Field test to evaluate load transfer for soil anchors

Essai in-situ pour estimer le transfert de charge des tirants dans les sols

G.M.DENBY, Associate, GeoEngineers, Inc., Bellevue, Washington, USA
A.A.BANGASH, Graduate Student, Washington State University, Pullman, Washington, USA
J.E.BRIGHAM, Project Engineer, GeoEngineers, Inc., Bellevue, Washington, USA
R.FRAGAZSY, Associate Professor of Engineering, Washington State University, Pullman, Washington, USA

SYNOPSIS: A field test anchor program was undertaken to identify the load transfer mechanism responsible for the variability in soil anchor capacities. The test program consisted of monitoring the stresses, displacements and pore pressures around four anchors before, during and after the grouting operation. Measurements of downhole and pore pressures indicate hydrostatic pressure at the end of grouting. The anchor capacities are low and related to in-situ stress. Empirical formulae predict enhanced anchor capacities that imply radial stresses on the anchor several times the in-situ stress.

INTRODUCTION

The Washington State Department of Transportation has installed several permanent tieback walls along the I-5 highway through Olympia, Washington. These walls retain up to 14m of medium dense to dense fine sand and are designed as soldier pile walls with straight shaft anchors that are grouted using pressures ranging up to 1000kPa. During installation of these walls, several of the anchors failed at approximately 60 percent of the design capacity. The subsequent investigation concluded that the failures were due to inadequate grout pressure during installation. This raises questions of what is "adequate" grout pressure and if the grout pressure enhances anchor capacity by increasing the radial stress on the anchor, can the radial stress be reduced by densification of the soil during an earthquake?

The validity of the second question depends on whether the radial stress is in fact increased by grout pressure. Other load transfer mechanisms to account for the enhanced anchor capacity include physical alteration of the grout by enlargement, permeation or grout "fingers;" increase in adhesion; dry packing of the grout; and dilation during loading.

A field test program was undertaken to monitor the stresses, displacements and pore pressures in the vicinity of four field test (FT) anchors before, during and after grouting. The performance of the FT anchors is compared with two failed (P1) and four successful (P2) proof anchors from a nearby project.

PROJECT DESCRIPTION

The Capitol Boulevard Project is located in a 18.3m high 1-1/2 horizontal to 1 vertical cut on the Interstate 5 freeway in Olympia, Washington. The field test site is located 120m south of this project on the west side at the toe of the cut. The soil profile consists of medium dense to dense uniformly graded fine sand with occasional zones of interlayered sand and silt (Figure 1). The water table is deep. Standard penetration blow counts are typically 40 to 50 at the anchor elevation. The moist density is 2000kg/m^3; percent fines is 5 to 15 percent; angle of internal friction is 35 degrees.

All anchors were installed at 15 degrees to the horizontal using a Klemm drill rig and 0.133m outside-diameter casing. The cuttings were air flushed internally. The hole was grouted continuously as the casing was withdrawn. The cement/ water grout had a water cement ratio of 0.45 and a density of 1700kg/m^3. Typical grout takes (ratio of pumped grout to theoretical hole volume) were 150 percent and were relatively constant.

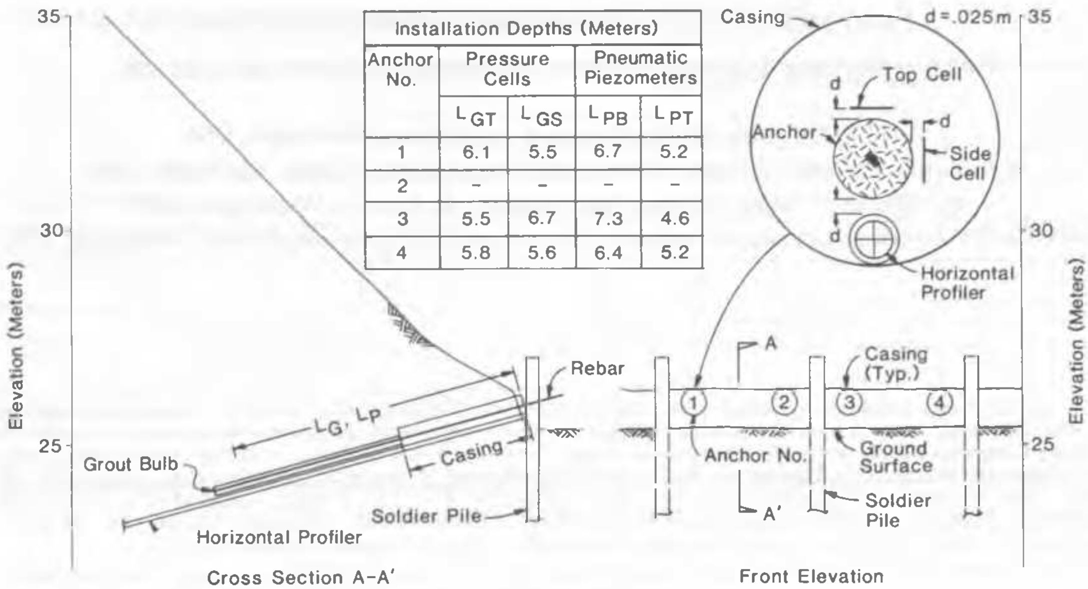
The two (P1) anchors were grouted with a screw-type pump operating at gauge pressures of 340 to 680kPa. The two anchors had a bond length of 13m and a total length of 18m and failed at 650kN and 800kN. Subsequently, four (P2) anchors with bond lengths of 15m and total lengths of 20m were grouted with a piston-type pump operating at gauge pressures of 1030 to 2060kPa. All four anchors were tested to twice design load of 1070kN with no sign of failure.

FIELD TEST ANCHOR PROGRAM

The FT anchors were installed in the same soil and using the same grout mix and anchor installation techniques as the P1 and P2 anchors, except that withdrawal of casing and grouting was stopped every 0.3m to allow recording of data. The piston-type pump was used at gauge pressures of 680 to 1030kPa. The holes appeared to be self supporting after 2m.

The test layout consisted of three soldier piles and four steel casings 0.6m in diameter and 3m long at 15 degrees to the horizontal (Figure 1). The instrumentation and subsequently 4.6m long anchors were installed concentrically with the casing.

The instrumentation consisted of a horizontal profiler, two pneumatic piezometers and two push-in Glotzl total pressure cells for each anchor as shown in Figure 1. The piezometers



Anchor No.	Pressure Cells		Pneumatic Piezometers	
	L_{GT}	L_{GS}	L_{PB}	L_{PT}
1	6.1	5.5	6.7	5.2
2	–	–	–	–
3	5.5	6.7	7.3	4.6
4	5.8	5.6	6.4	5.2

Figure 1. Field Test Layout

were fastened to a Dywidag bar. The pressure cells were inserted in 0.3m increments.

FIELD TEST ANCHOR RESULTS

The soil pressures before grouting measured by the pressure cells are shown in Table 1. The measured pressures increased steadily during insertion and once inserted, decreased. Prior to drilling the average pressure measured by 3S and 4S (horizontal stress) was 180kPa and by 3T and 4T (vertical stress) was 119kPa. For a level ground surface the vertical stress is calculated to be 107kPa. The pressure cell data indicate a K_0 greater than 1.6 which is possible for this deposit. The pressure cell data may also be high due to high cell stresses during insertion and consequent zero shift.

Table 1 - Pressure Cell Measurements kPa

Anchor Number, Cell Location	Immediately After Insertion	Start of Grouting	After Grouting
1 Side	214	77	89
2 S	390	369	--
3 S	255	181	232
4 S	207	179	157
3 Top	110	121	122
4 T	152	118	139

The downhole grout pressures measured by the two pneumatic piezometers in FT1 during grouting and the gauge pressure at the grout pump are shown in Figure 2. The pressures are plotted versus the casing depth during extraction. The plots should therefore be read from right to left to track the pressures during grouting.

The pressures were measured before and after each 0.3m increment of casing extraction. "Before" readings should correspond to the residual pressure in the grout after pumping stopped and "after" readings should correspond to hydrostatic grout pressure. Peak readings were also made during grouting. The readings represent a lower bound on the grout pressures as they were taken before the gauge stabilized.

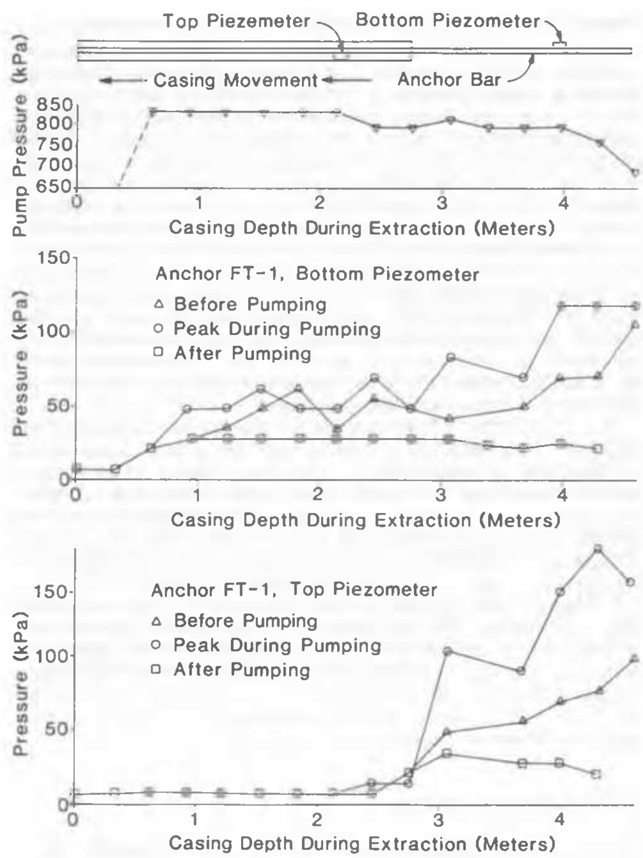


Fig. 2 Grout Pressures During Installation

FT1 bottom piezometer "after" readings indicate a hydrostatic grout pressure of about 28kPa during grouting. This is consistent with calculated hydrostatic pressures considering the grout in the casing. The top piezometer "after" readings indicate a value of about 10kPa during he latter stages of grouting. The reason for his is unclear. The "before" readings indicate variable pressures ranging up to 140kPa. The peak readings ranged up to 140kPa for the bottom piezometer outside the casing and 170kPa for the top piezometer while still inside the casing. The bar stuck inside the casing in FT3 and the piezometers were also extracted.

A test was subsequently conducted to validate the piezometer readings. A small flow of grout was pumped through the casing with a restrictor valve at the end of the casing. Piezometers placed in the casing correctly measured the average pump pressure.

The reason for the low grout pressures is considered to be grout flowing upwards between the casing and the hole.

The earth pressures measured during grouting are presented in Figure 3 in the same format as Figure 2. The earth pressures plotted correspond with the "after" piezometer readings and represent the net change in pressure from the initial reading prior to grouting. The measured earth pressures are less than 50kPa and are highly variable.

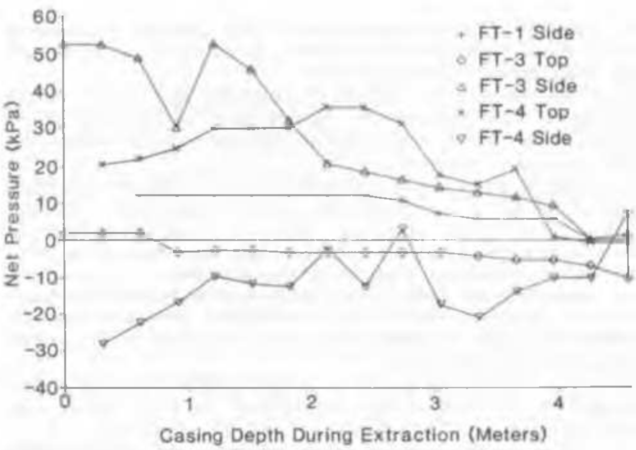


Fig.3 Net Pressures Measured by Pressure Cells

The anchor displacements measured by the horizontal profiler were observed to be less than the sensitivity of the device.

The anchor capacities of FT1 and FT3 were 120kN and 100kN, respectively. Transfer of load from the anchor to the casing resulted in an invalid test for FT4. The load/displacement curves for the FT and P1 anchors are shown in Figure 4. The capacities are significantly lower than expected and correspond to 26 and 52kN/m for the FT and P1 anchors, respectively. The P2 capacities are in excess of 71kN/m.

Visual inspection of the exhumed FT anchors indicated very little expansion of the anchor. The anchors were covered with a uniform 0.002m layer of cemented fine sand.

Significant physical enlargement should occur when the grout pressure exceeds the limit pressure. Baguelin (1978) indicates the limit pressure for sand at this depth is about 690kPa. Thus, the 690 to 1030kPa grout pressure should have caused physical enlargement of the anchor hole. The lower pressures measured by the piezometers are consistent with no enlargement.

The grout density was measured to be 1780kg/m^3 indicating a slight decrease in water content from the initial value of 1700kg/m^3. Permeation of the grout has been investigated by Scott (1964) in terms of standard filter criteria. The .002m cemented sand layer observed is consistent with Scott's findings.

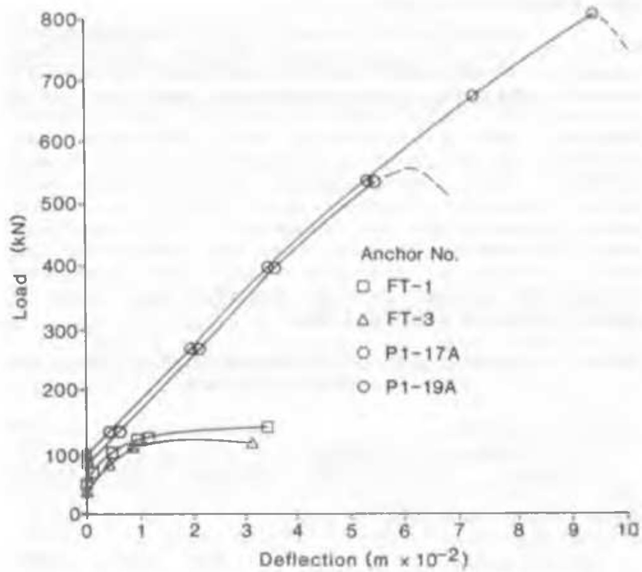


Fig. 4 Load Test Results

ANCHOR CAPACITY-CALCULATED VS OBSERVED

There are essentially three methods to calculating anchor capacity in sand - two rational and one empirical. The first method is based on grout pressure and the second on in-situ stress. The first method is generally used for high grout pressure anchors, although it may also be appropriate for tremied anchors. In the second method, the in-situ stress may be any combination of the vertical and horizontal stresses. Both approaches base the capacity on the anchor perimeter area, the grout pressure or the in-situ stress σ_o, and the soil strength. The third method is based on the anchor length, an empirical constant, N^1, and the soil strength. Nicholson (1978) recommends a value of N^1 from 130 to 167kN/m for grouted anchors of 0.18 to 0.20m diameter where the grout pressure is not sufficient to physically expand the anchor size. When Nicholson's average values for diameter and N' are substituted into the second method, the back-calculated in-situ stress is 250kPa.

Calculated capacities for the FT, P1 and P2 based on the three different methods are presented in Table 2. For the grout pressure method the normal stress on the anchor is the grout hydrostatic pressure. The grout pressure method significantly underestimates the observed capacities for all the anchors

The in-situ stress method slightly over-predicts the observed capacities for the FT and P1 anchors, but appears to under-predict the P2 anchors. The empirical method significantly over predicts the observed capacities for the FT and P1 anchors, but may be reasonable for the P2 anchors. It may be concluded that the in-situ stress method provides the best agreement with the FT and P1 anchors which may be considered low pressure anchors. The empirical method predicts significantly higher capacities which may be justified by the P2 anchors.

LOAD TRANSFER MECHANISM DISCUSSION

Based on the low observed anchor capacities relative to the empirically predicted values, it appears that the grout pressures were not high enough to yield the enhanced capacities. However, the piezometric data indicate that downhole peak grout pressures were at least 140kPa. Thus, there appears to be a threshold grout pressure, higher than 140kPa, at which anchor capacities are enhanced. It is unclear what the change in load transfer mechanism is, but a review of the potential load transfer mechanisms noted in the introduction leads to some tentative conclusions.

Table 2 - Comparison of Observed Anchor Capacity with Evaluation Method

Anchor Capacity Evaluation Method	Method Description	Anchor Capacity kN FT L=4.6m	P1 L=13m	P2 L=15m
Grout Pressure	$Q=L\pi DP_G\tan\phi$			
a) During Installation		56	220	280
b) After Installation		13	130	190
In-situ Press.	$Q=L\pi D\sigma_m\tan\phi$ σ_m-vert.stress =( )kPa	150 (110)	870 (220)	1090 (240)
Empirical (low pressure)	$Q=LN'\tan\phi$ $N'=150kN/m$	480	1400	1600
Observed	--	120	650-800	>1070

Note: P_G=Grout pressure
ϕ=Angle of internal friction
L=Bond length of anchor
D=Diameter of anchor

Enlargement of the anchor is possible though considered unlikely. The size of the field test anchor would need a fourfold increase in diameter to increase the observed capacity to the empirical value (120 to 490kN).

Enlargement of the anchor by grout permeation is considered unlikely in this soil due to its low permeability. Very little permeation occurred in the field test anchors based on the 0.002m layer of cemented sand.

An increase in adhesion at the grout/soil interface is not able to yield a fourfold increase in capacity.

Formation of grout "fingers" may relate to more pervious layers or hydraulic fracture. This mechanism does not appear to be applicable in this case based on capacities that were obtained for other comparable anchors.

Increase in Radial Stress: The piezometers installed in the FT anchors indicate that the grout remains essentially fluid during installation. The radial stress should therefore be close to the hydrostatic grout pressure which is about 10kPa. However, based on the rational method, the effective radial stress at failure is 10 times this amount and for the enhanced capacity is 30 times this amount.

The radial stress must therefore increase either before loading, due to stress readjustment/creep around the anchor and/or during loading, due to dilatant behavior. The capacities of both the FT and P1 anchors are consistent with the radial stress increasing to the in-situ stress.

The enhanced capacities indicate the radial stress increases above the in-situ stress. For this to occur without physical alteration of the grout body, the grout must stiffen sufficiently to lock in the increased stress before the end of grouting. This is commonly known as dry packing or "flash set" and occurs when water is extruded from grout under pressure.

CONCLUSIONS

The anchors discussed in this paper were installed in a dense fine sand deposit. The anchors were installed using the casing method and low grout pressures. The anchors are essentially shaft anchors with minimal physical alteration during grouting. The conclusions are limited to these conditions as follows:

1. Downhole measurements of grout pressure indicated hydrostatic grout pressure at the end of the anchor installation.
2. The measured grout pressures are too low to support the observed capacities which are more consistent with the radial stress being equal to the in-situ stress.
3. Based on visual inspection of the FT anchors and review of possible load transfer mechanisms, dilation and/or stress readjustment is considered responsible for increasing the radial stress to the in-situ stress.
4. Empirical formulae indicate enhanced capacities equivalent to a radial stress up to several times higher than the in-situ stress for low grout pressures.
5. The load transfer mechanism for enhanced capacity at low grout pressure is not clear but may be due to "dry packing" of the grout.
6. The question of the effect of earthquake loading on anchor capacity still appears to be relevant and further study is planned.

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