tics of the soil and those after the load test. The observation of the limit pressures shows three different zones in the treated ground:

1. From 0-30 ft (9 m) very high densification with average limit pressures in excess of 16 tons/ft² (1.6 MPa)
2. From 30-37 ft (11 m) slightly lower results with average around 10.5 tons/ft² (1.0 MPa), and
3. Below the water table 37 ft (11 m) down, a regain of the characteristics with limit pressure around 15 tons/ft² (1.5 MPa).

Tests immediately after treatment and two weeks after treatment showed that the characteristics are improving with time (Figure 5). This is explained by the very fine composition of this material and, therefore, the time required for the water pressure to dissipate.

The analysis of results obtained after treatment enabled the determination of the parameters that govern the relation between energy and depth. The equation used is

$$H = cd\sqrt{E} \tag{1}$$

where

H = depth of influence of the treatment (m);
c = coefficient depending on soil conditions;
d = coefficient determined as follows, for free drop $d = 1$ or for crane drop $d = 0.9$; and
E = energy per blow (Mg•m).

For the very high densification zones described above, c is 0.6. For the lower densification below that zone, c is 1.

By using all these parameters, a 40-ton pounder can be calculated to highly densify 60 ft (18 m) and collapse 100 ft (30 m) of mestastable soil. These numbers for a 150-ton pounder would be 107 ft (32 m) and 173 ft (52 m), respectively.

CONCLUSIONS

The extensive efforts that were put into the design, treatment, and control of this test section have largely improved the knowledge and understanding of the behavior of mine spoils. The pressuremeter tests were found to be a reliable and practical method of evaluating the engineering characteristics of this material, which are usually difficult to assess due to their heterogeneities.

The Dynamic Consolidation treatment improved this very fine and heterogeneous material dramatically. Depending on the type of construction and depth of the mine spoil, different machines with different energies per drop can be used. Densification of soils 30- to 100-ft deep can be achieved. Postconstruction total and different settlements are largely reduced to tolerable levels.

A Dynamic Consolidation treatment on mine spoils can be successfully completed for a fraction of the cost of other conventional techniques, such as overexcavation and backfill or piles.

Wick Drains, Membrane Reinforcement, and Lightweight Fill for Embankment Construction at Dumbarton

JOSEPH B. HANNON AND THOMAS J. WALSH

The use of special features to permit embankment construction over soft bay mud is reported. These features included reinforcing fabric, lightweight fill (sawdust), and vertical wick drains, a system that allows construction to proceed on schedule without major foundation failures. An instrumented test embankment that incorporates these features was constructed by the California Department of Transportation (Caltrans) in 1979 at the bridge head of the east approach to the new Dumbarton Bridge. The successful performance of the test embankment provided data for developing specifications for construction of the 2.4-mile embankment contract across the soft bay mud deposits. The fabric provided initial support over the bay mud, lightweight fill reduced loading to ensure ultimate stability, and vertical wick drains accelerated foundation consolidation, which allowed up to 7 ft of settlement to occur in 1 year as opposed to about 50 years under the same loading with normal drainage conditions. Polyvinyl chloride (PVC) wick drains were found to be 40-50 percent as efficient as Alidrains in accelerating consolidation. Their efficiency increased with greater hydrostatic pressures. Instrumentation monitored and controlled the rate of embankment placement. These special features were successful in maintaining stability during construction that would not have been possible by the use of conventional construction techniques.

Embankment placement over soft compressible foundation soils has caused many perplexing problems for the transportation engineer, both during construction and in long-term pavement maintenance. The selection of a particular foundation treatment to aid construction and reduce future maintenance depends on the extent and character of the soft foundation deposits. (i.e., Can they be removed and replaced with more competent material or improved by consolidation and resultant strength increase?)

The soft muds in the San Francisco Bay area have presented a challenge to California Department of Transportation (Caltrans) engineers on a number of occasions. The most recent challenge is the embankment construction for the approaches to the new Dumbarton Bridge. The existing low-level lift span bridge carries traffic for CA-84 across the southern portion of San Francisco Bay between Alameda and San Mateo Counties, in Caltrans district 4. The new 1.6-mile long, high-level bridge structure, which is under construction, should be open to traffic through detour connections in 1982. The approach embankments to this facility are also under construction. The east approach embankment extends from the bridgehead across the Newark salt ponds to Thornton Avenue, a distance of 2.4 miles. The west approach embankment construction covers a distance of 1.6 miles (Figure 1).

This paper discusses the construction and performance of a test fill, the east approach embankment construction, and the development of specifications for special features in embankment construction used

Figure 1. Project location.

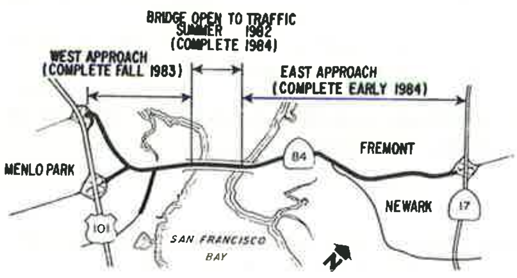
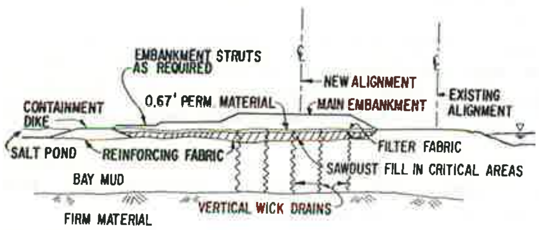


Figure 2. Cross section 04-ALA-84-PM 0.7/3.1.



to successfully execute the work within the contract schedule.

BACKGROUND AND ANALYSIS

The salt ponds through which the east approach alignment traverses lie over former marshlands. The average depth of water in the salt ponds is about 2.5 ft. The entire area is underlain by San Francisco Bay mud, which ranges in thickness from 20 ft near the toll plaza to 40 ft at the bridgehead. This material has an undrained shear strength of less than 100 $lb\cdot f/ft^2$ near the surface and increases to more than 300 $lb\cdot f/ft^2$ at greater depths. The proposed embankment, including surcharge, will reach a height of approximately 14-16 ft above mean sea level during construction. The bay mud foundation will not provide sufficient support without special foundation treatment and controlled rates of embankment loading.

As originally conceived, the approach roadway to the new bridge was to be built by conventional methods by allowing about four years of settlement time prior to pavement placement. However, local demands dictated that the bridge construction be undertaken first, followed by the approaches at intervals of one to two years. These time constraints and the need for a planned roadway profile at elevation +10 presented new stability and settlement problems.

To accomplish the embankment construction within the contract time, several features were incorporated that have had minimal prior use in the United States:

1. Reinforcing fabric to distribute embankment loading and prevent failure of soft foundation soil during construction and reduce differential settlement,
2. Lightweight fill (sawdust) to reduce total embankment loading,
3. Vertical wick drains to accelerate the consolidation process to less than one year, and
4. Filter fabric to prevent contamination of a permeable drain blanket for removal of pore water as part of the consolidation process (Figure 2).

Stability analyses of critical cross sections revealed only marginal to failure conditions by using conventional fill placement. The above features were needed to preclude multiple shear failures and the development of an extensive mud wave. Shear failure, long-term differential settlement, and inordinate maintenance requirements would result from the use of conventional construction techniques. The maximum heights attainable with conventional fill, based on previous Caltrans experience, would also be limited to about 8 ft. Laboratory studies indicated that normal consolidation would also require up to 50 years for ultimate settlement of about 6 ft to occur in a 40-ft depth of bay mud. Reinforcing fabrics have been successfully employed by Haliburton (1) and others to bridge soft materials. The use of fabric at Dumbarton would provide the uniform support necessary over the soft bay mud during the early stages of embankment placement. From computer analyses we estimated that the fabric would add about 10 percent to the overall stability of the embankment foundation system through additional unit cohesion provided by fabric tensile strength.

The sawdust would be incorporated in the main embankment as lightweight fill below elevation +5, which would keep it in a saturated state free from oxidation and, thus, deterioration. The experience of others (2,3) suggests that this condition, or a condition free from continuous wetting and drying within an embankment, will ensure long-term service life. After settlement of the bay mud foundation at Dumbarton, the sawdust would be submerged below the adjacent salt ponds.

The lightweight fill would be placed in areas of questionable stability and would require no compaction other than that provided by spreading equipment. Design parameters assumed for the sawdust were drained angle of internal friction ($\phi = 31°$), drained cohesion ($c = 0$), and wet unit weight ($\gamma = 40$ $lb\cdot f/ft^3$) based on experience (2). The conventional earth fill compacted wet density was specified at 145 $lb\cdot f/ft^3$ maximum.

Wicks or band-shaped drains were proposed for this project to accelerate consolidation and strength gain and to minimize postconstruction settlement. Wick drains are made of paper, plastic, or other synthetics and conduct water upward by excess hydrostatic pressure. They are about 4-in wide and vary in thickness from 1/16 to 1/4 in. With a wick spacing of 4 ft, we estimated that the settlement period could be reduced to less than a year.

Barron's method (4) was employed by assuming an equivalent sand drain diameter of 6 in for the wick drains based on work by Fellenius (5) and Hansbo (6). Values of site soil permeability were estimated from reference data and other project performance records. For design, the project was divided into units A, B, and C according to approximate depths of bay mud. Unit A had 40 ft of bay mud and required a wick spacing of 4 ft. Wick spacings of 5 ft for unit B with 30 ft of mud and 6 ft for unit C with 20 ft of mud were a compromise based on economics and similar postconstruction settlements in the three units.

TEST FILL

A 300-ft long test fill to a maximum height of 16.5 ft was constructed in 1979 at the east bridge abutment, prior to the main embankment contract. The test fill provided an evaluation of the applicability of the proposed special features prior to incorporation in the main contract. It was constructed by change order under the ongoing bridge contract.

Caltrans installed extensive instrumentation to monitor construction loading and prevent potential embankment failures. Instruments were located on two cross sections and at the corners of the embankment struts (buttress fills). The instrumentation consisted of settlement platforms, anchor postsettlement gauges, horizontal settlement profile gauges, piezometers, heave stakes, and inclinometers. In addition to the instrumentation, six test areas with different reinforcing fabrics were established.

Construction of the test fill was accomplished in the following sequence (see Figure 3):

1. Construct both longitudinal and cross-containment dikes by first placing reinforcing fabric across the salt ponds and end dumping and compacting earth fill to strut elevation +6,
2. Dewater area contained by dike and existing roadway embankment,
3. Place reinforcing fabric on bay mud for main embankment within containment area,
4. End dump lightweight fill (sawdust) and earth fill as required to establish a working table to elevation +5,
5. Place layer of filter fabric on working table,
6. Place permeable drainage layer,
7. Place layer of filter fabric on drainage layer,
8. Place 0.5-ft layer of earth fill,
9. Install wick drains,
10. Advance remaining fill at a controlled rate of 1-ft maximum/7 calender days to elevation +10 and 1-ft maximum/14 calender days for elevations above +10, and
11. Allow for settlement period whose exact length of time will be determined from instrumentation.

The test fill was constructed without serious problems and was considered a major success based on settlement performance. It also produced valuable information from which to develop specifications for the main east and west approach embankments.

The reinforcing fabrics placed under the test embankment included woven and nonwoven materials. A wide variation in the handling characteristics of these materials was noted. On the basis of strength, elongation, and handling characteristics, a woven reinforcing fabric of either polyester, nylon, or polypropylene was specified for the main approach embankment. Test requirements were a minimum grab strength of 200 lb, an elongation at failure of 35 percent maximum, and a minimum joint strength of 160 lb.

One-foot of earth cover was required over the sawdust for the main approach fill to prevent wind blown losses during long periods of exposure and to provide additional support. The first level of filter fabric was placed directly over this earth layer followed by the permeable drainage rock blanket.

For maximum efficiency of the wick drains, cutoff of the wicks was provided within the drainage blanket. Therefore, specifications were that approximately one-half thickness of the drain rock be placed, that the wicks be driven and cut off at this working table surface, and that the remaining portion of the drain rock then be placed.

The relief of the test fill drainage blanket by outletting at the toe of slope did not function as anticipated. In fact, the pore water that exuded up the wick drains from the bay mud foundation never appeared at the planned outlet although it was encountered in test holes at depths of 1-2 ft. This may be explained in part by the construction of the wick drains (Alidrains) that allowed pore water to exude at any vertical elevation, collect in the sawdust fill, and possibly escape along the plane of the reinforcing fabric.

The drainage relief plan for the main fill was changed to include a longitudinal underdrain system that would discharge into vertical riser pipe pumping stations at 500-ft intervals on centerline. The longitudinal collector used was a 1.5-in diameter slotted plastic pipe.

At the time of the test fill construction, only two wick drain materials were available in California: the polyvinyl chloride (PVC) wick, a Japanese product with no local installer, and the Alidrain wick, a product of Canada distributed in the United States through Vibroflotation Company of Pittsburgh. Vibroflotation was engaged in a wick drain project in southern California and so was selected by the contractor to perform the work for the test fill due to their availability. The wicks had to be driven through 1 ft of earth fill, 8 in-1 ft of drain rock, two layers of filter fabric, 3-5 ft of sawdust, a reinforcing fabric, and ±40 ft of bay mud to a total depth of 46 ft (see Figure 2).

The hydraulic powered wick driving machine, a Swedish design, was mounted on a backhoe. The installation proceeded smoothly with only normal delays and was completed in about three weeks. The test embankment was then placed to the planned height by using the controlled rates of loading previously described. The maximum fill height attained was 16.5 ft, including a surcharge.

Instrumentation was placed during the early stages of construction and was monitored continuously during embankment placement. The consolidation process accelerated immediately following installation of the wick drains in October 1979. Eleven feet of additional fill material was then placed above the wick level by using the controlled loading rates. By mid-February 1980, five months after the wick installation, nearly 100 percent of the primary settlement had occurred. Six months later the total recorded settlement was approximately 6.5 ft and the settlement curve was flattening (Figure 4).

As shown in Figure 4, which compares actual field settlement to the theoretical settlement curve with normal consolidation drainage, the wick drain performance met expectations. Figure 4 also shows a reasonably close comparison between predicted and actual settlement performance with wick drainage. The overall performance and appearance of the test fill was excellent. Two small cracks appeared in the first terrace of the embankment, which were probably the result of differential settlement between the main embankment and the outer strut. A small crack also appeared near the catch point between the new embankment slope and the existing roadway, which probably represented drawdown and rotation of the existing roadway.

The reinforcing fabric under the fill prevented the development of a mud wave in the soft bay mud foundation. High pore pressures were recorded during construction, which probably would have caused failure if reinforcing fabric had not been used. By contrast, an adjacent staging area constructed to elevation +6 by the end dumping method with no special treatments or controlled rates of loading produced a large mud wave.

EAST APPROACH EMBANKMENT

The east approach construction went to contract in June 1980. The original construction sequence, amended through experience with the test fill, was incorporated in its entirety. However, to facilitate the operation, the contractor was given the op-

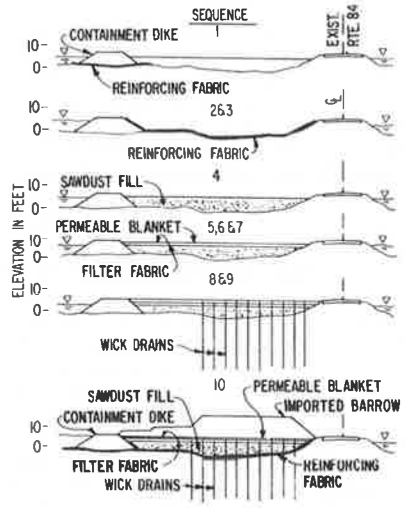


Figure 3. Construction sequence for test fill.

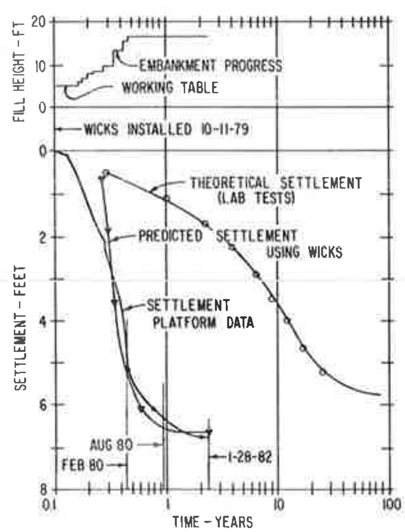


Figure 4. Actual versus predicted settlement for Dumbarton test fill.

tion of diking off subareas, which he elected to do (see Figure 3).

Reinforcing fabric was first placed by hand with the aid of a row boat across the open water, longitudinally with the dike. The salt ponds had conveniently been drawn down to about 1- to 2-ft depth, which aided placement. Imported borrow for the dikes was hauled in 10-wheelers, end dumped on the fabric, and pushed and advanced by D-6 dozers. The specified grading required a minimum of 20 percent passing the no. 200 sieve to ensure an impermeable dike.

Dewatering of the containment areas was accomplished by using 6-in trailer-mounted pumps supplemented by smaller suction pumps. Following dewatering of the subponds, reinforcing fabric was placed with stitched seams transverse to the main embankment. The woven Mirafi-500X reinforcing fabric was supplied by Celanese Corporation. It was floated on air in large sheets to cover the diked off subponds. Lead corners of the fabric were tied to light trucks that pulled the fabric over the dewatered ponds. One truck operated on the dike and the other from the shoulder of existing CA-84. This kite was about 200 ft wide and 800 ft long.

Complete evacuation of all the fluid muds near the surface was impractical. Consequently, much of this material was trapped as mud boils under the fabric. Loading and pushing of the fill materials on the fabric eventually forced the fluid mud out of the fill area, across the dike, and into the adjacent main pond. During this operation the fabric was able to contain mud boils up to several feet in thickness and a hundred or more feet on a front.

Earth fill for the embankment was hauled primarily in bottom dumps and pushed and spread by D-6 dozers. Sawdust for lightweight fill was trucked in from various saw mills in the Sierra foothills and northern logging counties. Sawdust was pushed and spread by D-6 dozers equipped with flotation tracks and side boards attached to the dozer blades. Drain rock for the permeable blanket was supplied locally and the nonwoven filter fabric was Mirafi 140S supplied by the Celanese Corporation.

The wick installation was done by Malcolm Drilling Company of South San Francisco. Malcolm elected to use the Japanese-manufactured PVC drain. For this project Malcolm built up two wick-drain-driving machines, which consisted of dual mandrels suspended from wide-tracked 70-ton cranes. Each pair of mandrels was driven by a single vibratory hammer powered off a generating package mounted at the rear of the cab. The mandrels could be adjusted to any drain field pattern.

The engineer's estimate called for 1 887 000 linear ft of vertical drains. On completion of the item approximately 95 percent of the estimated quantity had been installed. A portion of the remaining wicks was used through a change in contract order that required the contractor to drive additional wicks in an area where considerable amounts of fluid to very soft surface muds were encountered. The additional wicks were to provide early stabilization of the area and to the minimize damage potential to the adjacent KGO radio tower facility. This also provided wick spacings of 2.5, 4, and 5 ft for comparison within the same depth of bay mud (30 ft). The vertical drain work was done over a period of slightly more than four months, between October 1980 and February 1981.

The embankment was placed at the prescribed controlled rate to elevation +10 and excess pore pressures were monitored by piezometers in the 30- and 40-ft deep sections of bay mud. The contractor's embankment placement operation above elevation +10 required close control as a result of the slow dissipation of these excess pore pressures. Pore pressures were also monitored in the 20-ft section of bay mud, but only localized problems were experienced.

The waiting period required for dissipation of pore pressure exceeded eight weeks in some areas between placement of each additional 1-ft lift of embankment. The excess pore pressures in this case represented more than 50-60 percent of the total applied embankment load. A typical plot is shown in Figure 5.

Measured total settlement through November 1981 averaged about 3.5 ft in the 40-ft bay mud section. This represents about 55 percent of the predicted ultimate settlement in the deepest bay mud section. Figure 6 shows a comparison of settlement produced in the main east approach foundation by the PVC wicks and that produced in the test fill foundation by the Alidrains. The PVC wicks are not functioning as efficiently as the Alidrains; however, the rate of consolidation is accelerated somewhat with the PVC wicks driven on closer-than-normal spacings. This is illustrated in Figure 7, which compares field settlement with PVC wicks on spacings of 2.5, 4, and 5 ft.

Even though pore pressures were critical, with most of the embankment load being carried by pore water, no failures occurred in the 2.4-mile east approach embankment. However, one area did exhibit evidence of potential failure. A slope indicator located beyond the containment dike measured about 2 in of lateral movement. The outer portion of the main fill in the same area also subsided up to 1 ft over a distance of about 200 ft. No other signs of distress were observed other than settlement cracking. Failure would have occurred in this area and possibly other areas had the reinforcing fabric not been used.

Figure 5. Typical time plot for embankment progress, pore pressure, and settlement response in critical PVC wick drain area.

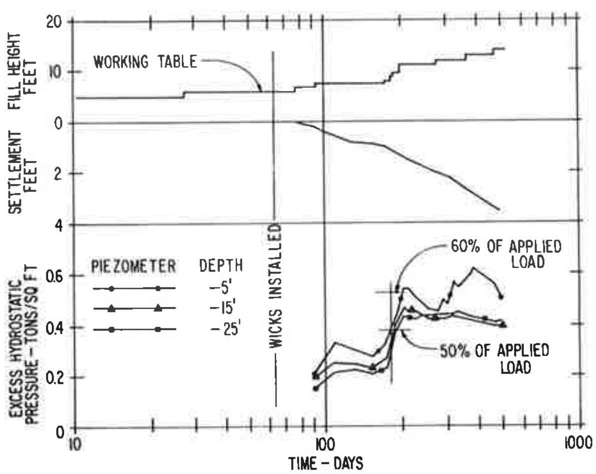


Figure 6. Settlement comparison, PVC wick drains versus alidrains.

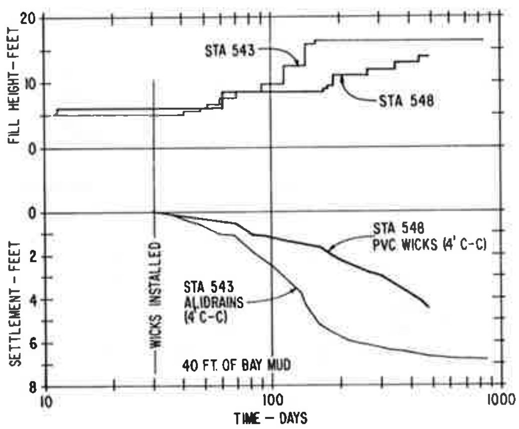
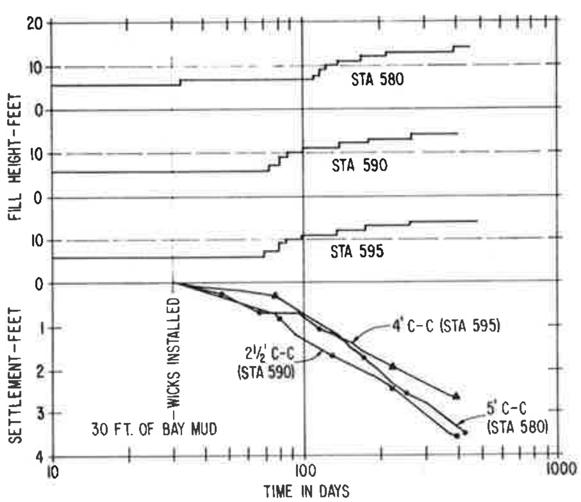


Figure 7. Settlement with PVC wicks on three spacings.



A determination of strength gain of the underlying bay mud foundation was accomplished by in situ vane shear tests. The results suggested an average shear strength gain of 0.1 tons/ft² (200 lb•f/ft²) after 40-50 percent consolidation in the 40-ft bay mud section (see Figure 8). Based on these results and some dissipation of excess pore pressure, a decision was made to resume embankment loading.

The PVC wick drains were observed to function more efficiently with additional embankment loading and greater hydrostatic pressure, which was consistent with laboratory wick drain performance test results. The problem of slow pore pressure dissipation minimized with the additional loading. However, waiting periods in excess of two weeks were still required for each additional 1-ft embankment lift. The east approach embankment was completed in January 1982.

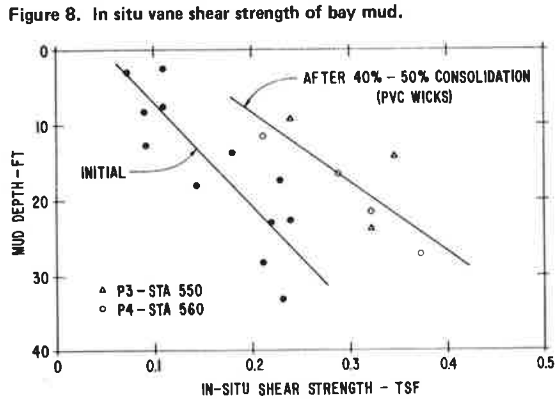
SUMMARY

This project provided the opportunity to introduce and evaluate various special features for construction of embankments over soft compressible bay mud deposits. The embankment construction was termed a major success because all previous construction experiences over bay muds were plagued with difficulty and embankment failures. These features comprised a system that, when carefully applied and controlled, resulted in successful construction. The system solved the problem of regulated construction approach, short-term stability, and long-term consolidation.

The reinforcing fabrics successfully bridged across the soft compressible bay mud foundation and provided adequate initial support for embankment construction. During the loading process, the reinforcing fabric held the fill together and prevented failure during critically high pore pressure conditions. Appropriate fabric specifications are now available as a result of this project.

Sawdust proved effective as lightweight fill and served to reduce both the initial and ultimate loading on the bay mud foundation and provided satisfactory support for construction equipment. Wick drains were effective in accelerating consolidation and proved to be an economical alternative to sand drains. The Alidrains in the test fill reduced the settlement period from 50 years under normal consolidation drainage to less than 1 year. The PVC drains in the main east approach fill were found to be 40-50 percent as efficient as the Alidrains in accelerating consolidation. The efficiency of the

Figure 8. In situ vane shear strength of bay mud.



wick drains increased with greater hydrostatic pressure due to embankment loading.

This project introduced the use of wick drains in California and several wicks are now available. Their application on other projects is proposed.

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Determining Maximum Void Ratio of Uniform Cohesionless Soils

JOHN E. WALTER, WILLIAM H. HIGHTER, AND ROBERT P. VALLEE

A testing program was conducted to evaluate several methods of determining the maximum void ratio of cohesionless soils. Preliminary test results indicated that a new procedure called the tube method was consistent in attaining reliable maximum void ratios. In performing the tube method, a long narrow tube or cylinder opened at both ends is placed upright in a mold of known volume. A quantity of dry sand sufficient to fill the mold is placed in the tube and then the tube is slowly extracted to allow sand to trickle into the mold until the mold overflows. The sand is then screeded level with the top of the mold and the void ratio is calculated from measured masses and volumes and the specific gravity of the sand. To evaluate various methods of determining maximum void ratios, a test series was carried out by using eight different test procedures on four sands. Statistical analyses of the data indicated that the tube method yielded higher values of maximum void ratio than did the other procedures. In addition, a testing program involving nine inexperienced operators demonstrated that by using the tube method an individual operator was able to reproduce results consistently and the operators were able to replicate one another's results well within limits mandated by practical applications.

Relative density (density index) is used to describe the state of compactness of cohesionless soils as a function of the loosest and densest states that the soil can attain. Knowledge of the density index (I_D) can give engineers valuable insight into the engineering behavior of a soil. However, since a particular soil can have different fabrics or arrangements of particles at the same void ratio, additional descriptors are required to characterize the soil.

The density index or relative density (D_r) of a soil at void ratio e and dry weight γ_d is defined in terms of void ratio as

$$I_D = D_r = [(e_{max} - e)/(e_{max} - e_{min})] \times 100\% \quad (1a)$$

and in terms of corresponding dry unit weight as

$$I_D = D_r = [\gamma_{d_{max}}(\gamma_d - \gamma_{d_{min}})/\gamma_d(\gamma_{d_{max}} - \gamma_{d_{min}})] \times 100\% \quad (1b)$$

where the subscripts refer to maximum and minimum states.

Dry unit weights or void ratios of the soil in the densest and loosest states are determined by laboratory tests. Errors in determining these values lead to significant errors in the estimation of the density index (1) and can mislead the engineer in an assessment of the likely in situ behavior of the soil under service loads. The determination of the loosest state of soil compactness has been particularly troublesome. The research reported here focuses on the procedures used in the determination of maximum void ratio for clean, medium to fine, uniform sands.

The choice of sands to use in the testing program was influenced by the findings of previous investigators. Burmister (2,3) reported the effects of particle shape, size, and gradation on limiting void ratios. Kolbuszewski (4) also studied parameters that controlled limiting void ratios in granular soils and found that particle shape and pluviation height have a strong influence on maximum void ratio. Youd (5) found particle size to have minimal effect on maximum void ratio. Studies by Dickin (6) and Norris (7) found particle shape to be the most influential soil factor in controlling maximum void ratio. Youd (5) prepared a plot from which maximum void ratio can be estimated from gradational and particle shape characteristics.

PROPERTIES OF SOILS TESTED

The four soils used in the investigation were medium to fine sands. The selection was based on particle shape, size, and gradation characteristics. Each sand contained less than 5 percent of particles finer than the no. 200 sieve and no more than 5 percent coarser than the no. 4 sieve. Because of the important effect of particle shape on limiting void ratio, a major criterion in selecting the soils was their shape characteristics.

Microphotographs of each candidate soil were obtained and, based on the method suggested by Wadell (8), the roundness of each soil was obtained. Wadell (8) defined roundness (ρ) as

$$\rho = (1/R)\left[(1/N)\sum_{i=1}^{N} r_i\right] \quad (2)$$

where the term within brackets is the arithmetic mean of the sum of N internal radii of the projected particle shape and R is the radius of the maximum