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# Upgrading Existing Footings With Micro-Piles

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SYNOPSIS The use of pressure-injected micro-piles to upgrade the load carrying capacity of existing spread footings and to limit settlement is described. Theoretical pile load capacity and predicted settlement have been verified by a full-scale pile load test.

## PROJECT DESCRIPTION

General Electric Company decided to add a mezzanine floor to an existing building at their manufacturing plant in Tyler, Texas. The two-story building had steel roof trusses on steel columns supported by spread footings. The layout of the existing footings is shown in Fig. 1. The proposed structural framing of the new mezzanine floor was designed as a rigid frame employing existing steel columns to carry vertical loads in addition to providing lateral stability to the building. Analysis indicated that if the existing footings were used to carry the proposed mezzanine loads, the bearing pressure would increase beyond the allowable value of 3,500 psf, and the resulting settlements would be in excess of those which the building frame could safely tolerate. It was, therefore, decided to upgrade the existing foundations by the addition of micro-piles, also known as pin piles.

Before this decision was made, a number of other options were considered. These options included:

a. Improvement of the underlying strata by cement grouting.

b. Underpinning by conventional underpinning techniques.

Any underpinning operation had to be predicated on the following considerations:

1. Low headroom for construction equipment inside the building.

2. Limited workspace due to plant and machinery inside the building.

3. The plant had to be kept in operation during all phases of construction; noise and vibration caused by construction had to be kept to a minimum level.

After taking into consideration the site constraints, local conditions, and geotechnical properties of the underlying soils, an underpinning scheme employing micro-piles, drilled through the existing footings, was selected as the most appropriate means of increasing the footing capacity and limiting settlements.

## SUBSOIL CONDITIONS

The existing spread footings were founded on a stratum of loose silty fine sand extending about 8 feet from ground surface or approximately 5 feet below the bottom of the footings. The allowable bearing pressure used for sizing the footings was limited to 3,500 psf. The standard penetration resistance ranged from 3 to 11 blows. Underlying this loose stratum was a thick layer of medium dense to dense silty fine sand with some clayey material for a depth of 25 feet. The standard penetration resistance for this stratum varied from 16 to 55 blows. Underlying this dense stratum was clayey material having a cohesion value ranging from 1,500 to 4,500 psf. The water table was about 30 feet below first floor level.

It was decided to construct micro-piles to develop full frictional resistance in the dense silty fine sand stratum below the upper loose fine sand layer. The length of piles was varied in direct proportion to the load on the footing, based on the frictional resistance value assumed. The tip of the piles was kept as high as practicable above the underlying clayey stratum so as to minimize long-term consolidation of this stratum under sustained pile loading.

The soils encountered, though somewhat erratic in nature, were predominantly sandy in nature consisting of silty sand (SM), clayey sand (SC) and clayey-silty sand mixtures (SM-SC). The generalized soil profile is shown in Figure 3. The following soil properties were used in the

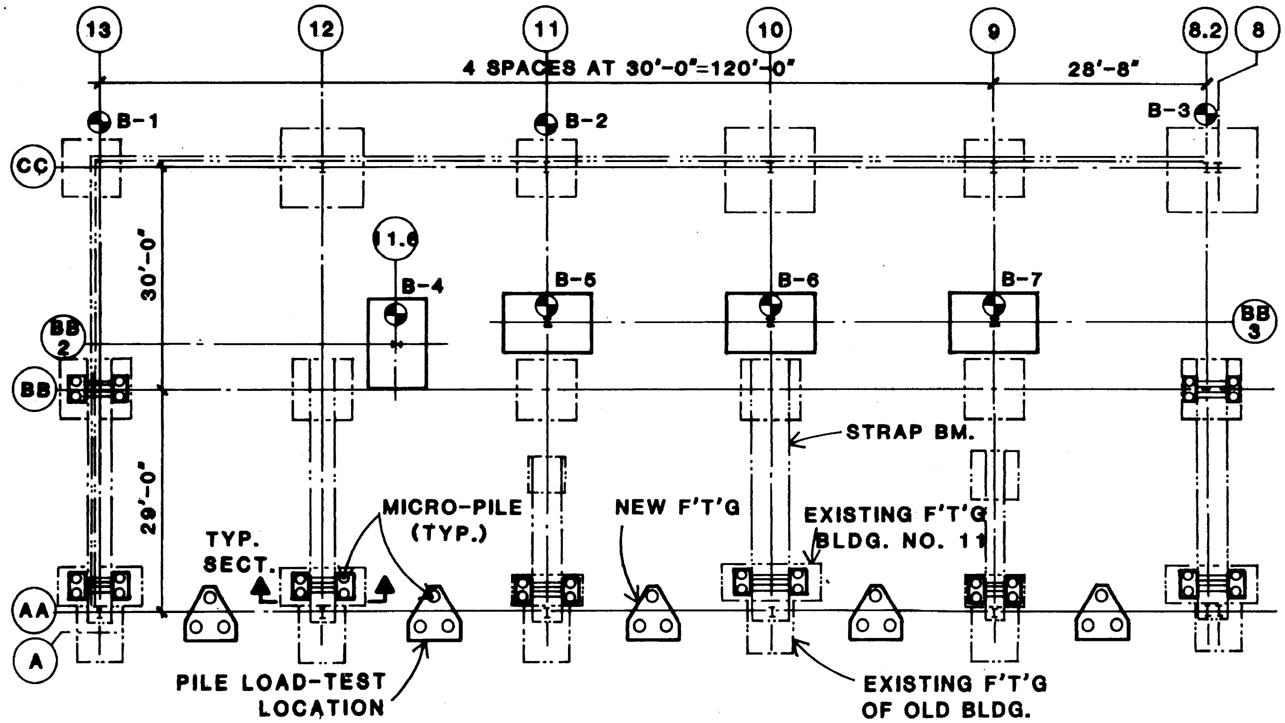



FIG. 1 - FOUNDATION PLAN

N

analysis of the theoretical pile load capacity:

Unit weight of soil, $\gamma$ for
Loose Silty Fine Sand = 105 pcf,
Dense Silty Fine Sand = 120 pcf

Angle of Internal Friction, $\phi$ = 36 degrees

Penetration Resistance Value, N = 30

## MICRO-PILE INSTALLATION

Micro-piles were installed by coring 6-inch diameter holes through the existing concrete footings using a diamond core bit. Below the bottom of the footing the hole was advanced with a 6-inch diameter hollow stem auger to the required depth. The hole was filled with neat water-cement grout using Type I Portland cement. Figure 2 shows the equipment and set-up for drilling and pressure-grouting the hole. A 1 1/4-inch diameter Dywidag steel bar and grout sleeve were placed in the center of the hole before the grout had set. The anchor zone, as shown in Figure 3, was pressure-grouted through the grout sleeve under a low pressure of 50 to 80 psi. Care was taken not to pressure-grout the top 5 to 8 feet portion of the pile immediately below the bottom of the footing, so as to prevent any possible heave of the surrounding floor slab or transmit any load to the top loose stratum. The purpose of pressure grouting the pile was to insure that the grout filled all voids and created an effective cylindrical bulb in the anchor zone, resulting in high values of skin friction between the pile and soil.

The hole in the footing containing Dywidag bar was filled with high-strength, non-shrink grout after the pile grouting operation was completed. 3-inch diameter holes were then drilled through the concrete/steel strap beams and 2-inch diameter through-bolts were set with high strength non-shrink grout. Fabricated steel brackets were then installed as shown in Figure 3 to transfer the load from the piles to the strap beam supporting the existing column. At the location of the new column, a new pile cap was poured directly over the micro-piles. The piles and steel bracket were designed to carry total column load. The arrangement of micro-piles and new footing layout are shown in Figure 1.

## THEORETICAL STATIC LOAD CAPACITY

The design of micro-piles and an estimation of the load capacity were based on the following considerations:

1. In situ soil conditions and standard penetration resistance values.
2. Static analysis predicated on elastic properties of pile and surrounding soil.
3. Pile load test results.

An estimation of the static pile load capacity was first made by considering the micro-pile as a friction pile. Secondly, an analysis was made treating the micro-pile as a tieback anchor. This was felt to be a rational but conservative comparison. Finally, strength evaluation of the pile was considered based on the material properties of steel bar used and neglecting the structural strength of surrounding cement-grout shaft.

### Vertical Load Capacity as a Friction Pile

The ultimate load capacity of a deep foundation is given by Bowles (1977),

$$P_u = P_{pu} + P_{si}$$

where,

$P_u$ = Ultimate load capacity of pile
$P_{pu}$ = Ultimate capacity for end bearing
$P_{si}$ = Ultimate capacity for skin friction

For a friction pile, the ultimate shaft resistance is given by

$$P_{si} = \Sigma A_s f_s \Delta L$$

where,

$A_s$ = Surface area of effective shaft
$\Delta L$ = Length of anchor zone
$f_s$ = Shaft resistance given by:

$$f_s = K(\bar{q} + q_s)\tan\delta$$

where,

$K$ = Lateral earth pressure coefficient
$\bar{q}$ = Average effective overburden pressure
$q_s$ = Surcharge loading
$\delta$ = Effective angle of friction between soil and pile material

For pressure grouted concrete pile, the value of K was assumed to be 1.6; the value of $\delta$ was taken to be 36 degrees. The length of pile required at each column was determined for total column load using the frictional resistance obtained from the above relationships. The length of anchor zone varied from 22 feet to 27 feet for column loads of 130 and 283 kips. A factor of safety of 2.0 was applied to the ultimate load capacity to compute allowable pile load. For a maximum pile load of 30.0 tons, anchor zone length was estimated to be 27 feet.

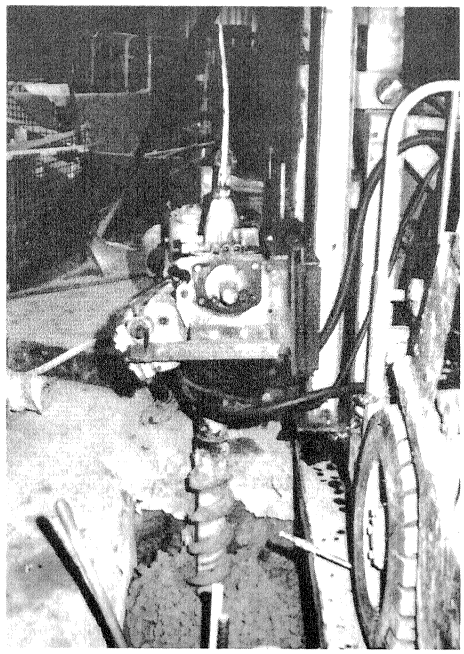


FIG 2. PILE INSTALLATION

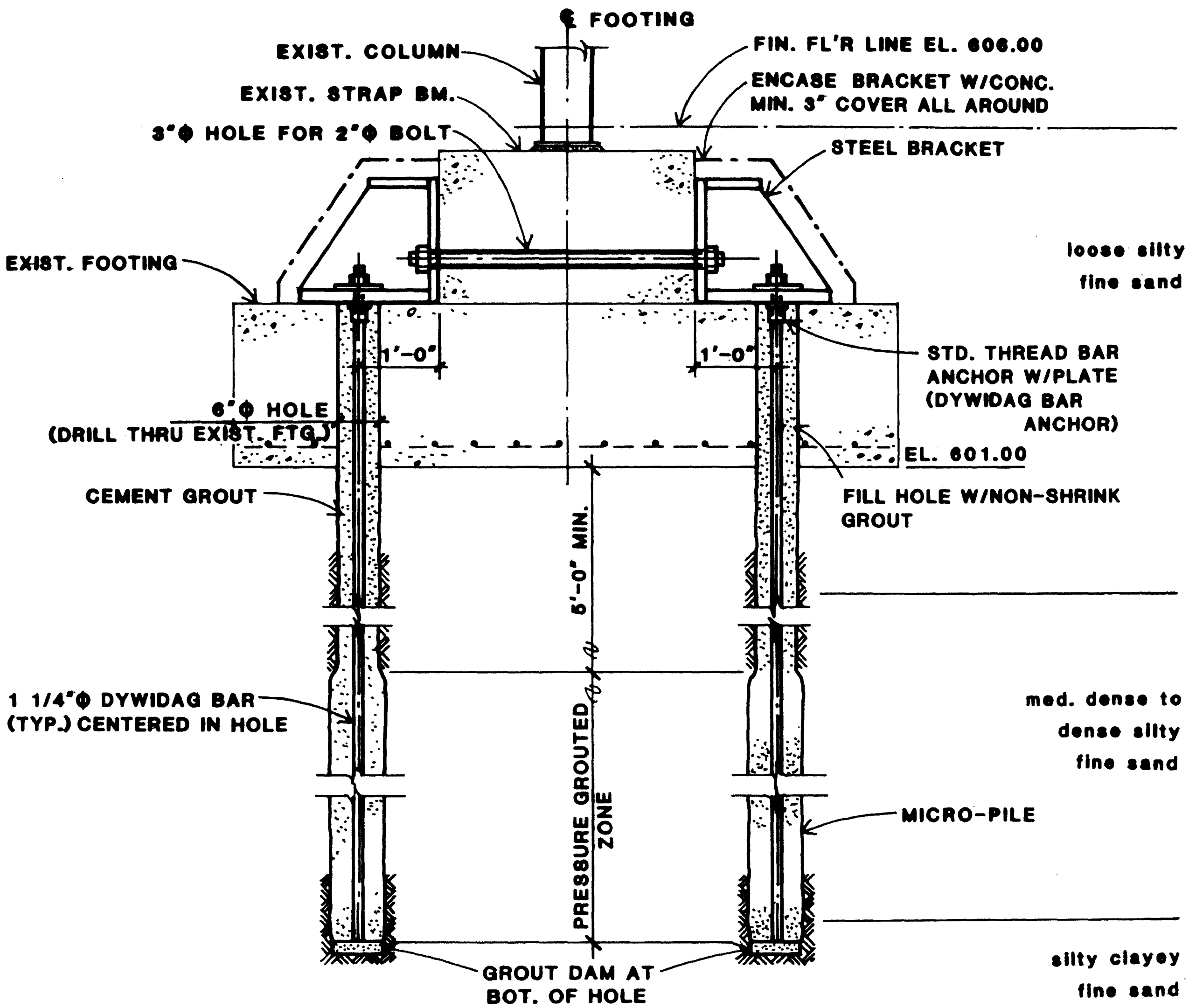



FIG. 3 - TYPICAL SECTION

Vertical Load Capacity Based on Pull-Out

Resistance (Earth Tieback Analogy)

Because the load from the pressure injected micro-pile is transferred to the soil by skin friction, the pile is capable of resisting axial loads in both directions. It can, therefore, be assumed that the load carrying capacity of the micro-pile is at least equal to its capacity as a tension tie. This assumption facilitates the computation of the theoretical load capacity based on empirical relationships for tieback anchors. In this case ultimate pile capacity is given by Goldberg et al. (1976),

$$Put = Pi\,\pi\, ds\, Ls \tan\phi e$$

where,

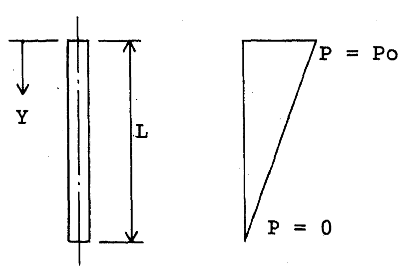
- Put = Ultimate pull-out capacity as tension tie
- Pi = Grout pressure
- ds = Effective shaft diameter
- Ls = Length of anchor zone
- Øe = Effective angle of internal friction

The effective shaft diameter used for computing pullout capacity was conservatively assumed to be 6 inches. With an effective angle of internal friction of 36 degrees, and a factor of safety of 2.0, the pullout values obtained were 25 to 40 percent more than the compression values for the same length of anchor zone used in pile load capacity computations.

THEORETICAL ESTIMATION OF PILE SETTLEMENT

An estimate of settlement of the piled footing was made using elastic theory assuming soil to be semi-infinite, homogeneous and elastic material. In general, the settlement of a pile footing is caused by (a) elastic deformation of pile and soil; (b) elastic and plastic deformation of soil due to consolidation.

The elastic deformation of pile and soil was computed by considering triangular distribution of the pile load along the shaft with a maximum value at the top and zero value at the bottom of the shaft as shown below.




PILE    LOAD DISTRIBUTION

The Dywidag bar alone was considered effective for computing the elastic shortening; the cement-grout shaft being neglected. For the load distribution shown above, the elastic shortening was obtained by,

$$e = \frac{0.5\ Po\ L}{AE}$$

where,

- e = Elastic shortening, inches
- Po = Pile load, kips
- L = Length of pile, inches
- A = Area of Dywidag bar, inch-2
- E = Modulus of elasticity of steel, ksi

The average elastic settlement computed for a maximum pile load of 30.0 tons and anchor length of 27 feet was 0.25 inches.

An estimation of the consolidation of soil below the pile tip was obtained by elastic analysis as given by Bowles (1977), considering stress distribution below the pile tip extending to a depth equal to the length of the anchor zone. Stress coefficients were obtained by assuming a linear variation of skin friction along the shaft. Poison's ratio for soil was assumed to be 0.3 and static stress-strain modulus was assumed to be 1,500 ksf. The settlement under total load was calculated to be 0.24 inches. Total elastic and consolidation settlement for the pile was estimated to be 0.49 inches. Long term plastic soil deformation was not calculated.

PILE LOAD TEST

A compression load test was performed on a single pile at the location of new column AA-11.5. The load test was performed in accordance with the requirements of ASTM D1143. Figure 4 shows plan view of test set-up while details of the load test are shown in Figure 5. The test pile and three anchor piles were installed by drilling 6-inch diameter holes in the ground and pressure-grouting as discussed earlier under micro-pile installation. The total length of test pile from the first floor was 37.5 feet and anchor piles extended to a depth of 60 feet below floor slab. The bottom 15 feet length of anchor piles was pressure-grouted to form the anchorage bulb at the bottom of the pile. The test pile was grouted for a length of 26 feet.

The test load on the pile was applied in five equal increments of 12.0 tons each for a maximum test load of 60.0 tons corresponding to maximum pile load of 30.0 tons. The first two increments of load were held for a period of 60 minutes each, while readings were taken at intervals of 1, 2, 5, 10, 30 and 60 minutes. Since the rate of settlement was less than 0.01 inch per hour, a shorter duration of loading (minimum 60 minutes, however) between each load increment was maintained. The third load increment at a test load

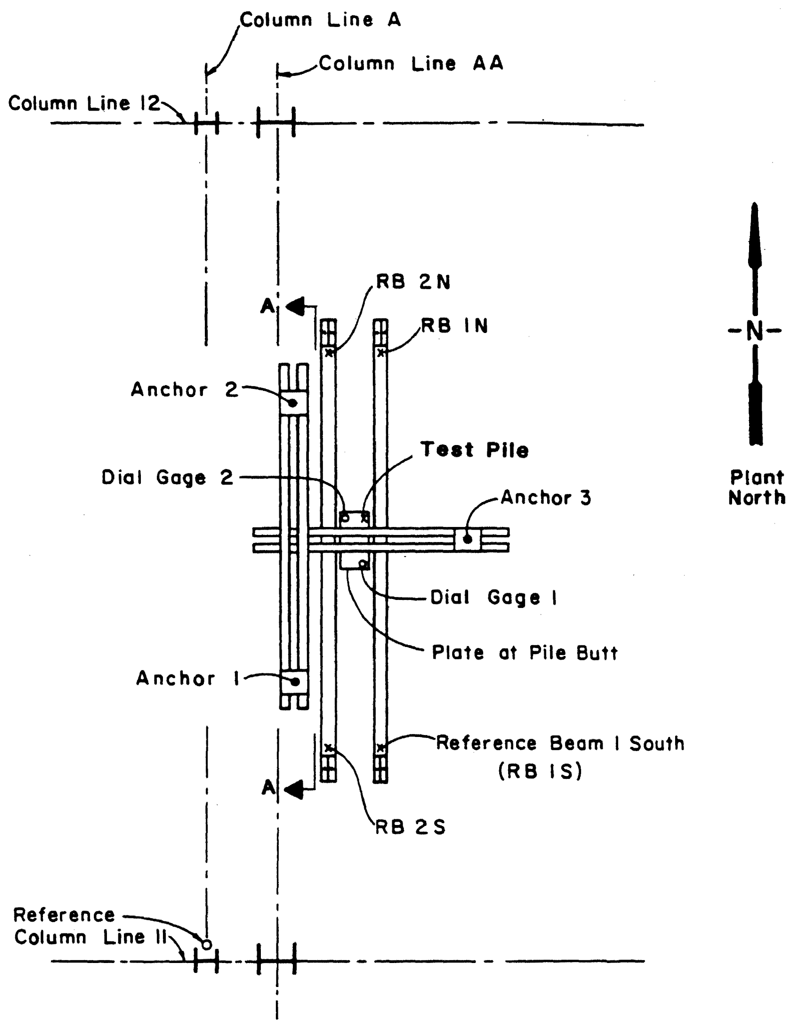
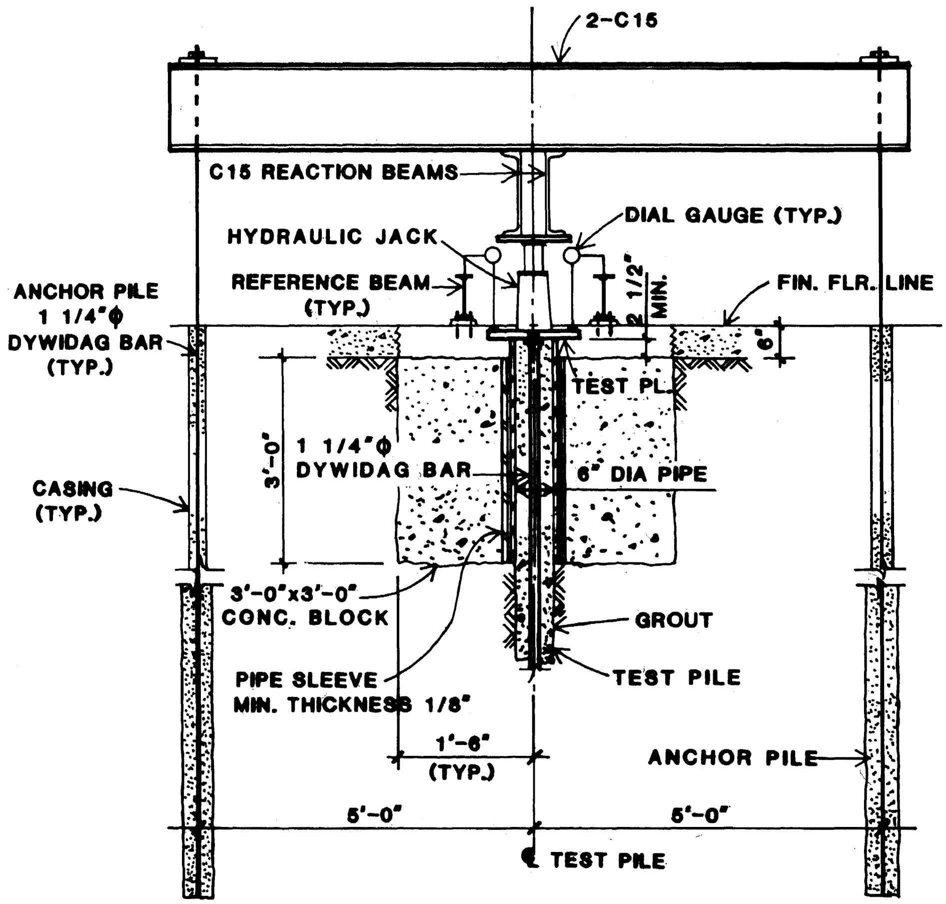



FIG. 4 - PLAN VIEW OF TEST SETUP




A - A

FIG. 5 - DETAIL AT TEST PILE

of 36.0 tons was held for 78 minutes while the fourth load increment for a test load of 48.0 tons was held for a period of 120 minutes. The final test load of 60.0 tons was held for a period of 19 hours and 41 minutes. The final settlement reading under test load of 60.0 tons was 0.432 inches. The rate of settlement at the end of this time period was 0.00061 inch per hour.

The unloading was carried out in five equal load decrements of 12.0 tons each at the rate of 60 minutes between each load decrement. Dial gage readings were taken at 1, 2, 5, 10, 30 and 60 minutes for each stage of unloading. The final settlement at the end of the test with all of the test load removed was 0.158 inches.

In addition to the dial gage readings taken on top of the cap plate on two opposite sides of the test pile, level readings were taken at each anchor pile and at ends of reference beams from a fixed reference point.

The tieback anchor piles performed well under test load of 60.0 tons. Except for heaving up of loose grout sticking to the tension bar around one of the anchors, there was no visible movement of ground around the anchor piles or the test pile.

## LOAD TEST ANALYSIS AND DISCUSSION

The results of the compression load test are shown in Figure 6, where load versus displacement curves have been drawn for the loading and unloading cases. The maximum downward displacement under a test load of 60.0 tons was 0.432 inches and the final reading after the test load was removed was 0.158 inches resulting in an elastic recovery of 0.274 inches. The shape of the load-displacement curve indicates that the failure load was not reached at the test load of 60.0 tons. The allowable load on the pile was estimated by two methods as described below.

As shown in Figure 7, the failure load in this case was obtained by Davisson's method (1975), by drawing a line parallel to the elastic curve of the pile at a displacement of 0.15 + 0.1*B inches, where B was the width of pile in feet, assumed to be 6 inches in this case. The elastic curve of the pile was obtained from the relationship $e = PL/AE$, where P was the pile load in kips; L, the length of pile in inches; A, cross-sectional area of the bar in square inches; and E, modulus of elasticity of steel in ksi. Since the load-displacement curve, based on total length of test pile, did not intersect with the elastic curve, a reduced length of one-half the pile length was used for the graph of Figure 7. Failure load was estimated to be 70.0 tons by this method.

In the second method, failure load was obtained from creep load based on time rate of settlement versus test load. Figure 8 shows a plot of slopes, obtained from displacement and log-time for various stages of test loads, and applied loads. The creep load was obtained from the resultant smooth curve by drawing tangents to the two portions of the curve. The allowable pile load equal to 36.8 tons was taken to be 80 percent of the creep load. The minimum allowable load obtained from pile load test was greater than 30.0 tons, the maximum load on any pile. The maximum displacement of pile under a working load of 30.0 tons was 0.11 inches as obtained from the load-displacement curve shown in Figure 6. This displacement was much less than the pile settlement calculated based on the theoretical length of the anchor size. Based on the results of the pile load test, it was possible to reduce the length of the theoretical anchor zone. However, due to the sensitive nature of the project with respect to differential settlements, micro-piles were installed as per the theoretical lengths.

## SUMMARY AND CONCLUSIONS

A case history is given for underpinning the foundations of an existing building using small diameter pressure-injected piles known as micro-piles.

As a result of a mezzanine addition to an existing building, loads on existing footings increased more than those stipulated at the time of original design. The new loads varied from 130 kips to 283 kips. These loads in some cases were in excess of those that could be safely supported by the subgrade under the spread footings. Settlement analysis indicated that the long term settlements would be in excess of 1.0 inch. These predicted settlements were considered to be detrimental to the integrity of the proposed rigid structural steel framing attached to the existing steel framing. A number of alternatives for strengthening the underlying strata and upgrading the existing footings were considered. Underpinning of the existing footings by the use of micro-piles was adjudged to be the most economical and feasible alternative for this site. The installation of micro-piles was carried out inside the building within the available headroom and without disruption of continuous plant operation. The need to excavate deep pits adjacent to heavily loaded footings was eliminated. By varying the length of anchor zone and number of piles under each footing, it was possible to develop uniform load on each pile group so as to minimize differential settlements between adjacent footings.

A full-scale load test was conducted to estimate settlement of the pile under proposed load and to predict allowable load on the pile. Results of the load test indicated that the length of piles as determined from theoretical analysis could be reduced without causing excessive settlements. However, due to the sensitive nature of the proposed framing, piles were installed to the depths obtained from theoretical analysis.

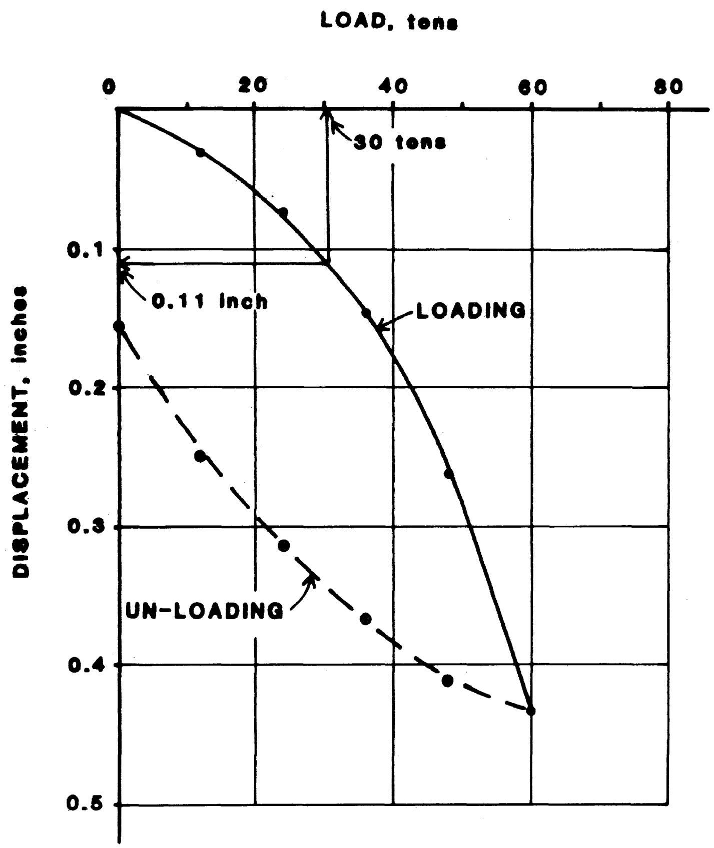



FIG. 6 - COMPRESSION TEST

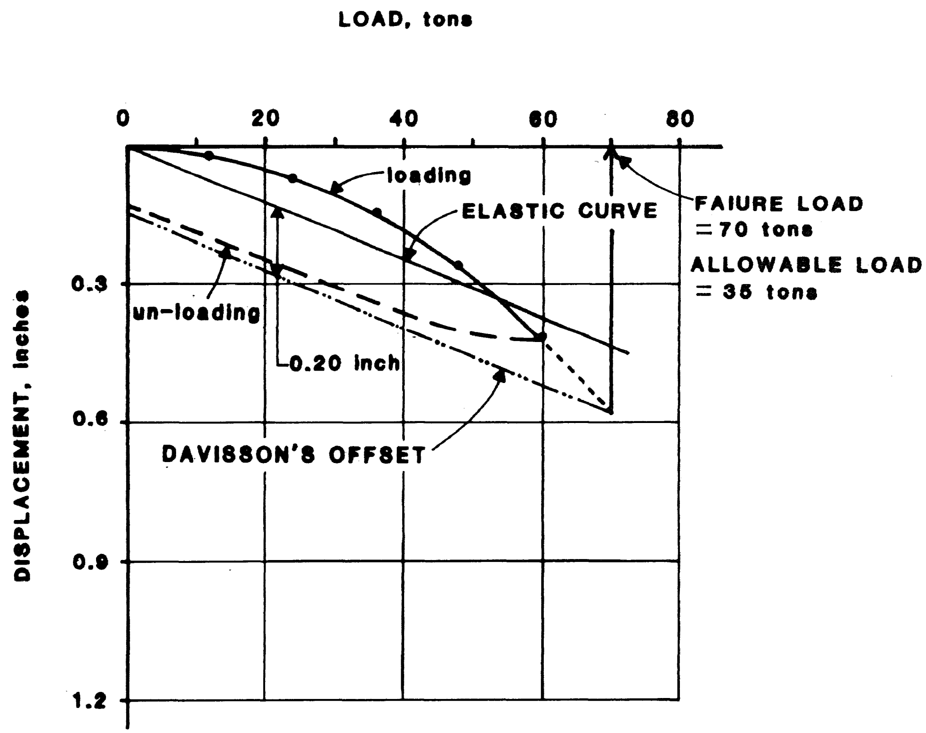



FIG. 7 - FAILURE LOAD BY DAVISSON'S METHOD

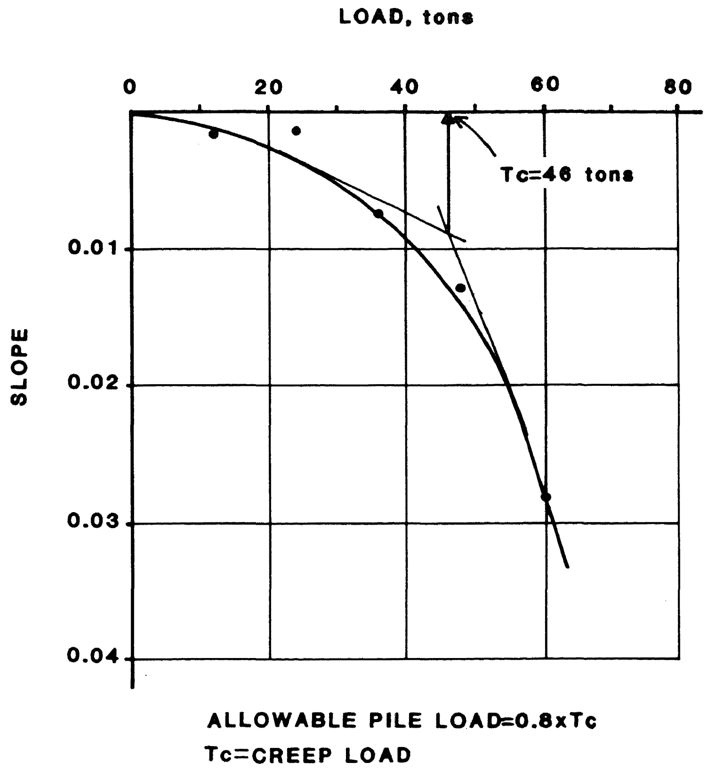



FIG. 8 - PLOT OF LOAD Vs. SLOPE

Micro-piles can be effectively used to underpin existing footings under constrained conditions. They can be installed rapidly and economically with small equipment. It is possible to predict the allowable load capacity of micro-piles by conventional analysis augmented by standard pile load test.

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