Foundation design in chaotic geomaterials: The H-3 project

Conception des fondations dans des sols cahotiques: le projet H-3

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ABSTRACT: The design of bored pile foundations for viaducts along the alignment of H-3, a major trans-island highway in Hawaii, required careful consideration of the extreme variability within the major unit of geomaterial into which the piles would be excavated A methodology is described for the design of foundations along one viaduct that included pooling all of the shear strength data along the alignment of the viaduct, defining design values based on the mean and standard deviation of the set of measured shear strength values (after discarding high values that could not be reliably counted upon in all piles), choosing design factors similar to those usually applied to mean values in less chaotic geomaterials, grooving the pile walls to increase the resistance in the presence of unknown soft seams, and verifying the design assumptions through load testing and deterministic modeling of the results of the loading tests.

RESUME: Le design des fondations de viaducs le long de l'alignement de H-3, route principale traversant Hawaii a nécessité une attention particulière et ce, due a l'extrême variabilité de la nature due sol dans lequel seraient excavées les piles. Une méthode concernant le design de fondations le long d'un viaduc est décrite et inclue la mise en commun de toutes les donnés de cisaillement le long de l'alignement du viaduc, la définition des valeurs de design basées sur la moyenne et l' ecart-type obtenus à partir de toutes les valeurs de cisaillement mesurées (après avoir écarté les valeurs élevées qui ne pouvaient pas être considérées fiables dans toutes les piles), le choix des facteurs de design semblables à ceux habituellement appliqués à des valeurs moyennes pour des sols moins chaotiques, le rainurage des côtés de la pile dans le but d'augmenter la résistance en présence de veines molles inconnues, ainsi que la vérification sous l'effet de charges des hypothèses de design, suivie d'une modélisation déterministe des résultats de ces essais de chargement.

1. INTRODUCTION

The foundation engineer is often faced with subsurface conditions that can only be described as chaotic, out of which design parameters must eventually be derived that are neither excessively conservative nor unsafe Such a condition existed for a series of viaducts along the alignment of a major highway project on the island of Oahu in Hawaii.

The highway, which carries the designation "H-3", traverses Oahu from near Pearl Harbor on the leeward side of the island to Kailua, on the windward side, as shown in Figure 1 The road ascends into the central highlands through rugged terrain in the Halawa Valley, which was eroded into a limb of an ancient shield volcano, and then carries traffic via twin tunnels through the bedrock of the Koolau Mountains, which once constituted the caldera wall of the volcano Emerging from the tunnels on the windward side, the highway descends to Kailua at sea level

H-3 is about 27 km long and required an expenditure of approximately 1,200 million $US, which made H-3 one of the most substantial public works projects in the United States during its period of construction, 1990 - 1997.

The concern of this paper is the foundations for the series of parallel viaducts leading up the Lower Halawa Valley toward the trans-Koolau tunnels The foundations for one viaduct consisting of parallel bridges, each 1 6 km long and denoted "Viaduct 1B," will be described

The valley has a narrow floor and steep sideslopes Figure 2 is a photograph along the alignment of the valley taken shortly after the contractor began to clear the vegetation. Cut-and-fill construction along the valley walls to accommodate the roadway would have required moving the Halawa Stream, which flowed down the valley floor, and burial of several archeological sites, both of which were unacceptable It would also have been costly, because the valley walls were unstable

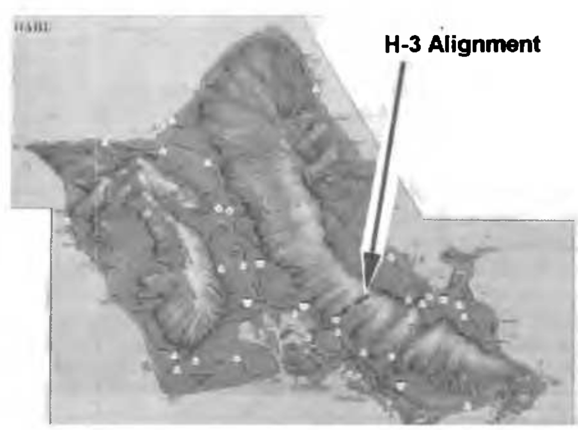

Figure 1 Location map for H-3

An important design constraint was that virtually all of the alluvium in the valley floor would be scoured away in extreme storms, requiring the use of deep foundations whose heads would be exposed after scour Up to 10 m of scour was forecast in some areas along the alignment of the viaducts. Simultaneous occurrence of seismic loading was also established as a design criterion Oahu lies in a seismic zone 3, and the design horizontal accelerations of the foundations were 0.17 g

The solution to the constraints was to construct a series of parallel viaducts, about 6 6 km in length in each direction, directly along the Halawa Stream to minimize disturbance to the landforms in the valley

Figure 2. Photograph along axis of Lower Halawa Valley

2. GEOMATERIALS

The geomaterials in the Lower Halawa Valley, idealized in Figure 3, consist of 5 to 11 meters of waterbearing alluvium and colluvium, overlying about 20 meters of saprolite and clinker (called here "saprolite"), which in turn overlies basalt of varying hardness. The intent of the design was to use groups of typically six bored piles to support each column of the viaducts that could easily penetrate the bouldery alluvium. In order to economize, the piles would terminate in the softer saprolite rather than in the harder basalt, whose surface elevation varied in an unpredictable manner. This made it imperative to quantify the properties of the saprolite as accurately as possible.

Compression loading from transverse seismic rocking of the viaducts was usually critical; however, in some bents uplift loading controlled the pile design. The pile diameters were fixed at 1.53 m. Since the geomaterials were porous and since construction was to be carried out under the level of the stream, boreholes would be concreted under water. Base resistance was therefore limited to a small value, so the piles perform essentially as friction piles.

The saprolite is a residual earth material that was differentially weathered from sequential lava flows with variable chemical and physical properties. The flows emanated from the caldera of the ancient shield volcano on the windward flank of the valley. Embedded within the saprolite are irregular deposits of clinker, which developed at the surfaces of aa flows. As a result of the weathering process, zones within the saprolite, which generally classifies as an ML according to the Unified Soil Classification System, had a consistency that varied from that of a medium stiff clay in highly weathered horizons to that of basalt in unweathered inclusions. A photograph of an exposure of the saprolite is shown in Figure 4.

The initial subsurface investigation program consisted of one wash boring at the location of each foundation. Samples were recovered where possible, and samples of appropriate length were subjected routinely to unconfined compression testing. Figure 5 shows the shear strength interpreted from the compression tests for all borings along a 1.4-kilometer-long path defining the locations of the central bents. The chaotic nature of the saprolite is evident. Spatial analysis of the data indicated almost no horizontal or vertical correlation of the data, with the following exception.

Two distinct stratigraphic zones were identified for design purposes. In the upper saprolite zone most of the samples exhibited compression strengths in the range of stiff clay, indicating relatively thorough weathering to the base of that unit. There were indications of harder inclusions, but these were infrequent and were assumed not to exist for purposes of design. In the lower saprolite zone, very few samples exhibited such low

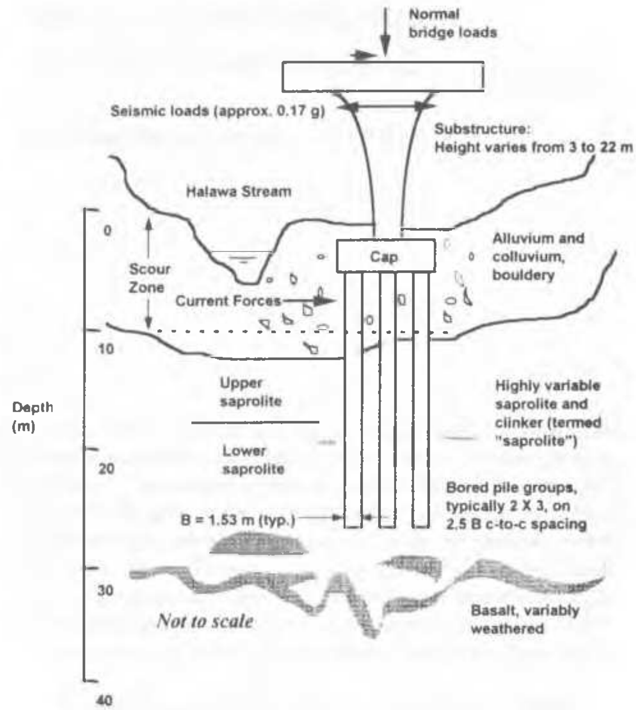

Figure 3. Foundation profile

Figure 4. Photograph of saprolite exposure in valley wall

values of compression strength, although the strengths were extremely variable. All tests with values of undrained shear strength in the lower saprolite exceeding 5 MPa were excluded from the population of data used to determine design values since such samples were irregularly distributed and probably represented thin, floating lenses of unweathered basalt whose existence could not be reliably predicted at the location of any specific pile and that were assumed to have no effect on the foundations.

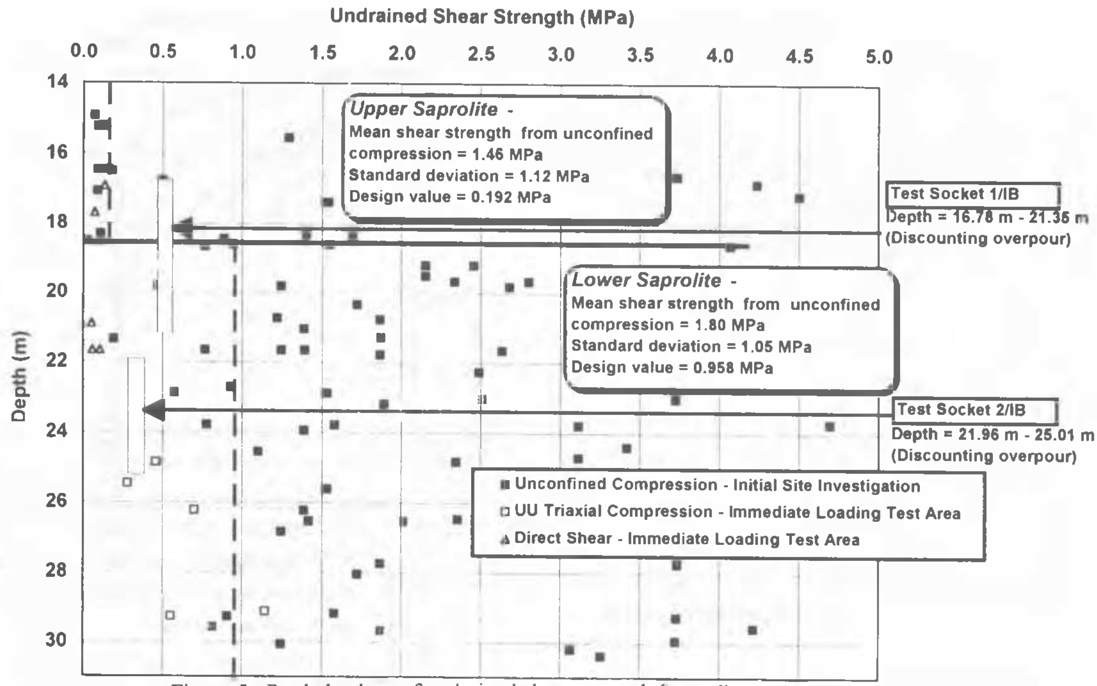

Figure 5 Pooled values of undrained shear strength from alignment of Viaduct 1B

The piles were designed using values of undrained shear strength, s_u The design value for shear strength in the upper saprolite, 192 kPa , was established from experience with stiff clays in well-defined soil profiles, discounting reinforcing effects of the harder inclusions, and the resulting limiting unit side resistance against the bored piles was computed using α factors that were consistent with experience elsewhere in stiff clay.

In the chaotic lower saprolite, where most of the axial resistance would develop, the design value of s_u was taken as the adjusted mean of all measured values along the alignment of Viaduct 1B (mean of all values excluding those larger than 5 MPa) minus 0 8 times σ (s_u), or 958 kPa. The displacement of the design value from the mean value was selected for consistency with the use of an allowable stress design method where a global factor of safety (F) of 3 was defined *a priori* by the structural designers and in which an attempt was made to maintain a level of reliability equivalent to using F = 3 in less chaotic geomaterial

Design values for shear strength were assigned for all of the foundations on Viaduct 1B as a unit, not on a boring-by-boring, or foundation-by-foundation, basis, because very few testable samples could be recovered from any one foundation location Greatly increasing the number of borings at each foundation location might have allowed for statistical analysis of the data for each individual foundation and the assignment of foundation-specific design values for shear strength, but the expense of taking borings in the rugged and heavily vegetated terrain prevented such a rigorous subsurface exploration program from being implemented The geomaterial was so variable that *in-situ* testing devices were not feasible for general use

The depth to basalt was difficult to define because the transition from saprolite to rock was irregular, and the hardness of the basalt was highly variable The average depth to basalt was about 27 meters

One major uncertainty remained Sample recoveries in the saprolite were about 70 %, and the rock quality designations (RQD) averaged < 0 1

It was uncertain whether the unsampled geomaterial might be significantly softer than the tested geomaterial and what effect such unknown soft inclusions would have on the resistance and settlement of pile foundations In order to minimize the effects of unknown soft inclusions, it was decided to groove the sides of the pile excavations in the saprolite with keys 50 mm deep and 75 mm high every 300 mm of depth The possibility of high settlements in the lower saprolite were excluded by conducting simple settlement analyses using oedometer results from the softer materials in the upper saprolite

3. FIELD TESTS

Validation of the design assumptions for Viaduct 1B was accomplished with two Osterberg-cell loading tests A test site was identified within the viaduct alignment at which, based on analysis of the recovery, RQD and shear strength data, recent weathering appeared to be most intense. New borings were made, and the shear strength profile was established by unconfined and direct shear testing, which indicated that the test site represented a location at which saprolite strengths were below the design values (open symbols in Figure 5)

Two full-diameter test sockets were installed, one 4 6 meters long that was partially in the upper saprolite, and one 3.1 meters long entirely in the lower saprolite Unconcreted zones above the sockets were filled with sand while the concrete was fluid to simulate the pressure of the overlying concrete.

The measured load transfer behavior can be summarized in terms of the familiar α values If the limiting unit side resistance is defined as α s_u (design), α = 0 6 for the lower saprolite, recalling that the saprolite was grooved, which is essentially equal to the value of α = 0.55 assumed for design. The method for characterizing the lower saprolite for strength limit state design purposes was therefore considered to be verified, and the design value of limiting side resistance of 527 kPa, was used to size the piles

Load-movement behavior was predicted using an approximation of the results of a FEM parametric study with a frictional, sinusoidal, interface roughness pattern between the socket and the geomaterial and inelastic stress-strain behavior in the concrete and geomaterial (Hassan and O'Neill, 1997). The constructed roughness, which was measured with mechanical calipers, crudely approximated a sinusoidal pattern A typical measured borehole roughness profile and the roughness profile that was used in the design model are superimposed in Figure 6 To account for the presence of soft seams within the saprolite, an empirical coefficient, denoted r, is used to reduce the shear strength of the sampled geomaterial to a depthwise mean unit side resistance Values of r as a function of RQD used in the design method derived from the FEM analysis are shown in Figure 7 r = 0 45 for RQD = 0 2, but reliable information is not available for RQD < 0 2 Unfortunately, most of the lower saprolite had RQD values < 0 2, even though triple-walled core barrels were used in the sampling process

The measured load-movement relation for the socket in the lower saprolite and predicted load-movement relations are shown in Figure 8 The graph is plotted in terms of settlement

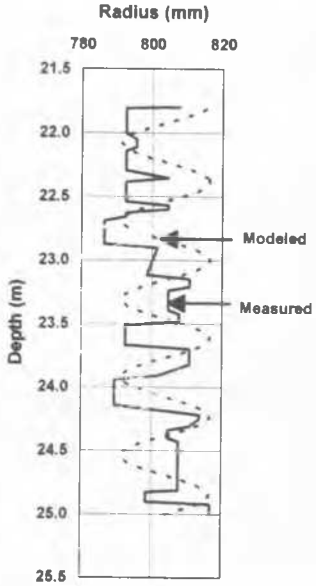

Figure 6 Measured and modeled borehole roughness profiles

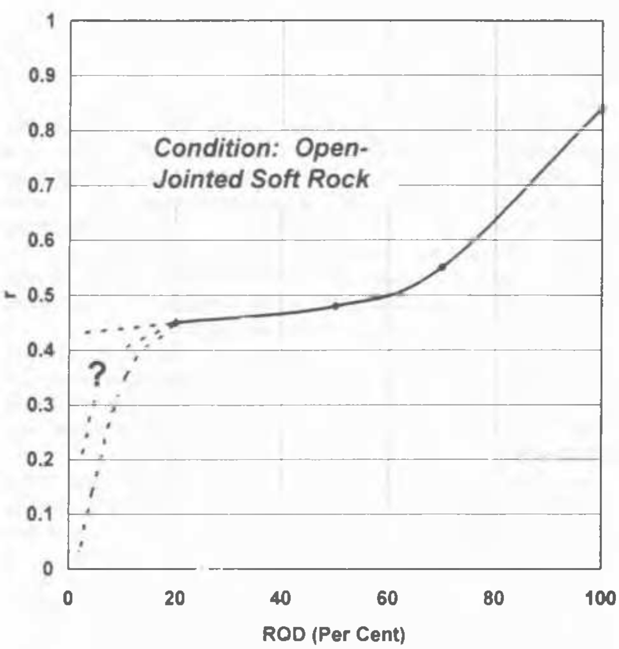

Figure 7 Parameter r versus RQD for the design method

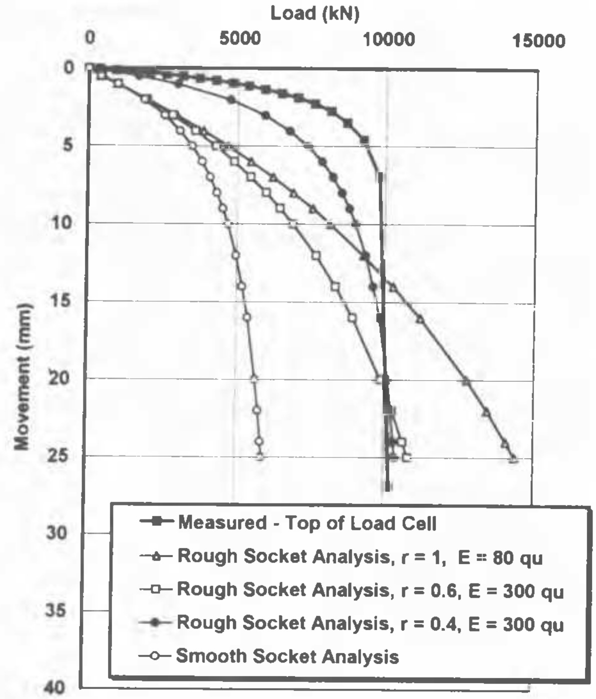

Figure 8 Measured / predicted load-movement benavior

When a value of r of 0.4 was used with an initial Young's modulus of 300 q_u (mean, adjusted value of the unconfined compression strength of the cores), the method simulated the defined limit state (settlement of 25 mm) well, although excessive settlements were predicted at smaller loads, possibly because the design method models compression loading, while the Osterberg cell test was a pushout test.

In the load-movement model, r = 0.4, which is slightly less than the empirical value for RQD = 0.2. This value is applied to q_u (mean, adjusted). r is not identical to α because α does not represent a complete shear failure state within the asperities of the geomaterial along the sides of the drilled shaft.

The foundations for Viaduct 1B were constructed successfully in 1995 and 1996. The only difficulty experienced by the contractor was that the basalt was encountered at a higher elevation than expected at some locations, which required that some piles be bored into the basalt.

4. CONCLUSIONS

The methodology employed for arriving at design parameters for axially loaded bored piles along one viaduct founded in a saprolite formation with chaotic patterns of undrained shear strength included:

- pooling all of the shear strength data along the alignment of the viaduct,
- defining design values based on the simple, first-order statistical parameters (mean and standard deviation) of the pooled set of measured shear strength values (after discarding high values that could not be reliably counted upon in all piles),
- choosing design factors similar to those usually applied to mean values of shear strength in less chaotic geomaterials,
- grooving the pile walls to increase the pile resistance in the presence of unknown soft seams, and
- verifying the design assumptions through load testing and deterministic modeling of the results of the loading tests.

The methodology, while requiring considerable engineering judgment, is rational and resulted in successful characterization of a chaotic saprolite to design foundations for major bridges economically.

5. REFERENCE

Hassan, K. M., and O'Neill, M. W. (1997). "Side Load Transfer Mechanisms in Sift Argillaceous Rock," *Journal of Geotechnical and Geoenvironmental Engineering*, Amer. Soc. of Civil Engineers, Vol. 123, No. 2, Feb. 145 - 152.

6. ACKNOWLEDGMENTS

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