Levee Stability Application for Deep Mixing (2) – Conclusions from Test Section Using Dry Mixed Soil Cement Columns

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The New Orleans District of the U.S. Army Corps of Engineers is considering Deep Mixing to improve the foundation beneath a planned earthen levee. To evaluate the performance of panels of soil cement columns in slope stability improvement, a full-scale test section using dry mix columns was instrumented and loaded to failure. Test data, results, and observations are presented. Conclusions and recommendations are offered by the authors in this companion paper.

INTRODUCTION AND BACKGROUND

Before deep mixing technology could be applied to flood protection slope stability, the New Orleans District of the U.S. Army Corps of Engineers invested in a full-scale test section to study design and construction processes, column performance, and quality assurance methods for dry mix columns. The test site was adjacent to the site of a planned replacement ship lock along the Inner Harbor Navigation Canal, located in urban New Orleans. A full description of the site, including relevant geologic and geotechnical data is provided in the companion to this paper (Cali et al., 2005). The test section was constructed in three phases. A bench scale study was conducted to optimize the lime and cement ratio and binder application rate for the four soil types within the foundation. In Phase I of the test section, ten-meter long, 800-mm diameter dry mix columns were constructed and tested using coring and either pressuremeter or laboratory testing of the cores. Also, Reverse Cone Penetrometer Tests (RCPT) were conducted on some columns. Design mix, mixing energy, shear strength testing methods, and construction processes were studied.

To obtain data verifying column/soil interaction assumptions made for infinite levee slope stability analyses upon which the actual flood protection levee design would be based, two test cells were constructed in Phase II of the project. Three column array replacement ratios were originally planned: 20%, 30%, and 40%, (Figure 1*). However, unexpected funding shortages delayed delivery of the dead load, which would then be insufficient to fail the column array at their predicted 90-day strength. This necessitated a reduction of scope of the test to two cells with replacement ratios of 12% and 20% of the loaded footprint, as shown in Figure 2. Columns were arranged in panels as recommended by Broms (Broms, 1999), or shear walls, oriented parallel to the axis of failure. All Phase II test columns were constructed using Type II Portland cement as the sole binder. The typical penetration rate was as stated in the companion paper.

Each cell was to be loaded separately and was to be isolated from side forces by means of side trenches parallel to the direction of failure. This became a practical impossibility due to probability of side slope failure into the trenches. A trench was excavated in front of the cells and provided the initial load on the test section. Columns were installed on the unloaded sides of the trenches to prevent trench collapse toward the loaded section. Dead load was applied until failure. Based on the Method of Planes slope stability analysis, the theoretical ultimate applied load capacity of the soil column matrix for failure at 30-ft depth would be 604,405 kilograms (128 kN/m^2) for the 12% cell and 972,500 kilograms (178 kN/m^2) for the 20% cell, compared to 105 kN/m^2 that would be exerted by the levee. To ensure failure of the test section, 1,134,000 kilograms of steel was stockpiled for use. Each cell was instrumented to measure load distribution between the soil and columns, pore pressure increase in the soil, and depth and inclination of the failure surface, in real time.

An untreated reference cell was also loaded to failure to assist in a clear and direct evaluation of the effects of the reinforcing columns. By extrapolating the data from the 12% and 20% replacement ratios, along with high quality untreated soil test data, an actual failure load for the untreated cell was estimated.

From earlier levee slope stability analyses, it was calculated that the shear strength of the soil mass in the failure prism would have to be improved from the in-situ average shear strength of 12 kN/m^2 to 96 kN/m^2 to yield the approved levee design safety factor of 1.30 with respect to failure into the new channel, (Figure 3).

A 30% replacement ratio of columns having an average shear strength of 290 kN/m^2 would yield a composite average of 96 kN/m^2. In laboratory bench scale tests, it was evident that this was easily achievable for all foundation deposits except the organic clay layer present from approx. el. –3.6 m to –5.5m. The test section ultimate applied load was then calculated using the Method of Planes slope stability analysis, assuming 290 kN/m^2 uniform strength for the entire column length.

**(Original project data were in imperial units, and for historical continuity these units are used where appropriate.)*

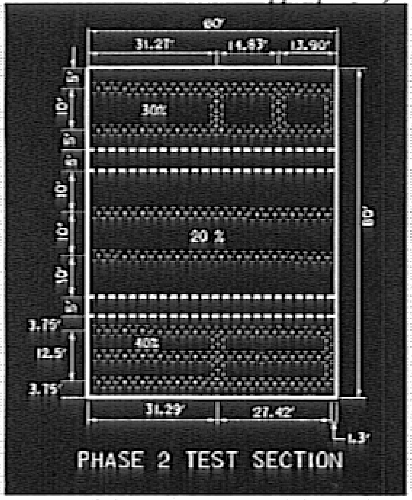

Figure 1. Foreseen Cell Layout with 20%, 30%, and 40% Replacement Ratios

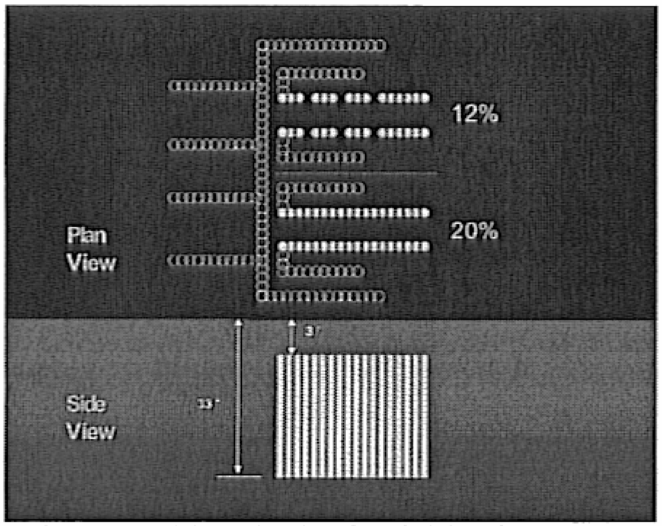

Figure 2. As Built Cell Configuration with 12% and 20% Replacement Ratios

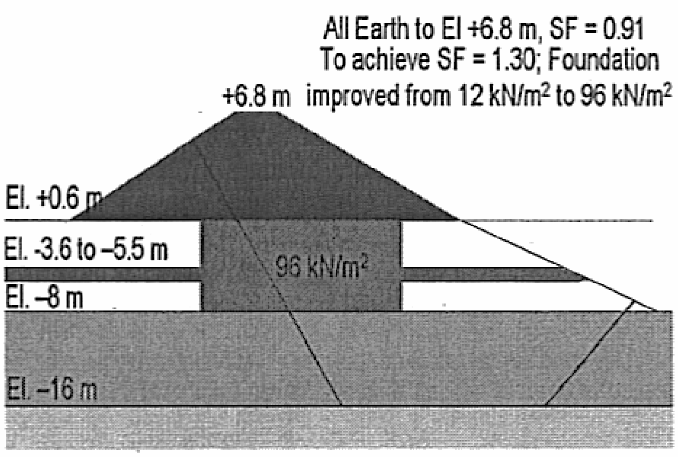

Figure 3. Slope Stability of Earthen Levee

LOAD TEST SETUP

Before subjected to dead load, the test cells were instrumented and reinforced in the weak direction using geotextile strips to resist possible failure perpendicular to the intended direction. 0.9m of soil was removed from the natural ground surface to the tops of the columns over the 7.6 m by 23 m test section footprint so that instrumentation could be installed. The upper meter was not treated since binder flow was turned off near the ground surface to prevent pressurized binder flow from entering atmosphere. After instrumentation was installed, the 7.6 m by 23 m area was backfilled in 15 cm lifts with four layers of 46 cm wide high strength geotextile strips to resist bearing failure in the weak direction of column reinforcement, (Figure 4).

Four In-Place (IP) inclinometers were installed at 2 m and 4.3 m from the front face of the columns in each cell to a depth of 20 meters. The automatic inclinometers had 15 sensors each, ten of which were placed at one meter intervals along the 10-m columns, with the remaining five sensors at 2 meter intervals below the column bottoms. Ten inclinometer casings were installed to a depth of 20 meters. Two of these could be manually read during installation, the rest were under the footprint of the excavation. Four piezometer tranducers were installed in the organic clay layer, one between the test columns, and one adjacent to the columns in each cell. Twelve earth pressure cells were installed. In each cell, four cells were placed directly on top of columns and two were place on untreated soil. With the exception of the ten-inclinometer casings, all the instrumentation was wired and connected to a data logger housed within an onsite trailer. The data logger was attached to two computers: one housing the database and one to serve as a web server so data could be viewed in real time as loading was taking place.

Columns fitted with inclinometer casings were cored with a 76 mm diameter Pitcher sampler. Column samples underwent unconfined compression tests, and three columns were subjected to pressuremeter testing, the results of which are presented in Figure 5.

Pressuremeter readings were interpreted using a model based on the assumption that the columns behaved as elastic cohesive materials. The data were also interpreted using the limit pressure model recommended in ASTM D4719. The cohesive model was used in test section data analysis since it gave more conservative values that better correlated to the laboratory test data.

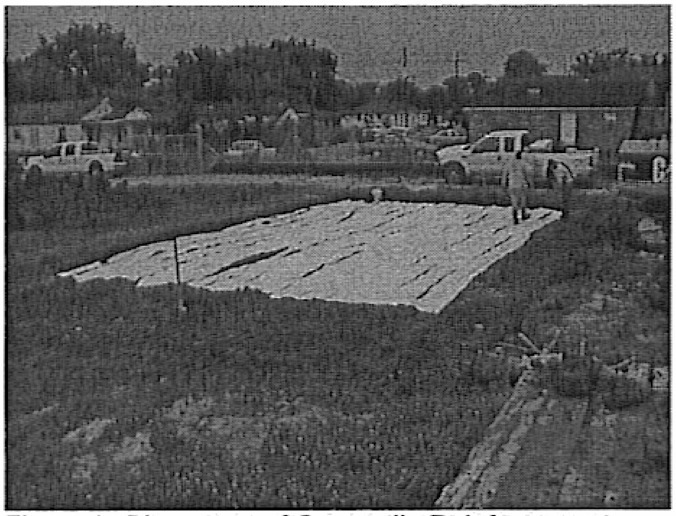

Figure 4. Placement of Geotextile Reinforcement Strips

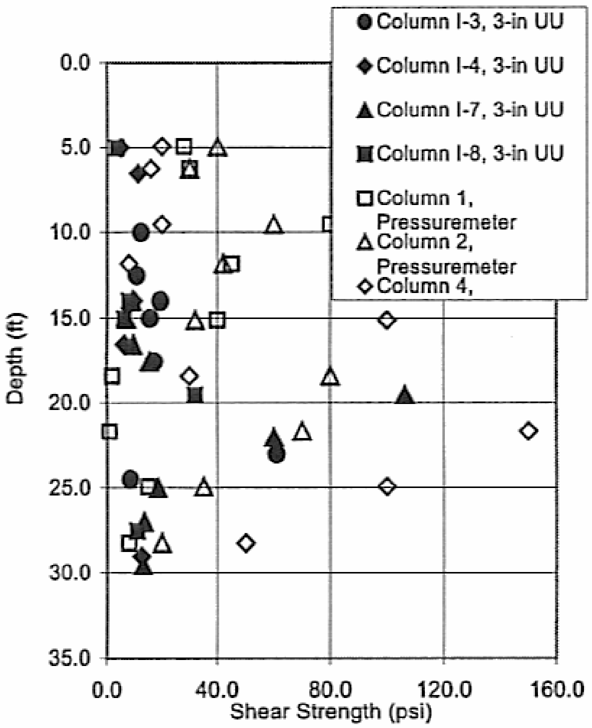

Figure 5. Column Shear Strength Testing Data

TRENCH CUTTING – FIRST STAGE OF LOADING

Trenches were cut across the faces of the test cells to minimize the steel required for the dead load and to insure failure in the planned direction. The trench excavation served as the initial load on the test section, with support columns guaranteeing no failure of the unloaded trench faces. Cell B (12% replacement ratio) was loaded first, followed by Cell A (20%). When the trench was excavated to a depth of 10 meters in front of Cell B, columns along the trench face 145, 146, 147, and B36 collapsed into the trench. Columns 145, 146, and 147 were 4.6m deep and slid into the trench in their entirety; B36 was 10m deep and broke off at a depth of 4.6m. When the trench was excavated to a depth of 10m in front of Columns 138, 139,140, and B18 a similar series of events occurred. The trench was then excavated to a depth of only 5.2m in front of Cell A.

DEAD WEIGHT – SECOND STAGE OF LOADING

Wooden mats were placed in alternating directions to create a 61cm thick pad over the 73 square meter loading area. The dead load applied was a stack of 15cm x 23cm by 7.6m long steel billets. Ultimately, a total of 116.5 kN/m^2 was placed on Cell B. The face of the dead load adjacent to the trench moved downward 1137cm during loading while the rear moved down 27cm. The entire load translated 51cm towards the trench in combined subsurface movement and surface sliding. After loading of Cell B was complete, the trench in front of Cell B was backfilled. Cracks were observed emanating from either side of the loaded footprint of Cell B during loading, some of which translated into Cell A.

Prior to loading Cell A, the trench north of the cell was loosely backfilled to prevent collapse of the facing columns as was experienced in Cell B. A total of 183.5 kN/m^2 was loaded onto Cell A, (Figure 6). By end of loading, Cell A had moved downward an average of 41.4cm; two hours later it had moved down an average of 46.1cm; and two days later (after half of the load had been removed from Cell A), the average downward movement of Cell A was measured as 47.5cm.

An unreinforced cell, Cell C, was loaded similarly to Cells A and B. A trench was cut in front of Cell C to a depth of 4.3m, then loosely backfilled. Timber mats and six layers of billets were placed for a total load of 63.6 kN/m^2. After 5 layers were placed the rear of the load had moved down 3.6cm while the front (adjacent to the backfilled trench) had moved down 14.2cm. Inclinometer readings were recorded after each load stage and four days after loading was completed.

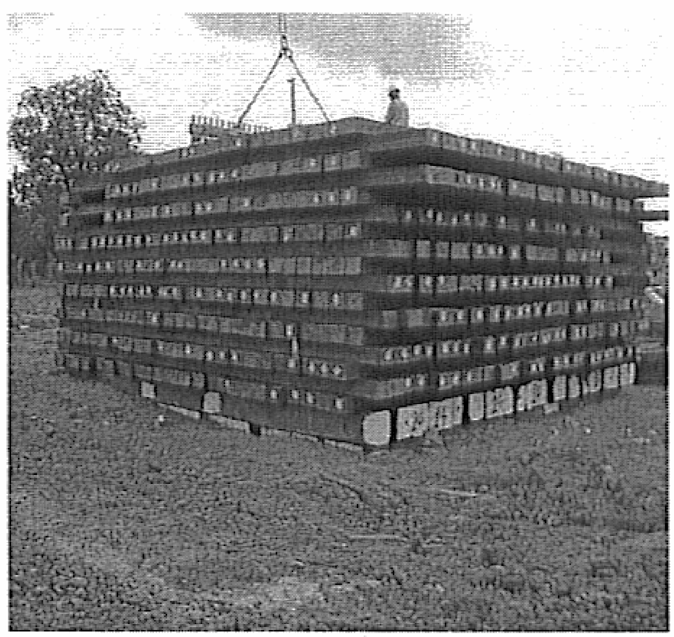

Figure 6. Cell A Fully Loaded with 1 million kilograms of Steel (177kN/m^2)

DATA ANALYSIS

Cell B (12% Replacement Ratio). Piezometers were installed in the organic layer between columns as shown in Figure 7. The data logger continuously recorded the pore water pressure increase corresponding to stage loading, Figure 8. Piezometric data along with load cell data provided the basis for conclusions reached regarding the soil/column load distribution. Figure 8 shows a drop in pore pressure immediately after the trench was cut, probably resulting from lateral expansion of the foundation beneath the cell footprint. As groundwater began to fill the lower portion of the trench, pore pressure rebounded slightly, and then responded predictably to placement of the mats. A similar (but lesser) drop in pore pressure was measured when a shallower trench was excavated for Cell A. As the geotextile strips were installed and the Cell B was backfilled (12 and 13 May), there was predictable response from the piezometers.

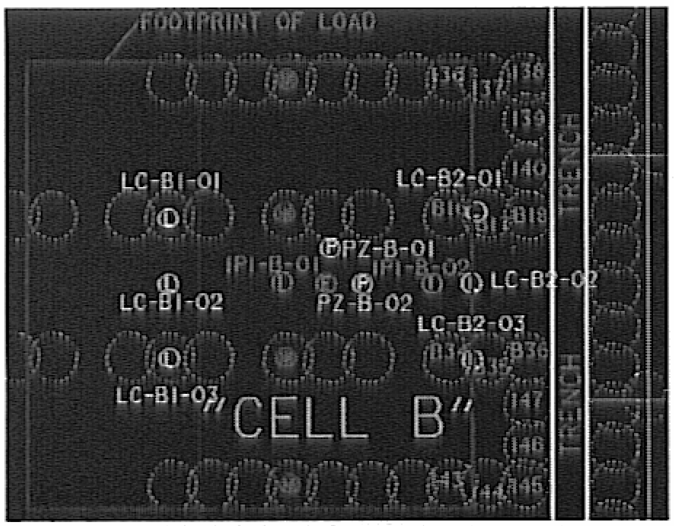

Figure 7. Cell B Instrumentation

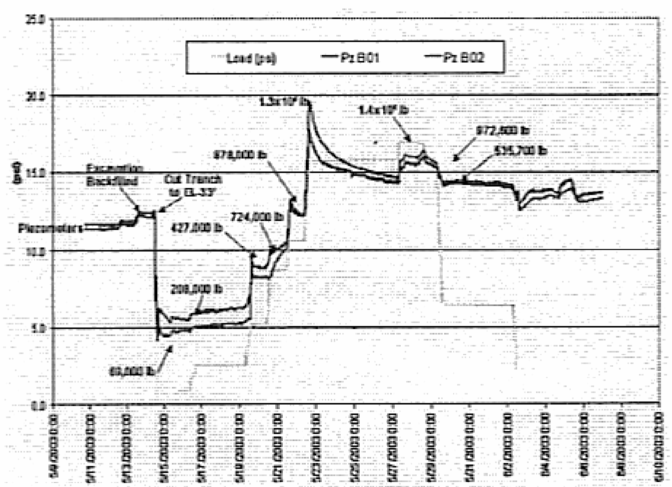

Figure 8 Cell B Piezometer readings

As steel was placed on the cell, the piezometric level rose correspondingly. The pore pressure rose slightly over the weekend after 94,545 kg had been placed on the cell. With every subsequent loading event (average of 109,000 kg per day), except after the 20 May 2003 loading event, the piezometric level was observed to dissipate overnight until loading started again the next morning. Pore pressures during unloading reflected the reduction in load, and the residual piezometric level was recorded as 0.6m higher than the initial reading.

The average responses from the load cells beneath Cell B are shown in Figure 9. Noticeable responses were recorded when the Cell was backfilled on 13 May and when the trench was cut on 14 May. Load cell responses are more clearly shown in Figure 10, showing the response of the six load cells under Cell B in the time frame of the trench cutting.

Figure 10 shows that the two load cells over columns next to the trench (LC B2 01 and LC B2 03, the two upper lines, up triangle and down triangle) indicate a marked increase in load transfer when the trench was cut to a depth of 5.2m between 1300 and 1340 hours and again when the trench was cut to a depth of 10m between 1540 and 1630.

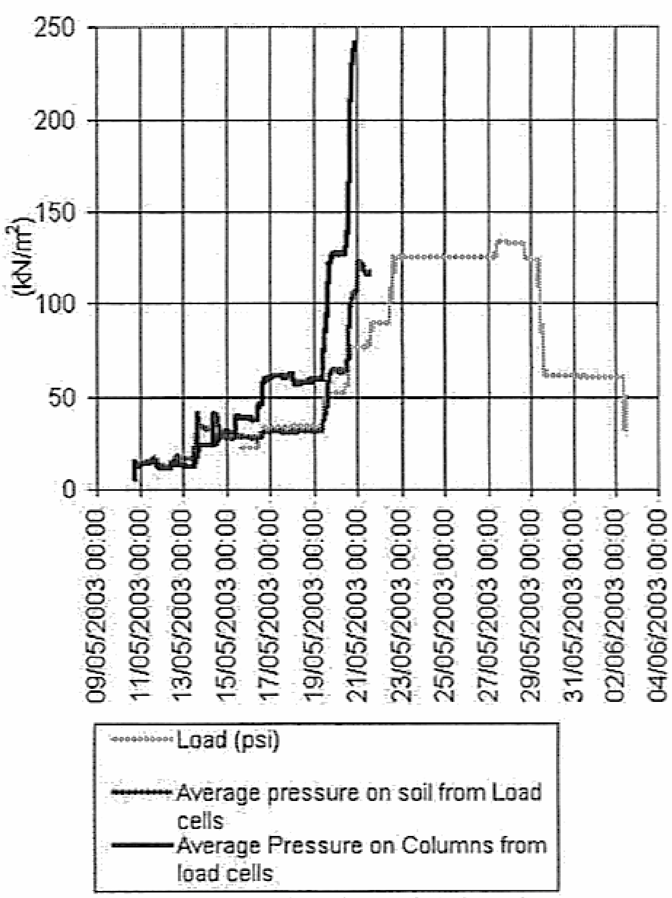

Figure 9 Load response for Cell B

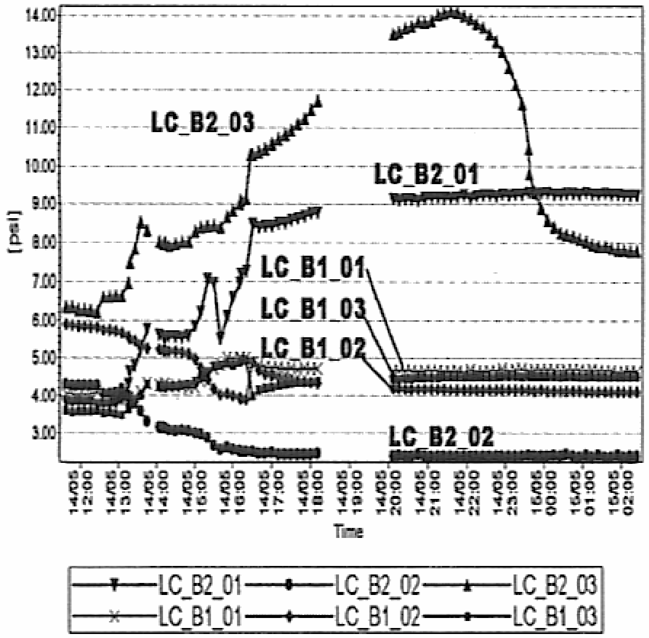

Figure 10 Cell B, Load Cell Data, 14 May 2003 (Trench cutting)

The column load cells to the rear of the cell (LC B1 01and LC B1 03, central lines, X and square) show less of an increase in load during trench cutting due to greater distance from the trench. It is interesting to note that during the cut to a depth of 10m, load cell (LC B1 03, denoted by square) readings jumped down as reading from the front two column load cells and rear soil load cell jumped up. The readings from the load cell over the soil near the trench (LC B2 02, lower line, circle) decreased during trench cutting, as did the load cell over the soil to the rear of the cell (LC B1 02, denoted by diamond) except when it picked up load with the front columns when some columns tipped toward the trench. The load cell data indicate that twice as much of the applied load was transferred to the columns as was transferred to the soil , a ratio that increased with increased load application until the load cells were lost before final loading.

Two slope inclinometers were installed between the columns to measure soil movement versus depth from which the angle of inclination of the failure wedge could be estimated. The 20m deep inclinometers were installed 10m below the bottoms of the columns with Inclinometer B2 situated 2m from the trench and Inclinometer B1 placed 4.3m away.

The inclinometer readings are presented in Figure 11 give insight into the soil column interaction. The points of inflection immediately after trench cutting occurred at an approximate depth of 9m, indicating that the entire soil/column matrix simply translated toward the trench.

This observation is logical since the dead weight of the soil would be transferred uniformly along the full length of the columns. However, when the steel dead load was subsequently applied, the points of inflection moved upward to depths of 4m for B-1 (solid lines) and 5m for B-2 (dashed lines). This scribes a failure arc or plane having an angle of failure of 30° from the horizontal with

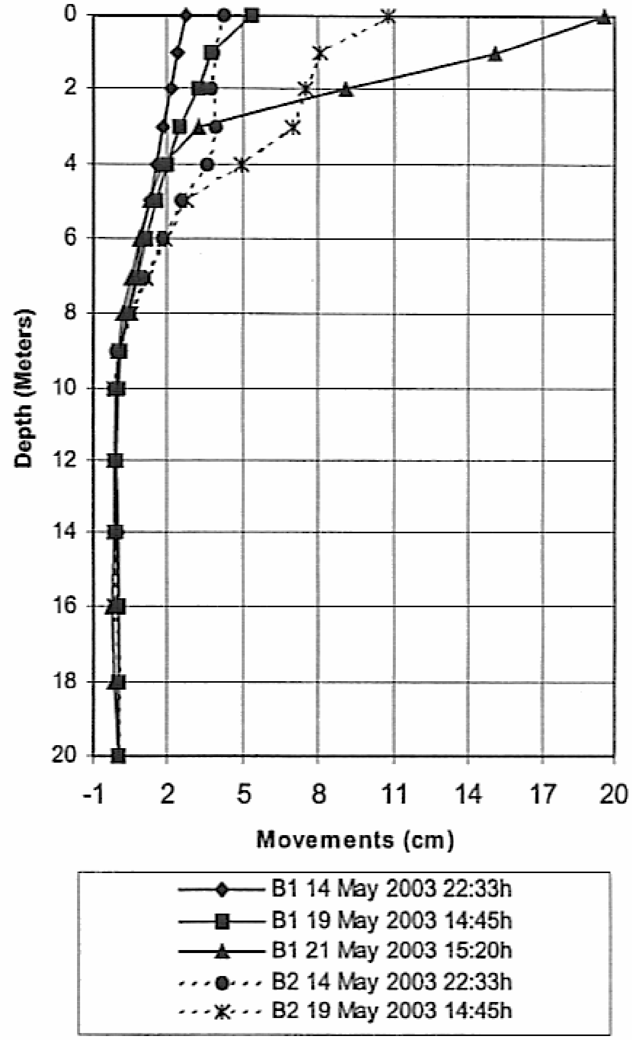

Figure 11 Cell B - Inclinometer Data

failure occurring in the organic deposit.

The authors believe that a complex load transfer occurred in the soil/column matrix. After the trench was cut, the total weight of the nearly saturated soil was transferred uniformly throughout the length of the columns. The applied steel dead load was transferred initially to the upper zone of the columns – in the organic deposit and above, where the columns were their weakest. Load delivered directly to the soil consolidated the clay in the foundation, as supported by the decrease in pore pressure during static load periods, such as from 13-27 May, Figure 8. As the soil consolidated near the soil/column interface, load was transferred from the soil back to the columns, not unlike the effects of negative skin friction on end-bearing piles. Ultimately, the applied dead load migrated down to the weakest zone where the system failed – in the organic layer between 4.2 and 6 meters of depth. If this truly occurred, the 2:1 column:soil load ratio measured by the Cell B load cells was only applicable to the upper zone of the columns and would be misleading if applied to the columns for their full length. At greater depth, such as near the apparent failure zone of 4 to 5 meters of depth, the load distribution may have been much greater, and conversely, below this zone, the ratio may have been closer to 1:1.

Coring a completed column and installing load cells at various depths to measure load transfer with depth could further investigate this theory of load transfer between the columns and soil.

Cell A (20 % Replacement Ratio Cell). In all, 1,001,965 kilograms (183.3 kN/m^2) of dead load was applied to Cell A, an amount that represented over 170% of the projected load from the earthen levee. As observed for Cell B, pore pressures in the soil dropped as a result of stress release during trench cutting, Figure 12, although not as significantly as for Cell A since the trench was cut only to a depth of 5.2m and the replacement ratio was higher for Cell A. Again, as for Cell B, the pore pressure in the soil increased with increases in applied load and dissipated during static loading between 29 May and 3 June, Figure 13, indicating consolidation in the clay layers.

Load cell data were available throughout loading and unloading of Cell A, Figure 14, where the columns supported half of the 1,001,965 kilograms of applied load. Figure 15 shows that the ratio increased with increased load to a maximum of 5:1 and retraced its ramp up during unloading. During unloading, the ratio was predictably higher than during loading due to the relatively larger modulus of the columns over the soil.

Like Cell B, two 20m deep slope inclinometers were installed between the columns to measure soil movement versus depth from which the angle of inclination of the failure wedge could be estimated. Inclinometer A2 was situated 2.1m from the trench and Inclinometer A1 was located 4.3m away. As the steel dead load was applied, the inclinometer points of inflection shifted from lower to higher depths 10m to 4m depth for A-1 (solid lines) and 3m for A-2 (dashed lines). Inclinometer data is presented in Figure 16. Like Cell B, this indicates a failure arc or plane having an angle of failure of 30° from the horizontal with failure occurring in the organic deposit. Only one

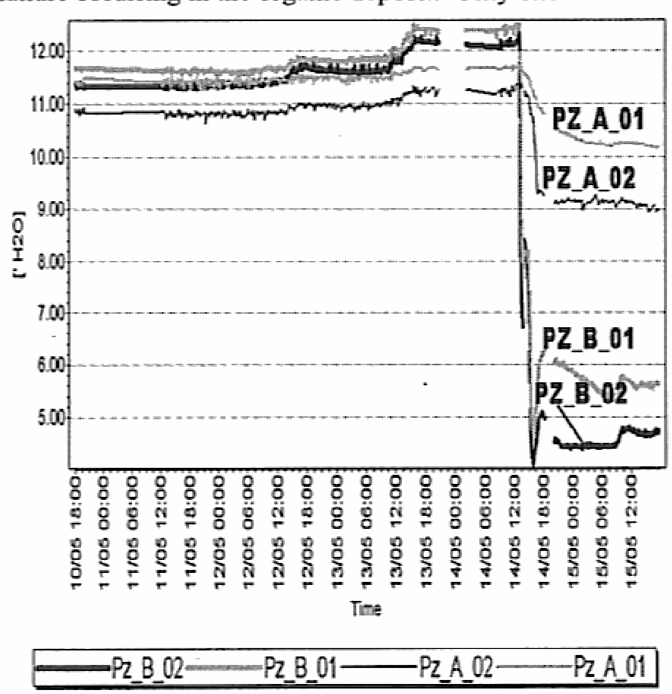

Figure 12 All piezometric data 10 May to 15 May 2003

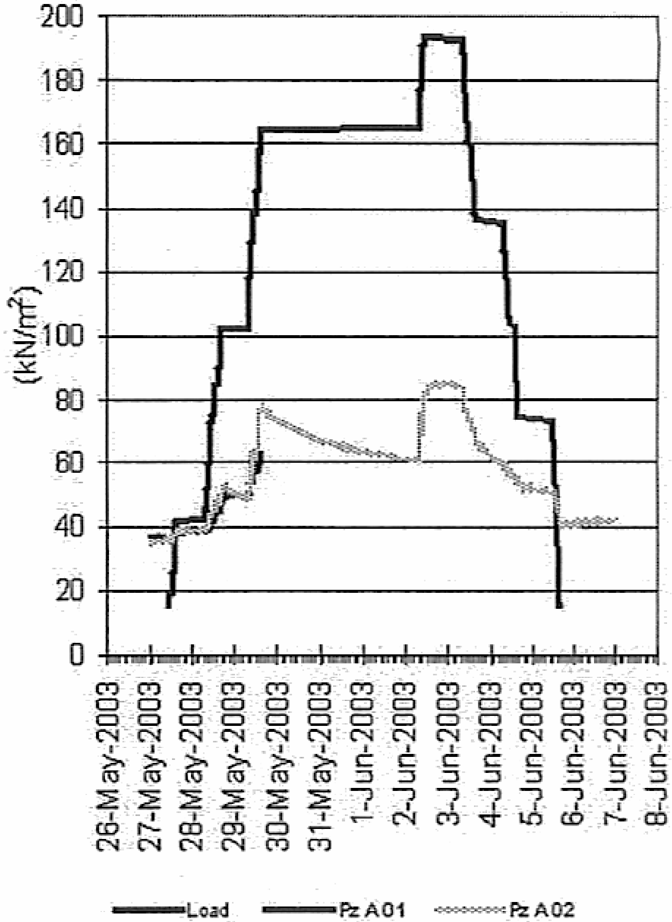

Figure 13 Piezometric data during loading and unloading of Cell A (20% Replacement Ratio)

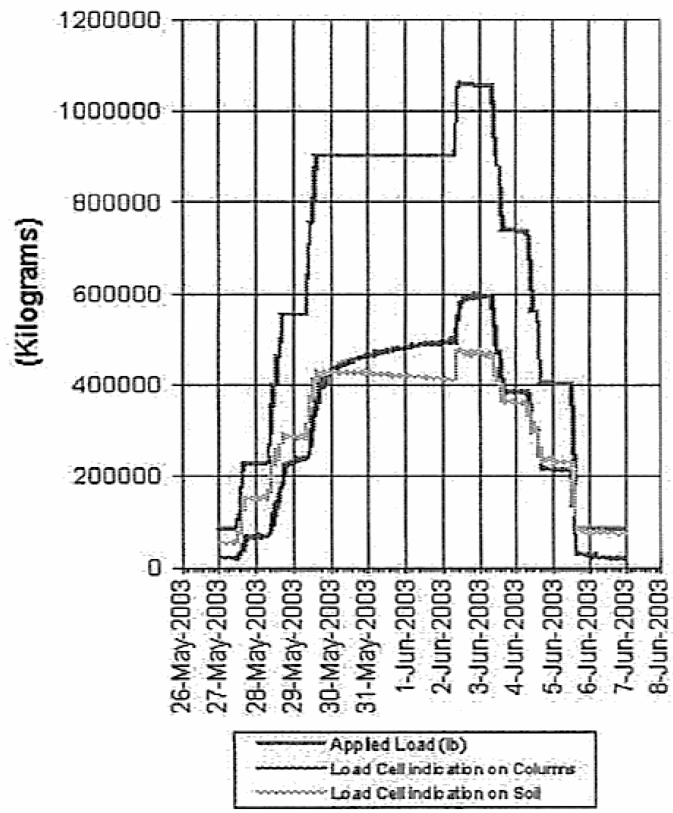

Figure 14 – Cell A – Load Cell Data

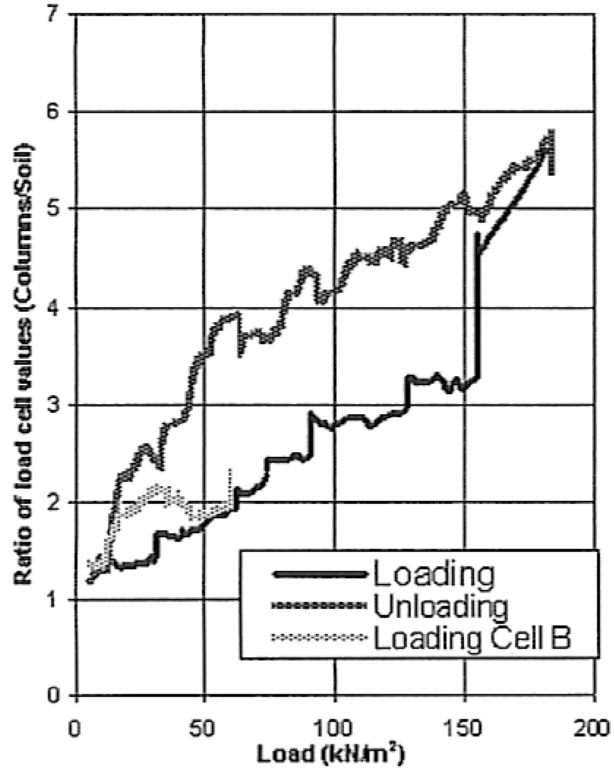

Figure 15 Cell A – Ratio of load cell values (Columns/Soil) vs. Load

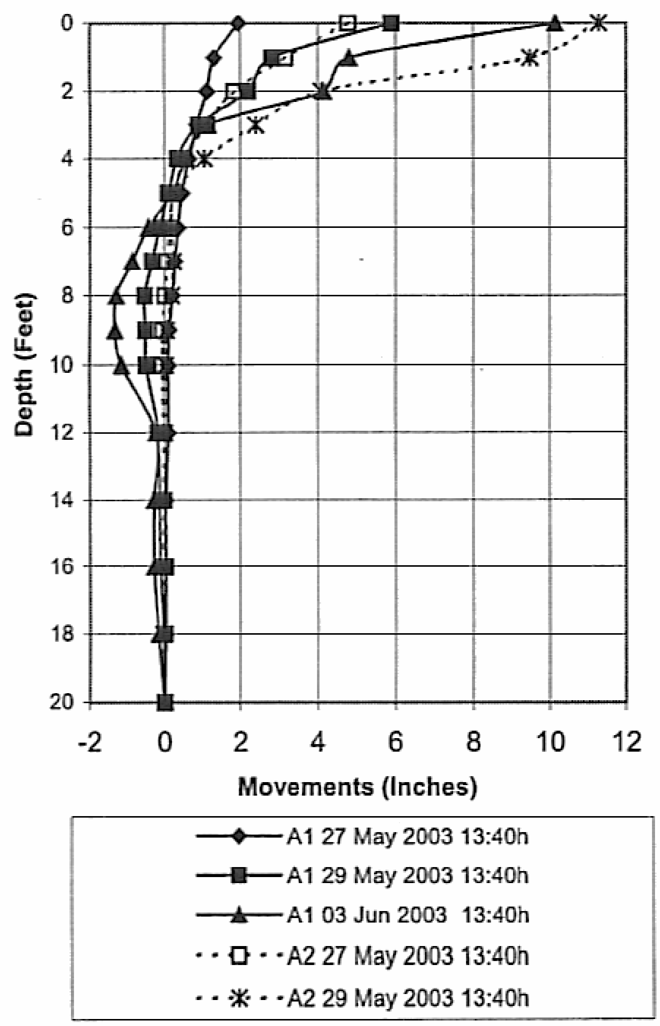

Figure 16 Cell A Inclnometer Data

inclinometer, which was installed into a column, could be read. It was located 4.3m from the trench face and just outside of Cell A. The measurements, shown in Figure 17, indicate bending in a similar mode as those in the soil between the columns, but without a distinct inflection point.

Cell C (zero replacement cell). A control cell was constructed with no reinforcement to assist in the evaluation of the data and loaded with 347,748 kg (63.6 kN/m^2). An inclinometer located 2.1m from the trench recorded lateral movement, but no piezometers or load cells were available for this phase. As for Cells A and B, a trench was excavated to release pore pressure in the soil, but was then loosely backfilled to prevent the trench sidewalls of the unreinforced cell from caving. In Figure 18, the initial reading before any actions were taken would be a vertical line at zero; the 7:35 am reading was after the trench was cut; the 8:05 am reading was after trench was backfilled; and the 9:06am reading was after the timber mats were placed. The next 5 reading were taken after each layer of steel was placed. The inclinometer was read on the next morning (6 June) and again on 9 June, after three days of static load. Failure occurred at a depth of 2.7 m at a distance of 2.1 m from the face of the trench. With only data point, an estimation of the failure arc or plane was not possible.

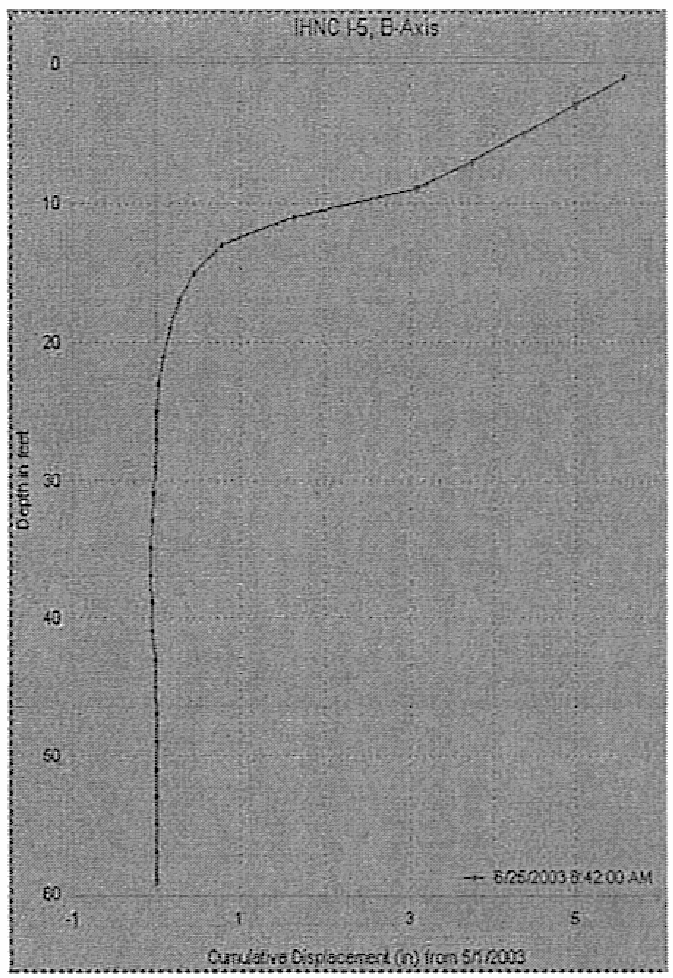

Figure 17 Manual Inclinometer Reading from Inclinometer within column

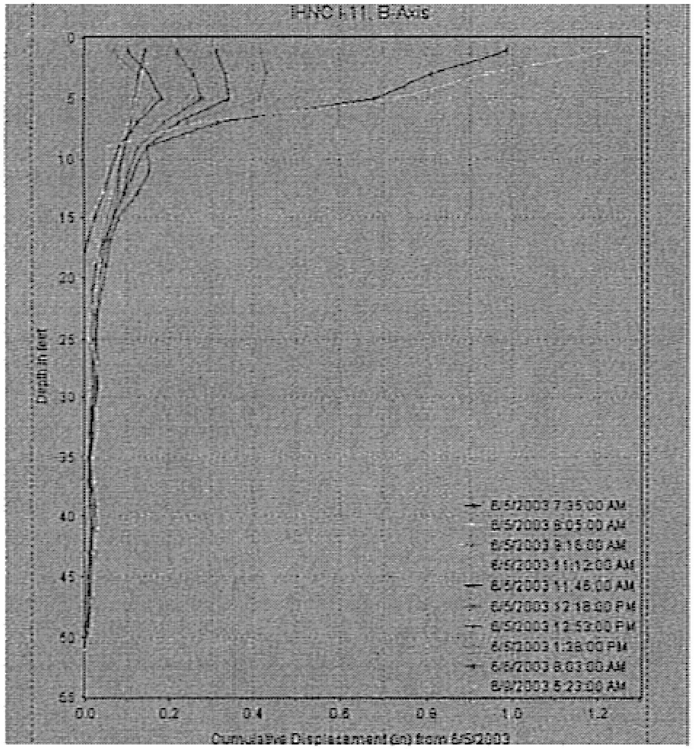

Figure 18 Inclinometer reading from Cell C (0% replacement cell)

MODE OF FAILURE

While the failure surfaces for Cells A and B were delineated reasonably clearly by the slope inclinometers, Figure 19, the mode of failure was not so clear. The Method of Planes was used to best represent the angularity of the failure plane, although an arc could be scribed through the inclinometer inflection points and could be argued as representative. Based on the pressuremeter tests conducted on Columns 1, 2, and 4, the average column shear strength within the failure zone was approximately 310 kN/m^2 according to the cohesive model but 420 kN/m^2 according to the ASTM model. To estimate the applied load at system failure, the cohesive model data were averaged and combined with the UU soil test data to create a composite strength and overall resistance to failure along the active wedge. Subtraction of the driving force of the soil column matrix from the available resistance force gave the necessary applied force to result in a factor of safety of 1.0. For Cell B, where 117 kN/m^2 was placed on the section, the system failed at the load equivalent of 74-kN/m^2 surface pressure using the cohesive model column strengths and 106 kN/m^2 using the ASTM. For Cell A, 184 kN/m^2 was applied, and the surface pressures at failure were 107 kN/m^2 using the cohesive model and 154 kN/m^2 using the ASTM model. Extrapolating these calculations for the 30% replacement estimated for the levee design and originally planned for the test section, the failure load could be predicted. Using the more conservative cohesive model, i.e., average column strength of 310 kN/m^2, the applied load at failure would equate to 164 kN/m^2 for Cell B and 183 kN/m^2 for Cell A. It is likely that failure of a 30% replacement test section would not have been realized using the available steel dead load. By substituting the average column strength of 310 kN/m^2 indicated by the cohesive model into the levee slope stability analysis, the soil/column matrix would have a composite strength of 102 kN/m^2, and the factor of safety would be 1.36.

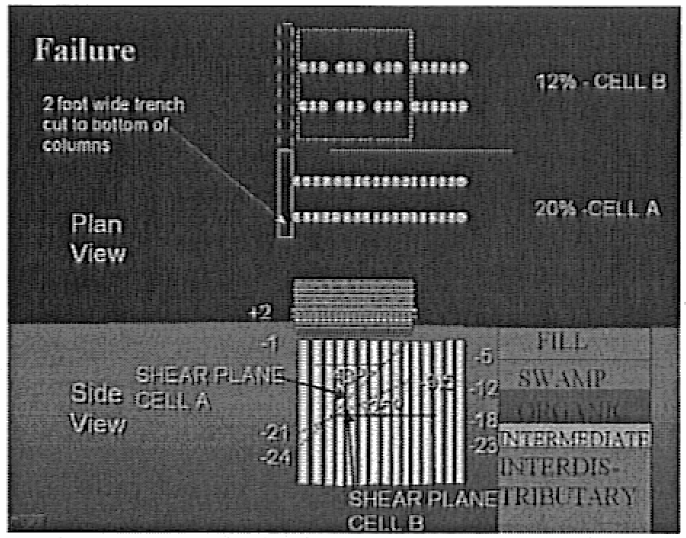

Figure 19 Failure Surfaces

OBSERVATIONS

Design and Analysis

Columns were not homogeneous and mixing was not uniform. Cement was concentrated in spiral bands. There was little bond between overlapped columns within panels.
The column:soil load distribution ratio increased with increased applied load, and retraced this pattern during unloading of Cell A. It averaged 2:1 for Cell B (12% replacement) and 5:1 for Cell A (20% replacement) at failure load.

Mix Design

The upper 1.4 to 1.8 m of the soil at the IHNC site was too dry to create a competent column, and introduction of water may be necessary for any production columns.
Organic clay layers required 200 kg/m^3 cement to produce columns having a shear strength of 276 kN/m^2. Double mixing in the organic layer produced stronger columns than single mixing.
Lime cement columns found to be weaker than 100% cement columns for equal binder dose, especially in the organic layer

QA/QC

Cone penetrometer testing worked well the first day of column life. Pressuremeter testing yielded more reliable data than did RCPT, CPT, or coring with laboratory testing.
Coring with a triple tube sampler did not produce samples adequate for testing; using a pitcher sampler produced better quality samples.
The unconsolidated-undrained triaxial tests conducted on the 76mm diameter cored samples yielded lower shear