Soil-Structure Interaction Under Extreme Loading Conditions

The Thirteenth Spencer J. Buchanan Lecture

By

Professor T.D. O’Rourke

Friday November 18, 2005

College Station Hilton
810 University Drive
College Station, TX 77840  USA

http://ceprofs.tamu.edu/briaud/buchanan.htm
Spencer J. Buchanan, Sr. was born in 1904 in Yoakum, Texas. He graduated from Texas A&M University with a degree in Civil Engineering in 1926, and earned graduate and professional degrees from the Massachusetts Institute of Technology and Texas A&M University.

He held the rank of Brigadier General in the U.S. Army Reserve, (Ret.), and organized the 420th Engineer Brigade in Bryan-College Station, which was the only such unit in the Southwest when it was created. During World War II, he served the U.S. Army Corps of Engineers as an airfield engineer in both the U.S. and throughout the islands of the Pacific Combat Theater. Later, he served as a pavement consultant to the U.S. Air Force and during the Korean War he served in this capacity at numerous forward airfields in the combat zone. He held numerous military decorations including the Silver Star.

He was founder and Chief of the Soil Mechanics Division of the U.S. Army Waterways Experiment Station in 1932, and also served as Chief of the Soil Mechanics Branch of the Mississippi River Commission, both being Vicksburg, Mississippi.
Professor Buchanan also founded the Soil Mechanics Division of the Department of Civil Engineering at Texas A&M University in 1946. He held the title of Distinguished Professor of Soil Mechanics and Foundation Engineering in that department. He retired from that position in 1969 and was named professor Emeritus. In 1982, he received the College of Engineering Alumni Honor Award from Texas A&M University.

He was the founder and president of Spencer J. Buchanan & Associates, Inc., Consulting Engineers, and Soil Mechanics Incorporated in Bryan, Texas. These firms were involved in numerous major international projects, including twenty-five RAF-USAF airfields in England. They also conducted Air Force funded evaluation of all U.S. Air Training Command airfields in this country. His firm also did foundation investigations for downtown expressway systems in Milwaukee, Wisconsin, St. Paul, Minnesota; Lake Charles, Louisiana; Dayton, Ohio, and on Interstate Highways across Louisiana. Mr. Buchanan did consulting work for the Exxon Corporation, Dow Chemical Company, Conoco, Monsanto, and others.

Professor Buchanan was active in the Bryan Rotary Club, Sigma Alpha Epsilon Fraternity, Tau Beta Pi, Phi Kappa Phi, Chi Epsilon, served as faculty advisor to the Student Chapter of the American Society of Civil Engineers, and was a Fellow of the Society of American Military Engineers. In 1979 he received the award for Outstanding Service from the American Society of Civil Engineers.

Professor Buchanan was a participant in every International Conference on Soil Mechanics and Foundation Engineering since 1936. He served as a general chairman of the International Research and Engineering Conferences on Expansive Clay Soils at Texas A&M University, which were held in 1965 and 1969.

Spencer J. Buchanan, Sr., was considered a world leader in geotechnical engineering, a Distinguished Texas A&M Professor, and one of the founders of the Bryan Boy’s Club. He died on February 4, 1982, at the age of 78, in a Houston hospital after an illness, which lasted several months.
The Spencer J. Buchanan ’26 Chair in Civil Engineering

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The text of the lectures and a videotape of the presentations are available by contacting:

Dr. Jean-Louis Briaud
Spencer J. Buchanan ’26 Chair Professor
Department of Civil Engineering
Texas A&M University
College Station, TX 77843-3136, USA
Tel: 979-845-3795
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<td>2:00 p.m.</td>
<td>Welcome by Jean-Louis Briaud</td>
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<td>2:05 p.m.</td>
<td>Introduction by David Rosowsky</td>
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<td>2:10 p.m.</td>
<td>A word from the GeoInstitute President by Stephen Wright</td>
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<td>Terracon Scholarship Presentation by George Cozart</td>
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<td>Introduction of Harry Poulos by Tanner Blackburn</td>
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<td>“Pile Behaviour - Consequences of Geological and Construction Imperfections” 2004 Terzaghi Lecture by Harry Poulos</td>
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<td>3:25 p.m.</td>
<td>Discussion</td>
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<td>3:35 p.m.</td>
<td>Introduction of Tom O’Rourke by Jean-Louis Briaud</td>
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<td>3:40 p.m.</td>
<td>“Soil-Structure Interaction Under Extreme Loading Conditions” 2005 Buchanan Lecture by Tom O’Rourke</td>
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<td>4:40 p.m.</td>
<td>Discussion with Darrow Hooper</td>
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<td>4:50 p.m.</td>
<td>Closure with Philip Buchanan</td>
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<td>5:00 p.m.</td>
<td>Photos followed by a reception at the home of Jean-Louis and Janet Briaud.</td>
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Professor T.D. O’Rourke

Professor O’Rourke is a member of the faculty of the School of Civil and Environmental Engineering at Cornell University. He is a member of the US National Academy of Engineering and an elected Fellow of American Association for the Advancement of Science. He has received several awards from professional societies, including the Collingwood, Huber Research, C. Martin Duke Lifeline Earthquake Engineering, Stephen D. Bechtel Pipeline Engineering, and Ralph B. Peck Awards from American Society of Civil Engineers (ASCE), the Hogentogler Award from American Society for Testing and Materials, Trevithick Prize from the British Institution of Civil Engineers, the Japan Gas Award and Earthquake Engineering Research Institute (EERI) Awards for outstanding papers, and Distinguished Service Award from the University of Illinois College of Engineering. He served as President of the EERI and as a member of the US National Science Foundation Engineering Advisory Committee. He is a member of the Executive Committees of the Multidisciplinary Center for Earthquake Engineering Research and the Consortium of Universities for Research in Earthquake Engineering Board of Directors. He has served as Chair of the Executive Committee of the ASCE Technical Council on Lifeline Earthquake Engineering and ASCE Earth Retaining Structures Committees. He has authored or co-authored over 290 technical publications. He has served on numerous earthquake reconnaissance missions, and has testified before the US Congress in 1999 on engineering implications of the 1999 Turkey and Taiwan earthquakes and in 2003 on the reauthorization of the National Earthquake Hazards Reduction Program. He has served as chair or member of the consulting boards of many large underground construction projects, as well as the peer reviews for projects associated with highway, rapid transit, water supply, and energy distribution systems. He has investigated and contributed to the mitigation of the effects of extreme events, including natural hazards and human threats, on critical civil infrastructure systems. His research interests cover geotechnical engineering, earthquake engineering, engineering for large, geographically distributed systems (e.g., water supplies, gas and liquid fuel systems, electric power, and transportation facilities), underground construction technologies, and geographic information technologies and database management.
SOIL-STRUCTURE INTERACTION UNDER EXTREME LOADING CONDITIONS

T. D. O’Rourke¹, J. E. Turner², S-S. Jeon³, H. E. Stewart⁴, Y. Wang⁵, and P. Shi⁵

ABSTRACT

Soil-structure interaction under extreme loading conditions includes performance during earthquakes, floods, landslides, large deformation induced by tunneling and deep excavations, and subsidence caused by severe dewatering or withdrawal of minerals and fluids during mining and oil production. Such loading conditions are becoming increasingly more important as technologies are developed to cope with natural hazards, human threats, and construction in congested urban environments. This paper examines extreme loading conditions with reference to earthquakes, which are used as an example of how extreme loading influences behavior at local and geographically distributed facilities. The paper covers performance from the component to the system-wide level to provide guidance in developing an integrated approach to the application of geotechnology over large, geographically distributed networks. The paper describes the effects of earthquake-induced ground deformation on underground facilities, and extends this treatment to the system-wide performance of the Los Angeles water supply during the 1994 Northridge earthquake. Large-scale experiments to evaluate soil-structure interaction under extreme loading conditions are described with reference to tests of abrupt ground rupture effects on urban gas pipelines. Large-scale tests and the development of design curves are described for the forces imposed on pipelines during ground failure.

INTRODUCTION

From a geotechnical perspective, extreme loading conditions are those that induce large plastic, irrecoverable deformation in soil. They are often associated with significant geometric changes in the soil mass, such as shear rupture, heave and void formation, and are accompanied by a peak, or maximum, interaction force imposed on embedded structures. Such loading takes soil well beyond the range of deformation related to the conventional design of civil structures. It applies to performance under unusual, extreme conditions. Such conditions include earthquakes, floods, landslides, large deformation induced by tunneling and deep excavations, and subsidence caused by severe dewatering or withdrawal of minerals and fluids during mining and oil production. Such loading conditions are becoming increasingly more important as technologies are developed to cope with natural hazards, human threats, and construction in congested urban environments.

Extreme loading conditions for soils are often accompanied by extreme loading conditions for structures. Examples include soil/structure interaction associated with pipelines subjected to fault rupture, piles affected by landslides, and soil failure imposed on underground facilities by explosions, flooding, and the collapse of voids. Such conditions induce large plastic, irrecoverable structural deformation that involves both material and geometric nonlinear behavior. Hence, analytical and experimental modeling

¹ Professor, Cornell University, Ithaca, NY 14853
² Engineer, Stephens Associates Consulting Engineers, LLC, Brentwood, NH 03833
³ Chief Researcher, Korea Highway Corporation, South Korea 445-812
⁴ Associate Professor, Cornell University, Ithaca, NY 14853
⁵ Graduate Research Assistant, Cornell University, Ithaca, NY 14853
for soil-structure interaction under these conditions requires the coupled post-yield simulation of both soil and structural response. Such behavior generally poses significant challenges to our analytical capabilities, thus requiring large-scale experimental and case history data to improve the simulation process and validate the models.

Extreme loading conditions, especially those associated with natural hazards and severe human threats, may affect large systems of structures. Consider, for example, Figure 1, which is a photograph of the corner of Wall and Williams Sts. in New York City in 1917. The congestion shown in this photograph has not improved in the last 88 years, and is indicative of the situation in a similitude of cities worldwide. The photo illustrates at least two important features of the built environment. First, much of critical infrastructure is located underground, and its fate is intimately related to that of the surrounding ground. Second, the crowded nature of urban and suburban developments increases risk due to proximity. Damage to one facility, such as a cast iron water main, can rapidly cascade into damage in surrounding facilities, such as electric and telecommunication cables and gas mains, with system-wide consequences. Soil surrounding critical underground infrastructure is frequently both the perpetrator and mediator of loading that can affect the systemic performance of an entire city.

In this paper, soil-structure interaction under extreme loading conditions is examined with reference to earthquakes, which are used as an example of how extreme loading influences behavior at local and geographically distributed facilities. The paper begins with the effects of earthquake-induced ground deformation on underground facilities, and then expands this treatment to consider the system-wide performance of the Los Angeles water supply during the 1994 Northridge earthquake. Large-scale experiments to evaluate soil-structure interaction under extreme loading conditions are described with reference to tests of abrupt ground rupture effects on urban gas pipelines. Large-scale tests and the development of design curves are described for the forces imposed on pipelines during ground failure. The paper covers performance from the component to the system-wide level to provide guidance in developing an integrated approach to the application of geotechnology over large, geographically distributed networks.
Earthquakes cause transient ground deformation (TGD) and permanent ground deformation (PGD), both of which affect underground pipelines. TGD is the dynamic response of the ground, and PGD is the irrecoverable movement that persists after shaking has stopped. PGD often involves large displacements, such as those associated with surface fault rupture and landslides. TGD can cause soil cracks and fissures triggered by pulses of strong motion that develop localized shear and tensile strains exceeding the strength of surficial soils. In these cases, crack widths and offsets are primarily a reflection of surficial ground distortion and gravity effects, such as local slumping. They should not be mistaken as an expression of PGD generated by ground failure mechanisms of larger scale.

The principal causes of PGD have been summarized and discussed by O’Rourke (1998). They are faulting, tectonic uplift and subsidence, and liquefaction, landslides, and densification of loose granular deposits. Liquefaction is the transformation of saturated cohesionless soil into a liquefied state or condition of substantially reduced shear strength (Youd, 1973). Liquefaction-induced pipeline deformation can be caused by lateral spread, flow failure, local subsidence, post-liquefaction consolidation, buoyancy effects, and loss of bearing (Youd, 1973; O’Rourke, 1998). It is widely accepted that the most serious pipeline damage during earthquakes is caused by PGD. Furthermore, it is well recognized that liquefaction-induced PGD, especially lateral spread, is one of the most pervasive causes of earthquake-induced lifeline damage (Hamada and O’Rourke, 1992; O’Rourke and Hamada, 1992).

Ground displacement patterns associated with earthquakes depend on PGD source, soil type, depth of ground water, slope, earthquake intensity at a given site, and duration of strong ground shaking (O’Rourke, 1998). It is not possible to model with accuracy the soil displacement patterns at all potentially vulnerable locations. Nevertheless, it is possible to set upper bound estimates of deformation effects on buried lifelines by simplifying spatially distributed PGD as movement concentrated along planes of soil failure.

Various modes of pipeline distortion caused by PGD are illustrated in Fig. 2. Pipelines crossing a fault plane subjected to oblique slip are shown in Fig. 2a. Reverse and normal faults promote compression and tension, respectively. Strike slip may induce compression or tension, depending on the angle of intersection between the pipeline and fault. Fig. 2b shows a pipeline crossing a lateral spread or landslide perpendicular to the general direction of soil movement. In this orientation, the pipeline is subject to bending strains and extension. As shown in Fig. 2c, the pipeline will undergo bending and either tension or compression at the margins of the slide when the crossing occurs at an oblique angle. Fig. 2d shows a pipeline oriented parallel to the general direction of soil displacement. At the head of the zone of soil movement, the displacements resemble normal faulting; under these conditions, the pipeline will be subjected to both bending and tensile strains. At the toe of the slide, the displaced soil produces compressive strains in the pipeline.

Fig. 3 shows a compressive failure at a welded slip joint on the Granada Trunk Line, a 1,245-mm-diameter steel pipeline with 6.4-mm wall thickness that failed during the Northridge earthquake because of lateral ground movement triggered by liquefaction near the intersection of Balboa Boulevard and Rinaldi Street in the San Fernando Valley. The PGD pattern and pipeline failure mode associated with this site are similar to those depicted in Fig. 2d. Similar compressive failures were observed in trunk lines during the 1971 San Fernando earthquake and in the adjacent 1,727-mm-diameter (9.5-mm wall thickness) Rinaldi Trunk Line during the Northridge earthquake. Loss of both the Granada and Rinaldi Trunk Lines cut off water to tens of thousands of customers in the San Fernando Valley for several days.
Fig. 2. Principal Modes of Soil-Pipeline Interaction Triggered by Earthquake-Induced PGD (O’Rourke, 1998)

Fig. 3. Welded Slip Joint Failure of the Granada Trunk Line During the 1994 Northridge Earthquake (photo by Y. Shiba)
Various simplified models for soil-pipeline interaction have been developed to account for the effects of abrupt ground displacement illustrated in Fig. 2. (e.g., Newmark and Hall, 1975; Kennedy, et al., 1977; O’Rourke, et al., 1985; and O’Rourke and Liu, 1999). Moreover, various finite element codes (e.g., ABAQUS, ANSYS, and PIPLIN) are applied frequently to model PGD effects on the post-yield performance of line pipe. A hybrid model, representing line pipe as a combination of beam and shell elements, has been developed recently to analyze PGD effects on pipeline elbows (Yoshisaki, et al., 2001).

LIFELINE SYSTEM RESPONSE TO EARTHQUAKES

The 1994 Northridge earthquake caused the most extensive damage to a US water supply system since the 1906 San Francisco earthquake. Three major transmission systems, which provide over three-quarters of the water for the City of Los Angeles, were disrupted. Los Angeles Department of Water and Power (LADWP) and Metropolitan Water District (MWD) trunk lines (nominal pipe diameter ≥ 600 mm) were damaged at 74 locations, and the LADWP distribution pipeline (nominal pipe diameter < 600 mm) system was repaired at 1013 locations.

The earthquake-induced damage to water pipelines and the database developed to characterize this damage have been described elsewhere (O’Rourke, et al., 1998; O’Rourke, et al., 2001; Jeon and O’Rourke, 2005), and only the salient features of this work are summarized herein. GIS databases for repair locations, characteristics of damaged pipe, and lengths of trunk lines according to pipe composition and size were assembled with ARC/INFO software. Nearly 10,000 km of distribution lines and over 1,000 km of trunk lines were digitized.

Figure 4 shows the portion of the Los Angeles water supply system most seriously affected by the Northridge earthquake superimposed on the topography of Los Angeles. The figure was developed from the GIS database, and shows all water supply pipelines plotted with a geospatial precision of ± 10 m throughout the San Fernando Valley, Santa Monica Mountains, and Los Angeles Basin. The rectilinear system of pipelines is equivalent to a giant strain gage. Seismic intensity in the form of pipeline damage can be measured and visualized by plotting pipeline repair rates and identifying the areas where the largest concentrations of damage rate occur. The resulting areas reflect the highest seismic intensities as expressed by the disruption to underground piping.

To develop a properly calibrated strain gage, it is necessary to select a measurement grid with material having reasonably consistent properties and a damage threshold sensitive to the externally imposed loads being measured. Figure 5 presents charts showing the relative lengths of LADWP and MWD trunk and distribution lines, according to pipe composition. As shown by the pie chart, the most pervasive material in the LA distribution system is CI. The 7,800 km of CI pipelines have the broadest geographic coverage with sufficient density in all areas to qualify as an appropriate measurement grid. Moreover, CI is a brittle material subject to increased rates of damage at tensile strains on the order 250 to 500 με. It is therefore sufficiently sensitive for monitoring variations in seismic disturbance.

Figure 6 presents a map of distribution pipeline repair locations and repair rate contours for cast iron (CI) pipeline damage. The repair rate contours were developed by dividing the map into 2 km x 2 km areas, determining the number of CI pipeline repairs in each area, and dividing the repairs by the distance of CI mains in that area. Contours then were drawn from the spatial distribution of repair rates, each of which was centered on its tributary area. A variety of grids were evaluated, and the 2 km x 2 km grid was found to provide a good representation of damage patterns for the map scale of the figure (Toprak, et al., 1999).
Fig. 4. Map of Los Angeles Water Supply System Affected by Northridge Earthquake (O’Rourke and Toprak, 1997)

Fig. 5. Composition Statistics of Water Trunk and Distribution Lines in the City of Los Angeles (O’Rourke and Toprak, 1997)
The zones of highest seismic intensity are shown by areas of concentrated contours. In each instance, areas of concentrated contours correspond to zones where the geotechnical conditions are prone either to ground failure or amplification of strong motion. Each zone of concentrated damage is labeled in Fig. 7 according to its principal geotechnical characteristics. In effect, therefore, Fig. 6 is a seismic hazard map for the Los Angeles region, calibrated according to pipeline damage during the Northridge earthquake.

Of special interest is the location of concentrated repair rate contours in the west central part of San Fernando Valley (designated in Fig. 7 as the area of soft clay deposits). This area was investigated by USGS researchers, who found it to be underlain by local deposits of soft, normally consolidated clay. Field vane shear tests disclosed clay with uncorrected, vane shear undrained strength, $S_{uv} = 20-25$ kPa, at a depth of 5 m, just below the water table. USGS investigators concluded that the saturated sands underlying this site were not subjected to liquefaction during the Northridge earthquake. Newmark sliding block analyses reported by O’Rourke (1998) provide strong evidence that near source pulses of high acceleration were responsible for sliding and lurching on the soft, normally consolidated clay deposit. The results of GIS analysis and site investigations have important ramifications because they show a clear relationship between PGD, concentrated pipeline damage, and the presence of previously unknown deposits of normally consolidated clay.

The records from approximately 240 free field rock and soil stations were used to evaluate the patterns of pipeline damage with the spatial distribution of various seismic parameters. Fig. 8 shows the CI pipeline repair rate contours superimposed on peak ground velocity (PGV) zones, which were developed by interpolating the maximum horizontal velocities recorded at the strong motion stations. Using the GIS database, a pipeline repair rate was calculated for each PGV zone, and correlations were made between the repair rate and average PGV for each zone. As explained by O’Rourke (1998), similar correlations were investigated for pipeline damage relative to spatially distributed peak ground
Fig. 7. Geotechnical Characteristics of the Areas of Concentrated Pipeline Damage After the Northridge Earthquake

Fig. 8. Pipeline Repair Rate Contours Relative to Northridge Earthquake Peak Ground Velocity (O’Rourke and Toprak, 1997)
acceleration, spectral acceleration and velocity, Arias Intensity, Modified Mercalli Intensity (MMI), and other indices of seismic response. By correlating damage with various seismic parameters, regressions were developed between repair rate and measures of seismic intensity.

The most statistically significant correlations for both distribution and trunk line repair rates were found for PGV. Such correlations are important for loss estimation analyses that are employed to assess the potential damage during future earthquake and develop corrective measures and emergency response procedures to reduce the projected losses (e.g., Whitman, et al., 1997).

Fig. 9a presents the linear regression that was developed between CI pipeline repair rates and PGV on the basis of data from the Northridge and other U.S. earthquakes. Fig. 9b shows repair rate correlations for steel, CI, ductile iron (DI), and asbestos cement (AC) distribution lines. The regressions indicate that the highest rate of damage for a given PGV was experienced by steel pipelines. This result at first seems surprising because steel pipelines are substantially more ductile than CI and AC pipelines. Steel distribution pipelines in Los Angeles, however, are used to carry the highest water pressures and are subject to corrosion that has been shown to intensify their damage rates in previous earthquakes (Isenberg, 1979).

The regressions in Fig. 9 were developed after the data were screened for lengths of pipeline that represent approximately 1.5 to 2.5 % of the total length or population for each type of pipe affected by the earthquake (O’Rourke and Jeon, 1999). This procedure reduces the influence of local erratic effects that bias the data derived from small lengths of pipeline. The use of this filtering procedure leads to statistically significant trends, but with the resulting in regressions only applicable for PGV ≤ 75 cm/s. For the Northridge earthquake, zones with PGV exceeding 75 cm/sec generally correspond to locations where PGD, from sources such as liquefaction and landsliding, was observed. Hence, this screening technique tends to remove damage associated with PGD, resulting in correlations relevant for TGD.
Of special interest is the work of Hamada and coworkers (Hamada, et al., 1986; Hamada and O’Rourke, 1992) in the use of stereo-pair air photos before and after an earthquake to perform photogrammetric analysis of large ground deformation. This process has had significant impact on the way engineers and geologists evaluate soil displacements by providing a global view of deformation that allows patterns of distortion to be quantified and related to geologic and topographic characteristics.

After the Northridge earthquake, pre- and post-earthquake air photo measurements in the Van Norman Complex were analyzed as part of collaborative research between U.S. and Japanese engineers (Sano, et al., 1999; O’Rourke, et al., 1998). Air photos taken before and after the earthquake were acquired by U.S. team members and analyzed through advanced photogrammetric techniques by Japanese team members. Ground movements from this initial set of measurements were corrected for tectonic deformation to yield movements caused principally by liquefaction and landslides.

The area near the intersection of Balboa Blvd. and Rinaldi St. has been identified as a location of liquefaction (Holzer, et al., 1999) where significant damage to gas transmission and water trunk lines was incurred. This location is the same area where the pipeline failure in Fig. 7 was observed. Ground strains were calculated in this area from the air photo measurements of horizontal displacement by superimposing regularly spaced grids with GIS software onto the maps of horizontal displacement and calculating the mean displacement for each grid. Grid dimensions of 100 m x 100 m were found to provide the best results (Sano, et al., 1999).

As illustrated in Fig. 10, ground strain contours, pipeline system, and repair locations were combined using GIS, after which repair rates corresponding to the areas delineated by a particular contour interval were calculated. Fig. 11 shows the repair rate contours for CI mains superimposed on the areal distribution of ground strains, identified by various shades and tones. In the study area, there were 34 repairs to CI water distribution mains and 2 for steel water distribution pipelines. There were 5 water trunk line repairs in the area. The repair rate contours were developed by dividing the map into 100 m x 100 m cells, determining the number of CI pipeline repairs in each cell, and dividing the repairs by the length of the distribution mains in that cell. The intervals of strain and repair rate contours are 0.001 (0.1%) and 5 repairs/km, respectively. The zones of high tensile (+) and compressive (-) strains coincide well with the locations of high repair rate.

In Fig. 12, the relationship between the absolute values of the ground strains and repair rates is presented graphically using linear regression. The repair rate in each ground strain range, 0-0.1, 0.1-0.3, and 0.3-0.5%, was calculated as explained previously. Ground strain contours, obtained by both air photo measurements and surface surveys, were used. The regression analysis shows that repair rates increase linearly with ground strain. In some instances, anomalously high repair rates were determined. Such values do not represent the actual distribution of damage, but are a consequence of locally high concentrations of repairs within a given cell. The occurrence of anomalously high values depends on the cell size and positioning of cells with respect to pipeline repair locations. It is important, therefore, to incorporate screening procedures to filter such erroneous types of data. For example, an investigation of the locally high data point in Fig. 12 showed that this repair rate was calculated for one particular positioning of the grid of GIS cells and not for others. As such, this locally high repair rate was not used in the regression analyses. A systematic investigation of cell size and positioning effects on GIS analytical results has been performed, and procedures for selecting an optimal cell sizes have been recommended (Toprak, et al., 1999).

Hamada and Wakamatsu (1996) showed a similar strong correlation between frequency of pipeline repairs and ground strains evaluated by air photo measurements after the 1995 Kobe earthquake. Seismic
Fig. 10. Procedure for Calculating Repair Rate in Each Strain Range (O’Rourke et al., 1998)

Fig. 11. Distributions of CI Repair Rate and Ground Strain (O’Rourke, et al., 1998)
Fig. 12. Correlation Between Ground Strain and CI Repair Rate (O’Rourke, et al., 1998)

Fig. 13. Experimental Concept for PGD Effects on Buried Pipelines with Elbows

design codes for gas pipelines in Japan have been developed on the basis of acceptable strain levels (Japan Gas Association, 2000). These values, in turn, can be related to the anticipated levels of ground strain triggered by an earthquake.

LARGE-SCALE TESTS OF GROUND RUPTURE EFFECTS

A key component of modern research involving geotechnical engineering for extreme loading conditions has been testing at very large scale. Large scale experiments sponsored by NSF through MCEER at Cornell in conjunction with Tokyo Gas, Ltd. were performed to evaluate the effects of earthquake-induced ground rupture on welded steel pipelines with elbows. The experimental set-up involved the largest full-scale replication of PGD effects on pipelines ever performed in the laboratory.

Many pipelines must be constructed to change direction rapidly to avoid other underground facilities or to adjust to the shape of roads under which they are buried. In such cases the pipeline is installed with an elbow that can be fabricated for a change in direction from 90 to a few degrees. The response of pipeline elbows, deformed by adjacent ground rupture and subject to the constraining effects of surrounding soil, is a complex interaction problem. A comprehensive and reliable solution to this problem requires laboratory experiments on elbows to characterize their three-dimensional response to axial and flexural loading, an analytical model that embodies soil-structure interaction combined with three-dimensional elbow response, and full-scale experimental calibration and validation of the analytical model.

Fig. 13 illustrates the concept of the large-scale experiments. A steel pipeline with an elbow is installed under the actual soil, fabrication, and compaction procedures encountered in practice, and then subjected to lateral soil displacement. The scale of the experimental facility is chosen so that large soil movements are generated, inducing soil-pipeline interaction unaffected by the boundaries of the test facility in which the pipeline is buried. The ground deformation simulated by the experiment represents deformation conditions associated with lateral spread, landslides, and fault crossings, and therefore applies to many different geotechnical scenarios. In addition, the experimental data and analytical modeling products are of direct relevance for underground gas, water, petroleum, and electrical conduits.

Experiment Description and Results

A 100-mm-diameter pipeline with 4.1-mm wall thickness was used in the tests. It was composed of two straight pipes welded to a 90-degree elbow (E). The short section of straight pipe (D) was 5.4 m long, whereas the longest section was 9.3 m. Both ends of the pipeline were bolted to reaction walls. The elbows were composed of STPT 370 steel (Japanese Industrial Standard, JIS-G3456) with a specified minimum yield stress of 215 MPa and a minimum ultimate tensile strength of 370 MPa. The straight pipe
was composed of SGP steel (JIS-G3452) with a minimum ultimate tensile strength of 294 MPa. About 150 strain gauges were installed on the pipe to measure strain during the tests. Extensometers, load cells, and soil pressure meters were also deployed throughout the test setup. The pipeline was installed at a 0.9-m depth to top of pipe in each of three experiments. In each experiment soil was placed at a different water content and in situ density. The experiments were designed to induce opening-mode deformation of the elbow. They were conducted with nitrogen pressure of 0.1 MPa in the pipeline. Details of the tests and experimental results are provided by Yoshisaki, et al. (2001).

Approximately 60 metric tons of sand were moved from the storage bin into the test compartment for each experiment. The sand was obtained from a glacisch-fluvial deposit, and contained approximately 2% by weight of fines. The water content of 0.5% for Test 1 is the hygroscopic water content, the lowest value possible without oven drying. Hence, the soil in Test 1 is dry sand, and is comparable to the dry sand used in previous soil-pipe interaction tests (Trautmann and O’Rourke, 1985). In contrast, Tests 2 and 3 were performed with sufficiently large water contents to investigate the effects of partial saturation. The grain size curve for the sand is shown in Fig. 14. The sand was placed and compacted in 150-mm lifts with strict controls on water content and in situ density, which are summarized in Table 1. The sand satisfied the standards for backfill specified by the Bureau of Construction of the Tokyo Metropolitan Government.

Fig. 15 shows the ground surface of the test compartment before and after an experiment. Surficial heaving and settlement can be seen in the area near the pipeline elbow and the abrupt displacement plane between the movable and fixed boxes after the test. In all cases, planes of soil slip and cracking reached the ground surface, but did not intersect the walls of the test compartment to any appreciable degree. One hundred and ten mm of surface settlement and 95 mm of surface heave were measured after the test. Fig. 16 shows an overhead view of the test compartment after soil excavation to the pipeline following Test 1. Leakage occurred at the connection between the elbow and the shorter straight pipe when the ground displacement was 0.78 m. Full circumferential rupture of the pipe occurred when the displacement was 0.94 m.

Analytical Model and Results

The pipeline was modeled with isotropic shell elements with reduced integration points. Average values of the actual thickness measured with an ultrasonic thickness meter were used for the elbow and straight pipes in the model. True stress-strain relationships from direct tension test data were approximated by multi-linear trends for the elbow and straight pipe. ABAQUS Version 5.8 was used as a solver for the analyses with geometric nonlinearity and large strain formulation. The von Mises criterion and associated flow rule were applied to the model. Since strains are in the same direction in strain space throughout the analyses, isotropic hardening was used in the model. An internal pressure of 0.1 MPa was also applied in the model. Soil-pipe interaction was modeled in accordance with Japan Gas Association guidelines (2000) and data presented by Trautmann and O’Rourke (1985).

Fig. 17 (a) compares the deformed pipeline shape of the analytical model with measured deformation of the experimental pipeline for Test 1. There is excellent agreement between the two, as well as close agreement between the analytical deformation and the overhead view of the deformed pipeline in Fig. 16. Fig. 17 (b) shows the measured and predicted strains under maximum ground deformation on both the compressive (extrados) and tensile (intrados) surfaces of flexure along the pipeline. Figs. 17 (c) and (d) show the measured and analytical strains around the pipe circumference in which the angular distance is measured from extrados to intrados of pipe, corresponding to 0 and 180°, respectively. In Fig. 17 (c), the data point with an upward arrow indicates the maximum strain measured when the gauge was disconnected during the experiment. Because the disconnection occurred before maximum deformation of the elbow, it is likely that the actual strain was larger than the value plotted. Overall, there is good agreement for both the magnitude and distribution of measured and analytical strains and deformation.
Fig. 14. Grain Size Distribution of Experimental Sand

Table 1 Properties of Experimental Sand

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water content, w (%)</td>
<td>0.5</td>
<td>3.1</td>
<td>3.4</td>
</tr>
<tr>
<td>Wet unit weight, $\gamma_{\text{wet}}$ (kN/m$^3$)</td>
<td>18.4</td>
<td>17.0</td>
<td>16.7</td>
</tr>
<tr>
<td>Dry unit weight, $\gamma_{\text{dry}}$ (kN/m$^3$)</td>
<td>18.3</td>
<td>16.6</td>
<td>16.2</td>
</tr>
<tr>
<td>Friction angle from slow triaxial compression tests (0.1%/min), $\phi_{\text{TXC-0.1}}$ (degree)</td>
<td>49</td>
<td>40</td>
<td>39</td>
</tr>
<tr>
<td>Friction angle from fast triaxial compression tests (5%/min), $\phi_{\text{TXC-5}}$ (degree)</td>
<td>51</td>
<td>43</td>
<td>42</td>
</tr>
</tbody>
</table>

Fig. 15. Overhead View of Test Compartment Before and After the Experiment (Test 1)
Fig. 16. Overhead View of Deformed Pipeline (Test 1)

(a) Pipeline deformation after the test
(b) Distribution of axial strain in the longitudinal direction
(c) Strain distribution at Section A-A
(d) Strain distribution at Section B-B

Fig. 17. Comparison Between Analytical and Experimental Results
The soil deformation patterns adjacent to the pipeline were different for the dry and partially saturated sands. During PGD, the dry sand in Test 1 tended to flow around the experimental pipeline, filling the spaces behind it as relative horizontal movement of the pipe increased. In contrast, the partially saturated sand in Tests 2 and 3 possessed apparent cohesion because of surface tension generated by interstitial moisture among the sand particles. As a result, relative movement of the pipe generated rupture surfaces rather than flow in the adjacent soil.

The large-scale experiments had three principal results. First, they were used to improve and validate a hybrid finite element model, which combines beam and shell elements for the pipeline with nonlinear p-y formulations to simulate soil-structure interaction. This model is now used by Tokyo Gas to plan and design pipelines for extreme loading conditions. Second, the analytical model was used to show that increasing the wall thickness of pipe, which is welded to the elbow, by 1.5 mm results in strain reduction of approximately 200% for abrupt ground rupture of 2 m. Simple, relatively inexpensive adjustments in pipeline fabrication, therefore can lead to substantial improvements in performance. Third, the strains induced in the experimental pipeline were markedly higher for tests in partially saturated sand than for those in dry sand, even though most other variables were held constant.

**SOIL-STRUCTURE INTERACTION DURING GROUND FAILURE**

To explore the effects of partially saturated sand on the lateral force conveyed to buried conduits due to relative soil-pipe displacement, a series of additional tests were performed on pipe of similar size and composition. The tests were designed to be similar to those performed by Trautmann and coworkers (Trautmann and O'Rourke, 1985; Trautmann, et al., 1985), who established design charts from which p-y and q-z relationships can be developed for analyzing soil-structure interaction in response to lateral and vertical PGD.

These design charts were developed on the basis of experiments in dry sand. However, the great majority of pipelines in the field are embedded in partially saturated soils. Shear deformation of partially saturated sand mobilizes surface tension, or negative pore water pressure, which increases shear resistance relative to that in dry sand under comparable conditions of soil composition, in situ density, and loading. Moreover, the geometry of the failed soil mass for partially saturated sand is significantly different than the flow and displacement pattern of dry sand around buried pipelines.

The experimental facility was constructed to model the effects of relative horizontal displacement between soil and pipe under conditions that duplicate the actual scale, burial depth, and soil characteristics encountered in the field. Horizontal displacement was applied externally to a pipe section in a manner that allowed unrestricted vertical pipe movement as well as adjustments in pipe weight to replicate different contents such as gas, liquid fuel, and water. The application of force and displacement on the pipe was designed to duplicate soil-pipe interaction for conditions in which PGD is imposed on underground pipelines during surface faulting, landslides, and lateral spreads.

The experimental facility was designed to induce maximum lateral displacement of 152 mm, with burial depths to 20 diameters. The experimental facility was composed of a test compartment, pipe loading system, instrumentation and data acquisition system, and soil handling equipment. Figures 18 and 19 show plan and profile photographs, respectively, of the test compartment.

The test apparatus consisted of a box with interior dimensions 2.4 m × 1.2 m by 1.5 m deep. A special collar was fabricated to fit on top of the testing apparatus (not shown in the figure) that extended the depth of pipe burial to 2.3 m. The apparatus was filled with a false wall that was removed when deep embedment depths (pipe depth exceeding 10 times pipe diameter) were used. Lateral force and displacement were conveyed to the pipe through a special yoke that allowed for unrestricted vertical
movement as the pipe was displaced forward. Loads were applied by means of a hydraulic cylinder, and were measured with a calibrated load cell. A counterweight system was used to adjust the experimental pipe weight to be consistent with pipe weight in the field. Lateral and vertical pipe movements were measured with extensometers, and soil movements were measured by means of wooden dowels, embedded in the soil mass, which were visible through the glass sidewalls.

Sand similar to that used in the large-scale experiments with the pipeline-elbow assembly was placed in 150-mm lifts and compacted. The grain size distribution of the experimental sand was nearly identical to that in Fig. 14. Frequent in situ density and moisture content tests were performed. Dry unit weight and moisture content in the sand mass were controlled to within ±2% and ±0.5%, respectively. The sand was placed dry and at moisture contents of approximately 4 and 8%.

Direct shear tests were performed on samples of the experimental sand at nominal moisture contents of 0, 4 and 8%. The direct shear behavior of tests with 4 and 8% moisture contents were essentially identical, and therefore, these tests were combined into one dataset. The direct shear test results show a nearly linear relationship between friction angle and dry unit weight for moist sand $\gamma_d \leq 16.4$ kN/m$^3$, and for dry sand $\gamma_d \leq 17.5$ kN/m$^3$. For dry unit weights greater these values, the friction angle increases rapidly with increasing dry unit weight. To capture this nonlinearity, the data were fit with bi-linear trends as shown in Fig. 20. Turner (2004) has shown that the high compaction energy required for preparing dense (moist sand $\gamma_d \geq 16.4$ kN/m$^3$, dry sand $\gamma_d \geq 17.5$ kN/m$^3$) samples results in the wedging of smaller angular particles between the larger ones, locking the soil structure and increasing the friction angle.

Compared to the direct shear data for dry sand, the moist sand friction angles are about 3 to 5° higher at a given dry unit weight. For a given friction angle, the dry unit weight of moist sand is about 0.5 to 1 kN/m$^3$ lower than that of dry sand.

The dry unit weight of each large-scale test specimen was measured using the Selig density scoop (Selig and Ladd, 1973) with typically 90 or more measurements per test. For dry sand and sand with 4% moisture content and $\gamma_d \leq 16$ kN/m$^3$, the dry scooped sample weight was calibrated to sand dry unit weight using the procedure described by Trautmann, et al. (1985). In this procedure, the operator applies downward force on the scoop while opening the handles to maintain contact between the scoop base plate and the soil surface, thereby consistently removing samples with a constant volume. For sand with 4% moisture at higher densities and sand with 8% moisture, the scoop tended to rise while closing the jaws due to dilatancy effects. Lifting of the scoop combined with greater variability in downward force applied by the operator resulted in smaller sample volumes and greater variability of the data.
An improved procedure was developed to minimize scoop movement and reduce operator variability in which lead blocks were stacked on the scoop base plate to provide a consistent reaction force. The weight of the lead blocks was varied proportionally to soil dry unit weight, as described by Turner (2004), to prevent the scoop from rising or punching into the soil while sampling. The calibration curves obtained for sand with 4 and 8% moisture contents using the new method, shown in Fig. 21, were nearly identical to the trend line obtained for dry sand, indicating that the new method limits scoop lifting and increases repeatability.

On the basis of previous field-scale experiments and an analytical model proposed by Ovesen (1964), Trautmann and O’Rourke (1983) developed a chart in which maximum dimensionless lateral force is plotted relative to dimensionless depth for various friction angles as determined with direct shear tests. These dimensionless charts were also published in a subsequent journal paper (Trautmann and O’Rourke, 1985) and the ASCE Guidelines for the Seismic Design of Oil and Gas Pipeline Systems (1984).
Figure 22 shows select plots of dimensionless force vs. dimensionless displacement for tests on partially saturated sand with dry unit weights between 16.3 and 16.6 kN/m$^3$ at ratios of depth to pipe centerline to external pipe diameter (H/D) of 6 and 8.5, respectively. The dimensionless force is the maximum measured lateral force, F, divided by the product of soil unit weight, $\gamma$, H, D, and length of pipe, L. This term provides a value that can be scaled to various depths, diameters, and soil conditions of practical interest. Table 2 summarizes information for each moist sand test shown in Fig. 22, including dry unit weight, water content, friction angle, and selected values of maximum dimensionless force, $N_q$. The characteristic displacement, $Y'_f$, corresponding to maximum force is shown for each curve with an arrowhead. The term $Y'$ is the ratio of the horizontal displacement, Y, to D.

For comparison with the moist sand test results, the figures also show force-displacement curves for dry sand obtained from current tests and tests by Trautmann and O’Rourke (1983, 1985). The dry unit weight of the tests by Trautmann and O’Rourke (1983, 1985) was 16.4 kN/m$^3$. The dry unit weights obtained during the dry sand tests by Turner (2004) were 16.7 and 16.9 kN/m$^3$.

The force-displacement curves for moist sand tests reached a peak at relatively small displacement, typically at $Y'$ between 0.1 and 0.2, and then decreased to a lower constant value at larger displacements, typically at $Y'$ of 0.2 to 0.3. The maximum dimensionless force, $N_q$, for all moist sand tests and the corresponding dimensionless displacement, $Y'_f$, were selected at the initial peak in the curve. As shown in Fig. 22, force-displacement curves for dry sand with similar dry unit weight as the moist sand tests did not exhibit peak behavior. Maximum force was selected for these tests using a horizontal asymptote to the force-displacement curve, and $Y'_f$ was selected using Hansen’s (1963) 90% criterion as described by Fellenius (1980). To compare moist and dry sand test results at a second dry unit weight for H/D of 6, tests were also performed with dry unit weights of 15.7 and 15.8 kN/m$^3$, respectively, as described by Turner (2004).
Table 2. Summary Information for Tests with Dry Unit Weight of 16.3-16.7 kN/m$^3$ and H/D=6

<table>
<thead>
<tr>
<th>Line Symbol</th>
<th>Water Content (%)</th>
<th>Dry Unit Weight (kN/m$^3$)</th>
<th>Test No.</th>
<th>Friction Angle$^2$</th>
<th>N$_q$</th>
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<tr>
<td></td>
<td>0</td>
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<td></td>
<td>0</td>
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<td>38.6-39.4</td>
<td>21.4</td>
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<tr>
<td></td>
<td>4.2</td>
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<td>38.6-39.5</td>
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<td>4.4</td>
<td>16.6</td>
<td>19</td>
<td>40.5-40.6</td>
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<td>38.6-39.4</td>
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<td>16.4</td>
<td>28</td>
<td>38.5-39.3</td>
<td>23.2</td>
</tr>
</tbody>
</table>

$^1$ T&O = Test data from Trautmann & O’Rourke (1983)  
$^2$ Friction angle range, in degrees, determined from exponential and bi-linear fits to direct shear data
The force-displacement curves shown in Fig. 22 illustrate several important features of soil-pipe interaction. First, the test results for sand with 4% moisture are nearly identical to the results for sand with 8% moisture, including maximum force, displacement at maximum force, and curve shape. Second, for similar dry unit weight, tests in moist sand experienced about twice the maximum force associated with tests in dry sand. Third, displacement at maximum force, \( Y' \), was smaller for the moist sand tests compared to dry sand tests at the same density. Moreover, the initial curve slope, or stiffness, is greater for the moist sand test results. Also, for the same dry unit weight, the moist sand force-displacement curves reach a peak value and decrease, typical of dense, dilative dry sand, whereas the dry force-displacement curves approach a horizontal asymptote, typical of loose or medium dense dry sand.

Figure 23 summarizes values of maximum force vs. dimensionless depth, as determined from the experimental data. Test results for dry, medium dense sand from Trautmann and O’Rourke (1983, 1985) are also shown, and an interpretive curve is drawn through the moist test results and extrapolated to other H/D ratios. For H/D less than 6, this extrapolation was performed by multiplying the dry sand test results by the ratio of moist \( N_q \) to dry \( N_q \) determined at H/D of 6. For H/D greater than 8.5, the dry sand test data were multiplied by the ratio of moist \( N_q \) to dry \( N_q \) determined at H/D of 8.5. The interpretive curve between H/D of 6 and 8.5 was drawn as a line connecting the moist sand data points.

The force associated with partially saturated sand is approximately twice that generated under dry sand conditions. Direct shear test results show that increased shear resistance in partially saturated sand accounts only for about 30% of the increased lateral force relative to that for dry conditions. The principal cause of increased resistance can be explained with reference to Fig. 24, which shows the soil deformation patterns in dry and partially saturated sands. Dry sand deformation shows distinct zones of heave and subsidence, with continuous rotational movement between well-developed passive and active zones in front of and behind the pipe, respectively. In contrast partially saturated sand moves more like a coherent mass of soil that must be pushed forward and lifted by relative lateral movement of the pipe.

Figure 25 shows the maximum dimensionless force, \( N_q \), vs. dimensionless depth, H/D, that are derived for partially saturated and dry sand tests, using the experimental data of Turner (2004) and Trautmann and O’Rourke (1985). Note that predicted curves for a friction angle of 30° are not shown in Fig. 25. Loose, dry sand consolidates during lateral loading, which, in effect, increases the friction angle and \( N_q \) values, and results in larger horizontal displacement to attain maximum load. Moist sand placed in the loose condition typically consists of a bulked, collapsible structure with inconsistent density, for which a uniform mass friction angle is not appropriate. Lateral loading of pipes in loose sand will result in collapse of the bulked structure and compaction of the sand, thereby increasing the dry unit weight and friction angle. With the available evidence from this and previous studies, a percent increase in \( N_q \) from dry to moist loose sand cannot be reliably predicted. Further experimental investigation is needed to confirm the force-displacement behavior of loose moist sand.

These experimental findings have important implications for lifeline design and construction. They confirm significantly increased lateral loads in partially saturated sand compared to those for dry sand that are currently used in practice (ASCE, 1984). The findings also illustrate the value of full-scale experiments, which were used to calibrate a general purpose analytical model, confirm higher reaction forces than predicted with current models, and point the way to a simple and effective means of reducing strain concentrations at elbows through moderate increases in the wall thickness of straight pipe sections adjoining the elbow.

In general, large-scale experiments play an essential role in discovering new mechanisms for soil and structural response, as well as key behavioral phenomena that were not previously appreciated. Such outcomes stimulate advances in modeling procedures.
Fig. 23. Maximum Dimensionless Force vs. Dimensionless Depth for Varying Moisture Content, Dry Unit Weight As Shown.

Fig. 24. Soil Displacement Patterns for Dry and Saturated Sand
Substantial emphasis is now being placed on the physical and numerical modeling of components with large and novel facilities, such as the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES). This network is intended to unite a geographically dispersed system of equipment sites, users, modelers, and industrial partners through high performance Internet so that experiments at different sites can be coordinated, run, and numerically simulated at virtually the same time. Testing facilities for large displacement soil-structure interaction of lifeline components are being developed as part of NEES (Jones, et al., 2004). They provide a unique combination of large-scale and centrifuge modeling facilities. Large-scale testing duplicates pipe and soil behavior and the intricacies of soil-pipeline reactions. Centrifuge modeling provides an excellent complement, through which multi-g scaling is applied to extend the physical range of testing to larger prototype dimensions and rates of loading.
CONCLUDING REMARKS

Soil-structure interaction under extreme loading conditions includes performance during earthquakes, floods, landslides, large deformation induced by tunneling and deep excavations, and subsidence caused by severe dewatering or withdrawal of minerals and fluids during mining and oil production. Such loading conditions are becoming increasingly more important as technologies are developed to cope with natural hazards, human threats, and construction in congested urban environments.

This paper examines extreme loading conditions with reference to earthquakes, which are used as an example of how extreme loading influences behavior at local and geographically distributed facilities. The paper covers performance from the component to the system-wide level to provide guidance in developing an integrated approach to the application of geotechnology over large, geographically distributed networks. Specific topics covered include geotechnical earthquake loading, lifeline response to earthquakes, large-scale tests of ground rupture effects, and soil-structure interaction during ground failure.

Permanent ground deformation (PGD) is the most damaging consequence of an earthquake for underground facilities, including regional distribution networks for water and natural gas. The sources of PGD involve landslides, soil liquefaction, and surface faulting. The generic patterns of displacement for earthquake-triggered ground failure are similar to those for landslides, subsidence, and ground deformation associated with deep excavation, tunneling, and mining activities.

The systematic analysis of pipeline repair records after the 1994 Northridge earthquake show the locations of important seismic and geotechnical hazards and were used to identify zones of potential ground failure not recognized in previous explorations and risk assessments. Moreover, the systematic assessment of pipeline repairs with GIS resulted in regressions linking damage rates and various levels of strong motion. Such relationships are important for loss estimation studies of future earthquake impact to plan for and reduce the potential for seismic disruption.

Large-scale tests of pipeline response to abrupt ground rupture have resulted in analytical models that can simulate such behavior at critical locations, such as pipeline elbows, where local soil restraint and the three-dimensional distribution of deformation leads to increased risk of failure. Large-scale tests of soil-pipeline interaction performed at Cornell University helped show how increased pipe wall thickness near pipe-elbow welds can improve performance under extreme loading conditions by over 200%. They also showed that soil-structure interaction for partially saturated sand results in significantly greater concentration of pipeline strain than for dry sand. Full-scale tests of soil-structure interaction for buried pipelines subjected to large horizontal movements indicate that maximum lateral forces are approximately twice as high for large horizontal displacement in partially saturated sand as for dry sand. Design charts are developed on the basis of experimental results to predict maximum lateral load for different depths of burial, pipe diameters, and soil angle of shear resistance associated with partially saturated and dry sand.

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REFERENCES


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