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General Report Session No. 2: Case Histories of Slopes, Dams and Embankments

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Proceedings: Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, June 1-4, 1993, General Report Session No. II

Case Histories of Slopes, Dams, and Embankments

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CLASSIFICATION OF PAPERS

The 33 papers received for this session involve case histories which were classified into five general topic areas. The topic areas are listed below. The number of papers in each topic and subtopic area is shown in parentheses.

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1. DAMS AND LEVEES (10)
   - o Geotechnical Problems of Dam Sites (1)
   - o Earthquake Response of Dams (1)
   - o Performance History of Dams (1)
   - o Seepage/Stability of Tailings Dams (3)
   - o Seepage/Stability Problems in Levees (3)
   - o Cracking in Concrete Dams (1)
2. EMBANKMENTS ON SOFT FOUNDATIONS (6)
3. CONSOLIDATION/SETTLEMENT OF FILLS (2)
4. LANDSLIDES/CUT SLOPES (13)
   - o Landslides and Their Remediation (6)
   - o Unstable Cut Slopes in Expansive Clay (2)
   - o Earthquake-Induced Landslides (1)
   - o Rockfalls (1)
   - o Landslide Zonation (1)
   - o Mathematical and Physical Models (2)
5. STRUCTURE FOUNDATIONS (2)

DAMS AND LEVEES

Geotechnical Problems of Dam Sites

Paper 2.43 by Gangopadhyay describes geotechnical problems at dam sites associated with adverse geological conditions present at the sites. The author quotes Donald Deere and emphasizes the need to detect all possible defects of a dam site, document the defects, and develop suitable measures to mitigate the defects to ensure an adequately safe dam. The author describes 10 adverse geologic conditions which have affected the design and construction of dams in Eastern India:

1. Weak structural features.
2. Thick mantle of overburden.
3. Deep weathering of bedrock.
4. Karstic conditions.
5. Permeable boulder bed.
6. Soft sedimentary rock.
7. Buried channel.
8. Kaolinisation.
9. Old slides.
10. Reservoir siltation.

The author provides one to three case histories for each of the above adverse conditions and describes measures used to either correct the defects, adapt the structures, or change the site. The remedial treatments included:

- o Excavation of gougy fault zones and backfilling with concrete.
- o Consolidation grouting with inclined grout holes.
- o Backfilling depressions with concrete.
- o Use of large impervious blankets and cut-off trenches to reduce leakage.
- o Relocation of project to miss adverse geologic conditions.
- o Cancellation of project because of high costs associated with treatment needed to make an adequate foundation.

The paper presents good and interesting summaries of the issues involved, but the space available was not sufficient to provide many of the details. Possibly to remind readers of the seriousness of the issues, the author also cites two dams, Tigra and Kedarnala, which failed due to inadequate treatment of their soft sedimentary rock foundations.

Earthquake Response of Dams

Paper 2.2 by Mejia and Boulanger describes dynamic response analyses performed to simulate the response of Stafford Dam during the 1989 Loma Prieta Earthquake ($M_s = 7.1$). Stafford Dam is an approximately 24-meter-high compacted fill composed primarily of clayey sands with gravel, clayey gravels with sand, and silty gravels with sand. The dam is founded on approximately 12 meters of alluvial soils composed of sandy and silty clays, clayey silts, clayey sands, and interlayered sands, silts, and clays containing

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gravel. Below the alluvium lies sedimentary bedrock of the Franciscan Formation.

Stafford Dam was located approximately 130 kilometers away from the Loma Prieta Earthquake. Records of the earthquake motions were obtained from seismographs located on the dam crest and on rock in the right abutment. Peak transverse motions were 0.039g on the right abutment and 0.086g on the dam crest.

Two-dimensional finite element analyses were performed to simulate the response of the dam during the earthquake. Computer program FLUSH was used for this task. This program uses the complex response method to solve the equations of motion and the equivalent-linear method to model the non-linear modulus and damping properties of soil for different values of shear strain.

The transverse motion recorded on the right abutment was used to load the model. For many of the analyses, the input motion was first modified in an attempt to account for the difference between outcrop motions and motions actually occurring at depth below the dam. These modifications were done using computer program SHAKE to deconvolve the recorded motion through a column profile through the dam. Computer program SHAKE is a one-dimensional response analysis which also employs the complex response and the equivalent-linear methods to solve for ground motions. However, unlike a finite element analysis which usually employs a rigid base, computer program SHAKE employs an elastic halfspace. The process was described as follows:

A. The abutment record was input as a rock outcrop motion in a SHAKE analysis of a profile through the dam and foundation. The rock halfspace in the SHAKE profile in this portion of the analysis employed a shear wave velocity of 762 mps.

B. The computed response obtained at the surface of the profile was then input to the surface of another SHAKE profile. The motion was then deconvolved down the profile to obtain a base motion. This second SHAKE profile was identical to the first, except that the rock halfspace was assigned a very high shear wave velocity to simulate the rigid boundary in a finite element analysis.

C. The deconvolved base motion was then used as input for the FLUSH finite element analyses.

The finite element model employed assumed modulus reduction and damping curves selected from previous studies of similar materials. A limited set of shear wave velocity data obtained at the dam during previous investigations was also available to develop general values of maximum modulus values. Several response analyses were performed in an effort to calibrate the model. The different analyses varied maximum modulus values for different portions of the dam and foundation, along with different methods for deconvolving the input motion. The three alternative methods of deconvolution employed were:

1. Deconvolution using a soil profile located at the midpoint of the downstream face of the dam.

2. Deconvolution using a soil profile located at the upstream toe of the dam.

3. No deconvolution.

The results of the analyses showed that the initial trials were not very good at duplicating the motions recorded at the crest. Comparisons using the initial values of maximum modulus were described as only poor. However, after tinkering with the modulus values, very good agreement was reached in Trials 9 and 10. The results also showed that deconvolution had a large effect on the calculated response at the crest. Without deconvolution, the model significantly overestimated the response. Best results were obtained using the profile at the midpoint of the downstream slope for deconvolution.

This paper presents a well-written description of calibration analyses performed for the actual response of an embankment dam at small levels of acceleration. Although very good agreement was eventually reached by tinkering with modulus values, the analyses using the initial estimates of modulus were not very good. This puts into question our ability to predict the future response of dams when we do not have available a previous earthquake for use in calibration.

The authors also raise the issue of the proper way of inputing earthquake motions. The authors have attempted to address some of the limitations of the finite element programs by a deconvolution mechanism. The resulting input motion is almost certainly incorrect for the depth of input, but is intended to result in an overall good estimate of the response of the overall model. One wonders if using computer program SHAKE to simply obtain the sublayer motion from the midslope column with the elastic halfspace and using this motion as input to the FLUSH analysis would have resulted in a better match.

Performance History of Dams

Paper 2.18 by Yasuda, Itoh, and Fujisawa presents performance data obtained from 21 rockfill dams with central cores. These dams had maximum heights ranging between 40 and 150 meters. The authors have summarized internal vertical movements, pore pressures, and post-construction crest settlements and have correlated the data with time and dam height. The results can be summarized as follows:

- Maximum internal settlements developed during construction in the core and rockfill shell zones ranged between 0.5 and 2 percent of the total height of fill at the location being measured.

- Maximum internal settlement in either the core or rockfill shell zones occurred typically at approximately mid-height.

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- Maximum internal vertical strain during construction was 8 percent, with values generally comparable to between 3 and 10 percent of the final height of the dam.

- For dams without unusually high groundwater, maximum pore pressures in the core were generally less than 50 percent of the total height of the dam. Maximum pore water pressures correlated with the height of the dam, construction speed, and the differences between actual moisture content and optimum moisture content.

- The point where maximum pore water pressure developed in the core during construction was typically between 10 and 40 percent of the dam height. The point at which maximum pore water pressure developed tended to increase in elevation as the fines content of the core increased.

- Post-construction settlement of the dam crest was found to be reasonably estimated using the following equation:

$$dv=(0.41\cdot\ln T-0.157)H^{1.7}$$

The paper presents a valuable contribution in describing performance behavior of recent rockfill dams. Such studies are invaluable in setting limit values for determining the safe performance of new dams. The results presented for post-construction settlement are quite similar to those presented by Sowers et al. (1965), Lawton and Lester (1964), Soydemir and Kjaernsli (1979), and Clements (1984).

Seepage/Stability of Tailings Dams

Paper 2.36 by Werno, Dembski, Juszkiewicz-Bednarczyk, Mlynarek, and Tschuschke describes the history and general condition of Zelazny Most, the largest tailings disposal reservoir in Europe. The reservoir has been in operation since 1977 and is the most important element in copper production in Poland. Approximately 80,000 tons of waste is hydraulically discharged daily into the reservoir using the sub-aerial method. The reservoir was originally retained by starter dams composed of medium sand and ranging in height between 4 and 24 meters. As the tailings increased in height, retention was maintained by increasing the height of the dams using tailing materials with the upstream method of construction. The eventual maximum height is expected to be approximately 40 meters. The Zelazny Most tailings reservoir represents a structure which becomes potentially more and more hazardous in time and requires close monitoring. Although good drainage of the tailings is apparently occurring, contamination of the groundwater has also developed. Piezometric measurements indicate pore water pressures in the tailings to be equivalent to less than 50 percent of the hydrostatic levels. Two sets of pumping wells intended to remove contaminated water before entering the groundwater system are apparently having only limited effects, with the result of having additional contaminated water being discharged into the nearby Odra River. Static and dynamic stability analyses have resulted in adequate factors of safety. However, the seismic coefficients being used are very low at about 0.02g, with liquefaction of the tailings requiring a seismic coefficient of about 0.05g. The implication here that is not fully addressed by the authors is that any significant earthquake motion would result in tailings liquefaction, a significant decrease in stability, and a potentially devastating environmental contamination.

Paper 2.27 by Fourie and McPhail describes a stability problem involving a tailings dam retaining platinum tailings. The dam and reservoir was constructed using the ring-dyke method with tailings being deposited along the perimeter of the dyke system and allowed to flow inward. When the north dyke was approximately 32 meters high, a sliding failure occurred involving both the tailings and the soft clay foundation. A high phreatic line was associated with the failure. Both laboratory tests and backcalculations showed the foundation clay to have effective shear strengths of approximately $c'=25$ kPa and $\phi'=10$ degrees. Investigators considered possible variations in shear strength by using a probabilistic approach which randomly varied shear strength values for different potential slip surfaces. As a result of these analyses, other dykes were determined to be potentially unstable. Remedial treatments which were considered included horizontal drains to relieve pore pressures in the tailings, rockfill berms to buttress the slopes, and vertical drains to relieve pore pressures. Horizontal drains were rejected because trial installations showed them to be impractical to install as designed. Rockfill berms were considered to be the most beneficial, but were rejected for their higher cost. Vertical drains were selected as the remediation and were installed at 15-meter spacings along the tailings dams. After 4 months of pumping, the wells were effective in significantly lowering the phreatic surface and improve stability. However, the applicability of this method was probably dependent on the fact that the method of tailings construction resulted in a relatively coarse and permeable outer zone of tailings. An alternative procedure once employed by one of the co-reporters was to isolate the existing materials by placing an impervious blanket. Without the additional supply of water, the lower deposits would eventually drain naturally and stabilize.

Paper 2.35 by Cowherd and Perlea compares theoretical and actual seepage conditions in tailings dams composed of coal refuse. In the United States, the authors report that coal refuse is typically separated into coarse and fine portions. Coal refuse embankments are generally constructed using coarse refuse which typically has D_{50} values between 0.6 and 15 mm. The fine refuse or slurry typically has D_{50} values between 0.005 and 0.6 mm and is deposited near or against the face of the embankment. This results in a beach or delta of fine refuse covering the coarse refuse of the embankment. Normally, there is seldom any water impounded immediately against the coarse refuse embankment. According to the authors, the typical design procedure is to ignore the presence of the fine refuse and to assume a value of 9 for the ratio of horizontal to vertical permeability (K_h/K_v) in the coarse refuse embankment. The authors show phreatic

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surfaces measured in 6 coal refuse tailings dams with ages between 24 and 34 years and maximum heights ranging between 9 and 55 meters. Numerous seepage studies were also performed using finite element program SEEP for different assumptions of permeability and thickness of fine coal refuse. The authors report that:

1. Ignoring the presence of the fine coal refuse deposit was very conservative. Actual phreatic surfaces in the coarse refuse embankments were much lower than those which would be comparable to ignoring the presence of the fine refuse. The fine refuse effectively acts as an upstream "impermeable" blanket. In comparison with the fine refuse, the coarse refuse is a relatively free-draining material which maintains its drainage characteristics over time.

2. The use of a (K_h/K_v) value of 9 for the coarse refuse was also found to be overly conservative. Backcalculations of actual phreatic surface showed equivalent (K_h/K_v) values ranging between 1 and 4, with an average of 2.4. The authors recommended a (K_h/K_v) value of 4 for the design of new embankments. With the presence of the upstream fine refuse, there is no need to provide internal drainage in the coarse refuse embankment.

Although the authors make a good case concerning the apparent gross conservatisms currently being applied to seepage analyses of these embankments, it would seem still a prudent measure to have at least a minimal toe drain to collect seepage along each refuse embankment.

Seepage/Stability Problems in Levees

Paper 2.41 by Yang, Luscher, Kimoto, and Takeshima discusses the quality assurance problems associated with the installation of a slurry cutoff wall through a sandy levee. The levee is located on the east bank of the Sacramento River south of downtown Sacramento, California and is composed principally of loose to medium dense sands interbedded with firm to stiff silts and clays. Geotechnical investigations performed after heavy rainfall in 1986 concluded that a cutoff wall would be required to control seepage and prevent piping failures of the levee system. A cutoff wall was selected in part by limited working space and the presence of residences on the landward toe of the levee. The wall was designed to have a minimum width of 30 centimeters, a depth between 7 and 9 meters, and to have a maximum permeability of 1×10^{-6} cm/sec. The construction of the cutoff wall employed the SMW technique of mixing soils in situ with a slurry of cement, bentonite, water, and additives with the use of multiple shaft augers. A dispute arose regarding the quality/permeability of the installed cutoff wall. The specifications called for permeability tests of cored wall specimens to meet the 1×10^{-6} cm/sec permeability criterion. Although tests of bulk samples of the wall slurry and in situ packer tests in boreholes in the wall generally met this criterion, tests of cores recovered using Pitcher samplers yielded permeabilities of about 10^{-5} cm/sec. Although the contractor felt that the core samples were disturbed and cracked by the sampling operation and were not representative of the in situ quality of the wall, he was unable to convince the client of this. As a result, an additional parallel wall was installed. The slurry for the new wall incorporated a modified mix design intended to allow better sampling with the Pitcher samplers. The paper gave no details of the modified mix used, but suggested that the modified slurry mix resulted in less disturbance and cracking of the recovered core samples. As a result, permeability results from bulk and core samples of the new wall were much closer with both sets of results passing the permeability requirement. The authors stress that it is crucial for all members of the project team to have a clear understanding of the properties of various cutoff wall materials in order to develop reasonable criteria for acceptance of installed cutoff walls. Although the authors recommend the use of bulk samples of the wall slurry for quality assurance testing, the authors also recognize the need for establishing correlations between bulk sample test results and in situ performance of the cutoff walls.

Paper 2.31 by Wade and Davies describes the successful use of Dywidag anchor bars for permanent stabilization of road embankments. The embankments form two bridge abutments lying on the East and West sides of the Elbow River in Calgary, Canada. The abutments consist of about 25 meters of medium dense sand and gravel fill overlying clayey soils with a maximum thickness of about 5 meters. Beneath the clayey soils lies bedrock. Stability concerns consisted of the potential for river flows to scour out the foundations for the bridges, together with the low slope stability factors of safety resulting from the application of the bridge loads. To remedy these concerns, a new concrete revetment was placed against the abutment slopes to prevent erosion, and the revetments were tied down with post-tensioned 36-mm Dywidag ground anchors. The post-tensioned anchors also increased the shearing resistance in the abutment slopes by increasing the normal stresses on potential slip surfaces. On the West abutment, 60 anchors in three rows were installed in boreholes drilled through soil and into the rock. On the East abutment, 41 anchors in two rows were installed through soil and into rock. However, due to sloping geometry of the rock, the 26 anchors in third row on the east abutment were terminated only in the granular fill. In general, the spacings between the anchors were approximately 1.5 meters with anchor lengths varying between 20 and 60 meters. Each anchor had a working load of 500 kN, a lock-off load of 800 kN, and an ultimate load of 1050 kN. Lift-off load tests showed that stresses in the anchors diminished almost immediately, apparently due to settling and readjustments of the individual concrete panels comprising the revetment slabs. A total of 4 lift-off load tests were performed which showed that the current load relaxation rate required between 2 and 4 years for lock-off loads to diminish from 800 to 500 kN. During the first six months of service, revetment slabs had settled about 20-27 mm and had moved 20-25 mm laterally inward toward the abutment, consistent with the direction of applied tension. Results showed the need to periodically reapply tension

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loads in the anchors to maintain the same improvement in stability. Measurements indicate that as time passes, the time intervals between tension applications can be stretched out due to reduced creep rates. Measurements also indicate that the soil anchors in the East abutment tend to lose tension faster. This would suggest that it might be appropriate to schedule periods where only the the soil anchors are re-tensioned in order for all of the anchors in this abutment to maintain about the same tension load.

Paper 2.19 by Deutekom and Termaat presents a case history involving seepage problems through a sand and clay levee overlying a permeable foundation. The levee lies along the Rhine River approximately 20 kilometers east of Rotterdam, Holland. The levee foundation has a clay and peat cap overlying a thick stratum of permeable sand which daylights in the river channel. During high river stages, the levee deformed away from the river and exhibited distress which required repairs. The authors contend that the distress was caused by high pore pressures in the sand layer which caused uplift of the clay/peat cap at the landward toe. The authors postulate that the uplift caused shear stresses to be resisted only by the cap and that this caused the cap to buckle and result in the deformations observed. Supporting the authors' theory are observations which show pore water pressure in the sand reaching a limit equivalent to the overburden pressure during high river stages. In addition, inclinometer data show that deformations generally occur only when the river is high. The paper also spends a relatively high percentage of time showing the results of pressuremeter and laboratory stress test results which indicate that the shear modulus and in situ lateral stress have been increased above at rest conditions. However, the paper does not show how these results are used to design remediation. An improvement in stability was achieved by removing a significant portion of the levee and installing a toe drain to collect seepage water through the levee. However, this would probably have been the same fix if the distress was caused by creep or instability through the levee alone. Very little connection was made between the theorized problem and the solution adopted.

Cracking in Concrete Dams

Paper 2.47 by Ramachandran discusses possible causes of cracking and remedial measures performed in the right spillway portion of Hirakud Dam. Hirakud Dam is located in Orissa, India and was completed in 1957. Almost immediately after completion of the dam, horizontal cracks appeared in the operation gallery of the right spillway dam. These cracks were confined to areas adjoining construction joints. Additional cracks and wet spots were observed in 1973, 1974, and 1983. The author reports that different investigators have not agreed as to the cause of the cracking. Theories which have been offered to explain the cracking include:

1. Thermal cracking caused by the rapid placement of concrete without precooling, lack of construction control, lack of pozzolans, and the use of high alkali cement.
2. Alkali-aggregate reaction because of possible use of river gravels containing opal, chert, and chalcedony.
3. The asymmetrical coefficients of thermal expansion exhibited by the feldspar minerals present in the fine aggregate.
4. Possible effects of industrial pollution of the reservoir leading to highly alkaline water.

The remedial measures taken to date have included opening the cracks with chisels and jackhammers, and grouting them with cement and epoxy resin. The grout only penetrates about 3 meters into the crack. Guniting some portions of the spillway dam with waterproof cement has also been performed. The latest measures consist of adding steel sheets on the upstream side of the dam in an apparent attempt to reduce seepage and deterioration in the concrete. The author does not provide an indication of how serious the cracking represents to the integrity of the dam or how effective the remedial measures are perceived to be.

EMBANKMENTS ON SOFT FOUNDATIONS

Paper 2.8 by Almeida, Danziger, Almeida, Carvalho, and Martins describes settlement problems which resulted following the placement of a 24-meter-high fill over a 5-meter-thick layer of soft clay. The clay layer was supposed to be removed by forced displacements using bulldozers prior to placement of the fill. In the event, an average 2.5-meter-thick layer of disturbed clay was inadvertently left beneath the fill. This lead to undesirable settlements, settlement rates, and the halt to construction of structures on the fill. The authors describe investigations using surface monuments, piezometers, piezocones, and odometers tests in conjunction with methods developed by Asaoka (1978) to predict settlements. The authors were able to reach good agreement in determining values of the coefficient of consolidation, c_v, for the disturbed clay using three different approaches. These values, generally about 1 to 2×10^{-8} m^2/s, were less than 3 to 20 percent of the values for the intact clay. The investigation concluded that the disturbed clay layer had reached approximately 70 percent consolidation and would require between 2 to 6 years to reach 95 percent consolidation. Because remedial treatment was expensive, the owner decided to wait for the settlement to stabilize.

Paper 2.9 by Raymond discusses the performance of a 3.1-meter-high road embankment placed in 1967 on 45 meters of soft soil. The upper 6 meters of the soft soil consisted on granular peat, lake marl, and algae with undrained shear strengths measured to be about 8 kN/m^2. The design employed the use of staged construction, 12-meter-wide stabilizing berms, and the use of 1 meter of sawdust fill for the bottom of the central embankment between the berms. The ratio of base width to embankment height was approximately 15. The design also called for first placing a central layer of logs in the middle 12 meters along the centerline of the embankment. The outer berms were then built

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in an effort to confine the displacement of the soft foundation soils. After the berms were in place, the sawdust fill was then placed to a 1-meter height in the center portion over the logging. Finally, the 2.1 meters of granular fill comprising the upper portion of the central embankment was placed in stages. Total construction time was less than 3 months. Except for the development of a local failure in the instrumentation area caused by discontinuous construction procedures, the settlements and pore pressures were within predicted ranges. Settlement of the original ground surface along the centerline of the embankment was approximately 1.2 meters during construction. The paper describes an innovative method for staged construction, along with the use of light-weight fill and logging reinforcement. However, the details of the design are not fully discussed and the reader is referred to other references for further information. Many of the design dimensions seem to be developed from judgment and past experience. Although the author states that both undrained and effective stress methods were inadequate for use in designing the fill, the author does not fully discuss their inadequacies.

Paper 2.15 by Lo and Li presents the results of observed performances and backcalculations of responses for two alternative designs of a breakwater embankment placed on soft soil. The breakwater embankment is 7.5 meters high, is composed of crushed rock, and is founded on approximately 10 meters of soft marine clay in a bay near Shenzhen, China. The original design called for the removal of the upper 2 to 4 meters of soil, backfilling the excavation area with a blanket layer of sand, construction of stabilizing berms, and staged construction of the central embankment. The ratio of base width to embankment height was approximately 16. After several failures during construction of the central embankment, the design was modified. The basic modification consisted of installing a layer of woven geosynthetics in the sand blanket and significantly <u>decreasing</u> the size of the stabilizing berms. The resulting modified design had a ratio of base width to embankment height of only about 10. The modified design successfully allowed the completion of the breakwater in 1985. Analytical studies were performed following construction in an attempt to "predict" the behavior observed during construction. The three methods chosen for analysis consisted of:

- A limit equilibrium method using undrained shear strengths.
- A finite element analysis assuming strain hardening properties for the foundation clay.
- A finite element analysis assuming strain softening properties for the foundation clay.

The analysis results showed that the limit equilibrium analysis gave a factor of safety of 1.22 for the original design and a factor of safety of 1.12 for the reinforced design. These results did not match either the values or the trends of the actual performance. The strain-hardening finite element model gave a fair estimate of measured displacements for both designs, but was unable to show the instability of the original design. The strain-softening finite element analysis produced a good estimate of the measured displacements for both designs and was able to show the pending instability of the original unreinforced design.

Paper 2.21 by Santiago, Barrera, and Pastor describes the response of a test embankment placed on uranium tailings near Andujar, Spain. The test embankment was approximately 4 meters high with a base which measured 46 meters by 35 meters. The fill was placed in about 12 days on approximately 5 meters of tailings. The upper 2 meters of tailings were partially saturated and nearly saturated at greater depths. The embankment and tailings were extensively instrumented with devices to measure settlement, total stress, and pore pressure. Maximum settlements were less than 0.15 meters with over 80 percent of the settlements developed within 60 days of the fill application. Most of the settlement took place during construction. A finite element analysis employing strain-hardening properties was able to reasonably predict the settlements over time. However, due to the fact that the analysis assumed saturated materials, it overpredicted the small pore pressures generated in the unsaturated tailings.

Paper 2.28 by Huat and Ali describes the performance of a test pile embankment founded on soft clay in Malaysia. Due to the need to construct high roadway embankments on soft soils, the Malaysian Highway Authority often employs grids of timber or concrete piles to support portions of the embankments. In 1987-88, a test embankment was constructed and instrumented to verify design procedures. The site for the test embankment contained approximately 17 meters of soft, sensitive clay with a 1-meter surface crust of dessicated clay. Typical undrained shear strengths determined in the soft clay by vane shear equipment were between 9 and 36 kPa. The test embankment was designed to have a maximum height of 6 meters. A grid of precast concrete piles (94 in total) was placed beneath the central 22 meters of the embankment. Each pile was driven down 21 meters to a supporting layer of sand and fitted with a 1.8 meter by 1.8 meter cap. Beyond the central pile-supported portion of the embankment, 3-meter-high stabilizing berms were added on both sides of the embankment. The total base width including berms was 62 meters, with only central 22 meters supported by piles. After placement of the piles, the berms and central fill were placed in stages. On Day 64 when the central fill was 4.3 meters high, differential settlement became apparent at the junction between one of the unsupported berms and the pile-supported central fill. This differential settlement resulted in the formation of a tension crack parallel to the embankment axis. During the next 160+ days, the berm on this side settled as much as 0.65 meters. When additional fill was added and the central fill had reached a height of about 6.4 meters, the unsupported berm collapsed. Progressive failure led to slumping in the central portion of the pile-supported fill. The authors also present the results of a centrifuge study intended to model the failure. The centrifuge model did not show a collapse as did the test embankment, but did show a zone of overstress in the foundation,

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along with tension cracks at the junction with the model piles. Both the test embankment and the centrifuge model showed the false economy of limiting the piles to only the central area of the embankment. The paper indicates that the same number of piles, if placed at greater spacings across the entire embankment, would have resulted in a stable structure.

Paper 2.37 by Colleselli and Cortellazzo presents the results of two instrumented levees built on thick soft compressible soils of the Po Delta in Italy. Both levees are unreinforced and incorporate stabilizing berms. The Volta Vaccari levee has a maximum height of 6 meters, a base width of 55 meters, and is founded on 30 meters of predominantly soft, silty clay. The Malcantone Levee has a maximum height of 9 meters, a base width of 75 meters, and is founded on approximately 10 to 15 meters of soft silt and clay. During design, the levees were designed using limit equilibrium methods to assess stability, and odometer tests and pressure increments calculated using elastic theory to estimate settlements. Both levees were successfully constructed in the mid-1980's and monitored for pore pressures and deformations during the next four years. Following construction, backcalculations of performance were made with more sophisticated techniques, notably finite element analyses employing a modified Cam-Clay strain-hardening model. Although the classical methods used during design provided reasonably adequate predictions of performance, the more sophisticated analysis techniques provided more complete and accurate information regarding settlement rates and pore pressure generation. The authors caution that such good results depend on obtaining a high number of good quality in situ and laboratory test results.

CONSOLIDATION/SETTLEMENT OF FILLS

Paper 2.4 by Ulrich describes the results from two separate in situ investigations of tailings deposits. The first investigation was made in 1988 in tailings created using the sub-aerial method at a gold mining complex in northern Nevada. At this time, the tailings were approximately 5.5 meters thick. During the next three years, another 4.5 meters of tailings had been deposited over the initial testing site. In 1991, another in situ investigation was performed at the same general location. Tests performed in both investigations included the piezocone, self-boring pressuremeter, field vane shear test, and the Standard Penetration Test. The initial results suggested that the entire previous deposit had not significantly consolidated over the three years. However, a more careful examination of the data showed that the upper portion exhibited significantly higher penetration resistance, strength, and stiffness. The increases decreased with depth with the bottom of the original deposit showing only marginal increases. This was explained by the authors as possibly being due to the fact that some consolidation had already taken place in the lower deposits at the time of the initial field investigation. However, another explanation would be that drainage might be inhibited at the bottom of the tailings reservoir, preventing a significant increase in consolidation in the lower deposits during the three-year period. In either case, the paper presents a good set of in situ test results.

Paper 2.6 by Trow, Carrington, and Orpwood describes the monitored settlement of a 32-meter-thick compacted fill. The fill was placed in an abandoned brick quarry and was intended to provide a site for a residential development. Although the fill came from a variety of sources and included some plastic material, it was intended to be free of contaminants and to be compacted to have a relative compaction of 95 percent of standard proctor maximum density. The upper 9 meters of fill was given particular compaction care in order to provide a raft-like mat to help bridge over any zones of variable compressiblity existing at depth. Construction began in July 1984 and was completed almost 3 years later in 1987. Settlements were monitored until April 1991 with four magnetic extensometer and settlement gages. Some adjustments and estimates of initial settlement had to be made to the data because no measurements were made during the first 9 months. The measurements showed that significant settlements can occur in even reasonably compacted fill. Maximum settlements were observed to range from 3 to 6 percent of the total fill height placed. With a rate of construction that never exceeded about 1.4 meters per month, almost all of this settlement occurred during fill placement with relatively little settlement occurring following the end of fill placement.

LANDSLIDES

Landslides and Their Remediation

Paper 2.17 by Mooney and Bowders presents a case history involving a small, incipient landslide in colluvium in West Virginia. The slide occurred in December 1987 on a 3.3:1 slope above a residence. The surface of sliding was estimated to be along the contact of the colluvium and the underlying shale, generally about 8 feet in depth. The total movement was estimated to be about 0.3 meters and to have been triggered by rainfall. Although the method of calculation was not described, the authors backcalculated a residual frictional shear strength between 23 and 25 degrees. This appeared to be in reasonable accord with previous correlations relating to clay content and plasticity index. The solution of the problem consisted of installing an 8-foot drain trench above the slide to cut off subsurface water and reduce the ground water level within the slide mass. Since installation of the trench, the slide mass has been stable. The authors state that the installation of the drain trench was done without their guidance and indicate that the drain may become clogged over time.

Paper 2.23 by Gonzalez-Valencia and Herrera-Castenada provides an excellent description on the use of instrumentation in monitoring and remediating a large landslide. The landslide involved approximately one million cubic meters of volcanic breccia which threatened the Agua Prieta Hydroelectric Power House near Guadalajara City in Mexico. During project construction in 1987, the 35-degree natural slope was trimmed to provide berms in an attempt to increase stability. Inclinometers

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set into the slope showed only limited movements until 1990 when displacements increased to about 10 mm per month. Additional instrumentation and geotechnical investigations showed that the slide of volcanic breccia was moving on a layer of highly plastic clay derived by weathering of lacustrine tuff. Piezometers showed that the piezometric surface was relatively high within the slide mass and increased during periods of heavy rainfall. Remedial work initially consisted of providing two large drainage galleries into the rock. However, the slide was not stabilized until approximately 37,000 cubic meters was removed from the active portion of the slide. Inclinometer measurements showed that the removal of this material abruptly halted deformations. Particular items of interest included:

- The initial trimming operations performed during project construction were concluded to actually have destabilized the slope.

- Removal of 37,000 cubic meters from the active portion of the 1,000,000 cubic meter slide halted the slide movement. Additional excavation leading to a total of approximately 100,000 cubic meters was calculated to have improved stability by 25 percent.

- The use of bentonite mud during the installation of piezometers in the slide mass was believed to have created low permeability zones within the breccia block and increased deformations during the raining season.

Paper 2.29 by Stanculescu, Athanasiu, Chirica, Stanculescu, and Georgescu presents the results from a controlled failure experiment in stiff-fissured clay. The experiment consisted of cutting back a slope of Dobrogean red clay along the Danube-Black Sea Canal. The excavation was performed in two stages in order to examine a progressive failure induced by reduced toe confinement. Prior to the field work, pressuremeter and laboratory tests were performed and the slope was instrumented with inclinometers. As in previous investigations, laboratory results showed a low residual shear strength for the fissured clays. As a result of the first stage of excavation and reduced confinement at the toe, limited surface cracking and inclinometer movement was observed. Following the second stage of excavation, more cracking and significant inclinometer movement (16 mm) was noted and a slide plane was interpreted as having developed. Laboratory modulus and shear strengths were used in a computer program to predict factors of safety. Prior to excavation, the factor of safety was calculated to be 1.33. Following the Stage II excavation, the factor of safety was calculated to be 1.02, in reasonable accord with the observed distress. However, the analysis calculated a factor of safety of 1.32 for the Stage I excavation, a value that would seem to be too high in light of the observed deformations. Another question which arises is why laboratory modulus values were employed when pressuremeter data appeared to be available.

Paper 2.30 by Al-Saadi and Al-Jassar describes failures of rock slopes along highway cuts in Northern Iraq. The failures consist of dip slope slab sliding of limestone blocks along thin beds of clay and marl. Some of the block sliding is facilitated by jointing in the limestone. The authors quote shear strengths determined for the claystone and marl layers and conclude that weathering decreases the cohesion values of these materials and trigger sliding. The authors recommend slope trimming and rock bolting to stabilize the limestone blocks.

Paper 2.49 by Dodds, Burak, and Eigenbrod describes a landslide along the Nipigon River in Ontario, Canada. The slide occurred in glaciolacustrine plain and delta deposits consisting of sands and silts. An eyewitness account and the breakage of a fiber optic cable indicate that the 350-meter-long slide developed over a three-hour period in April 1990. Slope stability analyses performed using the results of laboratory tests performed on undisturbed samples indicated factors of safety generally between 0.85 and 1.1 for the steep river bank slope. The range in safety factors was caused by different assumptions of water level in the river and within the slide mass. The analyses and the three-hour duration of sliding indicated that the slide developed initially in a limited area along the steep river bank and retrogressed up the slope as slide material moved into the river. The investigation concluded that several man-made activities contributed to the slide. These include:

- Relatively rapid fluctuations of the river level (1.2 meters/day) caused by upstream hydropower releases of water.

- Logging and construction activities at the head of the slide which left the ground deforested and susceptible to infiltration of surface water.

Paper 2.54 by Hunt, Miller, and Bump describes investigations made to evaluate and remediate a large landslide in marine shales. The landslide is located along the edge of the Oahe Reservoir in Forest City, South Dakota and impacts an approach embankment and bridge which crosses the reservoir. Studies indicated that the landslide originally developed in early post-glacial times as the Missouri River eroded the channel slope. However, the filling of the Oahe reservoir in 1958 was thought to have induced recent movement by reducing effective stresses and softening the shales. Shear tests of the weathered shales indicated residual friction angles between 6 and 16 degrees, but backcalculations of shear strength indicated that the in situ strength was approximately 6 degrees. One of the more interesting features of the slide was that the approach fill was built on a knoll of shale apparently more resistant to movement as surveys showed less deformation toward the lake at this location than in the adjacent portions of the slide mass. This led to several remedial alternatives which considered this feature, including consideration of a slurry wall that would be built around the approach fill foundation to allow the landslide to slide around the more stable knoll. In the event, the investigators chose to unload the driving force above the approach fill and the authors report that excavation of approximately 7.6 million cubic yards of material has begun.

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Unstable Cut Slopes in Expansive Clay

Paper 2.40 by Zunjing, Qiuyan, Guanping, and Xiangfan discusses three case histories involving slides in canal cut slopes. In all three cases, portions of the excavated canal slopes were in expansive clay soils. Following excavation, these soils expanded, developed fissures, and slid. In two of the cases, sliding was initiated and/or aggravated by rainfall. Laboratory tests on samples of the expansive clay indicated very low residual shear strengths ($c' < 10$ kPa, $\phi' < 12$ degrees). Remedial measures included flattening the slopes, buttressing the slopes with fill, and installing grouted rubble arches and concrete piles to increase shear resistance. For some reaches of the third case history, the canal excavation was so unusable that the solution was to construct culverts to convey water.

Paper 2.48 by Guangbin and Zunjing describes two projects involving highway cuts in expansive soil. In both cases, the expansive soil had plasticity indexes between 22 and 29 and was reported to have extremely low undrained shear strengths following expansion. For the first project, several costly features were incorporated in the design of the slope to prevent sliding. These included 12-meter-long reinforced concrete piles to prevent sliding, multiple arch retaining walls, and trench drains running up the slope. The slope surfaces were also covered with revetments consisting of grouted rubble within frameworks embedded in the slope. These slopes have performed well since construction in 1990. The second project also employed revetments of grouted rubble within frameworks embedded in the slope. While these slopes have performed well, other slopes on the same project using only sod for surface protection have developed extensive shallow and deep retrograde sliding. The slides typically occurred during rainfall and slowed or stopped during the dry seasons. This sliding has continued to occur over the last seven years and shows the inadequacy of the sod protection for this expansive soil.

Earthquake-Induced Landslides

Paper 2.5 by Rodriguez and Poran presents a case history of two landslides which developed in weathered volcanic ash following the 1983 Popayan Earthquake in central Colombia. The earthquake had a magnitude of 5.5 which induced an estimated peak acceleration of approximately 0.2 - 0.3g at the landslide sites. A power canal providing water to a hydroelectric plant was located on both landslides. The first slide developed 10 days following the earthquake. The second slide occurred 8 months following the earthquake. In both cases, the slides occurred during an interval of heavy rainfall. This rainfall, together with water released from the ruptured canal during the first slide, facilitated the remolding of the volcanic soils and the conversion of the slide masses into mud flows which extended large distances from their original positions. Although the rainfall was significant at the time of the slides, the amounts were not unusual for the area. Geotechnical investigations showed that the in situ weathered volcanic ash could be classified as porous clayey and sandy silts with slight cementation. Slope stability analyses employing effective shear strengths obtained from undisturbed samples resulted in factors of safety between 1.4 and 1.6. However, when remolded in situ, laboratory tests showed that the soils exhibited significant strength losses. The investigators employed undrained shear strengths together with a Makdisi-Seed type of analysis to estimate an earthquake-induced deformation of 1 centimeter during the earthquake. This small deformation correlated with the observation that only limited, small cracks were observed immediately after the earthquake. However, the investigators conclude that this deformation resulted in sufficient remolding to significantly reduce the static shear strength and result in failure during the subsequent heavy rainfall periods. The authors also present the results of parametric studies related to the effects of three-dimensional sliding, anisotropic shear strengths, and the effect of vertical acceleration on stability. One question which arises is why the authors employed the Simplified Bishop Method, generally applicable to only circular sliding, when the slide geometries resembled wedge failures.

Rockfalls

Paper 2.44 by Bhandari, Jeyatharan, and Raviskanthan describes the dynamics of rockfalls in Sri Lanka. In many areas of Sri Lanka, boulders hurling down the hill along natural water courses result in the destruction of roads, bridges and other public utilities. The authors describe two examples of rockfall problem sites and present charts to estimate runout distances, velocities and potential damage. In order to use the charts, it is necessary to employ estimates of the coefficients of friction for the slopes where rockfalls are to slide and/or land. It is not clear how the charts were developed or how these friction values are to be estimated. Similarly, it is not clear if the charts are for use with rocks with only certain geologic characteristics.

Landslide Zonation

Paper 2.34 by Rao discusses landslide management and control in the Himalayas. The first half of the paper describes the use of a landslide risk analysis and zonation procedure used to select alternative highway routes through the North Sikkum region. The procedure is also used to prioritize existing areas for remedial measures. The author also briefly describes the use of geogrids composed of natural fiber (coir) to reduce surface erosion on soil slopes.

Mathematical and Physical Models

Paper 2.12 by Prochazka and Vacek describes mathematical and physical models used to simulate a landslide which developed in weathered gneiss, claystone, and clay during a mining operation. Very little information on the actual slide is presented and the results of the mathematical modeling was difficult to follow. The physical model was 180 centimeters long, 90 centimeters high, and 29 centimeters wide. Mixtures of sand, cement, gypsum, and water were used to simulate gneiss, claystone, and clay. Removal of materials was used to simulate mining operations and the authors

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report that the physical model was able to simulate the actual slide failure. However, the authors report that shear strengths and moduli significantly different from actual in situ properties had to be incorporated in the model in order to simulate the actual slide. This calls into question the practicality of the physical model to predict failures in advance.

Paper 2.14 by Jianguo discusses the development of landslides over time. The authors separates the development of creep movement of rock specimens into three stages: initial creep, stable creep, and accelerated creep to failure. Assuming that actual landslides have similar time-dependent stages of movement, the author describes a simple mathematical model to simulate time effects using frictional and viscous elements. Over time, the frictional and viscous resistances both are considered to degrade linearly. The author then applies the model to time and deformation data obtained for three actual landslides. Essentially no data related to the actual landslides are presented and it is not clear how the characteristics of each slide are used to develop the parameters used to model the linear deterioration in resistance.

STRUCTURE FOUNDATIONS

Paper 2.22 by Lindenberg, Plooster, and Janssen describes data collected beneath one of the piers of the mammoth Eastern Scheldt storm surge barrier in the Netherlands. This barrier consists of a series of mammoth piers constructed on a prepared foundation of sand for the purpose of preventing large surges of water from entering three channels of the Eastern Scheldt. Steel gates mounted on the piers constrict water flow to accomplish this. The structure was completed in 1986 and is intended to function during large storms for the next 200 years. To verify the design, one of the piers was instrumented to measure water levels on both sides of the barrier, wave heights and frequency characteristics, pier displacements, and foundation piezometers. An opportunity to verify predicted displacements and pore pressures came during the storm of February 26 - March 2, 1990. The results from the instrumentation data were as follows:

- Three years after the piezometers had been installed, only half of the devices remained functional. During the storm, cyclic pore pressures showed peak values of approximately 2 kN/m^2 as a result of wave loading. Piezometers placed on opposite ends of the piers showed phase shifts of 180 degrees, demonstrating the rocking motion of the piers. High frequencies of wave loading were damped out. Pore pressure values were less than half of those predicted during design for the wave loadings measured, indicating a conservative prediction.

- No detectable cyclic displacements could be measured. As design calculations predicted between 5 and 8 mm of displacement for the wave loading measured, this result also showed that the predictions were conservative.

Paper 2.11 by Ruhl provides an overview of geotechnical issues associated with the design of the Olmstead Locks and Dam Project. This is a billion dollar project proposed for the Ohio River and consists of twin 365-meter locks, a 670-meter long navigable pass section, and a 130-meter section of fixed weir. Much of the foundation and channel slopes include layers of clay with very low residual shear strengths. Issues discussed include landslide remediation on the Illinois side of the channel, pile design, liquefaction potential, the design of cellular cofferdams, and impacts to the environment. As a result of the design studies, the following determinations were made:

- The landslide on the Illinois side will be remediated mainly using excavation. Over a million cubic meters will be excavated for this purpose.

- The pile driving analyser (PDA) was used in conjunction with a test program to determine if the PDA would predict the load capacity of foundation piles. Observations and test results showed that a large setup developed after driving, resulting in the PDA underpredicting the actual capacity of the pile. However, if used after time, PDA restrike values gave reasonable results. Pile capacities will be designed using the procedure developed by Decort (1989) using SPT results. The PDA will be used as quality control during construction.

- The construction for the lock structure will employ cellular cofferdams. Due to the presence of the landslide on this side of the channel, the interior of the cofferdam will require a large buttress.

- Approximately 2.5 kilometers downstream of the site there exists a bed of endangered mussels. In order to show that construction activities would not create excessive turbidity and kill the mussels, a mathematical model was developed by the Waterways Experiment Station. Calculations showed that construction activities would not adversely affect the mussels.

TOPICS FOR DISCUSSION

The papers in this session provide information on several interesting subjects. Many papers show again the value of instrumentation to either confirm designs or warn of impending distress or failure. The following topics were derived from the papers in this session and are suggested for discussion:

1. Some papers employing mathematical or physical models to simulate measured behavior were not able to duplicate reality using initial estimates of in situ properties. Relatively good duplication was eventually achieved only because material properties were changed in the models in order to match measured behavior. Examples of this include the dynamic response analysis of Stafford Dam in **Paper 2.2** and the

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physical landslide model described in **Paper 2.12**. What does this say about our abilities to predict or simulate behavior in advance?

2. Several papers on the performance of embankments constructed on soft ground employed the use of sophisticated analyses to examine behavior after the embankments were completed. However, the embankments were typically designed using less sophisticated classical methods. Why are the more sophisticated approaches not used more during design?

3. **Paper 2.15 by Lo and Li, Paper 2.21 by Santiago et al.,** and **Paper 2.37 by Colleselli and Cortellazzo** all conclude that finite element analyses using strain-hardening soil models are able to reasonably predict behavior of embankments on soft soil when such embankments perform well. When such embankments experience large distress and/or failure, Lo and Li indicate that the strain-hardening model was unable to predict failure. However, their use of a strain-softening soil model was able to reasonably predict the failure. Has this result been experienced by others?

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