

Monorail piers on shallow foundations, settlement analysis based on Dutch cone data

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1 INTRODUCTION

A major expansion project at Walt Disney World in Florida includes about 5 miles (8 km) of new, elevated monorail. The new system will connect the existing monorail with the new development, located 3-1/2 miles (5.6 km) to the south.

The existing monorail system was supported on piles because the upper soil profile was found to be generally poor and variable. The piles were driven to bearing in the underlying limestone formation, which was encountered at a depth of 50 to 60 feet (15 to 18m).

In contrast, along the new alignment the upper 25 to 50 feet (8 to 15m) of the subsurface profile generally consists of moderately dense fine sands, which are free of the clayey layers present along the old system. The surface of the limestone is also deeper along the new alignment, ranging in depth from about 60 to 100 feet (18 to 30m). For these reasons, it was decided to evaluate the possibility of supporting as many of the piers as possible on shallow foundations rather than on piles. The controlling aspect of the design was settlement performance.

Settlements of shallow foundations in sand are known to be difficult to evaluate, particularly when the sand profile is nonhomogeneous. This is especially true when looser sands are present and there is a high water table, conditions which make it extremely difficult to obtain good-quality samples. Therefore, any realistic settlement analysis must depend heavily on in situ testing, such as Standard Penetration Tests (SPT), Dutch Cone Penetration Tests (CPT), plate load tests, etc. Laboratory triaxial tests following realistic stress paths (Lambe and Whitman, 1969) or with the

objective of establishing a relationship between elastic modulus and confining pressure (Janbu, 1963) have also been found to provide realistic settlement results for fairly homogeneous, medium dense to dense sand layers. However, for large sites with varying soil conditions, it is convenient to establish a relationship between in situ test data and compressibility, rather than perform these sophisticated tests for each individual footing location.

For many years, engineers have used SPT blowcounts (empirically correlated to plate load tests and/or actual footing performance) for settlement predictions in sands (Terzaghi and Peck, 1967; Meyerhof, 1965; Peck, et al, 1974). However, inconsistencies in performing the SPTs (Fletcher, 1965), including the use of different drill rigs and even different operators, can affect the quality of the data. There is also reason to believe that dynamic tests such as the SPT may induce partial liquefaction in looser, saturated fine sands below the water table. Therefore, the results may indicate soil conditions worse than actually exist. The SPT is also dependent on a drill rig, and each test requires an interruption of the drilling procedure. If the sand profile is variable, numerous tests and considerable time are required to develop a complete description of the profile.

CPT soundings, on the other hand, are considered to overcome most of the disadvantages of the SPT. The CPT is a fast, relatively inexpensive in situ testing technique which also provides more continuous information than the SPT. For these reasons, it was decided to use the CPT for investigating monorail pier locations.

2 PROJECT CONSIDERATIONS

The new monorail beamway consists of a system of multispan concrete beams. The beams are supported about 26 to 28 feet (8 to 9m) above grade on 276 piers, spaced about every 100 feet (30m). Typical cross sectional views of the piers are shown on Figure 2. Once in place, every five to six individual spans are post-tensioned to form a continuous beam. The central pier in each continuous beam is designed to provide the primary resistance to longitudinal forces acting along the beamway.

Each pier is analyzed for several combinations of dead load, train load, impact, wind and breaking forces. The deflection performance of adjacent piers must be compatible; differential displacements could affect the beamway alignment and the stresses developed in the beams. Pier loadings and the resulting deflections are shown in a general manner on Figure 1.

The generalized subsurface conditions along the alignment are shown on Figure 1. The subsurface profile consists of three main strata: the Upper Sands, the Hawthorn formation, and limestone. The Upper Sands consist of fine to medium sands generally containing less than 5 to 7 percent silt or clay size material. Typical grain-size distribution curves are shown on Figure 3a. The thickness of this strata ranges from 25 to 45 feet (8 to 14m). Except for the upper 2 to 3 feet (1m), these sands are generally moderately dense. Layers of weakly cemented, silty fine sands (hardpan), typically 5 to 10 feet (2 to 3m) thick, are frequently present within the upper 20 feet (6m). The shallow ground water table exists within 5 feet (1.5m) of the ground surface.

Below the Upper Sands is a strata of loose to moderately dense, fine sandy silt to silty fine sand. This strata, locally referred to as the Hawthorn formation, is typically 35 to 50 feet (11 to 15m) thick. Typical grain-size curves for these soils are shown on Figure 3b. Laboratory consolidation tests indicate this formation is moderately overconsolidated (OCR approximately 1.5).

Limestone is found at depths ranging from 60 to 100 feet (18 to 30m). The limestone is typical of the cavernous material found throughout central Florida. The quality of the limestone can vary from almost crystalline rock to lightly cemented sands and silts.

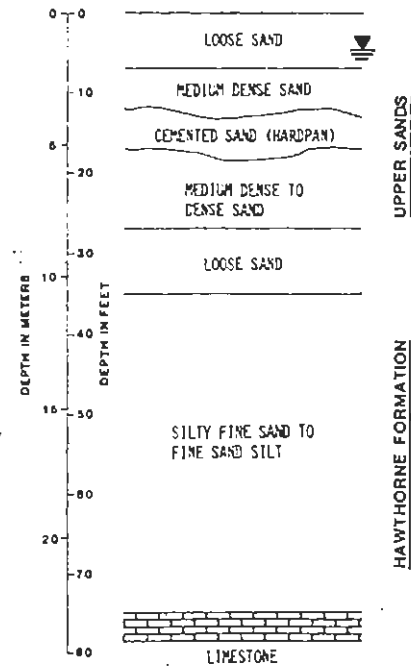


Fig. 1 Subsurface Profile

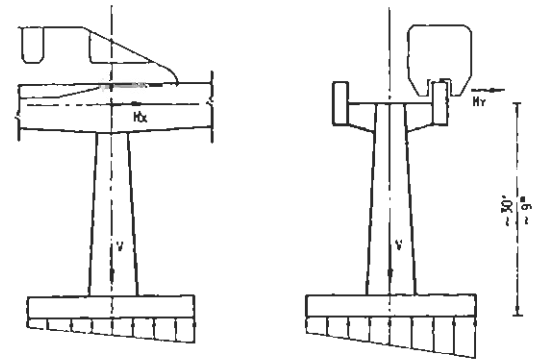


Fig. 2 Monorail Pier

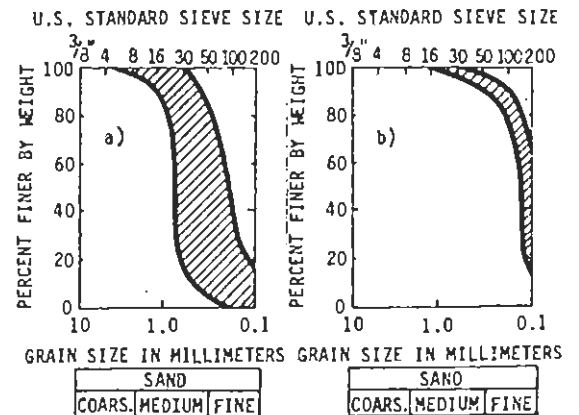


Fig. 3 Grain-Size Distribution

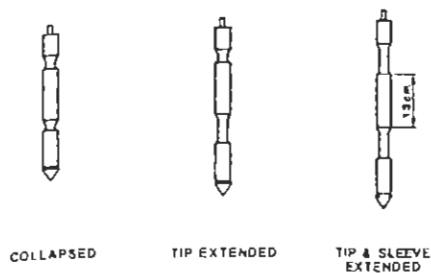


Fig. 4 Mechanical Cone

3 THE DUTCH CONE PENETRATION TEST

A CPT sounding was performed at each pier location where shallow foundations appeared feasible based on a preliminary study. A mechanical friction-cone was used. The tip is a 60 degree hard steel cone with a projected area of 10 square centimeters (cm^2). The friction sleeve has a circumference of 11.2 cm and is 13.35 cm long (friction area = 150 cm^2).

Figure 4 shows the operation of the mechanical friction-cone. The cone is first advanced to the desired depth by pushing on the outer rods of a two-rod system. Then the cone, attached to the bottom of the inner rods, is pushed into undisturbed material at a constant penetration rate of 1 to 2 cm/sec. The measured thrust divided by the projected cone area gives the cone bearing capacity, or penetration resistance, q_c . Continued pushing of the inner rods engages the steel friction sleeve which, upon further extension, measures the thrust on the cone plus the friction sleeve. The sleeve friction is obtained by subtracting the cone thrust and dividing by the sleeve area (this assumes the cone thrust did not change in the process). The outer rods are then pushed to collapse the telescoped apparatus, and it is advanced to a new depth where the process is repeated. Readings are usually taken at 20 cm depth intervals. Therefore, the test essentially provides a continuous record of cone and friction resistances.

During the investigation at pier locations, a total of 195 cone soundings were performed. Each sounding was advanced at least 10 feet (3m) into the Hawthorn formation. From a settlement standpoint, the lower portion of the Hawthorn formation was not a significant factor because of its depth and the transient nature of the most critical loading condition (wind). To simplify the settlement analyses, eight generalized

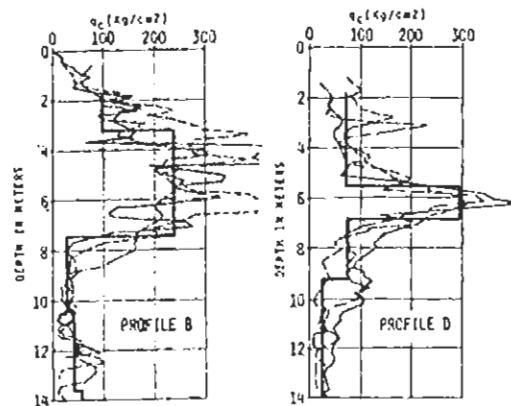


Fig. 5 Generalized CPT Profiles

cone sounding profiles were developed to represent the various conditions along the alignment. Two of these generalized profiles are shown on Figure 5. The sounding at each individual pier was then matched with one of the eight generalized profiles.

4 LABORATORY TESTING

Correlation factors relating CPT data to compression moduli have been studied by several researchers (DeBeer, 1967; Schmertmann, 1970; Garga, 1974). As part of this study, a series of triaxial compression tests was performed to check the validity of these correlation factors for this site. These tests were performed on representative samples of the three typical sand groups present within the Upper Sands: the upper loose fine sand, the upper medium dense to dense sand, and the lower loose fine sand.

Two types of triaxial tests were performed: consolidated-undrained (TXCU), and consolidated-drained (TXCD). For both tests, the samples were allowed to consolidate fully under the applied confining pressure prior to the start of the test. The volume change during consolidation was determined by measuring the water and/or air expelled. For the TXCU tests, no drainage was permitted during shearing, and total stresses were measured. Intergranular (effective) stresses were measured directly by performing TXCD tests. During the TXCD tests, drainage was permitted, and the shearing performed slowly enough to prevent the buildup of excess pore water pressures. Typical stress-strain curves measured during the TXCD tests are presented on Figure 6a.

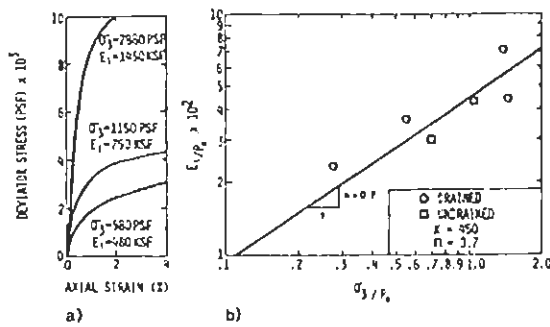


Fig. 6 Triaxial Test Results

Janbu (1963) provides a relationship between initial Young's modulus (E_1) and confining pressure (σ_3):

$$E_1 = K p_a (\sigma_3/p_a)^n \quad (1)$$

where K and n are dimensionless parameters, and p_a is atmospheric pressure. This relationship can be graphically solved using a log-log plot of σ_3/p_a versus E_1/p_a . A straight line drawn through the data points will have an intercept K at $\sigma_3/p_a = 1$, and a slope n , as depicted on Figure 6b.

The initial modulus was obtained by graphically fitting a tangent to the stress-strain diagram obtained from the triaxial tests (Figure 6a). In tests where the initial portion of the stress-strain curve was questionable, due to end disturbance etc., E_1 was estimated by fitting a hyperbola through data points at 70 to 95 percent of the peak stress, as suggested by Kulhawy, et al., (1969).

A comparison was made of the deformation behavior of the sands under drained and undrained conditions. The stress-strain curves of the undrained tests considerably stiffened shortly after initial straining, accompanied by a buildup of negative pore pressures. However, the initial moduli still fall in the same range for both the drained and undrained tests, as can be seen on Figure 6b.

5 SETTLEMENT ANALYSIS BASED ON CPT

The most widely recognized methods of predicting settlements based on the CPT are those developed by Schmertmann (1970, 1978) and by DeBeer (1967). Both methods are based on empirical relationships and, provided a realistic correlation between cone resistance and soil stiffness, can be established, they both yield reasonable results confirmed by numerous field

measurements (Schmertmann, 1970). Schmertmann proposed a triangular distribution of a strain influence factor which, multiplied by the constant E_s/p (E_s = equivalent Young's modulus for sand in compression, and p = footing pressure) gives the compressive strain. In a recent paper Schmertmann (1978) suggested a correlation of $E_s = 2.5 q_c$ for square footings, and $E_s = 3.5 q_c$ (with somewhat different strain factors) for long rectangular footings. However, because Schmertmann bases his method on the vertical strain distribution under a uniformly loaded foundation, it is not readily applicable for nonuniform stresses resulting from moment loading.

DeBeer's method (1967) utilizes the compressibility index, C , which is given by the empirical relationship, $C = 1.5 q_c/p'$, where p' is the effective overburden pressure, and q_c is the CPT resistance. The settlement, s , for any layer thickness, H , is obtained from:

$$s = (H/C) \ln ((p' + p_1)/p') \quad (1)$$

where p_1 is the increase in effective stress in the layer due to the foundation load. Because DeBeer's method works with soil stresses rather than strains, it is convenient to use for nonuniform foundation pressures.

The design of the shallow monorail pier foundations is controlled by a criteria which limits the transverse deflection at the beamway height resulting from wind, centrifugal, and train loading forces, rather than by the allowable bearing values or a purely vertical settlement criteria. For this reason, DeBeer's method of settlement analysis was chosen for this study.

The empirical relationship of the compressibility index $C = 1.5 q_c/p'$, as suggested originally by DeBeer, has been found to result in generally conservative settlement estimates. Meyerhof (1965) for a similar type of settlement analysis suggested a correlation factor of 1.9. Although used in different formulas, Schmertmann (1970) equates his stiffness definition with the definition of DeBeer, which suggests a factor of 2.5 for DeBeer's correlation of C to q_c . Garga (1974), based on extensive screw plate tests and plate load tests of up to 2.5 x 2.5 meters (8 x 8 feet) performed on fine to medium sands, found a correlation factor of 2.9. For the moment deflection analyses performed for this

Table 1. Elastic Moduli from CPT Data and Triaxial Compression Tests

Soil Group	Depth d (ft)	Triaxial Tests		CPT Data
		K n	E_1^* (tsf)	$E = 2.5 q_c$ (tsf)
Upper loose	5.	450	119	100-180
fine sands	10.	0.7	194	
Upper medium	15.	850	365	400-600
dense sands	30.	0.7	545	
Lower loose	30.	70	40.	40-80
fine sands	45.	0.9	54.	

$$E_1^* = K p_a (\sigma_3/p_a)^n$$

$$\sigma_3 = 0.5d \gamma_{eff}$$

study, $C = 2.5 q_c/p'$ was used. This correlation is supported by a comparison made with the triaxial compression test results, as shown in Table 1.

For the finer-grained soils of the Hawthorn formation, the compressibility was derived from consolidation tests. Considering the transient nature of the pier's moment loading, the use of consolidation test results is somewhat conservative. However, because of its depth, the contribution of the Hawthorn formation to the overall deflections is small. By equating Terzaghi's (1943) settlement formula,

$$s = HC_c \log ((p' + p_1)/p') \quad (3)$$

to DeBeer's formula (1), an equivalent cone resistance q_c was calculated from C which could then be used in DeBeer's formula.

Settlement analyses were performed for each of the eight generalized cone profiles for a variety of footing sizes and loading conditions. The stress distribution for each case was calculated using the Boussinesq formulation, as employed by a standard settlement computer program. The program was modified to calculate compressibilities directly from cone data. Nonuniform contact pressures were approximated with a step function consisting of six areas of different uniform pressure. Dead and live loads were applied in separate increments. The analysis resulted in a slightly bowl-shaped settlement profile, implying some bending of the foundation. This bending would not actually occur because of a rather stiff footing design. Therefore, the tilting of the footing was approximated by considering the differential settlement between two points located on the centerline of the footing at a

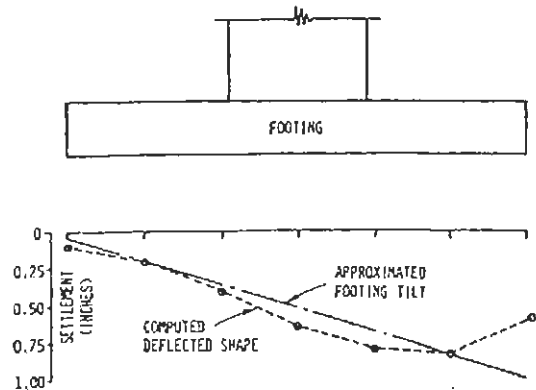


Fig. 7 Settlement Profile

distance of one-third of the width from the center (Figure 7). The deflection at the beamway level was then calculated using this slope for the footing tilt and multiplying by the height of the beamway above the footing base. The deflections of structural members were not considered in this analysis.

A design curve based on an applied transverse moment of 1,000 foot-kips was developed for each of the eight generalized cone profiles for use in sizing the footings for the entire alignment (Figure 8). For the range of moments anticipated, it was assumed that the foundation tilt would be directly proportional to the applied moment. Therefore, having assigned a generalized cone profile to represent the soil conditions at each

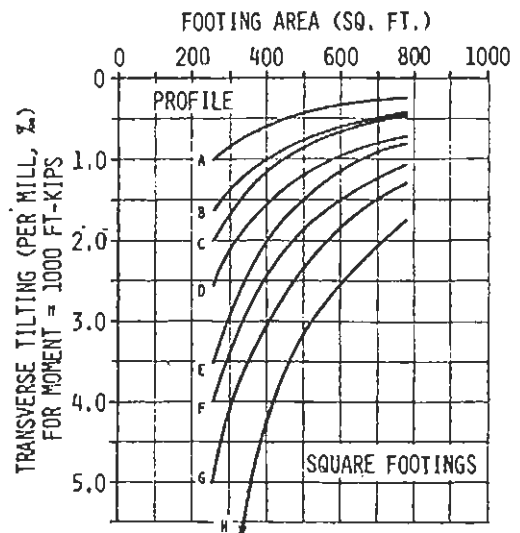


Fig. 8 Design Curves

pier, and knowing the magnitude of the applied moment and the height of the beamway above the footing base, the proper design curve could be used to select the footing size to maintain the beamway deflection within the specified tolerance. To remain consistent with analysis assumptions, the footing size was then checked to prevent uplift.

6 SUMMARY AND CONCLUSIONS

The Dutch Cone was used to aid in the design of shallow foundations along 5 miles (8 km) of new, elevated monorail alignment at Walt Disney World in Florida. The general subsurface conditions along the alignment consist of 25 to 45 feet (8 to 14m) of randomly varying sands overlying 35 to 50 (11 to 15m) feet of finer-grained soils, overlying limestone. The foundation design was based on a criteria to limit tilting of the foundation.

A settlement analysis was performed for each of eight generalized cone profiles using DeBeer's method. Triaxial compression tests were used to develop a site-specific correlation between CPT data and in situ compression moduli. The results of the settlement analyses were presented in the form of design curves relating base tilt to load and footing size for each of the generalized cone profiles.

In summary, the following statements can be made:

1. On large projects with varying soil conditions it is more convenient to establish a relationship between in situ test data and compressibility, rather than perform borings and laboratory tests for each individual location.

2. The CPT overcomes many of the disadvantages (dynamic side effect, inconsistent operating procedures, etc.) of the SPT.

3. The CPT can be performed quickly and relatively inexpensively.

4. The CPT provides essentially a continuous record of penetration resistance, which, when correlated with data from sampled borings and laboratory test results, allows for a direct evaluation of in situ soil properties such as density, shear strength and compressibility.

5. Published correlation factors for CPT data can vary depending on local soils and other site-specific conditions. Therefore, such factors should be confirmed by laboratory testing on samples taken from representative correlation borings.

7 ACKNOWLEDGEMENT

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