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Uplift Testing of Prototype Transmission Tower Footings

By

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SUMMARY.- This paper describes a series of uplift tests carried out by the State Electricity Commission of Victoria, Australia, on full size transmission tower footings. Tests were conducted on bored piles in basaltic clay and buried slab footings in basaltic clay and in a granitic soil. Comparison of the pile test results is made with a "tension limited shear" theory developed from the results of an earlier series of uplift tests on piles of smaller capacity. (Ref. 1 and 2.) The effect of compaction and compressibility of backfill above concrete slab footings subjected to uplift loading is discussed and the results are compared with a modified empirical cone theory.

I.- INTRODUCTION

Footings for power transmission towers are required to restrain large vertical loadings at the tower legs within small margins of differential movement to prevent overstressing of the tower structure. This is one of the prime factors to be considered in the design. Other factors include the nature of foundation materials, the type of footing and the method of construction.

Transmission lines in Victoria traverse country with cohesive and non-cohesive type soils which can vary significantly along the route, even at one tower site. The type of footings used are the bored cast in situ pile and buried concrete slab and are designed to satisfy a range of loadings and soil conditions.

The design of tower foundations by the State Electricity Commission of Victoria takes into account both the ultimate uplift resistance and the load-displacement characteristic for a particular type of foundation. The actual methods based on soil mechanics principles have been developed to predict the ultimate uplift capacity of tower foundations but heavy reliance has also been placed on full scale testing of actual footing designs to check the theoretical approach and to obtain experience of their load-displacement behaviour. The results of such tests carried out for 220 kV tower foundations have been published (Ref. 1 and 2) and this paper deals with tests carried out in 1965/1966 on 500 kV tower footing designs which require uplift capacities of up to 500,000 lb.

It was necessary to rationalise the tests to determine the behaviour of pile footings in a clay, and buried slab footings in clay and in weathered granitic soil. These soils represent those types generally encountered within Victoria.

For overall economy the footing must be simple in form yet suit a wide variety of soil conditions and loadings. Flexibility of dimensional change to suit unexpected site conditions during construction is

essential.

II.- OBJECTIVE OF TEST SERIES

The objectives of the 1965/66 test series were as follows:

- (a) To extend the testing of prototype footings up to a capacity of 500,000 lb uplift.
- (b) To compare the test results for the bored type footing in clay soils with the theoretical values calculated by the "tension limited shear" theory postulated by Paterson and Urie in 1964 (Ref. 2).
- (c) To check the validity of the empirical "cone of earth" method of design for buried slab type footings.
- (d) To check the effect of the degree of compaction of the backfill and the value of fine crushed rock placed over the slab type foundation on the uplift capacity and load-displacement characteristic.

III.- SELECTION OF TEST SITES

In order to exclude as many variables as possible, two sites were selected on the basis that one consisted of a cohesive soil and the other a non-cohesive soil and that the in situ materials were reasonably uniform to a depth of 18 feet.

The sites selected were at Keilor Terminal Station (approximately 8 miles N-W of Melbourne), and near Corryong (Tower 100 on 330 kV line, Snowy-Dederang).

- (a) **KEILOR SITE - IN SITU CLAY** - The in situ soil at Keilor is a residual deeply fissured basaltic clay. Five bores were put down to 20' depth over the site and undisturbed samples (1.5" diameter) were taken at intervals of one foot. No water table was detected.

The soil properties for the in situ and remoulded states are given in Table I.

The indirect tensile strength of the clay was determined on undisturbed samples 1.5" x 1.5" diameter by the Brazilian method using a standard unconfined compression testing machine as the loading mechanism. The procedure is simple and tests were made in parallel with the unconfined compression strength test on samples cut from the same tube. Some scatter in the properties occurred throughout the soil profile with indication of a very slight increase in their mean value below 9' depth where the indirect tensile strength values increased from about 4 to 6 lb/in², apparent cohesion strengths increased from about 11 to 20 lb/in² and moisture contents reduced from 30% to 20%. Overall mean values of 5 and 15 lb/in² for tension and cohesion strengths respectively were used in the theoretical calculations of uplift capacity.

- (b) **CORRYONG SITE - IN SITU SILTY SAND** - The soil at this site is a residual silty sand derived from weathered granite. It is poorly graded, and firm in the undisturbed state. Light usage of a pneumatic spade was required during excavation but the spoil collapsed to a cohesionless silty sand. Some slight fracturing of the soil mass occurred and light timbering was used to prevent minor collapses. Again no water table was detected within the 14' deep excavations. The soil properties are shown in Table I.

IV. - METHOD OF APPLYING TEST LOAD

Test loads on the bored piles were applied by a hydraulic jack and yoke arrangement set at the centre

of a 20' loading beam which was simply supported on timber packers at each end. The capacity of the loading jack and beam was 240,000 lb.

For the higher capacity buried slab footings the larger test beam and jacking system shown in Figure 1 was used. The beam, 36' long and 5' deep was supported clear of the ground on timber packing at each end. An uplift force of up to 500,000 lb was applied by hydraulic jack to a central yoke attached to the footing. A pair of 125,000 lb jacks at each end of the beam allowed a total uplift displacement of 10" at the test footing.

The load was read directly from a calibrated test pressure gauge fitted to the central jack. Displacements were measured by a scale fitted to the footing and a marker attached to an independent static beam. Displacements were also checked by survey level located some 30' from the footing.

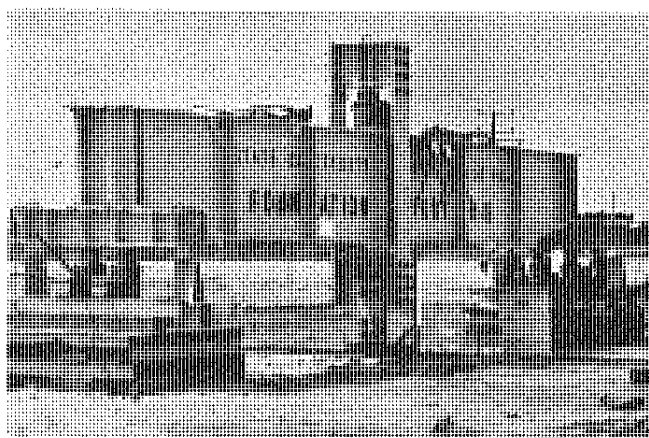


Fig. 1 Loading beam arrangement at site; 500,000 lb uplift capacity.

TABLE I

SOIL PROPERTIES AT KEILOR AND CORRYONG TEST SITES

Site and Soil Type	Moisture Content (%)	Dry Density γ_d (lb/ft ³)	Apparent Cohesion (lb/in ²)	Indirect Tensile Strength (lb/in ²)	Liquid Limit (%)	Plasticity Index (%)	Percentage Passing		
							Clay .002 mm (%)	Silt .06 mm (%)	Sand 2.0 mm (%)
(a) KEILOR Basaltic clay (CH)									
In Situ	28 av. 17-32	93 av. 86-104	15 av. 8-29	5 av. 3-9	105 99-110	80 69-87	60	34	6
Re-moulded	25 Optimum	95 γ_d max.*							
(b) CORRYONG Granitic silty sand (SM)									
In Situ	12 av. 9-15	96 90-102	3.0 $\phi = 34^\circ$	0	Generally Indeterminate		9	25	64
Re-moulded	13-18 Optimum	118-106 γ_d max.*							

* γ_d max. = Maximum dry density (Aust. Standard A89 - 1966)

Two rates of load application were used in this series of tests. In the pile tests, excluding test Nos. 3, 7, 10 and 13, the load was applied at a constant rate of 20,000 lb per minute. For the slab footing tests and pile test Nos. 3, 7, 10 and 13 an incremental method of loading, as specified by the Conference Internationale des Grande Reseaux Electriques a Haute Tension, was used. (CIGRE Committee No. 7.) The load was applied at a uniform rate in approximately five increments each of 5 minute duration, and the load was held constant during intermediate rest periods of 10 minutes each. Each of the 5 incremental loads was predetermined on the basis of 20% of the estimated ultimate footing capacity and displacements were measured during the loading and rest periods.

The loading rate differs in each of these methods and the effect of this is discussed in Section VI.

V.- FOOTING CAPACITY

The uplift capacity of a transmission tower footing is defined for these tests as the maximum uplift load applied to effect a 0.5 inch vertical displacement of the footing. This figure was originally chosen arbitrarily, but it has since been supported by an overall assessment of secondary stresses in the towers due to footing movements.

No load factor is specifically applied in determining the design capacity of the footing except those which are inherent either in the adopted design approach (for the footing) or in the structural analyses of the tower.

VI.- BORED PILES AND TEST RESULTS

Tests were conducted in 1965 at Keilor on seventeen bored concrete piles in clay; of which seven were plain cylindrical shafts and ten with enlarged bases. The pile dimensions and test numbers are given in Table II.

TABLE II

TEST DATA FOR BORED PILES IN BASALTIC CLAY

Bored Concrete Pile				Uplift Load (lb x 10 ³)			
Test No.	Type	Diam. (in)	Depth (ft)	Test		Theoretical	
				At 1/2" displacement	Ultimate	Ultimate	80% of Ultimate
1	Plain	24	9.2	75	75	80	64
2	"	"	12.2	96	99	120	96
3	"	"	15.1	135	137	160	128
4	"	"	18.0	200	212	202	161
5	"	30	9.0	85	87	98	78
6	"	"	12.1	114	114	150	120
7	"	"	15.0	170	173	202	161
8	Under-cut	16/30	12.2	109	120	135	108
9	"	20/36	9.5	110	120	105	84
10	"	"	11.9	165	173	164	131
11	"	"	15.0	229	244	234	187
12	"	24/42	9.0	112	117	136	109
13	"	"	11.5	157	165	181	145
14	"	"	13.7	203	208	236	189
15	Bulb	"	8.9	108	113	130	104
16	"	"	11.3	175	181	188	150
17	"	"	13.8	220	222	242	193

NOTE: The diameters of pile shaft and under-ream are shown 16/30, 20/36 and 24/42 inches.

Augering was carried out by rotary Pengo type equipment for the plain cylindrical piles (test Nos. 1-7) and the shafts for those with enlarged bases (test Nos. 8-17). The spoil was removed frequently from the auger bit to prevent excessive plastering of the walls with remoulded clay.

Two methods of forming the enlarged bases were used. A cylindrical bucket with expandable side cutters was used for reaming the undercut holes (tests 8-14). The bulb-shape enlargements (tests 15-17) were reamed out by a special cutter with several vertically-mounted expandable cutting strips which could be adjusted at the drill head to form the required bulb dimensions. The bulb-shaped enlargements were cleanly cut with less disturbance to the in situ clay than those formed with the bucket. Typical shapes of the enlargements are shown diagrammatically in Fig. 3. The diameters of the reamed sections finished 1" oversize for test Nos. 10, 14, 16 and 17; 2" oversize for test Nos. 11 and 12 and 4" undersize for test No. 9. Concrete was placed into the holes and vibrated to ensure intimate contact with the in situ soil.

The test results are shown in Table II and the shape of the load-displacement curves related to the theoretical ultimate capacity is indicated in Figs. 2 and 3.

Differences in the curve characteristics for the cylindrical and under-reamed piles are clearly evident. In general the cylindrical pile reaches its ultimate capacity at 0.30 inches displacement, and 50% of its ultimate capacity at 0.05 inches, whereas the under-reamed piles displace 1.0 inch at ultimate capacity and 0.10 inches at 50% of ultimate capacity. The cylinder piles are more rigid in behaviour with little reserve capacity at the design displacement of 0.5"; approximately 2% reserve for cylinder piles and 6% for under-reamed piles.

In tests using the incremental loading method, creep effects were noted during the 10-minute periods when the load was held constant. In test Nos. 3, 7, 10, 13, the amount of movement resulting from creep at 0.5 inches total displacement was 0.05", 0.0", 0.10" and 0.10" respectively. Creep effects beyond this point could not be observed readily due to a rapid increase in displacement at the ultimate test load. It was not possible to determine creep effects in the constant loading rate method (20,000 lb/min) which was used for the other pile tests.

In comparing the test capacities given in Table II with respect to the depth of each pile type, the effect of loading rate is not apparent. In the incremental method the average loading rate over the whole test period was about 2,000 lb/min and during the loading period it varied from 5,000 to 10,000 lb/min. These rates are much less than that used in the constant rate method.

It is also noted that the capacities of piles with 42" diameter enlargements formed with the bucket (tests Nos. 12, 16 and 17) were only 3% greater than the smaller 36" diameter piles also formed by the bucket. There was also no apparent effect on the test capacity resulting from oversize or undersize enlargements noted above. However, piles with 42" diameter enlargements formed with the bulb cutter

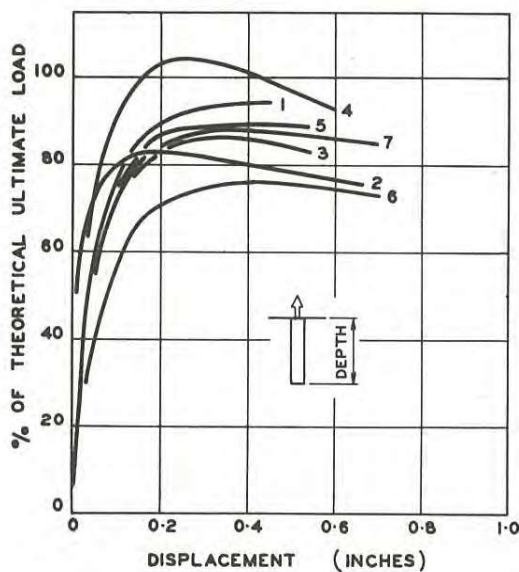


Fig. 2. Plain cylindrical Bored Piles in Basaltic Clay (Keilor, Oct., 1965). Test Nos. 1-7. Uplift load expressed as percent of theoretical ultimate load versus displacement.

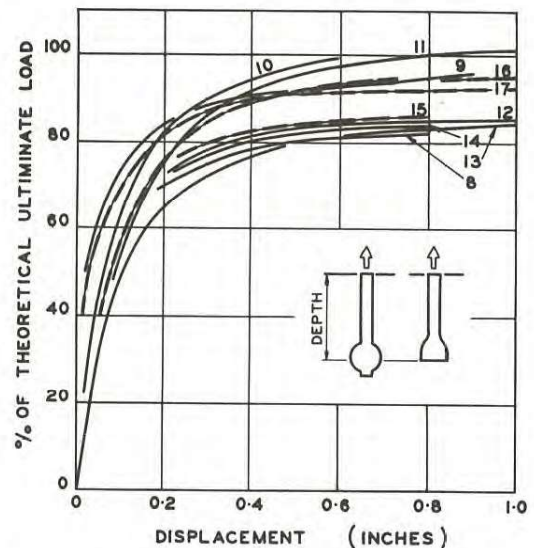


Fig. 3. Bored Piles with Enlarged Bases in Basaltic Clay (Keilor, Oct., 1965). Undercut pile test Nos. 8-14. Bulb Pile test Nos. 15-17. Uplift load expressed as percent of theoretical ultimate load versus displacement.

(test Nos. 16 and 17) generally contributed 10% more capacity than those formed with the bucket (tests 13 and 14).

VII.- COMPARISON OF PILE TESTS WITH "TENSION LIMITED SHEAR" THEORY

The design capacity of the test piles was based on a "tension limited shear" theory proposed by Paterson and Urie in 1964 (Ref. 2) of which a brief summary is given in Appendix 1.

Figs. 2 and 3 show curves for the ultimate capacity of the cylindrical and under-reamed piles expressed as percent of the theoretical capacity versus the measured displacements and it is seen that the test capacities at 0.5 inches movement fall within the range of 76% to 104% of the theoretical capacity. The low value occurred in pile test No. 6 which was probably due to plastering of the augered hole with remoulded clay for the top 3 feet of its total 12 feet depth. Less variation (80% - 96%) resulted in the piles with under-reams.

The variations in the other tests could be attributed to an incorrect choice of mean values for the in situ tensile and cohesion strength parameters of the soil. In determining the theoretical ultimate capacities, the mean values adopted were 5 lb/in² for tensile strength and 15 lb/in² for cohesive strength. Precision in determining the ultimate capacity is dependent on the accuracy of the assessment of these values. For example, in test No. 14 the theoretical capacity is reduced by 10% if the assumed average value of the tensile strength is 4 lb/in² instead of 5 lb/in². For a cohesion strength value of 14 lb/in² instead of 15 lb/in² the loss in the theoretical capacity of the pile is only 2%.

In comparing the actual ultimate capacity with that calculated by the "tension limited shear" method (Figs. 2 and 3) it is seen that the ultimate test capacity of all the test piles, with the exception of No. 6, equalled or exceeded 80% of the theoretical capacity at 0.5 inches displacement.

As a result of the experience gained from these tests the procedure adopted for the design of bored pile footings by the State Electricity Commission is based on a modified version of the "tension limited shear" theory. Footing capacities are designed to exceed 80% of the theoretical ultimate capacity determined by the "tension limited shear" method. However, bored pile footings with enlargements are used only in the suspension or light strain type of towers. They are not acceptable at towers subjected to long-term uplift conditions because of their low reserve capacity at the design displacement limit.

VIII.- SLAB FOOTINGS AND TEST RESULTS

The concrete slab footings tested in this series were constructed to uniform dimensions for the purpose of studying the effect of the density of backfill material on their uplift capacity.

Reinforced concrete slabs of 7' x 7' x 2' thickness were set at 14' depth and backfilled with 12' depth of compacted materials. The slabs were formed in the bottom of 7' x 7' x 14' deep excavations with vertical side walls. Open timber bracing was provided for the safety of personnel. The concrete was cast against the in situ soils at the walls of the excavation and no under-cutting of the walls of the excavation was permitted, particularly at the slab, so that the effect of the degree of compaction of the backfill materials could be observed during the test.

Backfilling was compacted in 4" layers into zones of predetermined densities representing various standards of compaction. Fine crushed rock material consisting of a well graded basaltic stone up to 3/4" size was compacted in 6" layers for 2' and 4' immediately above the slab (Zone A) in test Nos. 23, 27 and 28; the remainder of the hole was filled with compacted soil (Zone B).

The standard of compaction is referred to as the dry density of the compacted fill expressed as percent of the maximum dry density determined from a standard compactive effort of 12,375 ft lb/ft³ (Australian Standard A89-1966).

A total of 11 footings were constructed, six in basaltic clay at the Keilor site (Nos. 18-23) and five in granitic soil at the Corryong site. (Nos. 24-28.) The soil properties are given in Table I. The 500,000 lb capacity test beam described in section IV was used for each test and loads were applied by the incremental CIGRE method. Actual loading rates varied from 8,000 to 21,000 lb/min and on the basis of overall time to reach ultimate capacity, the rate varied from 3,000 to 9,000 lb/min. No specific effects on the performance of the footings could be attributed to variation in loading rate.

The load capacities and displacements determined in the tests and the standards of compaction of the backfill within Zones A and B are shown in Table III.

TABLE III

TEST DATA FOR SLAB FOOTINGS IN BASALTIC CLAY AND GRANITIC SILTY SAND

Concrete Footing (7' x 7' x 2' slab at 14' depth)			Uplift Load (lb x 10 ³)			
Test No.	Compacted Backfill ($\gamma_d/\gamma_{d \max}$) x 100		Test		Theoretical	
	Zone A (%)	Zone B (%)	At 1/2" displacement	Ultimate at stated displ.	"Cone of Earth"	80% "Cone of Earth"
BASALTIC CLAY (12' depth of compacted backfill)						
18	75	75	163	225 @ 5"	340	272
19	84	84	250	380 @ 6"	"	"
20	87	87	220	357 @ 8"	"	"
21	93	93	384	450 @ 1.6	"	"
22	2' zone - fine crushed rock	86	306	352 @ 3.6	"	"
23	4' zone - fine crushed rock	87	326	385 @ 2.2	"	"
GRANITIC SOIL (12' depth of compacted backfill)						
24 (a)	no backfill - slab only		130	170 @ 1.1	-	-
(b)	85	85	220	*495 @ 2.1	340	272
25	90	90	280	*495 @ 1.7	"	"
26	92	92	315	*460 @ 1.1	"	"
27	4' zone 92	90	320	*496 @ 1.5	"	"
28	4' zone - fine crushed rock 95	90	430	*500 @ 0.6	"	"

*less than ultimate load

NOTE: $\gamma_{d \max}$ = maximum dry density of soil (Aust. Standard A89-1966). γ_d = dry density of soil.

The load-displacement curves plotted in Figs. 4 and 5 clearly indicate the difference in behaviour of footings in a clay and a granular material. At 0.5 inches displacement, the capacity of the footings in clay ranged from 163,000 lb to 384,000 lb (236%), whereas in the silty-sand 220,000 lb to 430,000 lb (195%). The footings in clay provide less reserve capacity beyond the design displacement level to the ultimate capacity (12% to 62%) than that offered in the granular soil (estimated 60% to 100%).

In tests 18 to 21 it is evident that a high standard of compaction of the backfill significantly increases the footing capacity although tests 19 and 20 are inconsistent. The effect of a low compaction standard (75%) resulting in a low capacity is indicated in test No. 18.

The use of fine crushed rock in Zone A for footings in clay and silty sand, increased the capacity of the footing significantly at 0.5 inch movement from 220,000 lb to 326,000 lb (test Nos. 20 and 23) and from 320,000 lb to 430,000 lb (tests 27 and 28). The gain in capacity in each case was about 40%.

In Test 22(a) the load was applied to the slab before backfill was placed in order to ascertain the restraint offered by the slab. The average shear stress at the edges of the slab was 16 lb/in² and no suction effect was evident in the unsaturated clay. It was conducted in association with a theoretical study of displacement with respect to the compressibility of backfill related to its standard of compaction. Reference is made to this study in Section X. Backfill was then placed over the slab for test 22(b) in which the uplift load was released at 0.75 inch displacement then re-applied (Test 22, Fig. 4) incrementally after 80 minutes elapsed. On re-loading further displacement increased 0.5 inches from 0.4 inches to 0.9 inches at a capacity equivalent to 80% of that initially recorded at 0.5 inch displacement.

Observations of displacements due to creep effects could be made only during the 10 minute periods when the load was held constant. The total creep observed until the design displacement limit of 0.5" was reached indicated no clear trend with respect to the standard of backfill compaction or type of material. At footing displacements of 0.5 inches, the creep component recorded at Keilor was 0.10", 0.17", 0.06", 0.09" and 0.11" (tests 18-23 respectively) and at Corryong 0.02", 0.07", 0.07", 0.02" and 0.05" (tests 24-28 respectively). The creep effect was greater in the clay which is significant for long-term loadings.

At ultimate test capacity when failure is occurring it is difficult to isolate creep effects with precision. However, values recorded in tests 18, 19, 21, 23 at Keilor and tests 24 to 28 at Corryong are 0.36", 0.54", 0.39", 0.24" and 0.26", 0.22", 0.17", 0.09", 0.06" respectively. Creep could not be isolated in tests 20 and 22 beyond the 0.5" displacement capacity. These results indicate the reduction of the creep displacements as the compaction standard of the backfill increases, particularly in the silty sand materials and also with the use of fine crushed rock in Zone A. The total time taken to reach the ultimate capacities varied from 80 to 110 minutes which is dependent on the loading method.

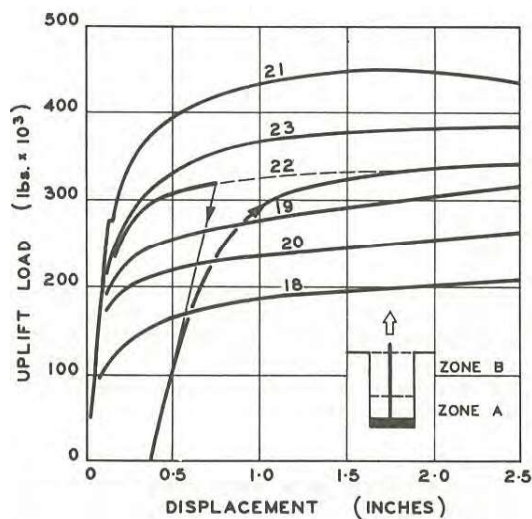


Fig. 4. Concrete Slab Footings in Basaltic Clay. (Keilor, Dec., 1966). Test Nos. 18-23.

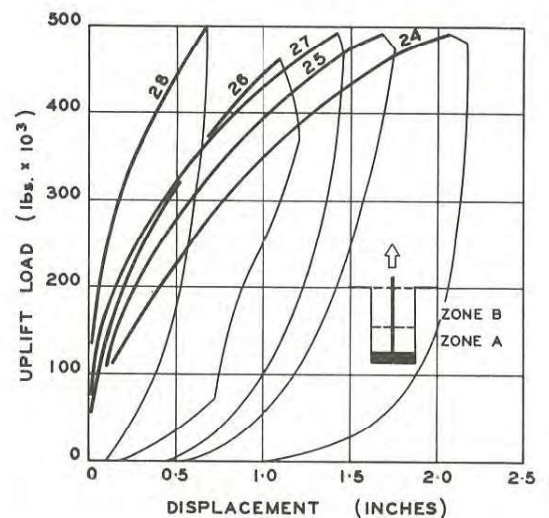


Fig. 5. Concrete Slab Footing in Granitic Silty Sand (Corryong, Nov., 1966). Test Nos. 24-28.

IX.- DISPLACEMENTS WITHIN BACKFILL AND IN SITU MATERIALS

Observations of movements within the backfill and the in situ soils were made to study the compressibility of the backfill and to indicate the location of any potential failure planes. Levels at sub-surface points were taken by means of steel rods set freely into 1" diameter holes. In backfill materials the rods stood on steel plates (12" x 12") which were set at various levels during compaction. For measurements in the in situ soils the lower end of the rods were cast into a handful of cement mortar.

Fig. 6 illustrates the distribution of displacements observed in the clays at Keilor during tests 19 and 23 for slab displacements of 0.8" and 3.0". The effect of increased restraint from the in situ soils can be noted when a 4' depth of fine crushed rock is included in Zone A above the slab.

In test 23 at 3.0" slab movement, a potential failure plane can be interpreted from Figure 6 to commence at the top of Zone A and rise on a 2 on 1 slope to the surface. In test 19 failure can only be interpreted to have occurred along the walls of the original excavation.

Maximum strains of approximately 50% at 3" slab movement were developed within the backfill immediately over the slab (test 19) and above the fine crushed rock (test 23) and reducing to zero at the surface. No strains within the fine crushed rock were observed. In comparing strain-depth plots not shown in the paper, it is apparent that the total strain energy applied to the footing, less shear forces developed at the slab edge, is absorbed in the clay backfill (test 19) and in both the backfill and the mobilised in situ material (test 23).

In comparing displacements in the clay footings without fine crushed rock (tests 18-21), some mobilisation of in situ material was only observed in test 21 in which the backfill was compacted to 93% of

standard. In the granitic materials ultimate capacities of the footing were beyond that of the test loading beam but displacement patterns similar to that shown in Fig. 6 (test 23) were observed in all tests except 24(b) in which case the slab had been pre-loaded and then backfilled with compacted silty sand to 85% of standard.

X.- COMPARISON OF SLAB FOOTING TESTS WITH "CONE OF EARTH" THEORY

The "cone of earth" method is an empirical method for determining the ultimate uplift capacity of a slab footing and relies on the weight of earth lying within a failure plane assumed to commence at the edge of the slab and reach the surface on a plane of slope 2 on 1.

In adopting a mean value for the density (115 lb/ft³) of the backfill and in situ materials at Keilor and Corryong sites, the ultimate capacity of the test footings determined by this method is 340,000 lb. However, to meet the design requirements, displacements should not exceed 0.5 inches.

Only one footing at each site (tests 21 and 28) fulfilled these requirements. However, most of the tests achieved 80% of the cone of earth capacity within the 0.5" displacement limit provided the dry density of the backfill after compaction was greater than about 90% of the standard maximum dry density. Tests 18, 19, 20, 24, failed and were unacceptable for this reason.

The effect of a 4 ft layer of compacted fine crushed rock in Zone A (tests 23 and 28) considerably increased the uplift capacity at 0.5" displacement. The dilatant behaviour of a dense granular material is considered to be important in transferring increased shear forces to the in situ materials.

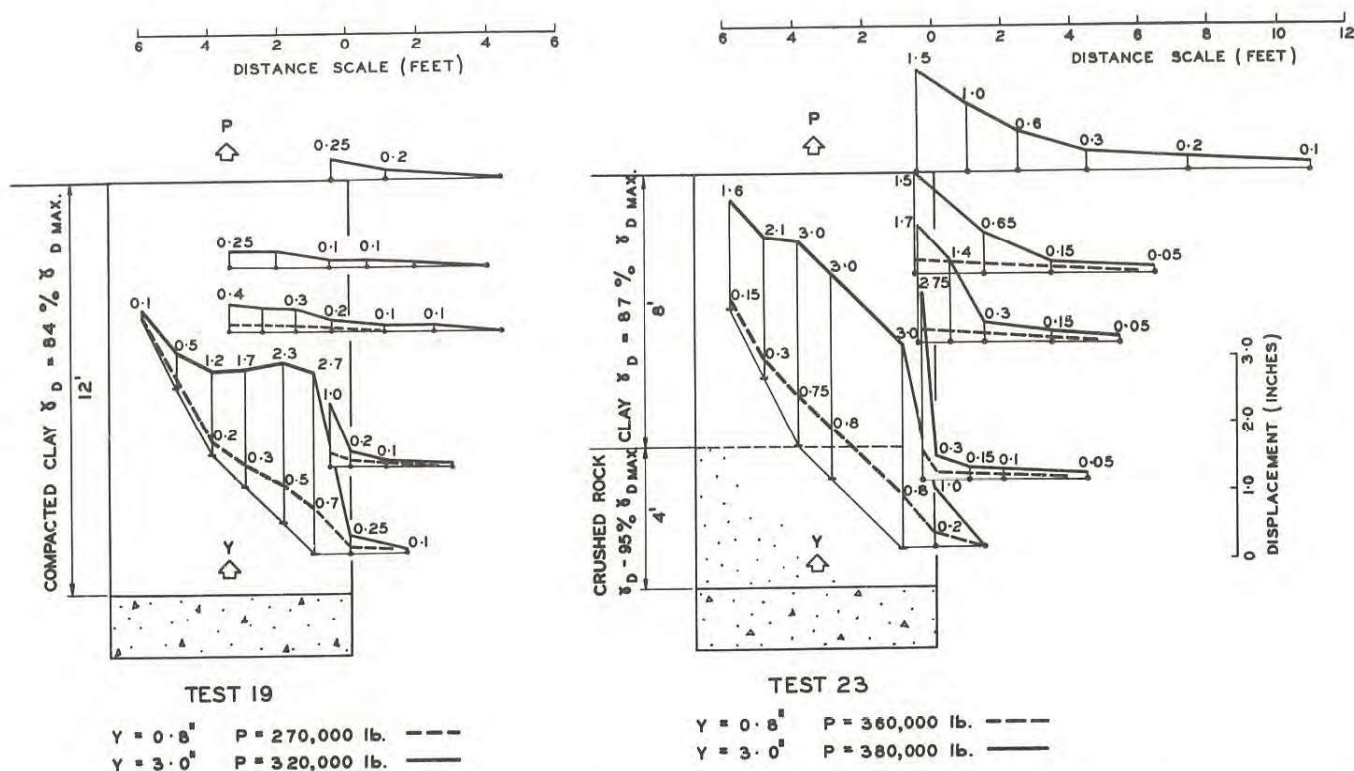


Fig. 6. Concrete Slab Footings in Basaltic Clay. (Keilor, Dec., 1966.) Observed displacements in compacted backfill and in situ soil for slab displacements of 0.8" and 3.0". Displacement values shown at top of ordinates which rise from the sub-surface point observed.

From observations during these tests, failure is interpreted to commence with a shear failure at the edge of the slab with some restraint being offered by the compacted backfill until the weight of the slab is taken up. At that stage the backfill is further compressed and shear forces are developed on the walls of the excavation, less restraint is now being provided by slab shear as lower residual strength values result. As the backfill now becomes fully mobilised it can either move as a plug or induce vertical movement of the in situ soils and at this stage the ultimate level of uplift restraint is reached.

An attempt was made to correlate the test results with a method of evaluation based on the compressibility of the backfill material. Limited agreement was achieved in the footings at Corryong but the method failed when applied to concrete slabs in basaltic clay at Keilor. In the latter case another method of calculation was tried, based on shear stress developed along the sides of the excavation, but this also proved unsatisfactory. Apart from their unreliability, these methods are not favoured on economic grounds because of time and expense required to gather precise data on the characteristics of the soil.

XI.- CONCLUSIONS

In conducting the tests valuable experience and data on the field behaviour of bored piles and slab footings in uplift loading was gained resulting in a

strengthened confidence in extending designs for tower footings to larger capacities up to 500,000 lb.

BORED PILES - The "Tension limited shear" theory consistently overestimates uplift capacity of the bored piles tested in stiff basaltic clay. However, footing capacities based on 80% of the "tension limited shear" resistance provides an adequate basis for determining the design capacity at displacements not exceeding 0.5 inches. The determination of cohesive and indirect tensile strength of the in situ clay can be readily obtained with sufficient accuracy from 1-1/2" diameter samples tested in a standard unconfined compression machine.

Bored piles have little reserve capacity at the design ultimate loading and are not recommended for use in conditions of long-term uplift loading. Some 20% of the design limit of 0.5 inches displacement resulted from the effect of creep. At least 10% loss in the design capacity can result from poorly excavated enlargements at the base of a bored pile.

BURIED SLAB FOOTINGS - The cone of earth method for the design of buried slab footings subjected to uplift loads overestimates the design capacity at the 0.5 inch limit of displacement. The uplift load-displacement characteristics are significantly improved with a higher standard of compaction (or stiffness) of the backfilled soils and a further improvement is gained from fine crushed rock, compacted in a zone immediately above the slab.

For the overall economy and simplicity of design, the capacity of slab footings at 0.5 inches displacement based on 80% of the "cone of earth" resistance is recommended, provided that the backfilling is compacted to a dry density not less than 90% of the standard maximum dry density and that a zone of compacted granular material is placed immediately above the slab. Slight under-cutting of the excavation at the edges of the slab is also recommended.

The "cone of earth" method is limited particularly in the case of loose or variable backfill material. Quantitative extrapolation of footing behaviour beyond the dimensional realms of the tests should be regarded with caution as the influence of scale was not investigated.

A slab footing provides adequate reserve capacity beyond the design capacity at 0.5 inches displacement and is recommended for use in conditions of long-term uplift loading.

XII.- ACKNOWLEDGEMENTS

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APPENDIX 1

"TENSION LIMITED SHEAR"

SUMMARY:

A brief description of the theory by Paterson and Urie (Ref. 2) is given.

NOTATION:

d,	Diameter of Pile Shaft.
d ₀ ,	Diameter of pile base.
h,	Depth of pile.
h _c ,	Critical depth (tension limited shear in soil).
γ,	Soil density.
γ _c ,	Concrete density.
γ _{av} ,	Average density of soil and concrete within failure surface.
f _t ,	Tensile strength of soil.

C,	Cohesion strength of soil.
V,	Uplift resistance of pile.
S,	Uplift resistance due to shear strength of soil.
W,	Uplift resistance due to weight of concrete and soil.

The theory is based on the observation of failure planes developed during uplift testing of bored piles and postulates that the shear strength of a fissured clay in a confined state is not controlled by the shear strength at the fissured surfaces. This is born out by noting that tensile cracks emanate from the cylindrical failure planes during the 1960 test series on cylindrical and under-reamed bored piles. It is suggested that uplift restraint is limited by the tensile strength of the soil rather than adhesion to the pile or shear strength on the failure plane of an under-reamed pile.

In developing the initial stress-strain relationships within the soil for the full depth of pile, the theory assumes the lateral stress level in the soil is raised by the fluid pressure exerted by vibrated concrete during placement and that the earth pressure coefficient K approximates to the ratio of the densities of the concrete and clay, i.e. K = 1.25. In the case of the under-reamed pile, K is adjusted by the ratio of the diameters of the shaft and pile enlargement, i.e. K = 1.25 x d₀/d = 0.75.

During uplift of the pile, shear forces developed along the cylindrical failure plane (projected plane for under-reamed piles) and, if it is assumed that no increase in horizontal and vertical stress levels occur, then the limiting shear stress can be determined from the Mohr envelope for all depths of the pile.

It is shown that at greater depths the maximum shear stress is controlled by the cohesion strength of the clay. Above a critical depth a failure in tension occurs and below this depth failure in shear occurs. i.e. $h = h_c \leq \frac{2}{1+K} \frac{1}{\gamma} (C - f_t)$

The maximum uplift resistance of the pile can be expressed as the sum of the weight of the soil and concrete within the failure surface plus tension limiting shear stress on the failure plain.

$$\text{i.e. } V = S + W$$

where

$$S = \pi d h_c \left(f_t + \frac{1+K}{4} \gamma h_c \right) + C \pi d (h - h_c)$$

and

$$W = \frac{\pi}{4} d^2 h \gamma_{av}$$

For plain piles,

$$K = \frac{\gamma_c}{\gamma}$$

For under reamed piles,

$$K = \frac{\gamma_c}{\gamma} \cdot \frac{d}{d_0}$$