top ring (m)
Baseline model – Kobe 0.26 g event
Response of Ottawa sand beneath embankment (location of P1): Kobe 0.26 g event
NDA results for baseline model:

Kobe 0.54 g event
Baseline model – Kobe 0.54 g event
Baseline model – Kobe 0.54 g event

- **P11** ($\sigma_{v0} = 451$ kPa)
- **P12** ($\sigma_{v0} = 313$ kPa)
- **P22** ($\sigma_{v0} = 18$ kPa)
- **P1** ($\sigma_{v0} = 504$ kPa)
- **P7** ($\sigma_{v0} = 358$ kPa)
- **P10** ($\sigma_{v0} = 56$ kPa)
Baseline model – Kobe 0.54 g event
Baseline model – Kobe 0.54 g event
Baseline model – Kobe 0.54 g event

Response of Ottawa sand beneath embankment (location of P1): Kobe 0.54 g event
Sensitivity analyses
Percent of peak $S_u$ assigned to soil-cement

- 80% of peak $S_u$ for soil-cement gave best overall results
- Over-estimating crest settlement in Kobe 0.54g event in all cases
Over-estimating crest settlement in Kobe 0.54g event

- Rotational deformations in embankment appear to be greater in the simulations than in the test.
- Strengthening the embankment ($D_R, n_b$) or loose sand ($D_R, h_{po}, n_b$) reduces the over-prediction of crest settlement in this event, but results in under-prediction of deformations for other events/locations.
Simulations without the treatment zone showed

- crest settlements of 3.1 m (4.7 times the value measured with soil-cement walls)
- berm horiz. displacements of 5.0 m (3.6 times the value measured with soil-cement walls)
Discussion
Complexity of deformation mechanisms

- Equivalent 2D model does not account for a number of mechanisms
  - Extrusion of sand between soil-cement walls – greatest at downstream end with local slumping of the overlying berm
  - Irregular cracking and offsets that varied along the length of the soil-cement walls and between adjacent walls
  - Offsets along irregular cracks likely accompanied by local fluctuations in normal stress, which would contribute to changes in excess pore pressure in the shear zones/cracks
  - Excess pore pressures in soil-cement shear zones are likely negative based on the strength of the soil-cement
  - Expect pore water to flow from the liquefied sand between the walls into the cracks in the soil-cement walls; rate should depend on hydraulic conductivities, crack apertures, wall thickness, and the hydraulic gradients
  - Shear resistance along the cracks likely degrades progressively during shaking as the cracks grow and offsets develop
Conclusions
Conclusions

- The 2D NDAs were in reasonable agreement with the responses recorded in the centrifuge model, providing support for the use of these numerical modeling procedures in practice
  - Soil cement walls had limited cracking in the 0.26g Kobe event, but were sheared through in the 0.54g Kobe event
  - Deformation in the treatment zone included complex cracking patterns, offsets along wall cracks, & extrusion of liquefied soil
  - Analyses represented the treatment zone with area-weighted properties, a non-degrading Mohr Coulomb model, and an average soil-cement shear strength \( S_u = 0.8 \left( \frac{q_{ucs}}{2} \right) \)
  - Analyses do not recreate the complex local mechanisms of deformation in the treatment zone, but appear to approximate the global behavior reasonably well

- 2D nonlinear dynamic analyses of the embankment without the treatment zone indicate that the soil-cement walls were effective in reducing embankment deformations even though the walls were extensively damaged in the 0.54g Kobe event

- Soil-cement walls/grids can be an effective remediation strategy for embankment dams with liquefiable or weak foundation soils
Acknowledgments

- Funding for the centrifuge facility
  - National Science Foundation (NSF) through NEES & NHERI
- Funding for the centrifuge tests
  - Pacific Earthquake Engineering Research (PEER) Center
- Funding for the numerical analyses
  - California Department of Water Resources (DWR)
- Assistance with centrifuge tests:
  - W. Yunlong (China Earthquake Administration)
  - Amber Pulido (CGM undergraduate assistant)
  - CGM staff (Chad Justice, Tom Kohnke, Anatoliy Ganchenko)
References


References


