Large Model Footing Tests for the Evaluation of Structural Fills

Essais d'Empattements à Echelle Reduite pour Evaluer des Remblais Structuraux

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SYNOPSIS Large model footing tests having areas over 3m^2 and developing pressures up to 2400kPa were used to evaluate compacted fills placed over shallow rock for the support of two power plants. Estimates were made of the "elastic" moduli of the fill and the attenuation of stress with depth for the prediction of the behavior of footings and mats. Testing methods and procedures are described, typical results are given, and comparisons are made between the observed settlements and those predicted using classical elastic theory and a semiempirical method.

INTRODUCTION

When the depth to competent rock is just beyond normal footing depths, it is frequently more economical and expedient to construct a compacted fill of select material between the rock and the footings instead of using piling or piers. Such a foundation system has been successfully used for two power plants in the U.S.A., one in Wyoming and one in Missouri. The Wyoming plant utilized a mixture of on site soils compacted over sandstone having wide variations in cementation, strength and density. Wet bottom ash, a waste product of an existing plant, stablized with a small amount of Portland cement was compacted on a jointed and weathered limestone at the Missouri site. Since power plants have very heavy column loads as well as equipment sensitive to settlement, proper design and construction of the structural fill is essential For foundation design and predictions of settlements, model footing tests were found necessary to provide quantitative data to: (1) determine the average "elastic" moduli of the compacted fill and, if required, the underlying rock, (2) predict the stress attenuation with depth, and (3) establish the lower boundary values for the ultimate bearing capacities of footings.

PRELIMINARY EVALUATIONS

Site conditions and structural requirements must be evaluated prior to the formulation of a testing program.

Depths to rock relative to the proposed floor or machinery slab elevations must be ascertained. In addition, the characteristics and engineering properties of the rock must be determined. Water conditions that may occur during or after fill placement are also important.

An inventory must be made of the qualities and quantities of possible fill materials with their relative costs. This requires extensive laboratory testing. For the power plant site, highly compacted materials having high shear strength (a minimum angle of internal friction of 35°) and low compressibility (a compression index of less than 0.01) were investigated.

Minimum foundation requirements must be established to set model footing test criteria. For power plants a minimum allowable bearing value of about 720kPa is required to prevent individual footing stress interference. Column loads greater than 8MN generally require combined footings or mats. Machine foundations may have small settlement tolerances, however, these are of consequence only after the machines are operational.

MODEL FOOTING TEST PROCEDURE

Excavations were made to a predetermined depth at selected locations on each site and into these excavations test fills were constructed of materials deemed feasible for use. In some cases, the excavations were extended and the rock was also tested by model footings. Fills were constructed using full sized compactors under controlled conditions. Two holes of minimum diameter were drilled through the fill into the underlying rock, bundles of prestressing cables were installed and then grouted into the rock. Reinforced concrete footings, with sleeves to accommodate the cable bundles and settlement rods, were formed and cast directly on the fill or rock. Large footings were used in an attempt to avoid the scaling problems brought out by Bjerrum and Eggestad (1963) and subsequent researchers. A surcharge equal to footing depth, about 1m, was placed around the footing. Loads were applied by two calibrated hydraulic prestressing jacks of up to 4450kN capacity reacting against the grouted prestressing cables. Settlement measurements were made during loading with a gage frame and direct reading dial gages (Wyoming tests) or with a precise leveling instrument (Missouri tests). In addition, settlements of the soil at two elevations under each pad were monitored. The configuration of the principle components of a typical load test is shown in Figure 1. A small pressure, approximately 200kPa, was recycled at the beginning of each test to

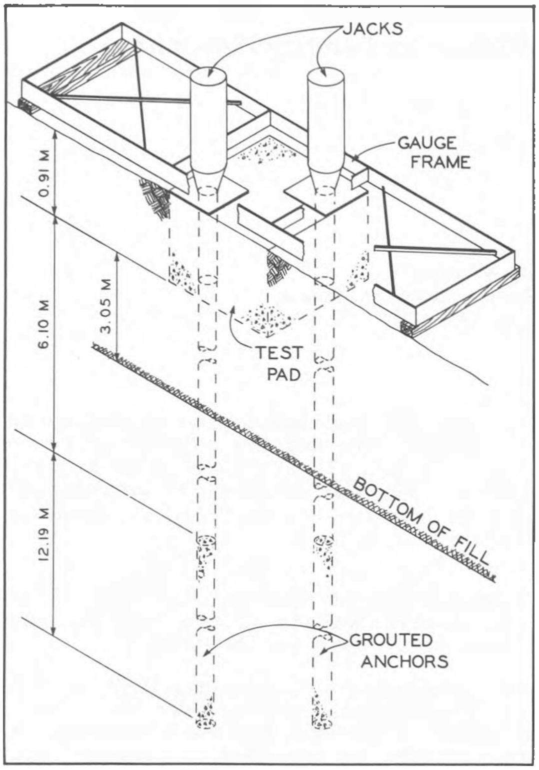

Fig.1 Model Footing Test Arrangement

minimize the effects of soil disturbance due to construction activities. Loads were also recycled to permit a better evaluation of the "elastic" modulus.

TEST RESULTS

The settlement of the footing at each load increment was computed as the average of the settlements of the four corners. Small erratic differential settlements, tilting, of the footings took place, which is indicative of stress redistribution. Due to the maximum safe jacking system capacity, it was not always possible to achieve the ultimate footing capacity. Typical load-settlement curves are presented in Figures 2, 3, and 4. Four other load tests on the Wyoming site and additional test data are presented in a thesis by Houghton (1979).

It was noted during the tests and from the data that, although high density and high shear strength materials were used under these footings, general shear failures did not occur. These materials behaved as essentially linear elastic masses up to a certain stress level and then underwent a progressive local shear failure, as described by Vesic (1973).

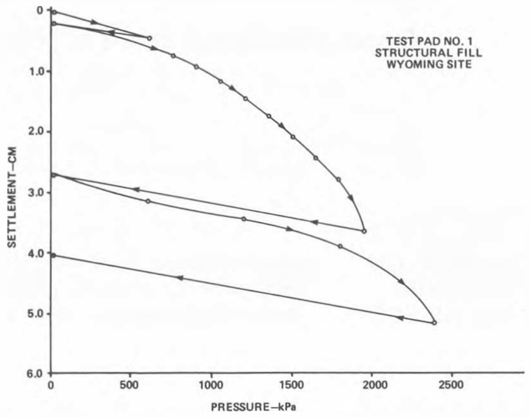

Fig.2 Load-Settlement Results, Soil Fill

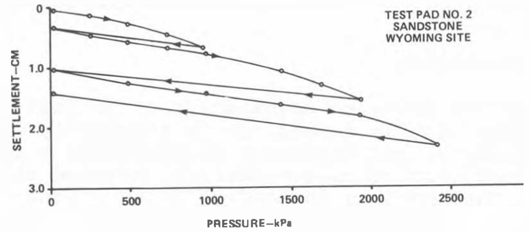

Fig.3 Load-Settlement Results, Rock

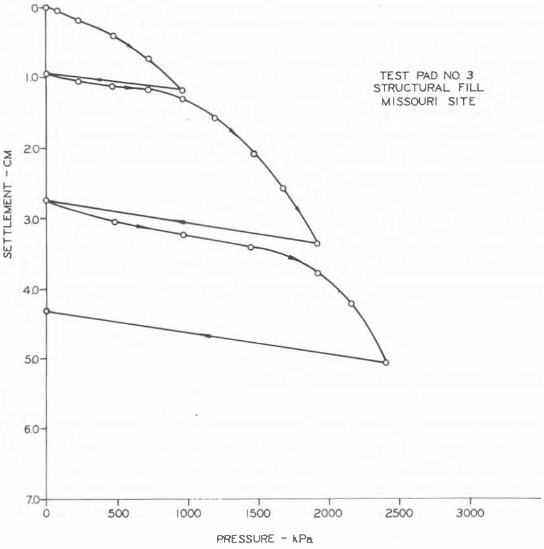

Fig.4 Load-Settlement Results, Bottom Ash

ANALYSIS OF DATA

Test data was analyzed to evaluate an "elastic" modulus of the material to predict settlement and estimate stress attenuation with depth for the prototype shallow foundations.

"Elastic" moduli were calculated using the slopes of the initial tangents of the first load cycle curve or the slopes of the rebound-reload curves. The moduli cannot be determined directly from these slopes since other factors influence the relationship between the load and settlement, among them are: (1) size and shape of loaded area, (2) distribution of loads, (3) rigidity of the loaded area, (4) position of the loads with respect to the surface, (5) extent of the elastic mass, (6) anistropy and (7) inhomogeneity. Exact solutions considering all combinations of these factors are not available; therefore, classical elastic theory and semiempirical methods were used to determine moduli. The well known solution by Schleicher (1926) for the determination of the elastic settlement of a point on or near the surface of a semi-infinite, homogeneous, elastic half space subjected to a uniform circular or rectangular load was used. Also, the Butterfield and Banerjee method (1971) for determining the vertical displacement of a rigid, rectangular area embedded within a semi-infinite, isotropic, homogeneous, elastic half space was applied. In addition, the semiempirical approach of Schmertmann (1970) for the prediction of elastic settlements of shallow foundations on sands was utilized. Table I lists the values of moduli obtained by these methods for two tests at each of the sites. Since the moduli increase with increasing depths, these moduli are regarded as average moduli.

Table I

COMPARISON OF ELASTIC MODULI

Modulus of Elasticity (kPaX10^3)

Test Number	Load Cycle	Schleicher	Butterfield and Banerjee	Schmertmann
		(Wyoming 1.2m x 1.2m)		
1	1	118	100	99
	2	124	104	104
	3	129	109	108
2	1	154	130	130
	2	194	164	163
	3	211	178	177
		(Missouri 1.8m x 1.8m)		
1	1	45	32	42
	2	129	100	129
3	1	110	100	84
	2	330	299	251

Attenuation of stresses with depth under the footings was inferred from the monitoring of settlements at points below the test pads. These settlements, for a contact pressure of 720kPa, were used in Figure 5 as a plot of the ratio of the cumulative settlement above the monitoring point to the total settlement of the pad as a function of depth relative to the pad width. Curves representing the integrations of strain influence factors with depth using elastic theory (Ahlvin and Ulery, 1962) and a semiempirical method (Schmertmann, 1970) are also shown on this figure. All tests indicated that the majority of settlement took place within a depth of twice the breadth of the footing.

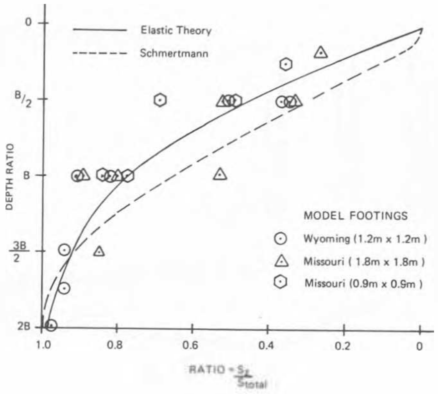

Fig.5 Settlement Profiles with Depth

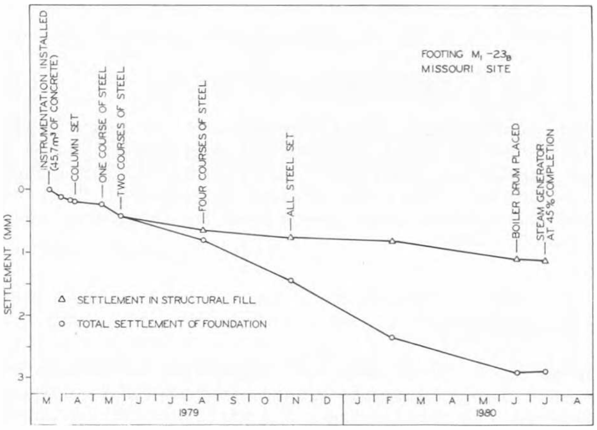

Fig.6 Time-Settlement Results for Footing

COMPARISON OF PREDICTED AND OBSERVED SETTLEMENTS

Selected foundation elements of the power plants were instrumented to monitored settlement with mechanical extensometers seated both at the base of the structural fill and at depth in the bedrock. These settlement measurements were taken for about one and one half years at the Missouri site, see Figure 6, and for three years at the Wyoming site, see Figure 7. Settlement estimates were made for selected foundation elements by both the Schleicher (1926) and Schmertmann (1970) methods.

At the Missouri site, settlement estimates for the column footings ranged from 8mm to 10mm. At approximately 45 percent of the design load, the measured settlement of a representative large footing is about 3mm. The majority of this settlement has occurred in the underlying shales and limestones rather than in the 1.8m of Portland cement treated bottom ash under the footing. Observed settlements to date have been essentially instantaneous with values less than those predicted.

At the Wyoming site, these estimates varied from 9mm to 22mm and from 14mm to 16mm for the turbine, which had 3m of fill, and heaviest column foundations, which had 3.7m of fill, respectively. As may be seen from Figure 7, measured settlement of the turbine foundation has been about 5mm. Observed maximum settlements of the column footings are approximately 10mm, about two-thirds

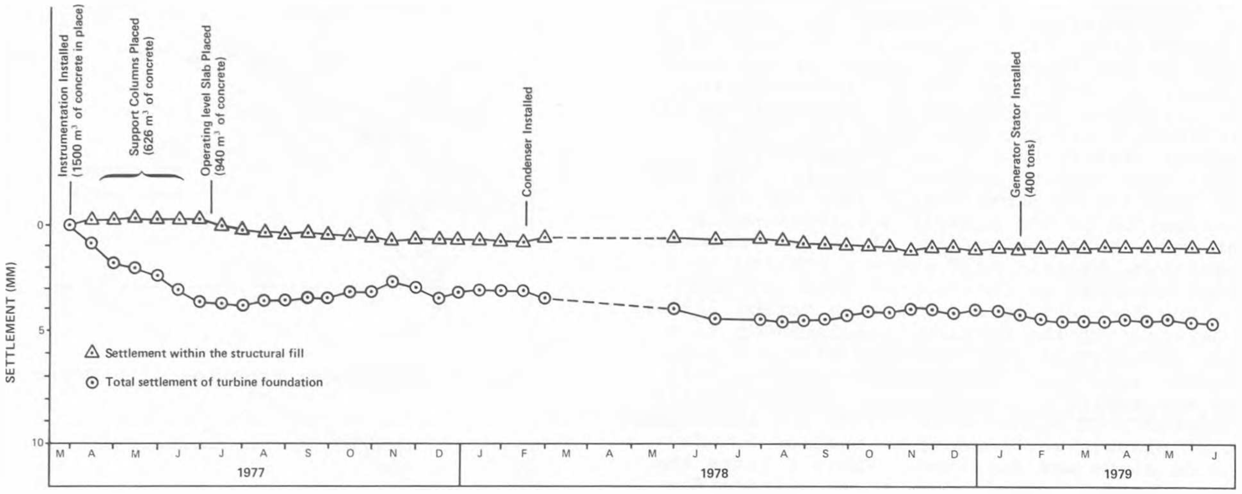

Fig.7 Time-Settlement Results for Turbine Foundation (Wyoming)

that predicted. These measurements also indicate that most of this settlement has taken place in the sandstone bedrock. Construction and compaction control details of the structural fill at this site have been described by Houghton and Leonard (1980).

CONCLUSIONS

Large model footings are required to adequately stress large volumes of compacted fill. This reduces the influence of localized variations in material or density on the measurement of a soil modulus. Also, the effects of scaling from these model footings to the prototype footing is reduced.

Even on well compacted materials a local shear mode of failure takes place, and the majority of settlement takes place within a depth of compacted fill equal to twice the breadth of footing.

Soil modulus is sensitive to the compacted density of the fill. Thus, stringent density control should be specified for these fills.

A significant proportion of observed settlements occur within the underlying rock. Thus, load tests are also required to ascertain rock load-settlement characteristics.

Predicted settlements using either the elastic theory or semiempirical methods are conservative but reasonable.

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