Liquefaction Hazard to Bridge Foundations

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Mechanism of Liquefaction
- Seismic waves propagating through soils generate shear deformations that collapse loose granular structures
- Collapses of granular structures transfer stress from particle contacts to the interstitial pore water
- Increasing pore water pressure and reducing effective stress
- When the pore pressure reaches a critical level, the previously solid granular soil transforms into a viscous liquid and liquefaction has occurred

Definition
- Liquefaction: “the transformation of granular material from a solid state to a liquid state as a consequence of increased pore water pressure and reduced effective stress”
  (ASCE Committee on Soil Dynamics, 1978)

Evaluation of Liquefaction Hazard

Fundamental Questions:
1. Will liquefaction occur (based on FS)?
   - No – no liquefaction hazard, no mitigation required
   - Yes – continue to question 2
2. Will liquefaction lead to detrimental ground deformation, ground displacement, or ground failure?
   - No – accept liquefaction hazard; no mitigation required
   - Yes – continue to Question 3
3. What mitigation is required to reduce liquefaction hazard to an acceptable risk?

Question 1. Will liquefaction occur?

Apply a verified evaluation procedure:
- Several procedures are in use worldwide for evaluating FS against triggering of liquefaction, including those of Youd et al (2001), Cetin, Seed et al (2004) and Idriss and Boulanger (2008) which are the most commonly used procedures in the US
- Although there are significant differences between the three procedures, they generally yield FS within +/- 30% of mean of the three procedures
- Although large, +/- 30% uncertainties are not usual in geotechnical engineering analyses
- For those instances in which FS is critical, more than one procedure should be applied, with conservative engineering judgment to select an appropriate FS

Simplified Procedure for Evaluation of Liquefaction Resistance

Seed and Idriss (1971) formulated the following equation for calculation of CSR:

$$ CSR = \frac{CRR}{CSR_{7.5}} = 0.65 \left(\frac{a_{av}}{a_{rms}}\right) \left(\frac{E_{cr}}{E_{cr,0}}\right) $$

(1)

where $a_{rms}$ is the peak horizontal acceleration at ground surface generated by the earthquake, $g$ is the acceleration of gravity, $a_{av}$ and $a_{rms}$ are total and effective vertical overburden stresses, respectively, and $E_{cr}$ is a stress reduction coefficient. The latter coefficient provides an approximate correction for flexibility of the soil profile.

This equation is used directly in the procedures by Youd et al., Cetin and Seed et al., and Idriss and Boulanger and nearly all other procedures.

The factor of Safety (FS) against triggering of liquefaction is:

$$ FS = \frac{FS}{CSR_{7.5}} \times MSF $$

Where:
- $CRR$ = cyclic resistance ratio, generally determined from field penetration tests such as SPT and CPT (capacity function)
- $CSR =$ cyclic stress ratio for $M = 7.5$ earthquakes (demand function)
- $MSF = $ magnitude scaling factor used to scale FS for magnitudes other than 7.5
Question 2. Will liquefaction lead to detrimental ground deformation, ground displacement, or ground failure?

Types of liquefaction-induced ground failure

- Flow failure
- Laterals spread
- Ground oscillation
- Loss of bearing strength
- Ground settlement
Flow failure, Half Moon Bay, Calif., 1906 San Francisco Earthquake

Aerial View of San Fernando Valley Juvenile Hall Lateral Spread area after 1971 San Fernando, California earthquake; ground slope across lateral spread zone about 1%

San Fernando Valley Juvenile Hall Damaged by Lateral Spread During 1971 San Fernando, Calif. Earthquake

Ground beneath standing part of building moved 1 m toward camera relative to stable ground beyond left end of building

Fissures and ground displacements (up to 2 m) generated by Juvenile Hall Lateral Spread

LATERAL SPREAD

Before earthquake

INITIAL SECTION

After earthquake

DEFORMED SECTION
Interior view of building pulled apart by lateral spread, San Fernando Valley Juvenile Hall

Diagrammatic view of building damage caused by San Fernando Valley Juvenile Hall lateral spread

Wall around San Fernando Valley Juvenile Hall pulled apart by lateral spread during 1971 earthquake; about 50 juveniles escaped through hole shortly after earthquake

Craters and flooding due to gas and water pipeline breaks, San Fernando Juvenile Hall lateral spread

Measured lateral spread displacement around N-Building following the 1964 Niigata, Japan earthquake

1985 photo of fractured piles beneath N-Building caused by lateral spread during 1964 Niigata, Japan earthquake
Diagrams of (a) post-earthquake pile configuration at N-building site and (b) plot of standard penetration resistance, N, versus depth.

Tall building supported on piles pulled apart at foundation level by lateral spread toward nearby island edge; building is located on, Rokko Island; damage occurred during 1995 Kobe, Japan earthquake (photo by Les Harder).

Bandai bridge pier displaced toward Shinano River during 1964 Niigata, Japan earthquake (M = 7.5); bridge deck acted as buttress causing the bridge pier to tilt away from the river rather than toward it.

Rio Bananitio railway bridge that tipped downstream during the 1991 Limon Province, Costa Rica earthquake (M = 7.6).

Lateral spread tilted the caisson toward river with capital block sliding off, allowing bridge truss to tip; note that in this instance the truss did restrain the tops of the caissons allowing them to tip freely toward the river.
11/29/2011

Ground Settlemnet

0.75 m of differential ground settlement between top of pile cap and surrounding ground due to liquefaction and compaction of 12 m of loose artificial fill during 1995 Kobe, Japan earthquake (M = 7.6)

Prediction of Lateral Spread Displacement

Empirical MLR Equations
Youd, Hanson and Bartlett, 2002

- Widely used for prediction of lateral spread displacement
- Based on case history data from several U.S. and Japanese earthquakes
- Data complied from about 500 lateral spread locations
- Equations regressed using multiple linear regression (MLR) procedure
Equations

T.L. Youd, C.M. Hansen and S.F. Bartlett

Free Face Conditions

\[ \text{Log DH} = -16.713 + 1.532 M - 1.406 \log R^* - 0.012 R + 0.540 \log T_{15} + 0.592 \log W + 3.413 \log (100 - F_{15}) - 0.795 \log (D_{5015} + 0.1 \text{ mm}) \]

Ground Slope Conditions

\[ \text{Log DH} = -16.213 + 1.532 M - 1.406 \log R^* - 0.012 R + 0.540 \log T_{15} + 0.338 \log S + 3.413 \log (100 - F_{15}) - 0.795 \log (D_{5015} + 0.1 \text{ mm}) \]

Where:

\[ R^* = R + R_o \]

and

\[ R_o = 10^{(0.89M - 5.64)} \]

Case history data compiled for MLR analysis:

Seismic parameters:

- \( M \) = Moment magnitude
- \( R \) = Horizontal distance from site to seismic energy source, in km

Topographic parameters

- \( W \) = Free face ratio, in percent
- \( S \) = Ground slope, in percent

Geotechnical parameters

- \( T_{15} \) = Thickness of layer with \((N_1)_{60} < 15 \text{ in } m\)
- \( F_{15} \) = Average fines content in \( T_{15} \) layer, in percent
- \( D_{5015} \) = Average mean grain size in \( T_{15} \) layer, in mm

Measured versus predicted displacements from revised MLR relationship

Collapsed bridge over Rio Viscaya; collapse caused by 3 m lateral spread displacement of floodplain deposits toward channel; ten bridges collapsed or were severely damaged due to liquefaction

Open fissures in roadway leading to collapsed Rio Viscaya bridge (behind camera); fissures opened due to soil tension caused by lateral spread of floodplain toward river

Case histories of liquefaction-induced lateral spread damage to bridges—1991 Limon Province, Costa Rica earthquake (M = 7.6)
Location map (courtesy of Google Earth™ 2011) with locations of damaged bridges analyzed by Kevin Franke (BYU PhD dissertation) and bridges discussed herein.

Highway bridge over Rio Cuba compressed by lateral spread of floodplain toward river channel.

View of Eastern abutment and bridge girders.

Eastern abutment bridge over Rio Cuba.

Sheared bridge seating due to compression of ground between abutments.

Bridge Plan, Rio Cuba crossing.
Foundation Description

According to bridge plans provided by the Costa Rican Ministry of Transportation, bridge is founded on a series of 14-inch square reinforced concrete piles. The abutments are supported by two rows of piles (8 piles in front row, 7 piles in second row) that are approximately 14 meters long and spaced at 4.1 diameters in the transverse direction and 2.5 diameters in the longitudinal direction. The dimensions of the pile cap at each abutment is 10.36 meters (transverse) x 1.90 meters (longitudinal) x 2.60 meters (vertical). (Kevin Franke, PhD dissertation, BYU)
Deterministically computed pile response for east abutment, Rio Cuba Bridge (courtesy Kevin Franke)

Collapsed Rio Estrella Bridge

Collapsed Estrella bridge with improvised temporary ramp

Central pier of collapsed Rio Estrella highway bridge

Shattered and spread highway embankment approaching eastern end of Rio Estrella bridge

Cracked and settled highway embankment at eastern abutment of collapsed Rio Estrella highway bridge (University of Costa Rica photo)
Eastern abutment of Rio Estrella highway bridge

TABLE 6-1 Comparison of planned and measured post-earthquake distances between bridge elements for the highway bridge of Rio Estrella.

<table>
<thead>
<tr>
<th>Distance between center of bridge units m</th>
<th>Planned Distance m</th>
<th>Measured post-earthquake distance m</th>
</tr>
</thead>
<tbody>
<tr>
<td>WEST Abutment and Pier 1</td>
<td>28.00</td>
<td>24.16</td>
</tr>
<tr>
<td>Pier 2 and EAST Abutment</td>
<td>75.00</td>
<td>75.24</td>
</tr>
<tr>
<td>WEST &amp; EAST Abutments</td>
<td>176.52</td>
<td>176.14</td>
</tr>
</tbody>
</table>

Note: liquefaction and lateral spread did not generate significant displacement of bridge piers or abutments at Rio Estrella bridge.

Foundation Data for Eastern Bent and Abutment

The eastern bent is founded on two 3.94-meter x 4.94-meter pile caps. Each pile cap is supported by twenty 12BPS3 steel piles (four rows of five piles) that are 20 meters in length and spaced at 1 meter intervals in both the transverse and longitudinal directions. The eastern abutment of the bridge was designed to be converted into a bent in the event of a bridge expansion. The bent is founded on two 3.96-meter x 5.96 meter pile caps. Each of these pile caps is supported by 24 12BPS3 steel piles (four rows of six piles) that are 20 meters in length and spaced at 1 meter intervals in both the transverse and longitudinal directions. Finally, the front row of piles at each abutment and bent are battered at approximately 5V:1H. (Franke, BYU PhD dissertation)
Borehole P1

Lateral Spread Displacement Calculation—Eastern Abutment Rio Estrella Bridge

Ground Slope Equation:

\[
\text{Log } Dh = -16.213 + 1.532 M - 1.406 \text{ Log } R* - 0.012 R + 0.338 \text{ Log } S + 0.540 \text{ Log } T15 + 3.413 \text{ Log } (100 - F15) - 0.795 \text{ Log } (D5015 + 0.1 \text{ mm})
\]

Free Face Equation:

\[
\text{Log } Dh = -16.713 + 1.532 M - 1.406 \text{ Log } R* - 0.012 R + 0.592 \text{ Log } W + 0.540 \text{ Log } T15 + 3.413 \text{ Log } (100 - F15) - 0.795 \text{ Log } (D5015 + 0.1 \text{ mm})
\]

\[
\begin{align*}
\text{Dh} & = \text{Horizontal Displacement, (meters)} \\
M & = \text{Moment Magnitude of Earthquake, } M_w \\
R & = \text{Horizontal Distance to Nearest Seismic Energy Source or Fault Rupture, (kilometers)} \\
R* & = R + R_0 \\
R_0 & = 10(0.89 M - 5.64) \\
W & = (H/L)*100 = \text{Free Face Ratio, (percent)} \\
H & = \text{Height of the Free Face, (meters)} \\
L & = \text{Length to the Free Face from the Point of Displacement (meters)} \\
S & = \text{Ground Slope, (percent)} \\
T15 & = \text{Thickness of Saturated Cohesionless Soils with } (N1)60 \leq 15, (\text{meters}) \\
F15 & = \text{Average Fines Content in } T15, (\text{particle size } < 0.075 \text{ millimeters, in percent}) \\
D5015 & = \text{Average } D50 \text{ in } T15, (\text{millimeters})
\end{align*}
\]

Procedures for Mitigation of Liquefaction and Ground Failure Hazards

- Avoid liquefiable sites
- Accept liquefaction hazard
- Strengthen the Structure
- Stabilize the ground
Avoid the Hazard

Valdez, Alaska, 1964 Alaska Earthquake

Valdez docks before earthquake After earthquake

Valdez docks were destroyed by flow slides and community was pulled apart by lateral spread during 1964 Great Alaska earthquake, Mw = 9.2

To avoid liquefaction hazard, Valdez was rebuilt on a non-liquefiable site

Diagrams showing depths and locations of liquefiable layers beneath highway I-15, Salt Lake City, Utah, interpreted from CPT data

Accept The Hazard

Log analyzed in next slide

Strengthens the structure

House pulled apart at foundation level by lateral spread during 1997 Varancia, Romania earthquake, causing partial collapse

Failure surface w/ FS = 1.46

Slope stability analysis for highway embankment underlain by liquefiable layers, Salt Lake City, Utah, USA; with FS = 1.46, liquefaction hazard was accepted without requiring remediation.
Ground fissures caused by lateral spread during 1995 Kobe, Japan earthquake; fissures continue beneath houses but caused no damage

Foundation under construction at time of 1995 Kobe, Japan earthquake; note foundation walls and grade beams are well reinforced creating a strong diaphragm. Strengthened foundations can withstand differential ground displacement without fracture

Cross section showing pile configuration for a building on Rokko Island shaken by 1995 Kobe, Japan Earthquake; liquefaction and ground settlement (up to 0.75 m) occurred without causing detectable structural damage to buildings

Pile foundations are an effective structural mitigation measure for sites with tolerable lateral ground displacement

As noted previously, liquefaction and lateral spread beneath Rio Estrella highway bridge did not generate significant displacement of abutments and piers founded on sufficiently strong 12BPS3 steel piles

➢ Stabilize the Ground

➢ Many procedures may be applied to compact, drain or cement liquefiable soils to increase strength and decrease deformation potential
➢ Bottom-fed stone columns, as shown above, is a commonly applied procedure in USA and other countries for stabilizing liquefiable soils

Compact soil with stone columns or other vibrocompaction techniques
Los Angeles County decided to rebuild the San Fernando Valley Juvenile Hall on existing site, but with soils stabilized by excavation and replacement or by soil grouting.

Example of excavation soil replacement and soil grouting.

Trench cut through Juvenile Hall site to investigate existence of an active fault; trench was backfilled with compacted soil to form a buttress against possible lateral spread displacement during future earthquakes.

Diagrammatic cross section showing ground stabilization that occurred at San Fernando Valley Juvenile Hall site prior to reconstructing facility.

Rebuilt San Fernando Valley Juvenile Hall was not damaged during 1994 Northridge earthquake. The 1994 earthquake shook the site as strongly as the 1971 San Fernando earthquake, but without generating destructive lateral ground displacements.

Rebuilt San Fernando Valley Juvenile Hall after the 1994 Northridge earthquake. Note open minor fissure along building wall indicating minor lateral spread was generated in unmodified ground during 1994 earthquake.
Summary:

1. Liquefaction may cause any of the following types of ground failure:
   - Flow failure
   - Lateral Spread
   - Ground Oscillation
   - Loss of bearing strength
   - Ground Settlement

2. Amount of potential liquefaction-induced ground deformations or displacements associated should be determined. If amount of ground displacement is tolerable to the structure, mitigation is not required.

3. The following mitigation measures may be applied:
   - Avoid the hazard
   - Accept the hazard
   - Strengthen the structure
   - Stabilize the ground