PERMANENT GROUND ANCHORS
Nicholson Design Criteria

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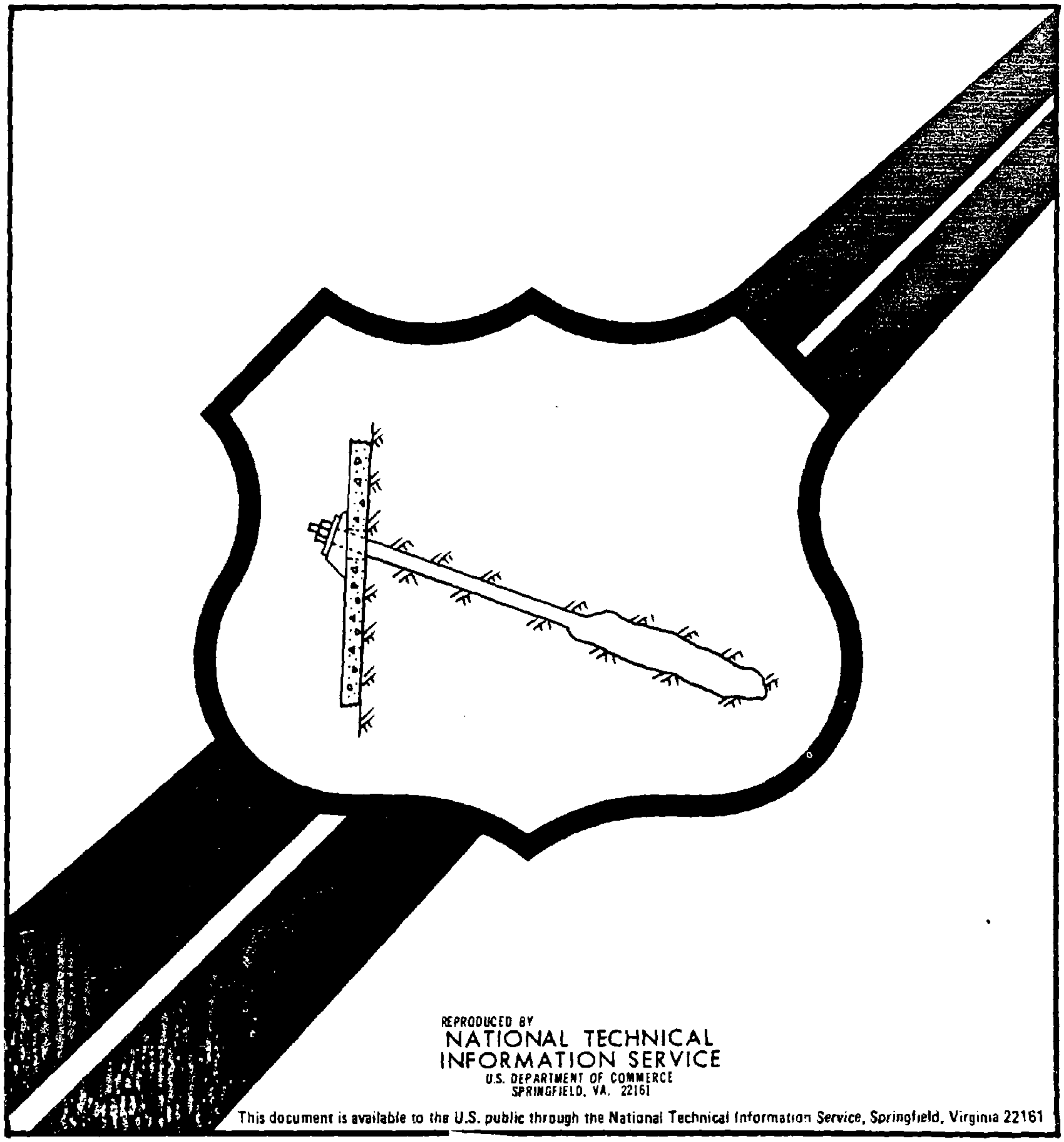

FOREWORD

This is one of three reports on the design of permanent ground anchors, written by the following internationally prominent ground anchor contractors:

Soletanche & Rodio, Inc.
Nicholson Construction Company
Stump/Vibroflotation

These reports are being used by the Federal Highway Administration in developing a design manual for highway engineers.

The design methods described herein were originally developed by the authors and company staffs for the sole use of each company. We are grateful to the company officials for sharing their design methods with us.

Copies of this report are being distributed by FHWA transmittal memorandum. Additional copies may be obtained from the National Technical Information Service, 5285 Port Royal Road Springfield, Virginia 22161.

Richard E. Hay, Director
Office of Engineering
and Highway Operations
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NOTICE

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16. Abstract

This study discusses the methods used by Nicholson Construction Company in the design of permanent ground anchors specifically as related to retaining walls. Basic soil parameters, design concepts, drilling and grouting methods for ground anchors are discussed. Particular emphasis is placed on anchors founded in soil rather than rock formations. Also, soil properties necessary for the proper design of anchored retaining walls are detailed.

The second chapter of the report is devoted to a general discussion of retaining wall and anchor design. In addition, a design example of an anchored retaining wall is presented in a step by step manner.

Three documented case histories of anchored retaining wall projects are presented. All are permanent walls constructed by different methods (steel sheet pile, H-beam and timber lagging with concrete facing, and tangent reinforced concrete drilled shafts) and all are supported by permanent ground anchors.

Companion reports on the design of permanent ground anchors are:
FHWA/RD-81/150 Soletanche Design Criteria
FHWA/RD-81/152 Stump Design Criteria

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PREFACE

The introduction and development of ground anchors as a civil engineering technique has proven to be one of the most important innovations for many years in the construction industry. Over the last twenty-five years significant advances have been made in both the theory and practice of anchor construction so that they no longer are regarded as temporary contract expedients but much more as tools of the design engineer, no more the method of last resort but a fundamental technique to be considered at the time of design. Permanent installations are now commonplace and are to be found incorporated into such major structures as dams and similarly critical situations. Specific uses are far too numerous to mention here and the potential for new applications is truly enormous. Suffice it to say that any problem involving tying up, tying down, tying back or generally anchoring in place is one that can be considered with the use of the new technology in mind.

But its use requires a basic understanding of the theoretical and practical considerations governing anchor installation and behavior. This report presents a review of the type of information considered necessary to design permanent anchor systems and the way in which that information is used by engineers and specialist contractors to determine anchor configuration and capacity.

It must be emphasized that assessment of anchor load carrying capacity is still in many cases affected by matters of engineering judgement and construction experience. This is reflected in the empirical formulae that are often used in the calculations. However, these have been tried and tested in the field and have, to a large extent, been proven to be conservative. They are changed or modified in the light of new experience or proof but generally serve well. As such they are quoted herein and form the basis of changing an art into much more of a science.

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APPLICABLE SI UNIT CONVERSIONS

	English		SI
LENGTH:			
	1 in.	=	0.0254 m
	1 ft.	=	0.3048 m
PERMEABILITY:			
	1 ft/sec	=	30.48 cm/sec
FORCE:			
	1 lb f	=	4.448 N
	1 kip f	=	4.448 kN
	1 ton f	=	8.896 kN
PRESSURE:			
	1 psf	=	47.88 Pa[1]
	1 psi	=	6894.76 Pa
	1 psi	=	0.006894 N/mm^2
UNIT WEIGHT (γ)[2]:			
	1 pcf	=	0.157 kN/m^3

[1] Pa = N/m^2

[2] In this report, the term density is used interchangeably with unit weight.

CHAPTER I - BASIC CONCEPTS

INTRODUCTION

As ground anchors can be constructed at any angle and in a wide variety of soils and rocks they are used for a multiplicity of purposes. One of their major uses is in the support of retaining walls and this study has been made with this particular usage in mind, although the basic concepts apply to all anchors.

Temporary anchors are not reviewed. However, the data required for the safe design of temporary works is similar to that necessary for permanent installations.

The simple view of a ground anchor is that it connects a man-made structure to a natural structure, i.e. a rock or soil mass. For this to be done successfully the nature and competence of the natural structure must be established. The basic requirements for accomplishing this are set out in this chapter together with the means whereby the data obtained from the ground investigations are used to estimate anchor capacity. Also discussed are tendon design, drilling and installation techniques, stressing and testing procedures and long term performance and monitoring.

GENERAL REQUIREMENTS

A permanent ground anchor system may be required as:

1. An integral part of the design concept of a project.

2. A means of resolving a problem revealed by a ground investigation carried out after finalization of project design.

3. A remedial or improvement measure to an existing structure.

The information on ground conditions that is available at the time of a feasibility study will largely depend upon the initial requirement. For instance there could well be much more data available for consideration on a project in Category 3 above as opposed to Category 1.

While there may be adequate data to indicate both the feasibility and advantages of a permanent anchor system, there may well be insufficient detailed information to permit safe and economic design and construction. The geometry of a ground anchor and its mode of operation requires in particular a detailed knowledge of ground conditions local to the anchor "bond" or "fixed" length. (See Figures 1 & 2 for nomenclature related to anchors).

SITE EXPLORATION

Ground investigations are most satisfactorily undertaken in a number of stages which can broadly be categorized as:

1. Initial desk and field study.

2. Main field and laboratory investigation.

3. Investigation during construction.

The data available at the time of considering ground anchor feasibility will dictate the stage at which the investigation process outlined above will be commenced.

Similarly the work involved in any one stage will depend upon the nature of the overall project. For instance, the designs for a simple rock bolt system may well be based principally upon visual field observations and mapping, whereas, those for a major retaining structure could require extensive field and laboratory investigations carried out in phases. The data obtained from one phase is used to determine the scope and extent of the work in the next phase. The aim of the investigation is to ascertain, by the most economic means, conditions within a block of ground that is influenced by or influences the installation of ground anchors.

Initially, office studies of geologic or soil survey maps should be undertaken to determine the general rock and soil conditions that would be expected to be encountered. If the office responsible for conducting the site exploration is familiar with the site or has had prior experience with other projects in the area, it may be possible to obtain very detailed rock and soil information before any site visit is made. In any case, as much local information as possible should be obtained from sources such as township engineers and public utilities.

Initial field reconnaissance will enable the designer to observe surface water runoff patterns, seepage, and vegetation characteristics of the site. This is helpful in assessing

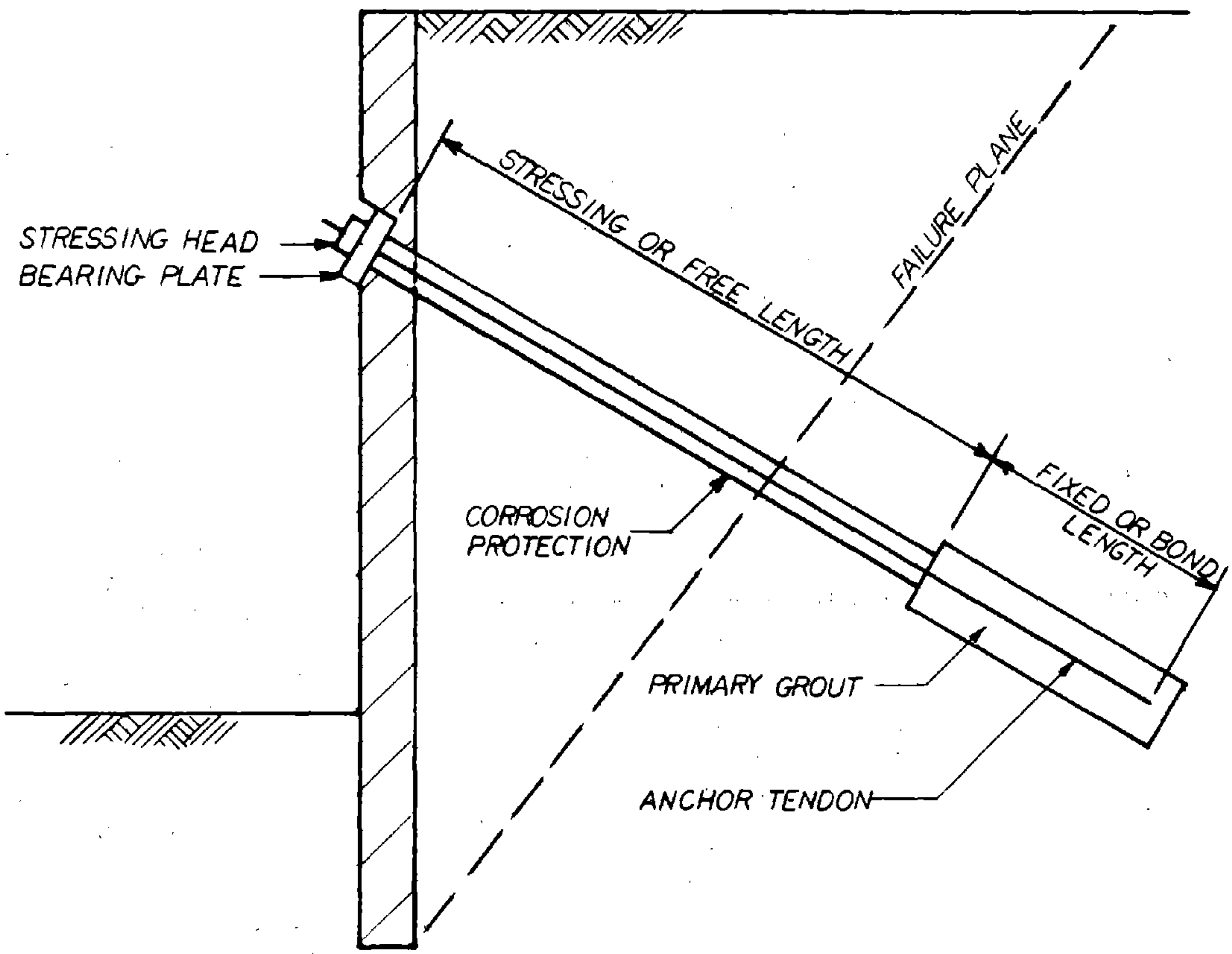


Figure 1. COMPONENTS IN GROUND ANCHORS

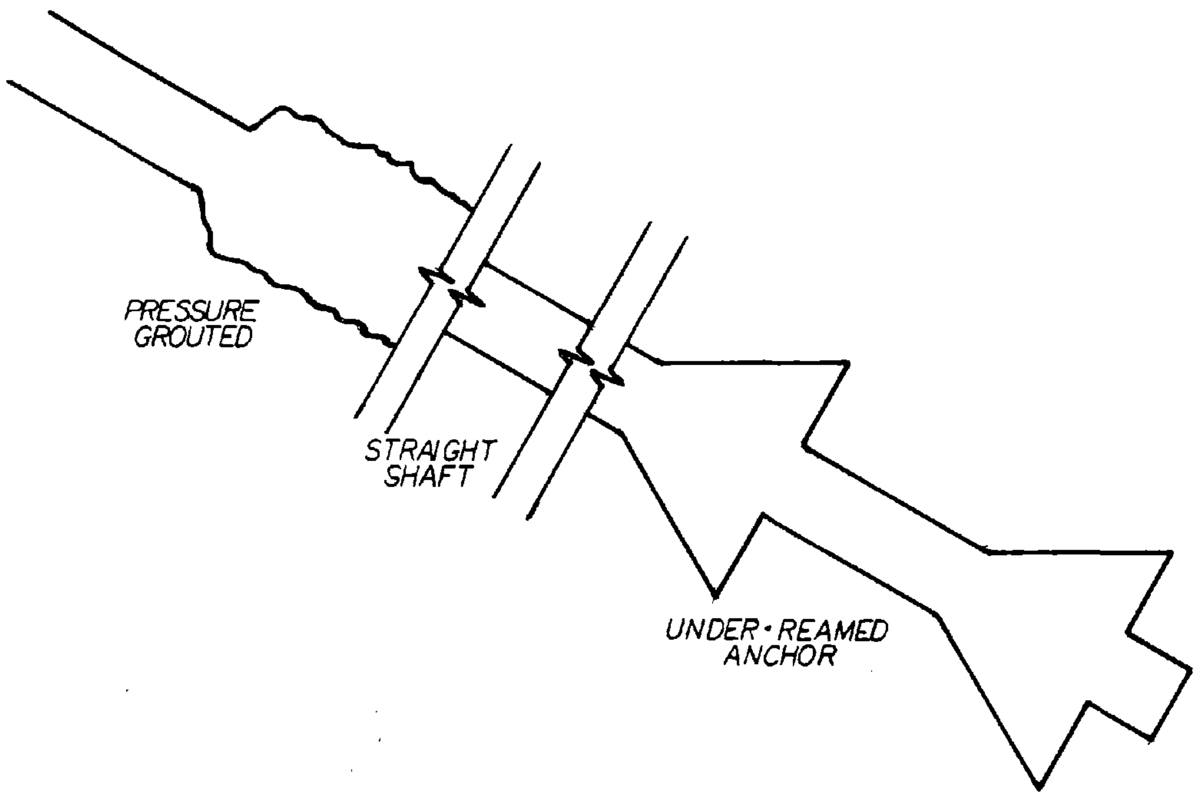


Figure 2. FIXED ANCHOR TYPES

drainage requirements to compensate for water pressure on the retaining wall. During the initial site inspection, surface geological features, including rock outcrops, cuts and excavation, can be inspected to obtain a preliminary concept of subsurface conditions. Also, surrounding environmental conditions can be evaluated to determine if a potentially aggressive environment exists, and what effect it might have on the ground anchors and retaining wall. Possible solutions to potential corrosion problems can begin to be developed. Another important feature that can be checked during the initial field study is the possible occurrance and location of landslides or crustal displacements which would increase wall loads, and affect wall toe elevations and anchor lengths.

Site access conditions for work forces and equipment can also be observed along with any existing adjacent structures and facilities, to check for possible interferences, which may cause problems during construction.

Where anchors have to be installed under adjacent structures or buildings it is vital, at this stage, to accurately determine the extent, nature and situation of any foundations, services, basements and sub-structures to those buildings. Also, where those existing buildings are not under common ownership with the new project, legal permission should be obtained prior to commencing the installation of anchors. Withholding of such permission is sufficient in itself to make a ground anchor system unfeasible.

Desk studies should include an investigation of the known plans or intentions for developing areas. For example, pile driving for a new building adjacent to the project site could result in changes in soil structure in the anchor bond zone due to vibration, liquifaction, or loosening of the soil. Also a pile could be driven right through an anchor tendon thus destroying it completely. Similarly future tunnelling or shaft sinking work could also have a profound effect upon the in-situ soil properties through alteration of watertable and disturbance due to construction activities.

After the initial field reconnaissance, a topographic survey should be performed to obtain necessary geometric parameters for design of the wall at typical cross sections. Then a test boring program to determine subsurface conditions at typical cross sections can be started.

The number and locations of borings are usually determined by the engineer based on available information obtained from previous experience with the area, and on observations made during the initial field reconnaissance. Thus, for example, on a site for which little previous information is available, primary boreholes should be sunk to identify geological sequences and those strata which are of particular interest from the point of view of overall stability of the design of the retaining wall and anchors. These primary holes should be located at the site extremities so that soil profiles can be interpolated between boreholes, rather than assumed from boreholes situated in the center of the site. The depth of these bores should be such to ensure that known geological formations are proved and that no underlying stratum exists which will affect overall design and stability.

These preliminary investigations can then be followed by further borings and/or in-situ tests to obtain more detailed information. The number of test locations will depend on the results of the preliminary investigations. Glacial drift materials will require more attention than a well-known competent geological formation. However, as has already been stated, minor structural changes in soils can have a substantial effect upon ground anchor performance so it is recommended that boreholes or test locations be at a maximum spacing of 60 feet (20m).

Generally, for a retaining wall, three borings per typical cross section are desirable at the following locations as shown on Figure 3:

1. On a line behind the wall at a distance equivalent to half to full wall height.

2. On the line of the probable anchor bond zone. (NOTE: The boring locations mentioned in Item 1 and 2 may be coincidental.) From this information anchor design will be accomplished as well as wall design.

3. As close to wall center line as possible in order to determine soil parameters and strengths on and under wall position.

4. From 1/4 to 3/4 of the wall height in front of the wall. This gives information on expected passive earth pressures, potential slip planes and/or ground heave.

Additional test borings should be taken where sloping ground or possible landslide areas exist. Also, where long anchors may be required, a number of test sites should be located to investigate drilling conditions above bond zones. The optimum drilling technique can be chosen from the results.

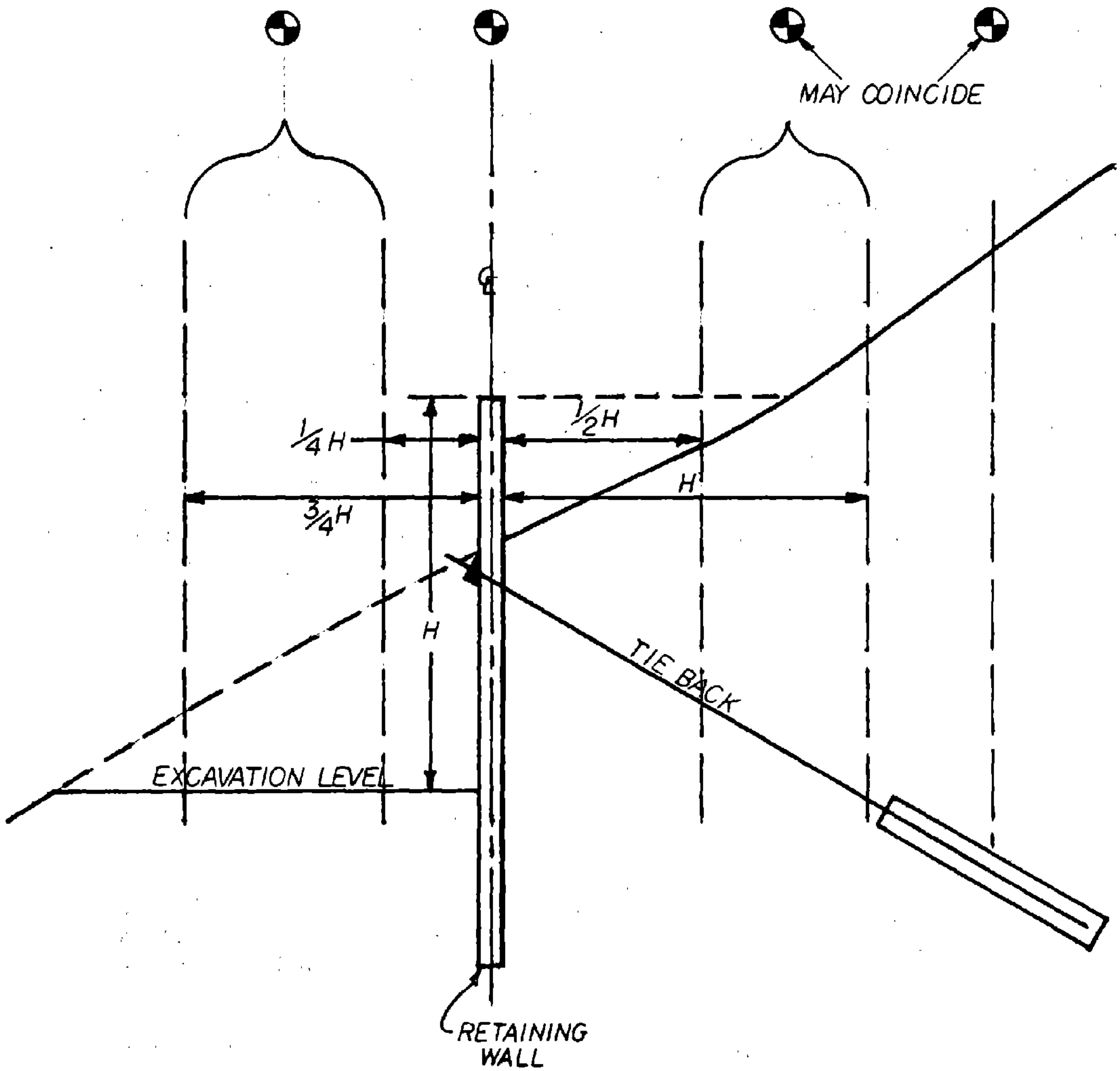


Figure 3. SUGGESTED SITE INVESTIGATION BOREHOLE LOCATIONS

The depth of the borings are determined by the general geology, but where bedrock is encountered typically they are taken to top of rock with a minimum core penetration of 10 feet (3m). This is to ensure distinction between boulders and bedrock. The core samples will give information on grout/rock bond. Behind the wall, borings should be at least as high as the wall if a soldier beam and lagging wall system is used, and deeper if sheet piling is used.

Samples should be taken by standard tube penetrometer, Shelby tube, or NX rock coring to obtain material for identification and testing, and for determining rock quality by R.Q.D. index. Field tests can be performed using the standard tube penetrometer to carry out standard penetration tests to establish soil density and consistency. Great care must be exercised to see that the S.P.T. tests are conducted strictly according to specification if meaningful results are to be obtained. Field vane or Dutch cone tests in cohesive soils will give undrained shear strength values. Dutch cone tests in cohesionless material give estimates of relative density.

To facilitate anchor design it is virtually essential that an indication of in-situ permeability be obtained from the boreholes in the bond zones of the soil or rock. In soil this provides data that is used to assess groutability of the material and thus size and pullout resistance of the fixed anchor.

In rock the results indicate rock quality and thus the need for a consolidation grouting to improve the stability of the rock mass and also to prevent infiltration or percolation of ground water into the anchor bond zone. Shutting off such moving water is a primary corrosion protection method as it prevents potentially aggressive or corrosive elements coming into contact with the anchor tendon. Permeability is determined by falling head or water pressure testing directly in the anchor borehole or assessed from the soil particle size distribution curves using the Hazen formula.

Site water levels need to be carefully monitored as they are essential to the calculation of horizontal pressures on a retaining wall.

SOIL PROPERTIES

The properties of a soil that must be determined in order to compute anchor capacity are:

1. Unit weight in natural conditions

2. Angle of internal friction ($\emptyset$)

3. Cohesion (c)

4. Particle size (in non-cohesive and mixed soils) (D, mm)

5. Density in-situ (γ)

6. Permeability (k, cm/sec)

7. Liquid and plastic limits (LL, PL)

8. Unconfined compressive strength of cohesive soils. (q_u, ton/s.f.)

For design of retaining walls the above values are used with the addition of information on water table levels, superimposed loads and construction sequences.

It is common practice for soils information to be presented to the designer based on samples obtained by split spoon samples together with the blow counts recorded during the driving of the sampler (S.P.T. results). While these are valuable data which can be used to give approximate relationship between penetration resistance and relative density, unconfined compression strength, angle of friction and unit weights, they are often found to be greatly in error. This is often due to the penetration test not being conducted according to specification. For instance there is less fatique on the driller if two or even three turns of the pull rope are rapped around the drive pulley of the drill rig during the test. This results in higher frictional resistance to free fall of the drive weight and thus a higher blow count. This in turn indicates a stronger material than actually exists. Another common violation is when a resistance material is encountered and the drive hammer is dropped more than the specified distance in order to speed up the test. This results in a lesser number of blows being recorded for the soil being investigated. The condition of the sampler shoe also affects results.

From this it can be seen that site testing should be very carefully supervised, preferably under the direction of the design engineer, and that the field tests should always be augmented and confirmed by laboratory tests.

An earth retaining wall in equilibrium resists horizontal pressure. While this pressure may be evaluated by the theory of elasticity, it is more practical to use empirical coefficients of active, passive and at-rest earth pressures. These coefficients are calculated from soils data and, as they are directly related to soil unit weight, apparent cohesion (c) and angle of shearing resistance (internal friction, Ø). These values must be obtained.

Angle of internal friction, (Ø), may be determined approximately from S.P.T. results, but should be confirmed by laboratory shear box tests for cohesionless free-draining materials. Plotted values for shear stress against applied compressive stress will give a straight line passing through the graph intercept and it is the angle formed between this line and the base line that represents Ø. See Figure 4.

For fine grained materials the shear strength depends mainly on the "cohesion" between the grains and does not vary with compressive load. So for cohesive materials (clays) where Ø = 0 the shear stress graph would be as shown in Figure 5. In an unconfined test, "c" would be taken as half the maximum compressive stress.

However, most soils are made up of a combination of frictional and cohesive materials and where a sample is stressed to failure and the results plotted, the line connecting the values obtained will not pass through the graph origin but will intercept the shear stress axis at a value equal to the apparent cohesion (c). See Figure 6.

It is usual for soil samples (except sands and gravels) to be tested undrained in a tri-axial compression tester as this enables an examination of soil stability to be made without knowledge of pore water pressure being required. Also as the effect of wall and anchor construction and stressing on the soil structure is relatively quick, the "quick" undrained test most nearly represents actual site conditions. The minimum factor of safety thus occurs in the short term undrained condition when strength is lowest. With time, excess pore water pressure induced by stress is dissipated and the soil structure stiffens and gains strength.

Note that if the plot in Figure 6 passes through the origin of the graph, then the cohesive material being tested will be normally consolidated and be of low bearing value for anchors at normal foundations.

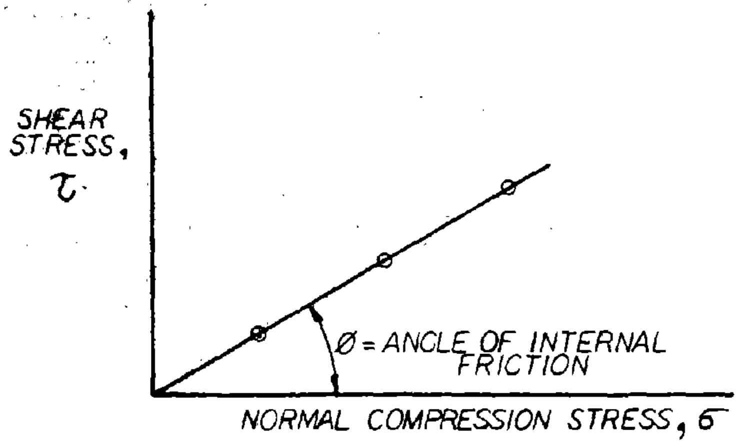


Figure 4. SHEAR TEST GRAPH FOR FRICTIONAL SOILS (SANDS AND GRAVELS)

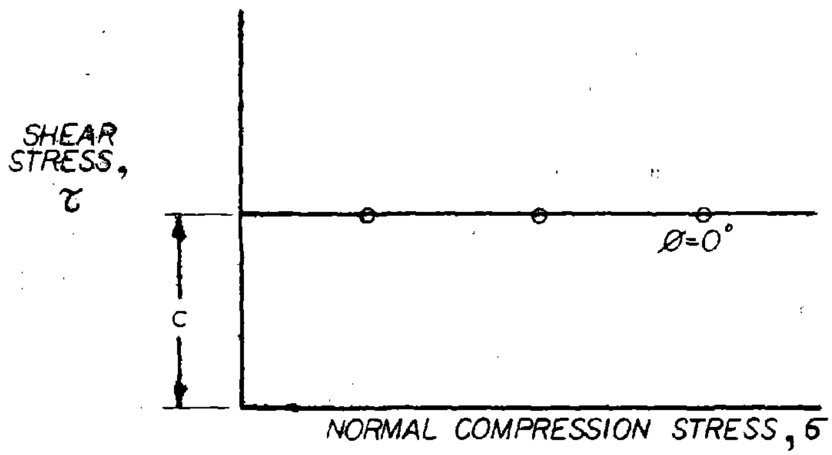


Figure 5. SHEAR TEST GRAPH FOR COHESIVE SOILS (CLAYS)

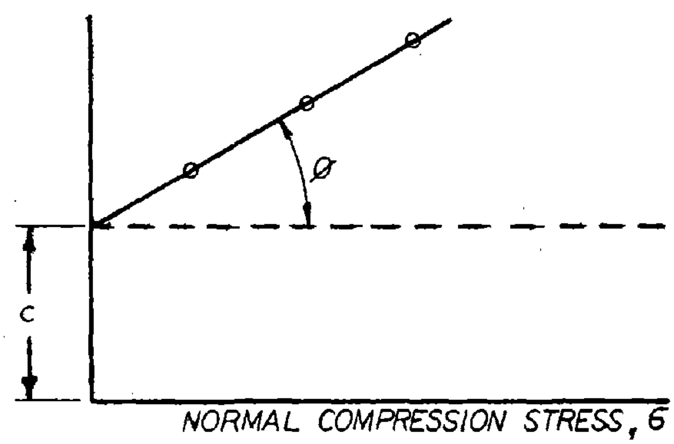


Figure 6. SHEAR TEST GRAPH FOR MIXED (FRICTIONAL-COHESIVE) SOILS

TABLE I Approximate relationship between penetration resistance and relative density, angle of internal friction and unit weight of soils.

GRANULAR MATERIALS

Compactness	Very Loose	Loose	Medium	Dense	Very Dense
Relative Density	<15%	15-35%	35-65%	65-85%	85-100%
Standard Penetration Resistance N = Blows/Foot	0-4	4-10	10-30	30-50	> 50
Ø, Degrees Approximately	<28	28-30	30-36	36-44	> 41
Unit Weight PCF Moist	<95	95-125	110-130	110-140	>130
Submerged	<60	55-65	60-70	65-85	> 75

TABLE II Approximate relationship between penetration resistance and unconfined compressive strength and unit weight of soils.

COHESIVE MATERIALS

Consistency	Very Soft	Soft	Medium	Stiff	Very Stiff	Hard
Unconfined Compressive Strength (TSF)	< .25	.25-.50	.50-1.00	1.00-2.00	2.00-4.00	>4.00
Standard Penetration Resistance N=Blows/Foot	0-2	2-4	4-8	8-16	16-32	> 32
Unit Weight PCF (Saturated)	<100	100-120	110-130	120-140	130-145	>140

NOTE: 1 pcf = 0.157 kN/m^3
1 tsf = 95.76 kPa

Where soils of high plasticity or compressibility are encountered and are to be retained, their long-term consolidation charateristics should be determined. From this information, it can be established whether there will be a loss of anchor stress due to a dimensional change in the retained soil mass. Thus the stress level below which the anchor will be stable can be decided, that is, where consolidation or creep will not occur.

Particle size distribution is of great significance in the design of anchors as the soil can be described according to the shape of the distribution curve, permeability and thus groutability can be assessed, and an indication of friction angle can be obtained. See Particle Size Distribution Graph, Fig. 7.

The effective particle size of a soil is governed by the size of the smallest 10% fraction of the soil. This size (D_{10}) is related to permeability and thus to permeability of the soil.

Soil permeability can be assessed from the particle size distribution curve. This is done by taking the square of the 10% fraction particle size (D_{10}) and assuming this to be the rate of flow. For instance for a D_{10} particle size of 0.2mm, permeability would be $(0.2)^2 = 0.04$ or $k=4 \times 10^{-2}$cm/sec.

To form a soil anchor grout bulb larger than the borehole diameter, it is necessary to force cement particles between the soil grains. The limit for this is at an effective particle size of approximately 0.2 mm or fine sand. This represents a permeability of 10^{-1} to 10^{-2} cm/sec. Below this grain size the cement will not penetrate easily, but will densify the sand strata in the immediate area of the bond zone to form a larger effective diameter than the borehole, but with a probable increase in the maximum ratio of 3/2 x R where R is borehole radius. Permeability of these soils would be $k=10^{-3}$ to 10^{-4} cm/sec. Soils with permeabilities less than $k=10^{-4}$ cm/sec and low cohesive strength indicate the presence of high proportions of silt. Such soils are not considered to be suitable materials in which to found ground anchors.

A comprehensive chemical analysis of the soil materials should be undertaken to determine whether an aggressive or corrosive environment will exist. Anchors particularly have to be protected from the effects of stress corrosion and the levels of such protection have to be decided during design stages. Samples of ground water must also be tested with sampling taking place at various depths. It is of great importance to ensure that the samples recovered are truly of the natural ground water and not of or contaminated by the drilling wash water. The analysis as a minimum should determine the sulphide and chloride content of the sample, pH value and the presence of any element

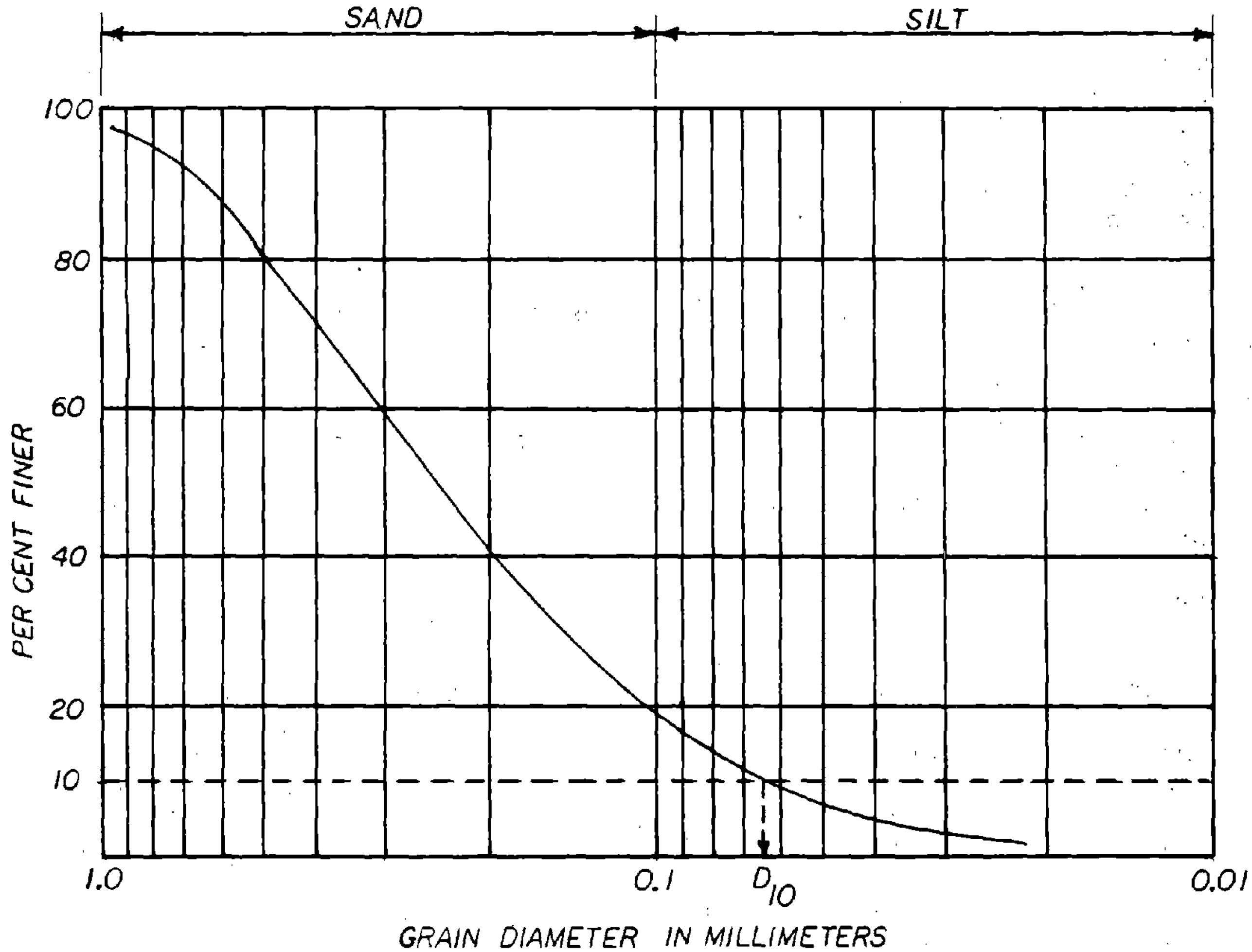


Figure 7. PARTICLE SIZE DISTRIBUTION

which may promote an aggressive attack upon the anchor tendon or grout or the wall itself.

During construction, continual observations of drilling conditions and cuttings recovered will confirm previous test drilling results. These observations will also reveal any anomalous conditions existing locally and permit any changes to be made if needed to ensure structural stability.

HORIZONTAL EARTH PRESSURES

To some extent the horizontal wall pressures on a tieback depend on the stiffness of the wall relative to the soil and to the lock-off anchor loads. Both of these can be controlled by the designer. Lock-off loads producing a total thrust equal or nearly equal to that developed by the active earth pressure must be selected to limit movement of the wall. Obviously, the horizontal pressures produced by the anchor wall system must not exceed the passive earth pressure.

Theoretically, it should be possible to use anchor loads and a wall system which should result in a horizontal pressure distribution very similar to the active earth pressures. This would require close control of construction procedure and periodic adjustment of anchor loads during and after wall construction. The method of construction most typically used results in a pressure distribution at best approaching the arching active case and more reasonably approaching the total thrust of the "at-rest" case. The ideal situation is where the retained earth mass never "knows" that its former support has been removed and replaced by a different structure. But construction sequencing often means that the loads on the wall imposed by stressed anchors can, in fact, produce wall movements towards the retained soil. This is typical of the case where a row of anchors is stressed fully prior to excavation to the next lower level. A check should be made of horizontal pressures on a wall once the construction sequence is known to determine if wall movements are likely to occur. Anchors can be stressed in stages if necessary to obviate problems arising from this source

The lateral pressure distribution used in the design will depend on the following:

1. Soil type, e.g. cohesive or non-cohesive.
2. Ground water conditions
3. Relative wall stiffness, e.g. concrete or H-beams and lagging.
4. Anchor locations and lock-off loads.

An earth retaining wall in equilibrium resists horizontal pressure. This pressure could be evaluated by the theory of elasticity, but more practically an empirical coefficient of earth pressure is used. If the weight of soil above any depth h is h, then the horizontal earth pressure at rest = $K_o \gamma h$. Where K_o is the "at-rest" coefficient, and $K_o = 1 - \sin \phi$ (where ϕ is soil angle of internal friction).

However, in practice many retaining walls move forward slightly. When this happens the pressure on the wall is reduced. The minimum value of this pressure at the moment of failure of the soil is known as the active pressure. It is estimated using Rankine's theory where the coefficient of active pressure

$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi}$$

Figure 8 shows the active thrusts developed on a wall retaining cohesionless soil. Where the wall is rigid the pressure distribution will be triangular as shown and the maximum value of active pressure will be: - $p_a = K_a \gamma H$ and the resultant thrust acting through the center of gravity of pressure triangle i.e. 1/3 from bottom of wall, will be:

$$P_a = 1/2\ p_a H$$

or $$P_a = 1/2\ K_a \gamma H^2$$

In computing earth presures for tied-back retaining walls it is more usual to use the coefficient K_o as this will give resultant anchor loadings which will prevent or minimize any wall movements.

Figure 9 represents a retaining wall formed by driving steel sheet piling. The movement of such retaining walls is opposed by the passive resistance of the earth into which the piling is driven. The value of passive resistance of a cohesionless material may be found by replacing the coefficient of active pressure K_a by its reciprocal K_p, the coefficient of passive pressure

$$K_p = \frac{1 + \sin \phi}{1 - \sin \phi}$$ and thus if

$K_a = 1/3$, then $K_p = 3$. Values of H are replaced by the depth of earth towards which the wall is driven (h in Figure 10). When the passive resistance thus computed is smaller than the active pressure, then the wall may fail if there are no other means such as stressed ground anchors to provide additional support and thus prevent failure.

The active pressure distribution on a wall is triangular if the wall is rigid. When the wall is flexible, and construction sequences cause incomplete or irregular bending then a different

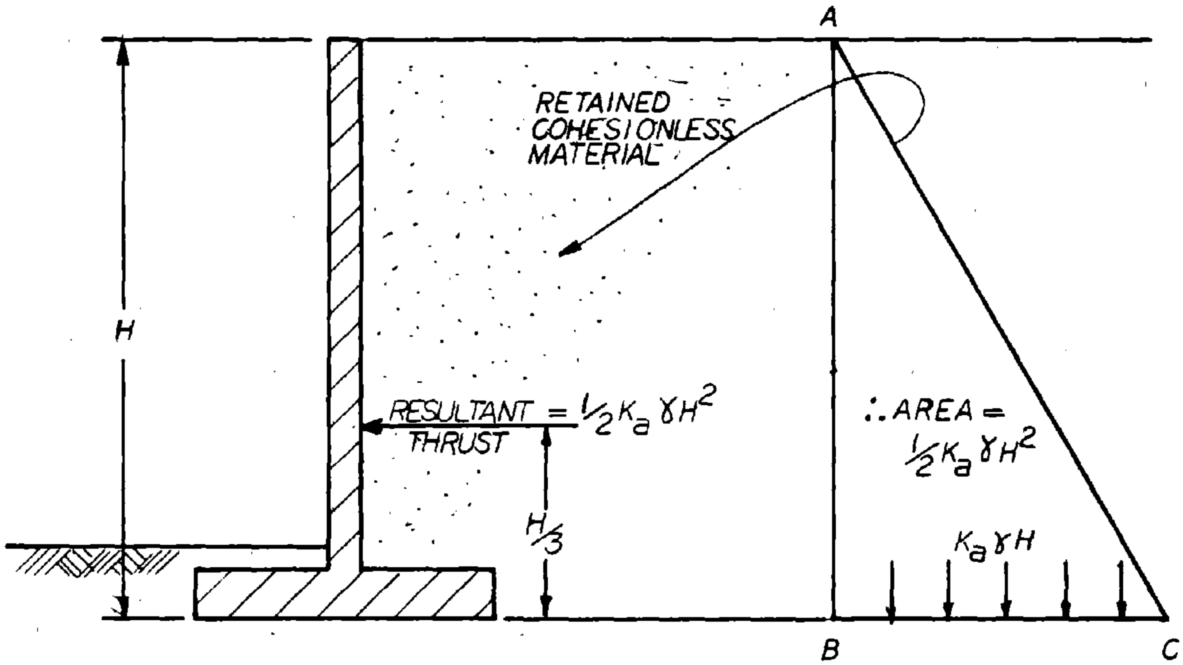


Figure 8. ACTIVE PRESSURE DISTRIBUTION

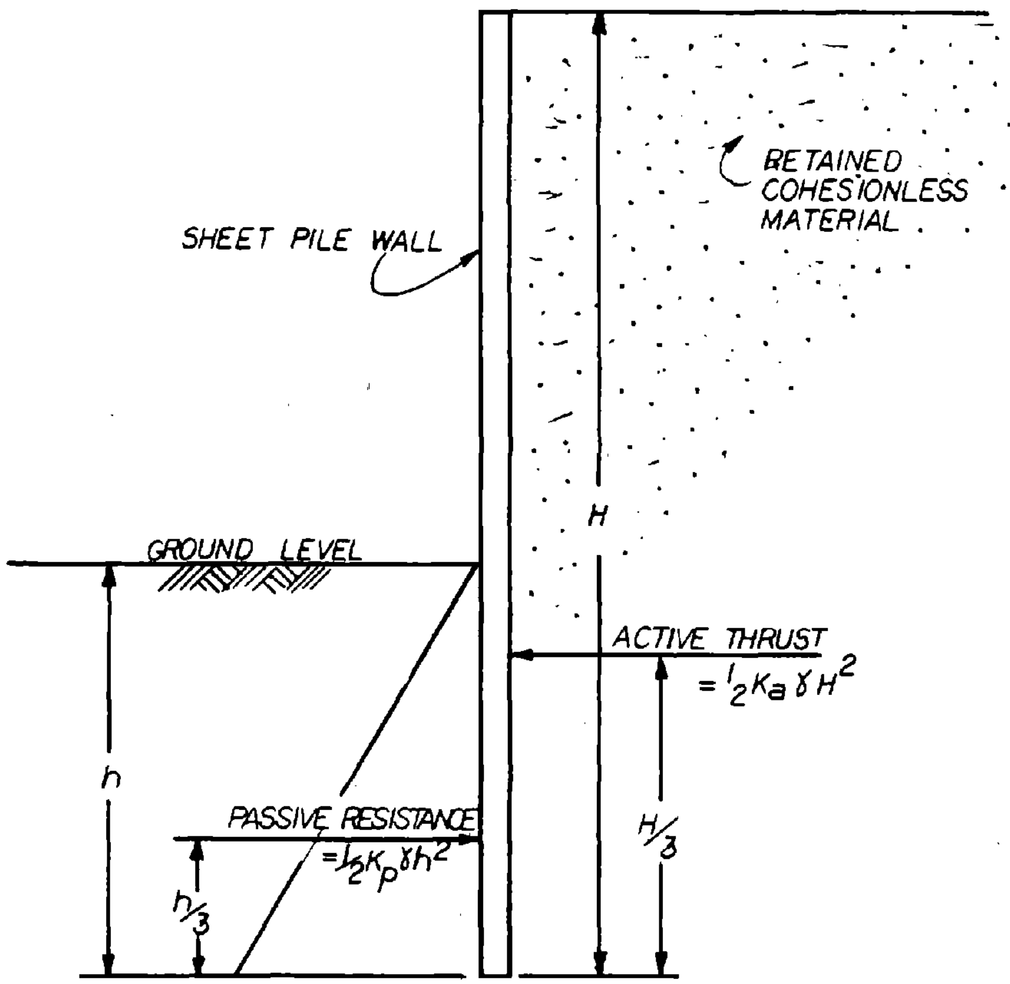


Figure 9. ACTIVE PRESSURE AND PASSIVE RESISTANCE

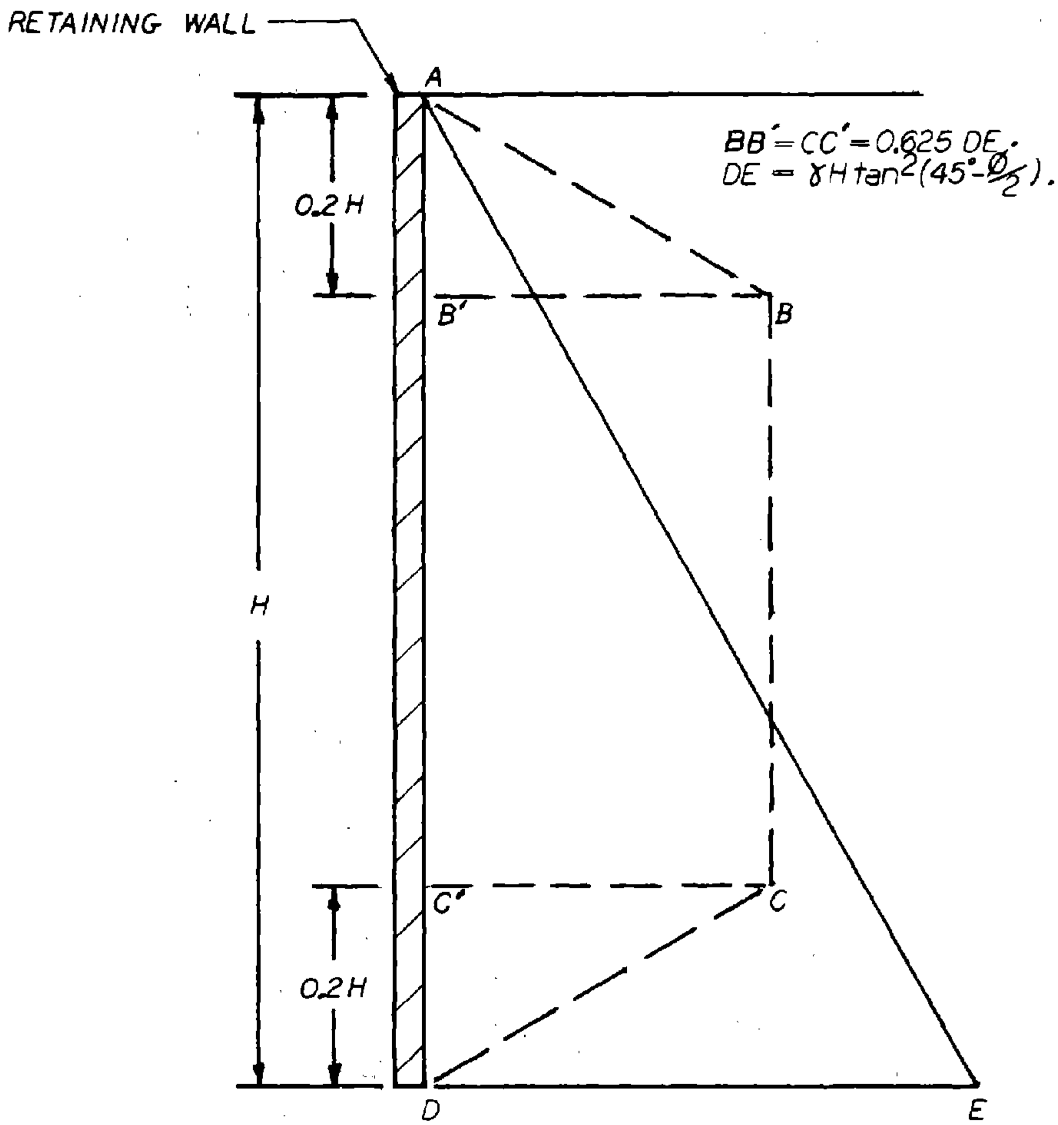


Figure 10. COMPARISON OF TRIANGULAR AND TRAPEZOIDAL PRESSURE DISTRIBUTIONS (After Terzaghi)

pattern of pressure distribution will take place. This was found experimentally by Terzaghi to be trapezoidal. Figure 10 shows this comparison. The area of the triangle A E D and the trapezoid A B C D should be equal since both express the same lateral pressure on the wall.

The preceding discussions refer to cohesionless soils and where cohesive soils are encountered the methods outlined must be modified. This is because the response of cohesive soils to changes in imposed stress is not instantaneous but is time dependent. Thus consideration must be given for the 'long-term' effect of stress changes as well as during the short-term construction period.

ANCHOR LOCATION SELECTION

In order to select optimum locations for ground anchors in an earth retaining system the following factors need to be considered:

1. The probable wall load and anchor forces.

2. The presence of a suitable strata in which to found the anchor fixed length.

3. Whether the vertical load component in the anchors will induce bearing failure beneath the toe of the wall.

4. The stiffness of the wall under design and thus whether a single or multiple tier system of anchors should be installed.

5. Site access and construction sequencing.

6. Expected drilling conditions, including water table and obstructions.

7. Existing services and structures adjacent to the project site together with an investigtion of any future work planned.

8. Structural analysis of the retaining wall at various stages of construction.

An initial study of the project design requirements and preliminary soils data will enable probable wall loadings to be calculated together with an estimation, based on experience at this stage, of the number of levels of anchors required.

The probable failure planes should also be determined at this stage as this will indicate probable anchor lengths. As a minimum, the fixed anchor length should commence 6 feet (2mm) beyond the calculated failure plane.

The soils data are then further examined to establish whether a suitable bearing strata exists that can be used for the construction of permanent anchors. Non-plastic soils and those of medium to low plasticity having high density together with granular materials such as silty sand or coarser can be considered suitable providing an adequate stratum is present. Adequate in this sense refers to the thickness of the deposit, and requirements will vary according to anchor load and angle. Anchors in plastic clays and silts, backfill materials or materials subject to normal consolidation only will probably not be successful anchorage subjects. Placing of the anchor bond zone in soils having a high organic content should be avoided. From this assessment, the utility of anchors can be decided. As an indicator, if the vertical distance from the anchor entry point through the wall to the closest suitable anchoring strata exceeds 100 feet (30m), the possibility of the economical use of anchors decreases rapidly. The 180 feet (55m) deep swamp near Meadville, PA, and the soft silty clays in excess of 100 feet (30m) in the area of Detroit, MI, are typical of situations where the use of soil anchors would be very costly and alternatives should be considered. If no alternatives are possible, then the cost has to be borne.

The soils information then should be reviewed to enable an assessment to be made of the possibility of vertical drawdown of the retaining wall under the loading imposed by the ground anchors. By simple trigonometry it is easy to arrive at the load component. Where suitable strata exist close to the surface, anchor angles shall be as flat as possible (minimum 15°) and thus exert low vertical loads. Deep foundation strata require anchor angles much steeper (45° - 50°) with corresponding greater vertical stress. The wall must resist vertical movement since any such change in elevation will result in 1) Loss of anchor stress; 2) Wall horizontal movement due to earth pressure; and 3) Probable eventual failure, but certainly surface subsidence and settlement behind the wall with attendant damage to roads, services and structures. Adequate bearing can be assured by driving sheet piles or soldier beams until pre-determined resistances are achieved. Where this cannot be done consideration should be given to increasing end bearing by setting H-Beams in concrete caisson sockets, construction of tangential caisson retaining walls or bentonite slurry walls.

Once the structural type of retaining wall is known, the number and levels of anchors can be considered. In a waterfront bulkhead, for instance, only one level of anchors is normally possible and they would be attached to a wale capping the retaining wall. For other walls in dry conditions, the number and tiers of anchors can be calculated by static analysis bearing in mind that soil anchor capacity is governed by the strata encountered and not by statics. As a general rule, the aim should be to construct anchors of maximum capacity within the limits imposed by soils and structural strengths as this is normally the most economical method. Other factors have to be considered such as the horizontal spacing of anchors to coincide with sheet pile profiles, spacings to coincide with lagging board width or H-Beam centers, the tangent points between contiguous caissons forming a wall, or the width of slurry wall panels.

Site access and construction sequencing can have an effect upon the number of anchors installed and also upon the drilling technique employed. For instance, structural analysis may suggest a certain optimum in vertical spacing for tie levels. However, this may not be possible to achieve if excavation can not also proceed to match these optimums because of the need to carry out construction work at higher elevations or, for instance, to install the next level of a dewatering system. Extra anchors may be needed to support temporary surcharge loads immediately behind the wall caused by the presence of cranes or stocked construction material. If the site access is restricted by the work plan or by the overall design, then the use of large drilling equipment, generally of the hollow stem auger type, will not be possible and the choice will be between small diameter rotary or percussive types.

DRILLING TECHNIQUES

The soils information can now be used in conjunction with the anchor load estimates to select the drilling technique. Three main drilling methods are available, namely, hollow-stem auger, percussive and rotary. Of these, the last is by far the most versatile as it permits fully cased holes to be drilled to virtually unlimited depths, even with limited access or headroom; it permits preservation of the soil structure, penetration of obstructions, prevention of damage to adjacent structures and services through impact, vibration or over-drilling (or mining), drilling below water table, inspection of drill cuttings for confirmation of strata, and accurate control of anchor placement and grouting. While rotary equipment is specialized, and thus not off the shelf, and expensive to operate, it does present the most certain technical method for accurate and safe control of the processes involved in ground anchor construction.

The drilling method must not damage the integrity of services and structures adjacent to the work both during installation and in the long term. The presence of deep foundations, sewers, railroad tunnels, etc. in the immediate area may make the construction of anchors difficult, if not impossible. Legal permission must be obtained for drilling in the properties of others. Also, care must be taken to find out if any future work will take place that could affect the stability of the system. For example, pile driving that could shear anchor tendons, earthwork that could either overload the wall through surcharge or limit anchor capacity by removal of overburden.

A structural analysis of the system will reveal if any reduction in load or overload of the wall or anchors will take place during various phases of construction, particularly excavation in front of the wall. Anchors can be stressed in stages according to the reaction loads required at any construction phase and additional anchors can be provided for severe local overload conditions.

TYPICAL ANCHOR CHARACTERISTICS

From the foregoing, it can be seen that optimum anchor spacing for a particular project can only be decided from the information available relative to that project. However, because many projects tend to be similar, some ground anchor parameters can be stated for a fairly typical permanently anchored retaining wall.

1. Design Load Between 50 Tons (445kn) and 130 Tons (1156kn) - An anchor tendon of this capacity can be handled without the need of heavy equipment (except bars more than 40 feet [12.5m] long) and the drilled hole size need be no larger than 4 inches (10cm). In addition, the stressing equipment can be readily handled without using power lifting equipment.

2. Length of Between 40 Feet (12.5m) and 70 Feet (21.4m) - Due to geotechnical requirements, there are few retaining wall anchors installed that are shorter than 40 feet (12.5m). A minimum stressing length of 20 feet (6m) should be adopted to avoid unacceptable high prestress losses in anchors due to long-term relaxation, creep in steel and soil, and anchor seating losses. Where possible, free length should be 25 to 30 feet (8-9m).

3. Angle of Inclination Between 15° and 45° from the Horizontal - It is difficult to properly grout an anchor at an angle less than 15°. In addition, shallow anchor angles can lead to a lack of overburden depth which in turn limits the capacity of the anchor. Relative to this, it is desirable that a minimum of 20 feet (6m) of unsubmerged overburden be above the fixed anchor. Most soil anchors are installed at an angle of between 15° and 30°. However, when a suitable anchoring strata lies at some depth, generally more than 30 feet (9m), an angle of 45° may be chosen as a compromise between length of anchor and decrease in the resultant horizontal force for a given anchor capacity. It must be kept in mind, though, that by increasing the angle of inclination, the vertical component of the anchor load also increases, thus increasing the vertical load on the wall members and the underlying foundation material.

4. Drilled Hole Diameter Between 3 Inches (7.6cm) and 6 Inches (15cm) - The vast majority of soil anchor work is performed using a cased hole. The weight of the casing and associated handling, and drilling problems related to larger casings at present makes 6 inches (15cm) the largest size in common use. Most usual sizes are 3-1/2 inch O.D. (90mm) used with percussion methods and 5 inch O.D. (12.5cm) used with rotary drills. Several methods can be used to install soil anchors without the use of casing but the most common is by hollow stem auger. This report does not describe the use of uncased holes, particularly those formed with auger equipment, because their use is so limited. They could not be used:

   a) In sites with limited or difficult access.

   b) In difficult soil conditions.

   c) In an urban or built-up environment where undermining of structures and services may cause damage.

ANCHOR LOAD DETERMINATION

Calculated lateral earth pressures are used to determine the retaining wall anchor loads as follows:

1. Single Tiered Walls - Triangular pressure distribution and surcharge applied either uniformly or dissipating at depth. Add hydrostatic loads as applicable. See Fig. 11.

2. Multiple Tiered Walls - Rectangular or trapezoidal pressure distribution. Add surcharge and water pressure. See Fig. 12.

Loading on the anchor is determined by either or both of the following methods:

1. Proportional Method - Where the anchor takes all the pressure above the top row of anchors and 1/2 the pressure from that anchor down to the next support point, another anchor or the toe of the wall if only one row of anchors is used.

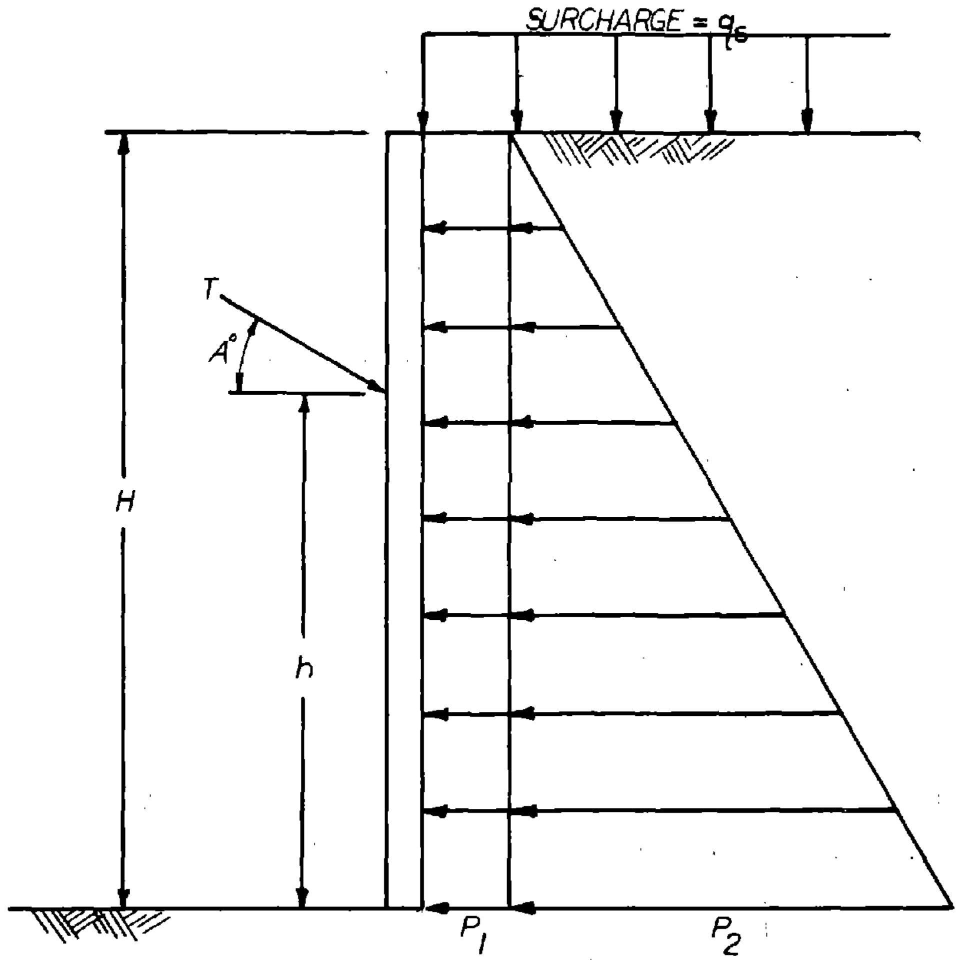
2. By taking moments at or a few feet below the base of the excavation of final grade in front of the wall - In some instances the moment developed by the pressure distribution and the anchor forces are required to sum to zero. In other instances, the design of the wall is such that the toe of the wall is permitted to take a portion of this moment. The anchor load must, of course, be increased by 1/COS A, where A is the desired angle of inclination of the anchor with respect to the horizontal. See Fig. 13.

SAFETY FACTORS

Once the theoretical anchor load is determined in the above manner, it is then necessary to apply the proper factors of safety to the various components of the retaining wall.

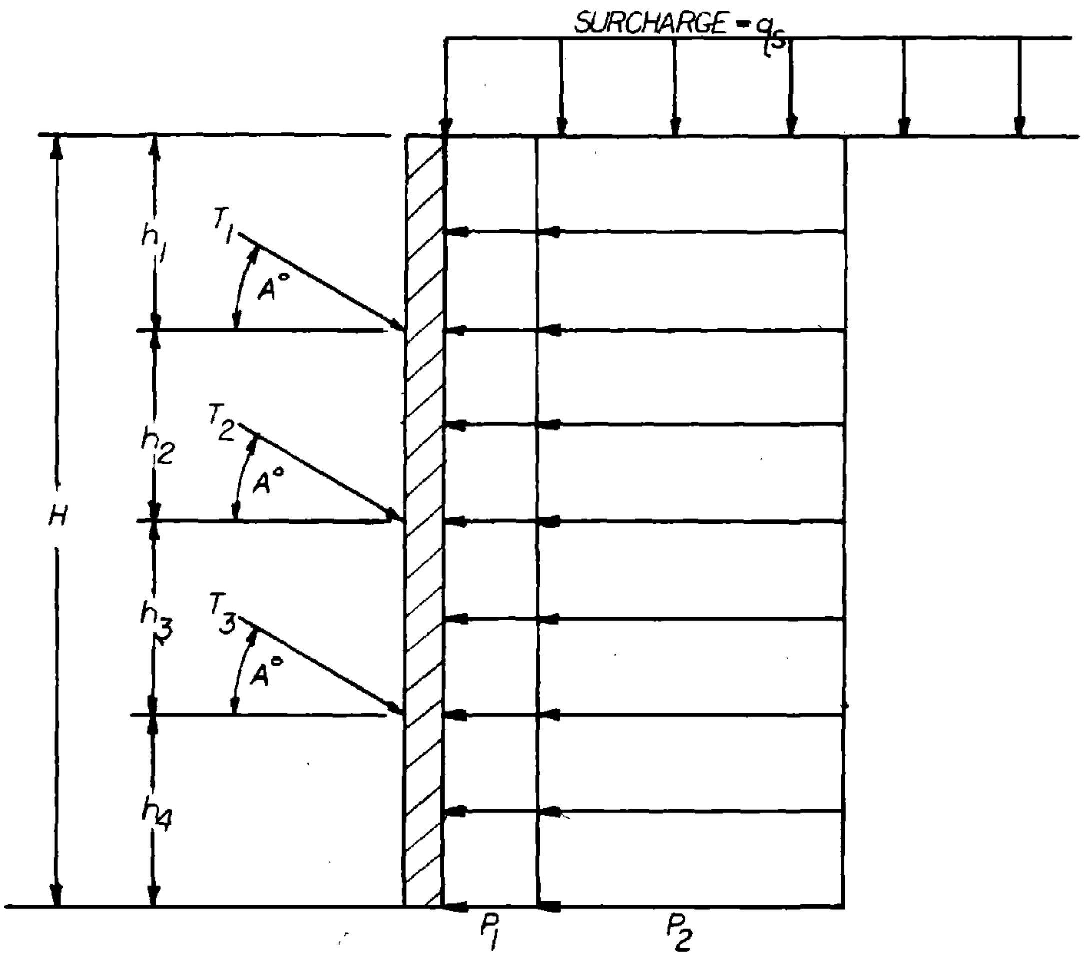
For temporary anchors in coarse grained soils, a minimum safety factor of 1.5 to 2.0 should be used.

In the case of permanent soil anchors, a minimum factor of safety of 2 over design load must be used. If the structure is an important one where serious economic loss or loss of life is likely as a result of a failure, and/or where corrective measures would be extremely expensive or impossible, adoption of a factor of safety of 2.5 is advisable. Also, if the probable



T = TIEBACK LOAD
P_1 = SURCHARGE LOAD = $K_a q_s$
P_2 = LATERAL EARTH PRESSURE = $\gamma K_a H$

Figure 11. SINGLE TIERED WALL



T_1, T_2, T_3 = TIEBACK LOADS
P_1 = SURCHARGE LOAD = $K_a q_s$
P_2 = LATERAL EARTH PRESSURE = $0.65\ \gamma K_a H$

Figure 12. MULTIPLE TIERED WALL

FOR FIGURE 11

1. PROPORTIONAL METHOD

$$T = \frac{P_1 H + \frac{1}{2} P_2 H}{\cos A}$$

2. SUMMING MOMENTS TO ZERO

$$T = \frac{P_1 H^2/2 + P_2 H^2/6}{\cos A \cdot h}$$

FOR FIGURE 12

1. PROPORTIONAL METHOD

$$T_1 = \frac{(h_1 + h_2/2)(P_1 + P_2)}{\cos A}$$

$$T_2 = \frac{(h_2/2 + h_3/2)(P_1 + P_2)}{\cos A}$$

$$T_3 = \frac{(h_3/2 + h_4)(P_1 + P_2)}{\cos A}$$

2. SUMMING MOMENTS TO ZERO

$$T_1(H - h_1) + T_2(h_3 + h_4) + T_3 h_4 = \frac{H^2/2\,(P_1 + P_2)}{\cos A}$$

loading on the structure cannot be determined accurately, or where soil conditions or properties are suspect, selection of a factor of safety greater than 2 may also be prudent.

Anchors founded in soils that are marginally suitable, such as soft fine-grained cohesive soils and those of medium to low plasticity, should be subject to an increased factor of safety of 3 or more. This will help to insure that anchor creep does not become a factor during the life of the structure.

ANCHOR TESTS

For all permanent anchor systems the installation of pre-production test anchors is recommended. These anchors should be tested to twice design load and then to failure where practical. This procedure will give the designer an indication of the actual factor of safety he can rely on with respect to anchor pull-out. Two or three such tests per project (depending on wall length or varying soil conditions) should be undertaken. In addition to the pre-production tests to 2 times design load or greater, testing of production anchors should be carried out. Testing of 10% of the production anchors to 1.5 times design load for non-critical structures and to 1.75 or 2 times design load for critical structures or where soil or loading conditions are suspect is recommended. The pre-production anchor test results can be used to provide envelopes of performance against which the production anchors may be judged.

ANCHOR TENDON STEEL DESIGN

ACI and other prestressing steel codes limit the maximum temporary allowable load applied to prestressing steel to 80% of guaranteed ultimate tensile strength. Some European codes are stricter and limit the maximum temporary load to 75% of g.u.t.s. In the United States conventional prestressing doctrine further specifies a maximum lock-off or transfer load of 70% of g.u.t.s. which, with a long-term allowance of 15% for load loss, results in a final effective prestressing force of 60% of g.u.t.s. This conventional doctrine is not truly applicable to anchors. If it is desired to check test some of the anchors to 150% or more of design load, then the test anchor tendons must be sized accordingly.

There have been instances of failures of strand tendons at approximately 85% of g.u.t.s. In addition, most prestressing steel hardware, by code, is required to provide only 95% of g.u.t.s. Therefore testing of anchors to 80% of g.u.t.s. of the tendon is too risky. A maximum test load of 75% of g.u.t.s. is recommended. If it is required that a percentage of the

NOTE: guaranteed ultimate tensile strength=g.u.t.s.

production anchors be tested to 150% of design load, without preselection of those to be tested, then it follows that the stress on the steel tendon at working load will be 50% g.u.t.s. This figure of 50% g.u.t.s. should be used for all permanent anchors. The adoption of this 50% g.u.t.s at design working load has the added benefit of providing a factor of safety of almost 2 with respect to failure of the steel tendon for all anchors.

Once the anchor design has been verified by testing, the required lock-off or transfer load must be determined. Again, typical prestressing doctrine has usually been adopted, i.e., stressing the anchor to design or working load plus an allowance for seating loss and long-term time-dependent relaxation losses. Seating losses vary from 1/8 inch (3.2mm) to 3/8 inch (9.5mm) depending on the type of tendon used. Long-term losses due to steel relaxation, concrete creep, bearing plate seating, temperature effects, etc., are usually quoted as between 10% and 15% of transfer load. As an example for an anchor tendon with a final required effective prestressing force of 150 kips (667kn), a free length of 25 feet (7.6m) tendon steel at a stress of 140 ksi (965N/mm) (51.8% of g.u.t.s. for a strand tendon with a g.u.t.s of 270 ksi [1862N/mm]^2) and a modulus of 28 x 10^6 the final elongation can be computed by the use of the following equation:

$$\Delta L = \frac{PL}{AE} \quad \text{or} \quad \Delta L = \frac{140 \times 25 \times 12}{28 \times 10^3} = 1.5'' \ (38\text{mm})$$

If the seating loss is 1/4 inch (6.4mm) and the expected long-term losses are assumed to be 7-1/2% of the design load, the required tendon elongation during stressing is computed as follows:

$$1.5'' \times 1.075 + .25'' = 1.8625'' \ (47\text{mm})$$

By inverting the above formula and solving for P/A, we arrive at a stress of 180.8 ksi (804KN/mm^2) or 64% of g.u.t.s. as a transfer load. If all other factors can be ignored, the final effective prestressing load in this tendon will be 140 ksi (965KN/mm^2).

A retaining wall does not necessarily react in the same manner as a prestressed concrete beam. The wall and soil response as well as potential surcharge, seismic, water level fluctuations and other variable conditions must be taken into account. In addition, on multiple tiered walls, the desired working load on the upper row of anchors may be much higher than the passive earth pressures during the early stage of the excavation. Prestressing to the full design load at this stage may move the wall back into the retained soil an amount that is undesirable. This is particularly true in loose or soft soils. It may, therefore, be desirable in some instances to temporarily lock the anchor off at a load somewhat lower than the final design load.

Final lock-off of anchors at a nominal or small load which is below that computed as being necessary to provide full horizontal restraint is a practice which should be avoided whenever possible. Where this is done, excessive wall movement can occur which would result in structural distortion of the wall, even collapse, and subsidence behind the wall with probable damage to services and adjacent structures.

ANCHOR DESIGN CAPACITY

Empirical Formulae have been developed as a result of theoretical soils considerations confirmed and/or modified by actual construction and testing experience in the field. They are used to calculate the pull-out resistance or ultimate capacity of anchors.

1. Clay Anchors - Figure No. 14 shows the bases for the design formula for calculating underreamed anchors in clay. As can be seen, at ultimate capacity the anchor could fail at three different places: (a) an adhesion failure at the grout/clay interface in the shaft region, (b) an end-bearing failure could occur in the clay which would be analogous to a pile end-bearing failure, and (c) there could be a failure of the clay in shear along the cylindrical plane joining the tips of each of the underreams. Having established, as with all other types of soil, the ultimate load capacity of a clay anchor, factors of safety against pull-out must now be applied. Due to the uncertainty which exists at this time regarding the long-term behaviour of anchors in clay, it is suggested that factors of safety to 2 be used in temporary works and 3 or more for permanent installations.

2. Rock Anchors - In the softer rocks, anchors would be formed in the same configuration as that shown in Figure 14. However, ultimate capacity is not governed by cohesive failure or plastic flow as in clay soils, but by crushing strengths of rock and grout, the grout/rock interface bond strength and the bond between the tendon steel and grout. In massive type rock with few bedding planes, it may

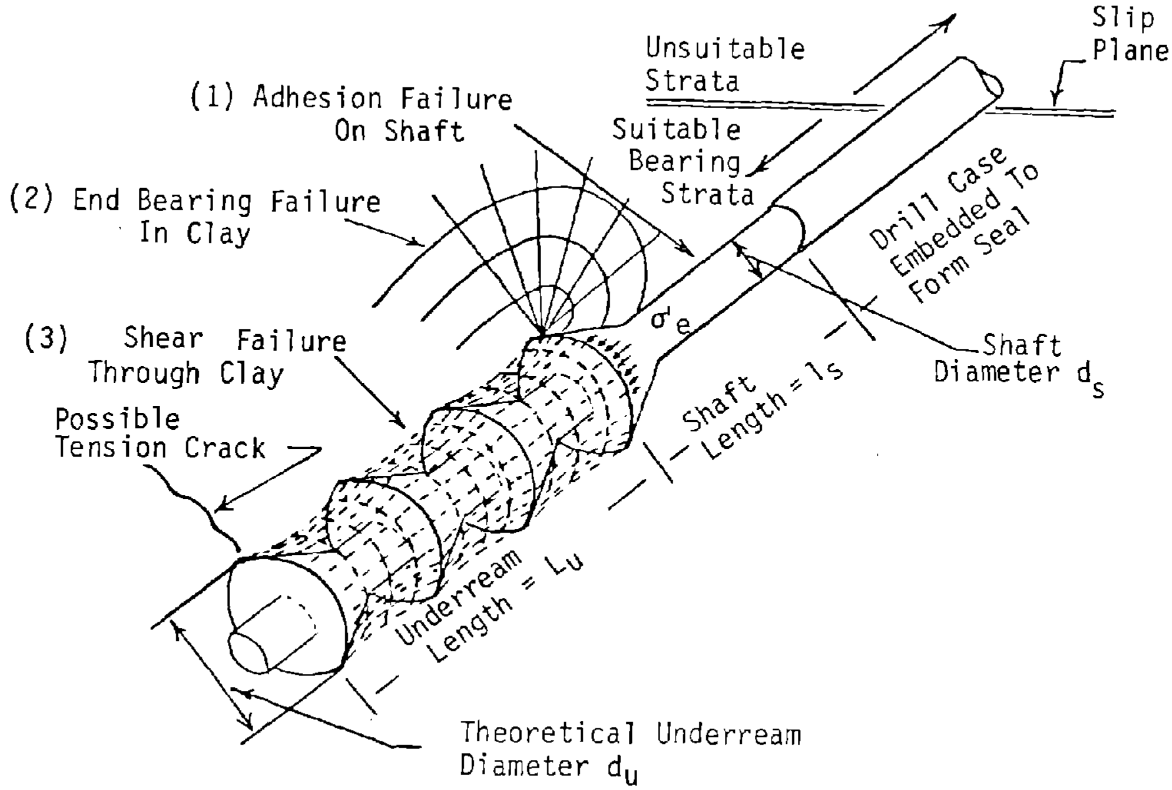


Figure 13. NICHOLSON UNDERREAM ANCHOR AT ULTIMATE CAPACITY

<u>Capacity of a Clay Anchor</u> - Ultimate Capacity $(T_{ult}) = T_s + T_e + T_u$

(1) Shaft Adhesion $T_s = \pi\ d_s \times f_s \times C_u\ (\text{Shaft}) \times l_s$

Area x Adhesion x Soil Cohesion Factor

(2) End Bearing $T_e = \frac{\pi}{4}\ (d_u^2 - d_s^2) \times (N_c \times C_{u(end)} + \sigma'_e)$

Area of End x (Bearing Capacity x Soil Cohesion + The Effective Stress Perpendicular to the End)

(3) Underream $T_u = \pi\ d_u \times f_u \times C_{u(underream)} \times L_u$

Area x Efficiency Parameter x Soil Cohesion of Underream

NOTE: No allowance has been made for water suction.

Where f_s = Adhesion factor
(0.3 to 0.6 dependent upon the type and quality of the clay, etc.)

f_u = Efficiency factor
(0.75 to 0.95 for disturbance caused by under-reamer tool and technique)

N_c = End bearing factor
(6 to 13 dependent on depth, but more usually between 6 and 9. Where lower values are used, a component σ'_e may be added equal to the effective stress perpendicular to the end due to surcharge soil)

σ'_e = Effective stress perpendicular to end of cone.

TABLE III - Typical Bond Stress for Rock Anchors

Type	Ultimate Bond Stresses Between Rock and Anchor Grout Sound, (PSI)	Non-Decayed (PSI)	N/mm
Granite & Basalt	250	450	1.73 - 3.10
Dolomitic Limestone	200	300	1.30 - 2.07
Soft Limestone*	150	220	1.03 - 1.52
Slates & Hard Shales	120	200	0.83 - 1.38
Soft Shales*	30	120	0.2 - 0.83
Sandstone	120	250	0.83 - 1.73
Concrete	200	400	1.38 - 2.76

* Bond strength must be confirmed by pullout tests which include time creep tests.

NOTE: For small load strand anchors (such as single strand) the bond between grout and strand might govern. The bond capacity between grout and strand is about 450 psi (3.10 N/mm).

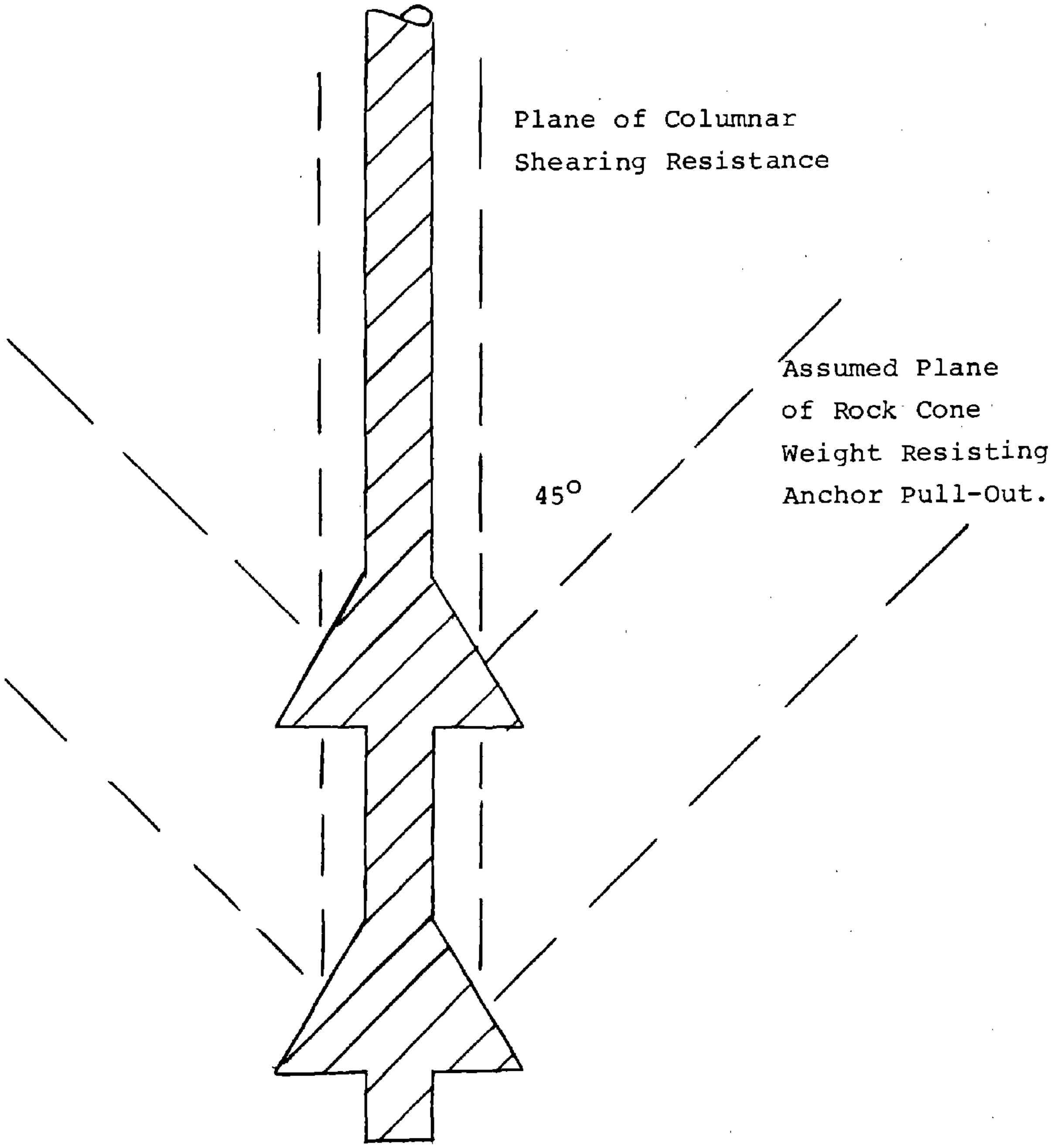


Figure 14. DIAGRAMMATIC REPRESENTATION OF AN UNDER-REAMED ANCHOR IN ROCK

be anticipated that each underream would exert forces on the rock that would be dissipated at a 45° angle from the direction of the pull. The conical stress fields so produced would overlap and form one large cone expanding upwards in the direction of the top of the hole. Figure 15 shows this. It is this large inverted cone that is providing the resistance to pull-out required in the rock anchor. The deeper the penetration of the anchor in the foundation material, the larger the cone becomes and, theoretically, the larger the possible ultimate anchor load. In a bedded or fractured rock, the cone size and shape will vary with the distribution of bedding and cleavage planes and the grout take in the fissures; while in the case of badly broken material, stress distribution patterns may be closer to those shown in Figure 14 for clay anchors.

Experiments have shown that the mode of failure in shallow anchors in rock, i.e., with a total length of less than 10 feet, is of the conical form described, although there is some difference of opinion as to the angle of the apex of the cone. High capacity anchors are seldom constructed in such proximity to the surface and so other factors become important in their design. These are the crushing strength of the grout, the tested values of the rock, the magnitude of the bond developed between the grout and the rock, and the tendon configuration. This last is the mechanism whereby the stress in the tendon is distributed to the grout and then the rock. This distribution should be as even as possible; and local concentration of forces should be avoided. For instance, a plain smooth bar terminating in in an end-plate is the worst possible tendon layout and can cause premature anchor failure through rock or grout crushing due to the intense pressure concentrated on and around the end-plate. These considerations apply equally to underreamed and straight shafted anchors.

The diameter of the drilled hole is largely governed by the size of tendon required to carry a specified tonnage. For anchor capacities up to approximately 250 tons (2224KN), a 4-1/2 inch (115mm) diameter hole is satisfactory. Depending on the type of rock and the load, underreams should be formed to approximately 12 inches (30m) diameter with either two or four underreams per anchor. The need for underreams diminishes as rock strength increases until, with hard massive rock, underreaming may not be necessary at all.

Provided that the tendon layout is designed to evenly distribute the anchor stress, e.g. a basketed and banded strand tendon, the fixed anchor length for straight shafts may be determined from a fairly simple empirical approach which takes into account grout and rock strength.

$$L = \frac{T_{ult}}{\pi \cdot d_s \cdot f_c \text{ (or } f_r \text{ where } f_r < f_c)}$$

where,

T_{ult}	=	Ultimate anchor capacity
d_s	=	Diameter of borehole
f_c	=	Crushing strength of grout x 0.1
f_r	=	Crushing strength of rock x 0.1
L	=	Fixed anchor length

In calculating pull-out resistances the lesser of the values for f_c or f_r is used up to a maximum of 600 psi ($42N/mm^2$). This gives a conservative factor of safety of 3 based on the normal range of grout/rock working bond stresses used having a maximum of 200 psi ($1.38N/mm^2$).

For underreamed anchors in soft rocks the same formula may be used but substituting 2/3 d_u for d_s. In this, the expression d_u = diameter of the underream bell.

The maximum figures given above are considered to be conservative at the moment, but continual investigation and widening of experience of anchor performance could well result in these design parameters being much more concisely defined.

3. Sand Anchors - In considering the loads obtainable from sand strata, it is necessary to obtain accurate soil data, including sieve analysis grading curves, angles of internal friction and strata thickness. Where this description indicates that the soil permeability would be $k=10^{-1}$

to 10^{-2} cm/sec, the fixed anchor formed where the grout pressure is a nominal 10 to 40 psi [$0.0069 N/mm^2$] (i.e., the hydrostatic head dependent upon a drilled depth) would not consist of a smooth grout cylinder since the sand would permit some permeation by a very fluid cement grout. Field trials have enabled an empirical rule to be established which is as follows:

$$T_{ult} = L\, N' \tan \phi$$

where,

T_{ult} = Ultimate load capacity of the anchor (kips)

L = Fixed length of anchor (feet)

N' = 27 to 41 kips per foot

ϕ = Angle of internal friction

In this equation the factor N' (27-41 kips per foot [36- 56kN per m]) automatically takes into account the depths of the overburden to the top of the fixed anchor (H = 20' to 45' [6-14m]) the effective diameter of the fixed anchor (d = 15" to 24" [380mm - 610mm]) together with a range of anchor lengths over which the rule has been tested. Where the description of the sand indicates that its permeability would be $k = 10^{-2}$ to 10^{-4} cm/sec., permeation of the sand by the cement grout would not occur and a somewhat smoother grout cylinder would result. The rule above can be adjusted to allow for this and the factor N' taken to be 9 to 11.5 kips per foot (12 - 16KN/m). In this case, the factor N' automatically takes into account the depths of overburden to the top of the fixed anchor length (H = 18' to 30' [5.5 - 9m]) and the effective diameter of the fixed anchor (d = 7" to 8" [28mm -31mm]) together with the range of anchor lengths over which the rule has been tested.

However, the use of pressure grouting techniques means that increased anchor loadings can be expected by virtue of the fact that greater penetration of the cement grout into the surrounding soil is achieved, consolidation or densification of the soil takes place and there is a residual "locked-in" grout pressure remaining after completion of pressure grouting in the fixed anchor length. A further empirical formula has been derived from field trials to express this, using the factors above:

$$T_{ult} = p' \cdot \pi \cdot d \cdot L \cdot \text{Tan}\,\emptyset$$

where,

T_{ult}	=	Ultimate load capacity of the anchor
p'	=	Grout pressure at 2 psi (0.014N/mm^2) per foot of over-burden above the top of fixed anchor taken as an average over the fixed anchor length.
d	=	Effective diameter of fixed anchor
L	=	Fixed anchor length

In the foregoing formula the factor p' has been used to express the known increase in grout/soil friction that occurs due to pressure grouting and is therefore related to known dimensions. The actual grout pressure used during anchor construction has been considered in attempting to verify anchor capacity, but this has been shown to be over optimistic in its approach as well as subject to variability due to site conditions. The insertion into the formula of the very high post grouting pressure used, for example, in association with the tube-a-manchette technique would make this calculation very unrealistic. As very high pressures cause ground heave and hydraulic facturing, consider this to be desirable. However, a more conservative approach is recommended on the grounds of safety and stability and thus the grout pressure used in the formula should also be used to guide the determination of grout pressures during actual construction.

The effective diameter of the grout bulb, d, is estimated from the soil permeability with rates of 10^{-1} to 10^{-2} indicating soil infiltration by the grout and 10^{-3} to 10^{-4} soil densification local to the borehole. Grout takes determined during construction can be a good guide to grout bulb size and eventual anchor performance.

For optimum anchor performance, grout should be plain cement and water mixes with no additives whatsoever unless there is an overriding and proven reason for their use.

It must be strongly emphasized that anchor grout is a main structural component of the total system. Therefore use of any material which could affect grout strength or competency should be disallowed. From long experience and specific research, it has been shown that additives in anchor grouts can reduce strength and adhesion, and expansive agents in particular, while reducing grout strength through unrestrained expansion in open boreholes, also raise the question of the effect of released hydrogen upon the tendon steel brittleness. Plain cement/water grouts have proved their reliability and consistency of performance over the years and so additives are superfluous and potentially damaging unless some overriding reason is present that dictates their use.

ANCHOR INDUCED VERTICAL STRESS ON RETAINING WALL

The computation of vertical forces in the wall system due to anchor stress is a simple task involving triginometrical functions. For instance, where a wall design indicates a horizontal tie reaction is required of 50 tons (445KN) and that the anchor angle is 15°, then total anchor design load will be

$$\frac{50}{\cos 15°} = 51.75 \text{ ton (460KN).}$$

The vertical component in the triangle of forces is, therefore, 51.75 x SIN 15° or 50 X Tan 15° = 13.4 tons (119KN) for 30° the same horizontal load would result in a total load of

$$\frac{50}{\cos 30°} = 57.75 \text{ tons (514KN)}$$

and a vertical load of 50 X Tan 30° = 28.9 tons (257KN) and for 45° total load

$$\frac{50}{\cos 45°} = 70.7 \text{ Tons (629KN)}$$

and vertical load of 50 X Tan 45° = 50 Tons (445KN).

Once the vertical load in the structure induced by anchor loads has been determined, a static analysis of the components of the wall sheet piles, wales, H-beams, etc. can be made to ensure that no component is over-stressed. Components can be sized according to the load requirements. Particular care must be taken to insure that the wall system has sufficient end-bearing to resist the vertical or compressive forces induced and, where soils data suggests a deficiency in this respect, additional measures must be taken to prevent draw-down of the wall components. Any vertical movements of the wall will change the stress level in the anchors and thus the horizontal reaction to earth pressure. Significant movements could ultimately lead to total failure of the entire retaining wall.

End bearings should be assessed from the soils information and checked during construction by means of sheet pile and H-Beam driving records, anchor and/or caisson drilling records and site survey of settlement monitors.

RETAINING WALL DEFLECTIONS

Estimates of wall deflections should be made at each anchor location at various construction stages as part of the full static analysis of the total wall system. Wall deflections, if they occur, will be accompanied by movements of the retained soil either by settlement or by heave. These effects are inter-dependent and should be considered together. The following paragraph deals with ground surface settlement.

SETTLEMENT OF THE GROUND SURFACE BEHIND THE RETAINING WALL

Earth pressure is the force per unit area exerted by the soil on a retaining structure. The magnitude of this pressure depends upon the physical properties of the soil, the size and character of the retaining wall and the loading conditions being imposed. This earth pressure is not a unique function for each soil, but rather a function of the total soil/structure system. Movements of the structure are primary factors in developing earth pressures. Calculation of these movements is highly indeterminate.

Two stages of stress in the soil are of particular interest in the design of retaining structures as they define the stress limits. These are the active and passive states.

If a wall deflects under the action of lateral earth pressure, each element of the soil adjacent to the wall will also expand laterally mobilizing shear resistance in the soil and causing corresponding reduction in lateral pressure. In other words, after movement occurs, the soil becomes more self supporting. The minimum value of this pressure at the point of movement or failure of the soil is known as the active pressure. On the other hand, where the wall is pushed towards the soil as in a bridge abutment, lateral pressure will increase as the shearing resistance of the soil is mobilized. The maximum value of this pressure at the point of failure of the soil is know as the passive pressure. Between these two pressure conditions is a third which may be described as the "at-rest" condition when ground movements are essentially minimal and changes are basically in internal stresses in the soil.

One of the many advantages offered by stressed anchors when used in a retaining wall is their inherent ability to maintain a state of equilibrium by exerting pressures equal to that of the soil being retained. In this way neither horizontal wall deflections nor any accompanying ground settlement behind the wall can theoretically take place. In other words, the soil can be maintained in its "at-rest" stress state. In practice, however, a certain amount of movement is unavoidable due to construction sequencing. This can usually be kept to a minimum and related to the elastic properties of the total soil and wall system components.

To control settlement of the ground surface behind a tied back wall, a construction procedure and anchor stressing sequence can be developed to induce an at-rest earth pressure stress condition in the retained soil mass. Changes in this imposed at-rest earth pressure will depend primarily on wall flexibility and anchor creep characteristics.

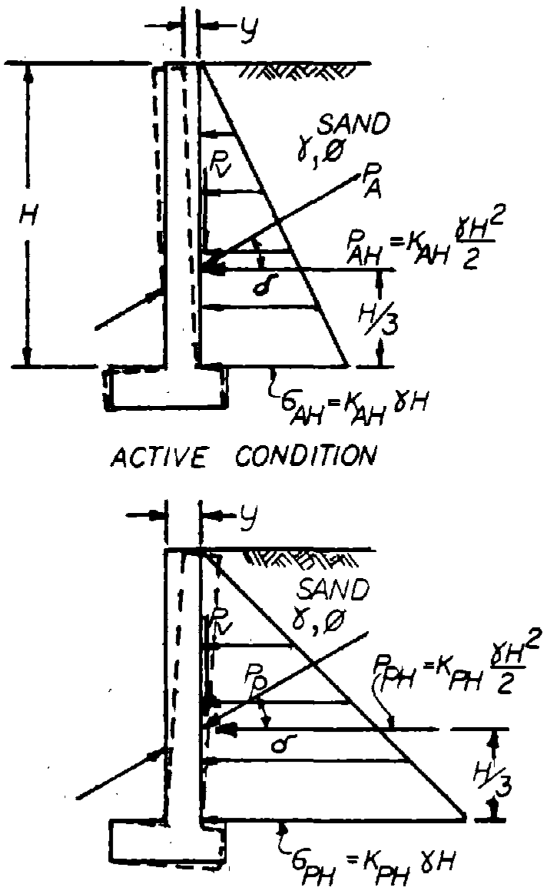
These changes will, however, be small if the wall is essentially rigid and the anchors are evaluated for long term creep on the basis of test results. Movements will be related to the elastic properties of soil and structure. It has been shown that a modest level of prestress induced in the soil mass through the tensioning of the anchors has a beneficial effect of increasing soil strength and friction between wall and soil. However, at no time should this stress level approach the passive pressure case.

Some settlement of the ground surface behind a tieback wall will occur during excavation to install the uppermost row of anchors. The wall movement will be consistent with the movements required to develop the active earth pressures which will probably be experienced above the first anchor row during excavation. An estimate of the magnitude of horizontal wall movement required to develop the active earth pressure condition in sands can be made based on field measurements and empirical charts. One such set of charts appears in Fig. 16 in NAVFAC DM-7. This chart relates the coefficient of horizontal earth pressure to wall movement and was prepared from measurements made by Terzaghi and Tschebotarioff. The magnitude of the ground surface settlement behind the wall can be approximated utilizing estimated horizontal wall movements required to develop active earth pressures, assumptions of wall deflection patterns, configuration and extent of the stressed soil zone and Poisson's Ratio of the soil.

Soil stress versus strain relations measured in laboratory triaxial compression tests can be used to better estimate horizontal wall movements due to earth pressures. Strains required to reach an active earth pressure condition in cohesive soils, as measured from the results of laboratory triaxial compression tests, are used to estimate the settlement of the ground surface behind the wall in the case of cohesive backfill or retained soil.

Refined predictions of the ground surface settlement behind the wall during and following construction may be necessary when facilities particularly sensitive to settlement exist in close proximity to the back of the wall. In such cases, a finite element model of the wall, soil, and anchor system can be developed. This model can incorporate soil yielding. Sophisticated finite element models are available which can predict excess pore water pressures developed due to anchor stressing and settlement which results from their dissipation. Results of stress path triaxial compression tests, field strength tests and field loading tests prior to construction can be used to establish an initial construction plan. Measurements using field instrumentation such as piezometers, slope indicator, earth pressure cells and tieback load cells can be made. These measurements can be compared with corresponding quantities initially predicted and the in-put parameters to the finite element analysis adjusted accordingly. The construction procedure may subsequently be adjusted if required based on the refined predictions.

The above studies are very refined and may not be warranted on certain straight-forward projects of an uncritical nature. The basic concept of calculating pressures and then applying resistive loads through the use of tensioned anchors is all important to the idea of maintaining equilibrium. Simply stated, the retained earth should never know that its original support has been removed and replaced by another type.



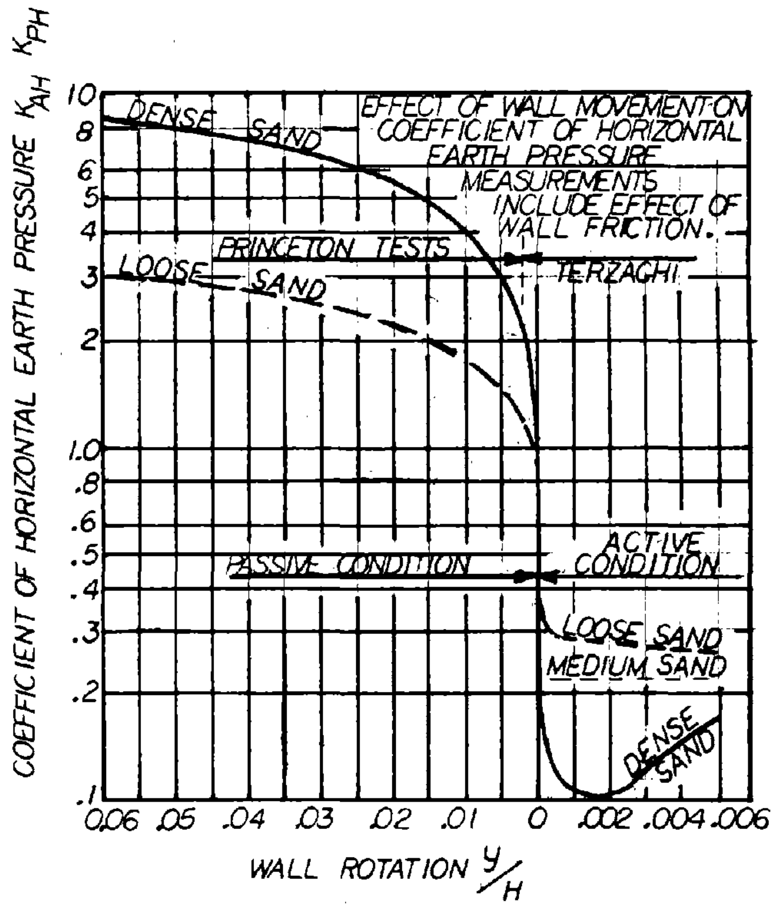
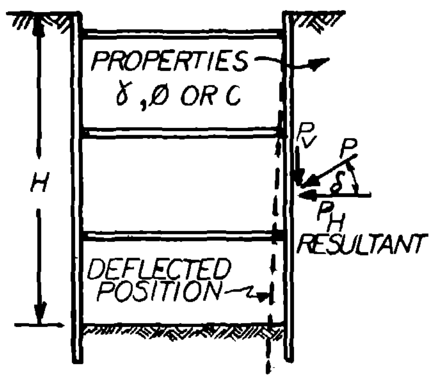


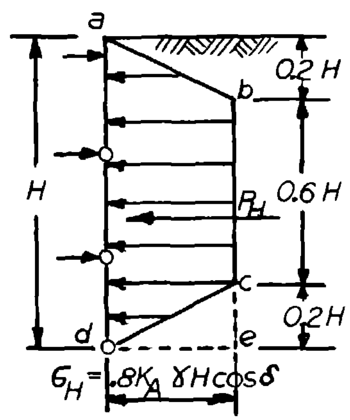
Figure 15. CONDITION OF ROTATION ABOUT BASE OF WALL



SHEET PILES OR SOLDIER BEAMS DRIVEN.

AS EXCAVATION IS DEEPENED WALES AND BRACES PLACED IN SEQUENCE.

WALL DEFLECTION INCREASES WITH DEPTH, RESULTING IN TRAPEZOIDAL PRESSURE DISTRIBUTION.



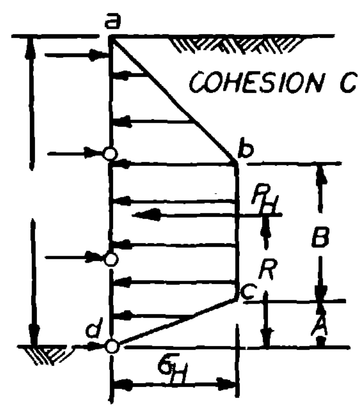
EXCAVATION IN SAND

AREA abcd PRESSURE DISTRIBUTION IN DENSE SAND:

RESULTANT, $P_H = (0.64) K_A \gamma H^2 \cos\delta$, ACTING 0.5H ABOVE BASE OF CUT.

AREA abde PRESSURE DISTRIBUTION IS IN LOOSE SAND:
RESULTANT, $P_H = (0.72) K_A \gamma H^2 \cos\delta$, ACTING 0.42H ABOVE BASE OF CUT.

K_A IS THE COEFFICIENT OF ACTIVE EARTH PRESSURE, INCLUDING EFFECT OF WALL FRICTION



EXCAVATION IN CLAY

AREA abcd PRESSURE DISTRIBUTION IS THE SHAPE OF THE PRESSURE DIAGRAM AND MAGNITUDE OF PRESSURES DEPENDS ON VALUE OF STABILITY NUMBER:

$$N_o = \frac{\gamma H}{C}$$

	$2<N_o<5$	$5<N_o<10$	$10<N_o<20$	$20<N_o$
P_H	$.78H\sigma_H$	$.78H\sigma_H$	$(2.1-0.55)H\sigma_H$	$.5H\sigma_H$
σ_H	$\gamma H-1.5(1+N_o)C$	$\gamma H-4C$	$\gamma H-(.8-.4N_o)C$	γH
A	$.15H$	$.15H$	$(.3-.015N_o)H$	0
B	$.55H$	$.55H$	$(1.1-.055N_o)H$	0
R	$.46H$	$.46H$	$.38H$	$.33H$

Figure 16. CONDITION OF ROTATION ABOUT TOP OF WALL

LONG TERM SOIL-ANCHOR CREEP

The stability of earthwork in fine grained soils is time dependent. This is because the average size of the interconnecting pores is so small that the displacement of pore water is retarded by viscous forces such as the surface tension of water. This resistance is measured in terms of flow rate through the soil and expressed as its permeabilty.

Permeability is the largest quantitative difference between soil of different time dependant stability. For example, a sand and a normally consolidated clay may exhibit similar effective stress shear strength parameters (c and tan Ø) but the permeability of the clay is several orders of magnitude lower. The stability of the clay under load is thus time dependent, whereas the more permeable sand reacts to loading changes almost immediately.

If a saturated clay is loaded, the immediate change in effective stress is minor with most of the load going into the pore water. With time, however, this excess pore water pressure is dissipated by drainage away from the area of increased pressure into the surrounding area of lower pressure which is unaffected by the construction process. This dissipation of pore water pressure causes an increase in effective stress and a time dependent reduction of soil volume in the zone of influence, i.e., soil consolidation. The soil structure will stiffen and give rise to decreasing settlements and increased strength. For the short term quick loading condition the stressed soil does not immediately change its water content or its volume. The load increment does, however, distort the stressed zone. The effective stresses change along with the change in shape of the soil structure. Eventually the changes in structural configuration may no longer result in a stable condition and this instability gives rise to plastic flow and the soil "fails".

Long term creep is related to permeability and soil grain size and is not normally a factor of great significance when coarse grained soils of a free drainage nature are considered.

In time-dependent soils, large creep displacements under constant load can take place before failure load is reached. Therefore, for the design of permanent anchors it is essential to know the load creep displacement relationship as a function of time. There is generally a relationship between displacement and time which is an exponential mathematical function, i.e., a straight line is obtained when results are plotted to a semi-log scale. The slope of this line can be considered as a creep

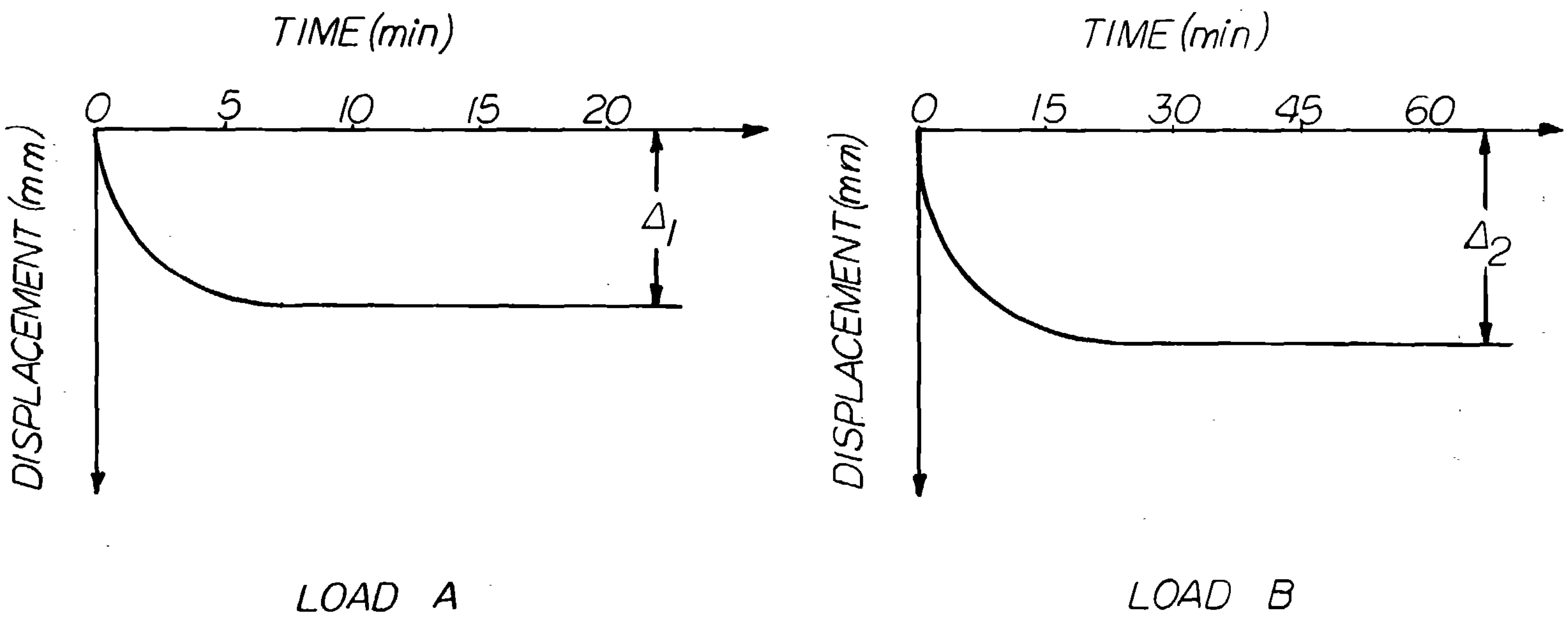


Figure 17. CREEP DISPLACEMENT

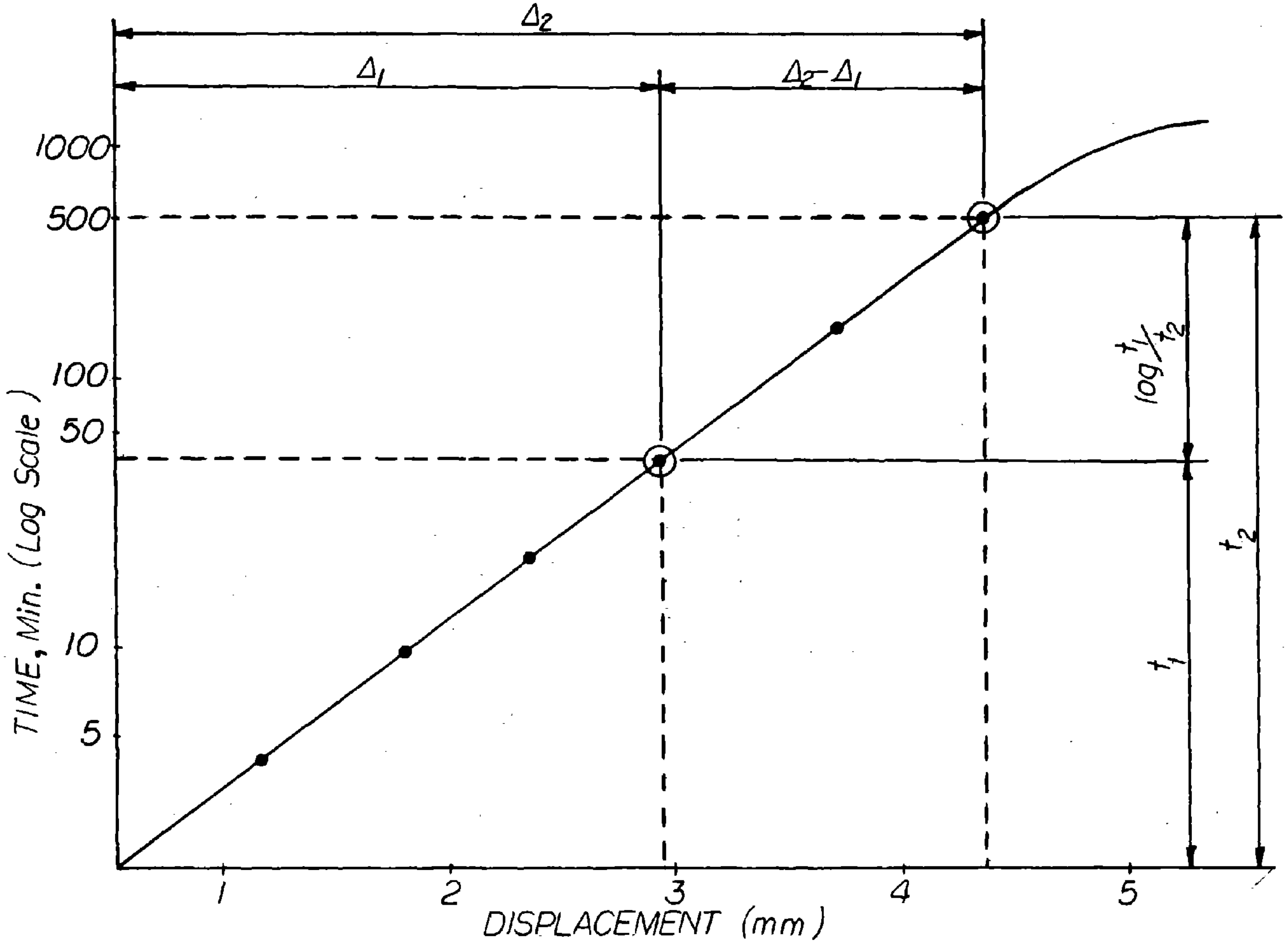


Figure 18. TIME-DISPLACEMENT CURVE

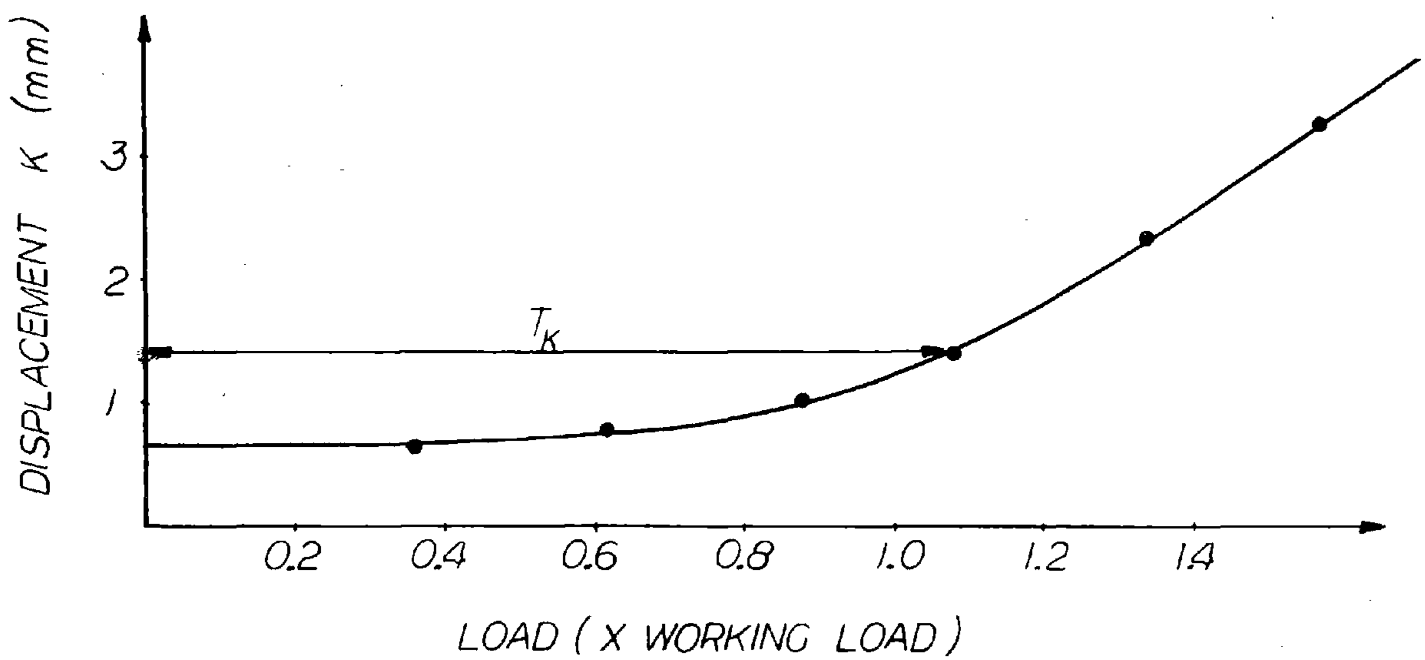


Figure 19. METHOD OF DETERMINING LIMIT FORCE, T_k

coefficient and the slope increases with each increase in load. When the ultimate load is reached, the displacements do not decrease with time, but movement is continuous for a given constant load. This gives a definition of anchor failure (a failure to accept more load) and the basis for assessing anchor working loads.

Pre-contract test anchors should be constructed to verify anchor design capacity and test loading sequences should be formulated so that observation of creep relative to time can be made at any load stage. One or two load cycles should be carried out so that anchor seating can occur and also so that elastic and plastic displacement can be judged. Anchors should then be restressed and the limit load for minimal or acceptable creep determined. The displacements must be measured under constant loading during a given period and the results can be plotted as in Figure 17.

Recommended minimum observation periods are stated in Chapter II Test Procedures, but these periods can be modified if necessary to make sure the trends are clear and the creep coefficent $K\Delta$, related to the displacement of the fixed anchor, can be determined.

The draft German code for permanent soil anchors (DW 4125 - 1974) recommends that creep be calculated as follows:

$$K_\Delta = \frac{\Delta_2 - \Delta_1}{\log \frac{t_2}{t_1}}$$

The values may be evaluated at different stages of loading and can be recorded as in Figure 18.

By definition, the limit anchor force T_k corresponds to a creep K_Δ of under 2 in the above example (where $t_2 = 10 \times t_1$). The limit force can be determined from plotting the results of creep measurements against load as shown in Figure 19.

In the above, that part of creep associated with creep of cement, long-term relaxation of steel, partial debonding of the steel/grout interface and other similar longer term sources of load change are not considered to have any significant effect upon the determination of the creep coefficient. In the long term, it has been estimated that those factors could correspond to a creep factor of up to 0.016 inch (0.4mm) depending upon the stress level in the steel.

STEEL RELAXATION

The rate of steel relaxation varies with initial stress and the type of steel. Relaxation from an initial stress of up to 50% g.u.t.s. may be considered negligible in practice. For initial stresses greater than 0.55 f_y the relationship is:

$$\frac{f_s}{f_i} = 1 - \frac{\log t}{10}\left(\frac{fi}{fy} - 0.55\right)$$

Where, f_s = residual stress after time t
f_i = initial stress
f_y = 0.1% proof stress at working temperature
t = time in hours after application of initial stress

It can be expected that long-term relaxation losses for an anchor tendon locked at 70% of g.u.t.s. may amount to about 7-1/2% of transfer load. If the anchor is loaded to 50% or less of g.u.t.s. at transfer load, as most permanent anchors are, the relaxation losses should be less than half this value or about 4%. For an anchor of 25 feet stress length loaded to 75 tons (667KN) or 51.8% g.u.t.s., the maximum calculated movement of the anchor tendon will be 0.04 X 1.50 inches (38mm) or 0.060 inches (19mm). Movements of this magnitude can be neglected in almost all structural applications relating to earth retaining systems.

WALL SYSTEM MONITORING

There is a basic disadvantage in the assessment of long-term anchor behaviour by relying on the creep test. An essential part of the test is the recording of displacement against time for a constant load. This can only be done under live test conditions and it is rare indeed for site conditions to be such as to allow the setting up of long-term tests. This is irrespective of the cost elements involved. It is much more usual to carry out lift-off tests to establish residual loads in the anchors and these can be performed on selected anchors until access to them is denied due to construction sequencing.

Long-term anchor and wall system monitoring can be carried out if preparations are made at the design stage. The information that can be obtained is invaluable and permits correlation of anchor load fluctuations with the performance of the structure and any environmental changes.

It is recommended that every critical structure which depends on soil anchors for its stability should be instrumented for long-term monitoring. The common types of instrumentation for retaining walls are:

1. Load Cells - A hollow center load sensing device permanently placed under the anchor head to monitor load changes of the anchor.

2. Earth Pressure Cells - Placed between retaining wall and soil.

3. Slope Indicators - or inclinometers placed at selected locations behind the wall to monitor wall and ground movements.

4. Precise surveys - of the wall system and surrounding areas.

LOAD CELLS

If load cells could be made fail safe they would provide invaluable information. The proper use of the load cells would give the designer greater faith in the use of permanent soil anchors. The history of load cells is that there is a relatively high incidence of malfunctioning and, therefore, a distrust of the results. Studies have been made of experiences on 7 different anchor projects on which load cells were used. One job involved 13 cells of two different types: vibrating wire gauge type and bonded resistance strain gauge type cells. The balance of the jobs involved the use of bonded resistance type strain gauges. In light of this experience, the bonded resistance strain gauge type is the preferred type load cell because they are less complex and less prone to malfunctions. However, the strain gauge type are subject to some problems and these are listed in order of importance:

1. Moisture infiltration and effects upon readings.

2. Electrical malfunctions.

3. End restraint conditions on the cell.

4. Resistance in the conductor cable reducing sensitivity when long lengths are used.

5. Temperature variations in uncompensated systems.

Because of the potential malfunctioning problems, it is necessary to design the load cell and the anchor connection so that the load cell can be removed, repaired and replaced. It is also necessary to provide for occasional lift-off checks that can be performed on the instrumented anchors to verify the load cell readings. In this manner, the validity of the load cell readings can be proved and much valuable data collected.

From experience with instrumented anchored retaining walls it appears that the specified anchor loads were usually higher than necessary for restraining the wall. This has been confirmed by the absence of load changes in the anchors during the period of time they were monitored (up to 3 years). Generally a 10% change in load can be expected due to ground water and temperature changes, and inaccuracies in the measuring system. A degree of change over and above 10% may well indicate that a fundamental change is occuring -- for instance, a decrease in load could indicate that the anchor is creeping, that the wall is being pulled back into the retained earth, or that the wall is moving vertically downward. An increase in load may be attributable to an outward movement of the wall or an increased surcharge load. If the correct lock-off load is applied to the anchor initally there should be little change recorded in the anchor load during the life of the structure.

Another method of monitoring anchor performance is an indirect method, the slope indicator. By placing an inclinometer tube in the wall or in the soil directly behind the wall, the deflections of the wall can be monitored. If deflections are recorded near anchor supports, it can be inferred that there is movement and/or changes in load of the anchor. Although this is an indirect method as concerns the anchor, it will serve as an early warning device for distress to the wall, and alert the engineer, as deflections are recorded, to potential problems.

Both methods should be supported by precise surveys of wall and surrounding ground for both alignment and elevation.

WRITERS' NOTE

In ten years of experience, he has not become aware of wall movement or deflection traceable to creep in a properly installed anchor. This experience relates only to granular, non-plastic soils and rock as the author has not been directly involved with the monitoring of permanent anchors in cohesive or plastic type soils. Numerous walls on which permanent anchors have been used have been monitored for movement for periods up to 10 years with no reported instances of anchor creep or wall deflection.

Examples are:

PROJECT: Roanoke Memorial Hospital, Roanoke, VA
GENERAL CONTRACTOR: Nello L. Teer Company, Durham, NC
TESTED ANCHOR LOAD: 120 Tons
TYPE STRUCTURE: Tieback Anchors for a Composite-Steel Beam Concrete Retaining Wall 58' High
SOIL CONDITION: Colluvial and Alluvial Silts and Sand

PROJECT: Montefiore Hospital, Pittsburgh, PA
GENERAL CONTRACTOR: Navarro Corporation, Pittsburgh, PA
TESTED ANCHOR LOAD: 200 Tons
TYPE STRUCTURE: Tieback Anchors "Tangential Caisson" Retaining Wall
SOIL CONDITIONS: Clay Shale

OWNER: Pennsylvania Department of Transportation
LOCATION: Moon Township, PA
GENERAL CONTRACTOR: Nicholson Pile Company, Bridgeville, PA
TESTED ANCHOR LOAD: 200 Tons
TYPE STRUCTURE: Tieback Anchors Restrain Existing Failed Retaining Wall
SOIL CONDITION: Silt Shale

PROJECT: Salem Nuclear Generating Station, Salem, NJ
OWNER: Public Service Gas & Electric of New Jersey
GENERAL CONTRACTOR: J. Rich Steers Co, NY and United Engrs. & Constr., Philadelphia, PA
TESTED ANCHOR LOAD: 120 Tons
TYPE STRUCTURE: Tieback Anchors Cofferdam for Intake Structure
SOIL CONDITION: Dense Cemented Sand

OWNER: U.S. Army Corps of Engineers, Detroit District
GENERAL CONTRACTOR: Bultema Dock & Dredge Co, Muskegon, WI
TESTED ANCHOR LOAD: 70 Tons
TYPE STRUCTURE: Tieback Anchors for Steel Sheet Pile Bulkhead
SOIL CONDITION: Sand, Silty Sand

PROJECT: North Anna Power Station, Unit No. 1, Mineral, VA
OWNER: Virginia Electric & Power Company
GENERAL CONTRACTOR: Stone & Webster Engr. Corp., Boston, MA
TESTED ANCHOR LOAD: 45 Tons
TYPE STRUCTURE: Tieback Anchors West Wall-Heating Boiler Room
SOIL CONDITION: Fine Sand Backfill

Measurements have ranged from the crude, i.e., visual, to precise surveys, slope indicator monitoring and load cells. In no instance is the author aware of actual changes in anchor load of greater than 10% or of movements of the wall of greater than 1/4 inch (0.64cm) due to anchor creep or loss of load in the anchor.

As was previously discussed, it is the opinion of the writers that, with proper installation techniques and stressing and testing procedures, wall movement due to anchor creep can be almost eliminated.

In cohesive or plastic type soils, there will probably be some movement of the anchor and associated deflection of the wall. These movements can be anticipated by rigid control of anchor testing procedures. By testing an anchor to 2 or 2-1/2 times the working load with an appropriate holding period of an absolute minimum of 24 hours, the creep of the anchor may be predicted by its behavior under the test loading.

GROUND WATER

Apart from the effects of water upon the cohesive properties of a soil, there will also be a decrease in active pressure below the water table since the submerged density of the soil is used:

$$Pa = K_a \gamma' H$$

where γ' is the submerged unit weight of the soil

However, the total pressure on the back of the wall will increase owing to water pressure. Suitable drainage is usually provided at the back of the retaining wall to reduce this hydrostatic head. It follows that account must be taken at the time of design to estimate the long term water table condition in the retained soil and to design for the worst case.

Retaining walls built in connection with waterfront facilities are subjected to maximum earth pressure where tide or river level is at its lowest stage. A receding tide or high water, or a heavy rain storm may cause a higher water level behind a sheet pile wall than in front depending upon the type of backfill used. If the backfill is fine or silty sand there will be a time lag between water levels equalizing either side of the wall and so the height of water behind the wall could be several feet. If the soil behind the wall is silt or clay, full hydrostatic pressure on the back of the wall should be assumed up to the highest position of recorded water level.

As has been stated, water level differences on either side of the wall cause additional pressure on the back of the wall and a reduction in unit weight of the soil, thus reducing active pressures and passive resistance. An additional reduction in soil unit weight is caused due to the upward seepage pressure exerted by the unbalanced head of ground water on the soils in front of the outer face of the retaining wall. From the terms in Figure 20 , it is possible to approximate this reduction by using:

$$\Delta \gamma' = \frac{20\ H_u}{D}$$

Where, $\Delta \gamma'$ = reduction in submerged unit weight of soil, p.c.f.

Hence, the effective unit weight to be used in computing passive pressure would be:

$$\gamma' - \Delta\gamma' = \gamma' - \frac{20\ H_u}{D}$$

Where, H_u = Unbalanced waterhead
D = Distance below excavation or dredged level to toe of wall

Downward seepage in the soil behind the wall and ground water percolation has only a very small effect on free draining soils and may, therefore, be neglected.

The effect of ground water upon anchor construction is not as marked as may be thought, provided that due consideration is given to the requirements for minimum overburden pressure for anchors close to ground surface. Otherwise, it seems that the presence of water in the soil and the installation of anchors below water table has no marked effect upon the load carrying capacity of those anchors.

This is probably due to the fact that ground anchors tend to defy the normal laws of soil mechanics in that, when tested they develop skin frictions in the bond length many times that of the effective overburden pressure. So, providing there is sufficient overburden to prevent ground heave or shallow foundation failure, and grout pressure is kept below that needed for hydraulic fracturing, it would seem, from years of experience, that the effect of the effect of ground water upon anchor performance is minimal.

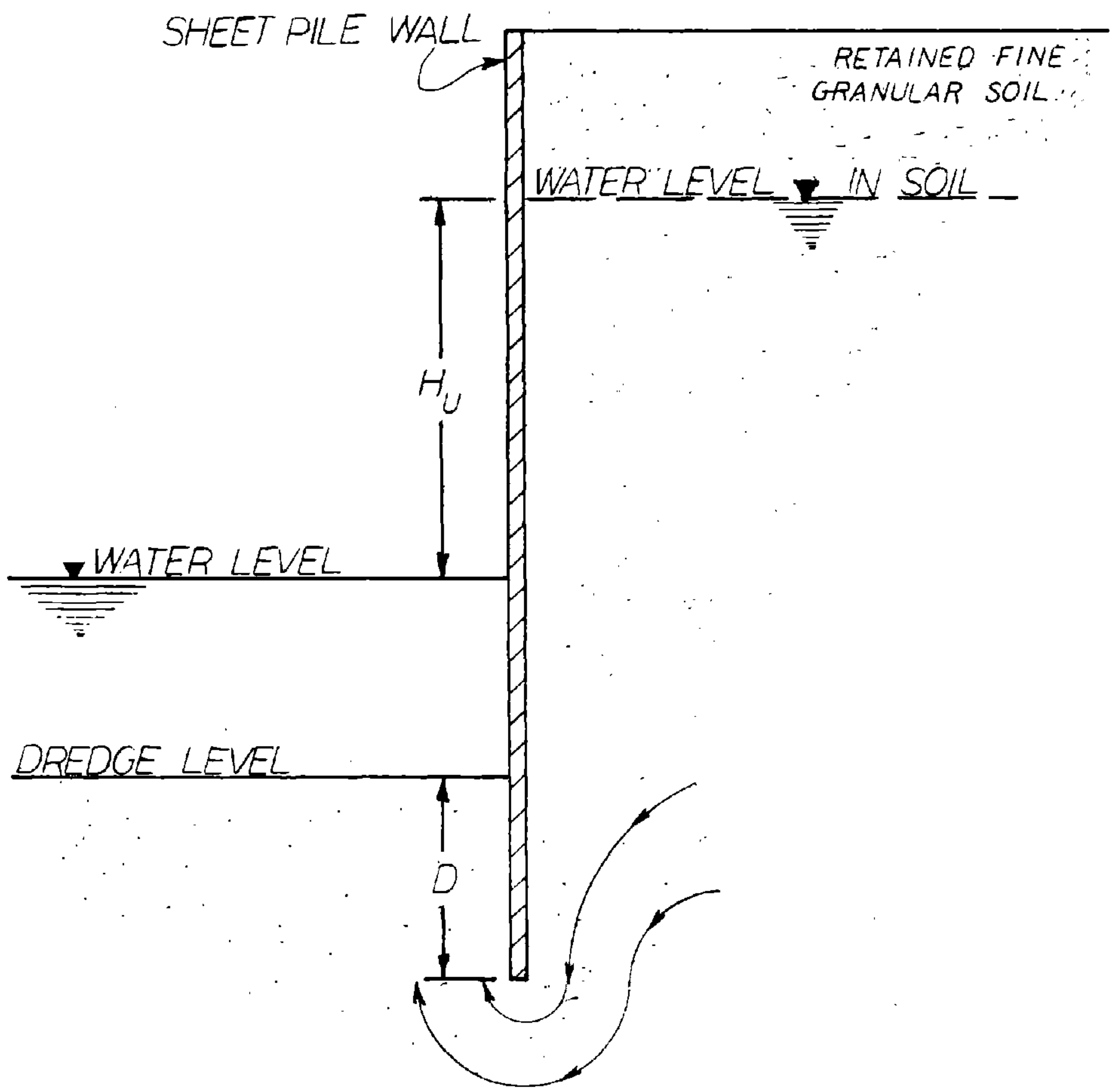


Figure 20. HYDROSTATIC AND SEEPAGE PRESSURES

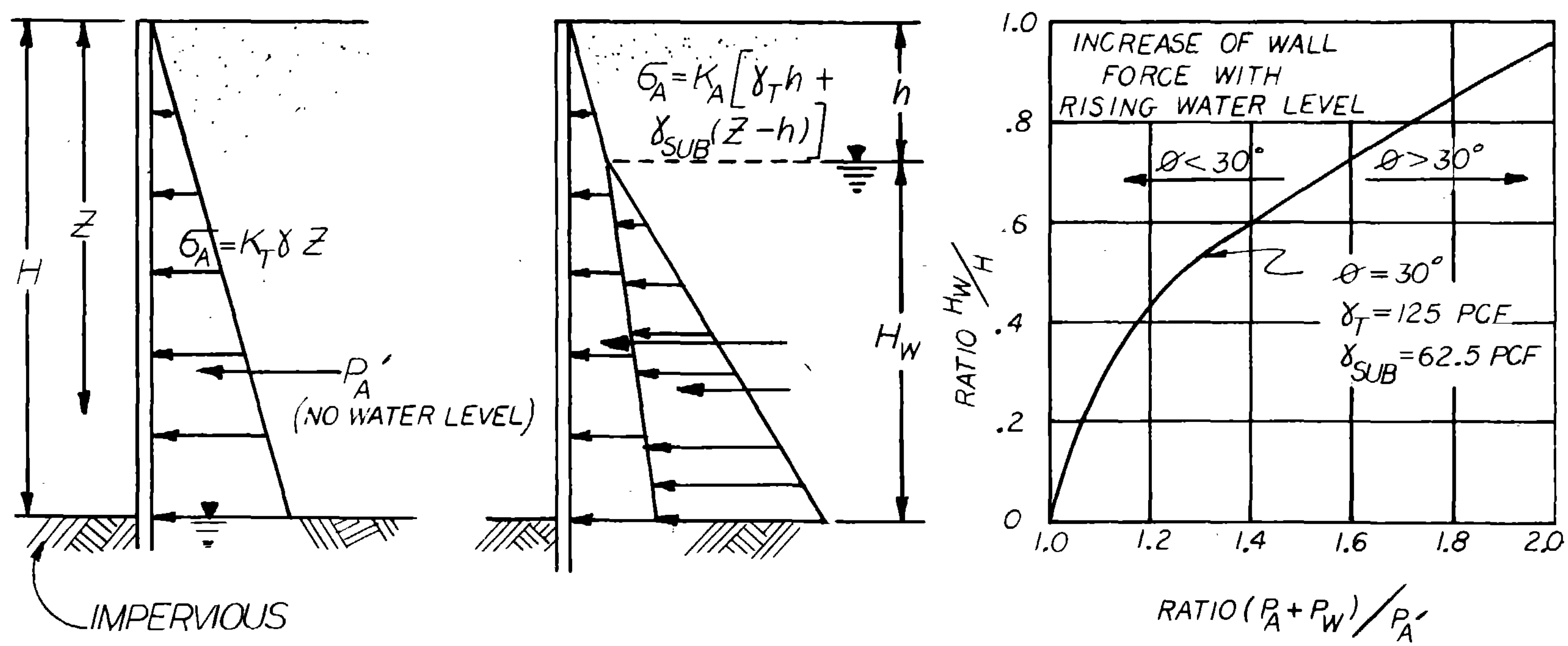
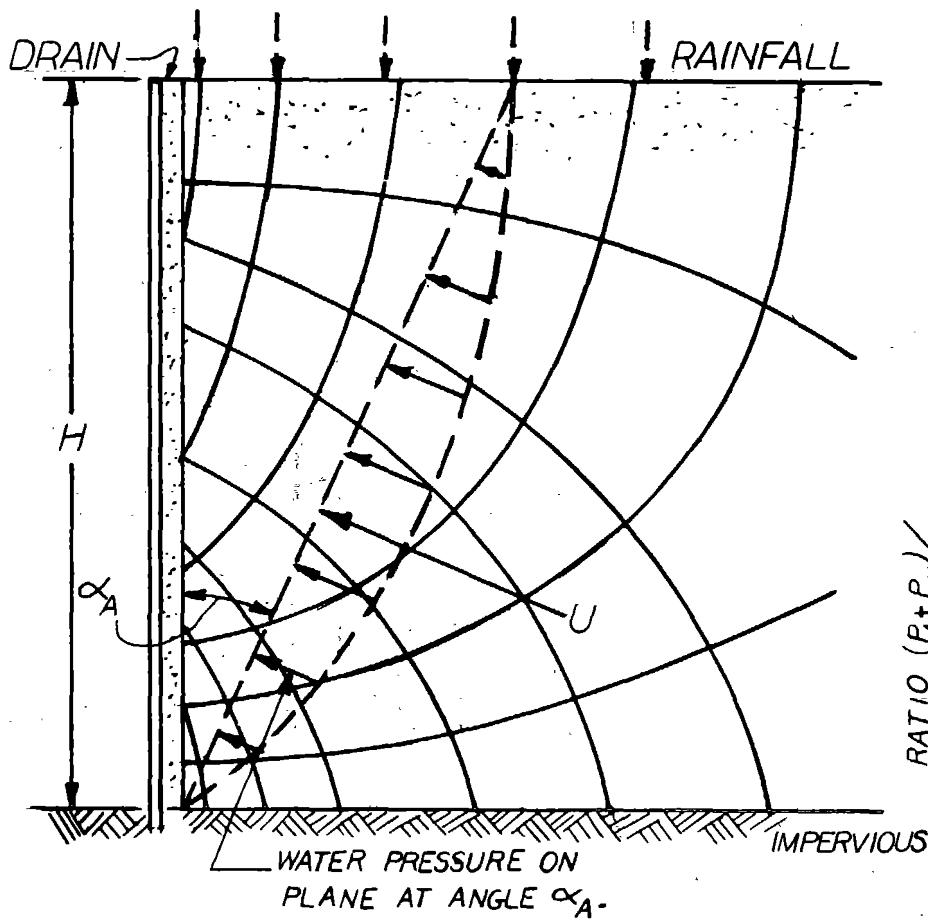


Figure 21. EFFECT OF STATIC WATER LEVEL



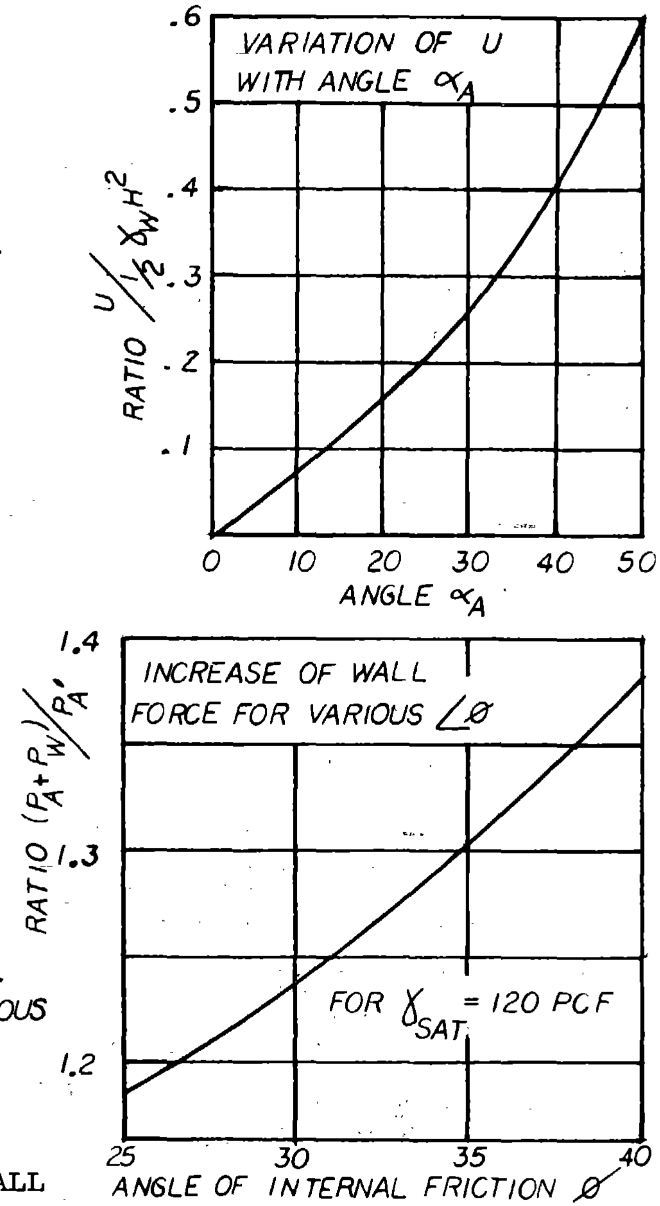


Figure 22. EFFECT OF RAINFALL ON DRAINED WALL

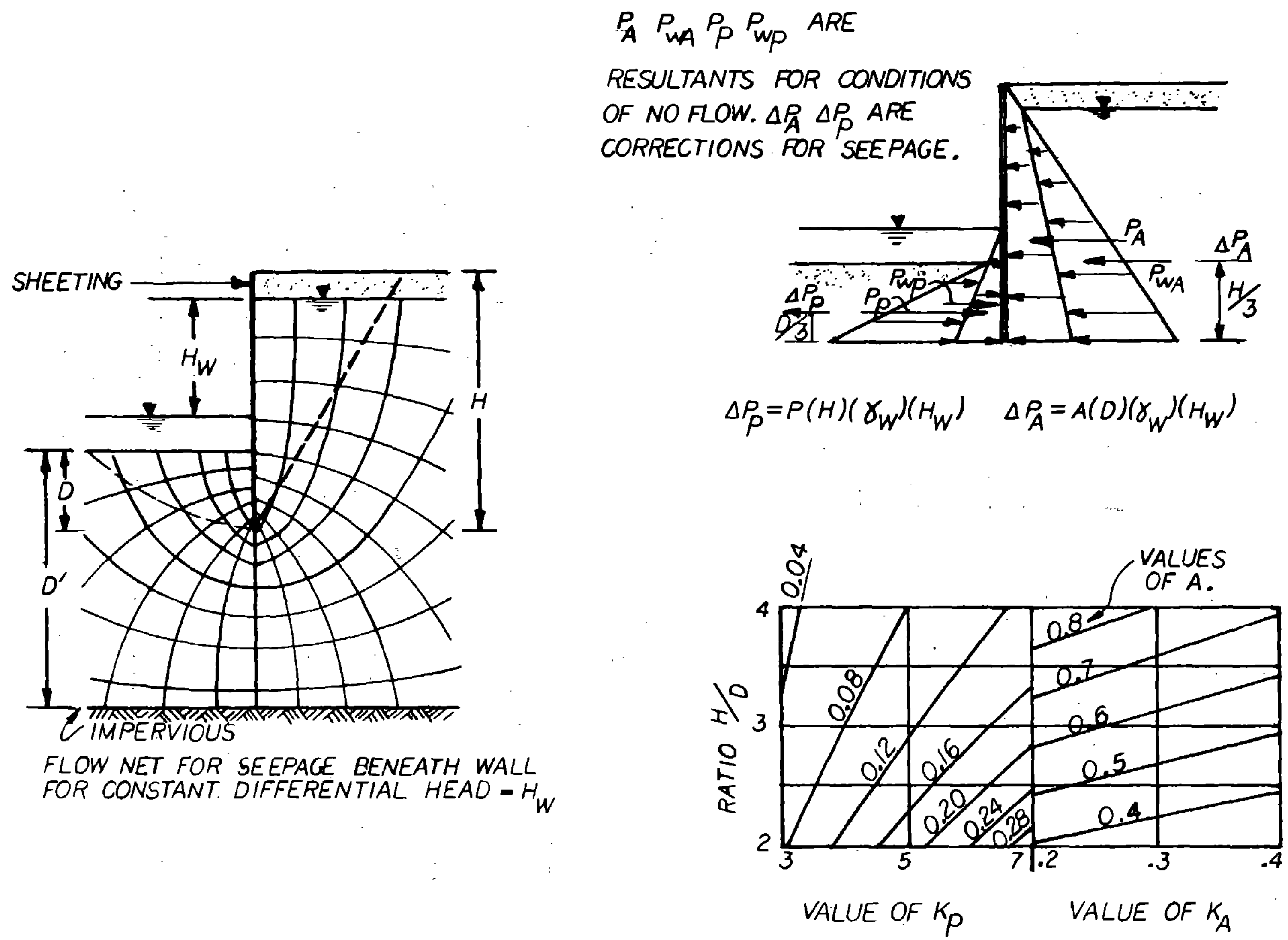


Figure 23. EFFECT OF SEEPAGE BENEATH WALL

What is of great importance, however, in considering the installation of anchors in soils below water table is the drilling and installation technique. It is vital that cased holes be drilled to avoid borehole collapse and disturbance to adjacent ground and foundations. For this reason, the hollow-stem auger method is not suitable. Apart from the large size of the entry hole needed through the retaining wall with attendant loss of component strength and difficulty in sealing against water inflow, the auger will tend to remove much greater quantities of soil from the hole than the net volume of the auger. This is particularly true in sands and gravels below water. This "mining" leads to considerable disturbance of the retaining soils and loss of support.

The method of casing must also be qualified. It is well known that the foundation soils structure should be maintained in as undisturbed a condition as possible and, for accurate pressure grouting, in contact with the casing for the majority of its length. Therefore, casing a hole by overdrilling and flushing outside the casing is a faulty technique that can lead to similar problems as with augers.

Two methods are acceptable. The first is where a closed or opened end casing is driven into the ground to depth, the inside cleaned out and then the anchor formed. This technique is useful for fairly short anchors, where obstructions are not present, where vibration will cause no problems and where examination of cuttings is not necessary. The majority of anchors constructed using the driven or rammed casing method involves the use of standard air track drilling rigs. However, the use of air flushing techniques in water bearing soils is potentially very dangerous and damaging to the soil structure and adjoining buildings and foundations. The reason is simply that the high pressure air flush ejects both drill spoil and ground water from the hole. The ground water carries fines with it thus disturbing the area around the borehole. The greatest problem occurs when the air flush is switched off to permit, for instance, the addition of an extra drill rod. At this time a considerable negative differential head pressure will occur in the borehole and the ground water will rapidly flow into the zone of low pressure bringing soil particles with it and in most cases triggering borehole collapse. Sometimes even the drill rods are blocked by this sudden inflow of material. When drilling recommences, all this collapsed material has to be removed from the borehole before proper flushing and drill penetration is re-established. It is not uncommon for considerable cavities to form when this drilling technique is used and several instances of building collapse have been recorded.

Probably the most versatile method is the installation of casing using the rotary method where cuttings are removed from the inside of the casing as drilling progresses. Water is used as the flushing medium so no danger exists of damaging the soil structure due to differential pressures. Also, the soil is maintained in intimate contact with the casing for its full length thus preventing any form of borehole collapse or ground loss through overdrilling and ensuring accurate control of subsequent grout placement and pressurizing.

CHAPTER II - DESIGN PROCEDURE

Based on the concepts described and developed in Chapter I, the following procedures are used in determining important ground anchor features:

1. Length of Anchors - Governed by soils information providing a suitable anchorage strata within a reasonable distance. Also by bond length calculated using empirical formulae which assume even distribution of bond over the fixed length. Experience and records show that little improvement is gained in ultimate capacity for bond lengths in excess of 40 feet (12m). Total anchor lengths should also be assessed on the grounds of economy and comparison with other possible methods.

2. Anchor Loads - Anchor loads are governed to a large extent by the soil conditions. Calculations should be made to assess ultimate capacity and then the required safety factor should be applied to determine the working load. Other factors that will be considered are allowable loads on the wale or other wall component and the effect of "drawdown" induced by the anchor vertical load component.

3. Number of Anchors - This is dictated by the total wall load, the maximum anchor load attainable in the conditions and the constraints upon anchor load imposed by the structure.

4. Spacing - Vertical and horizontal spacings may be determined from a study of the statics of the wall with the idea that the maximum spacing should be attempted commensurate with the structural strengths computed. Horizontal spacings are generally closer than vertical due to the fact that the anchors generally have to conform to modular dimensions in the structure such as lagging board widths. Vertical spacing will be predicated by the allowable bending moments in the structure. All spacings are subject to analysis for overstress of any part of the wall but there are minimum spacings also to be considered for anchors.

Experimental research has shown that pressure grouted anchors develop skin friction values well above normal effective stresses calculated from the overburden depth. But these "locked-in" stresses decay radially quite rapidly with the effect being lost at approximately 3 times the radius of the effective fixed anchor. So spacings should be designed to separate anchor bond length by at least 6 feet to 9 feet (2m -3m) depending upon soil permeability and structure. This should apply to vertical as well as horizontal spacings. Where surface entry points are closer than the minimum, then separation of the bond length can be achieved by alterations of anchor angle and/or anchor length. In fact, in multiple tiered walls there is merit in arranging for anchors to have varying lengths (by 5-10') as this breaks the tendency for secondary slip plane formation on a line through the distal ends of the anchor bond lengths.

5. Anchor Angle - Angle is governed by elevation of strata, presence of obstructions, adjacent services or foundations, and the need to achieve anchor bond length separation. The angle of 15° below horizontal is about the minimum that should be contemplated due to the profound practical difficulties that are inherent in constructing anchors at flattened angles. At this angle the vertical anchor load component is relatively small.

   Maximum angle for all practical purposes is between 50° and 55° as steeper angles impart too much vertical load to the structure for a useful horizontal load.

   The most common angle used is 30°$\pm$ as this is the optimum from the point of view of ease of installation.

ANCHOR DESIGN PROCEDURE

With the above concepts in mind, the design of ground anchors is accomplished as follows:

1. Compute total anticipated horizontal earth pressures acting per L.F. of retaining wall. Include any temporary and/or permanent surcharges.

2. Assume the wall section to be a beam, with loading conditions as determined by the horizontal earth pressures. Use support points at the tieback locations for multiple tiered walls or tieback location and a few feet below toe of retaining wall for wall sections with a single anchor.

3. Using statics, determine required tieback horizontal force. Several iterations may be required to find the optimum location for the tieback. Wall design as well as the tieback should be considered in finding the optimum anchor location.

4. Determine the most likely failure plane of material behind retaining wall. Locate top of anchor bond zone 5 feet (1.5m) minimum beyond this surface. If no other precise determination for failure plane location is known, assume a plane beginning 5 feet (1.5m) below the bottom of the excavation and extending upward at an angle to the horizontal equal to $45° + \phi/2$ for granular soils. For cohesive soils, assume a circular slip surface centered on the wall top and having a radius equal to wall height.

5. Select anchor angle of inclination. For ease of installation, a 15° - 30° angle from horizontal is optimum, providing that suitable anchoring strata is relatively close (within 30' [9m]) to the tieback elevation. The vertical component of the tieback is then checked to ensure that excessive forces are not imposed on the wall foundation. Check that anchors will not foul services or foundations adjacent to the project site. Check for presence of minimum depth of overburden above fixed anchor.

6. Determine the required anchor load by dividing the horizontal force by the cosine of the anchor angle of inclination. The number and spacing of the anchors can then be determined.

7. Determine allowable anchor loads and required bond lengths as follows:

   a) In granular material:

   $$T_{ult} = p' \cdot \pi \cdot d \cdot L \cdot \tan \phi$$

where,

T_{ult} = Ultimate load capacity of the anchor

p' = Locked-in grout pressure taken as 2 p.s.i. (0.013N/mm^2) per foot of overburden above the top of the fixed anchor length with a maximum of 80 - 100 p.s.i. (0.55-0.69N/mm^2)

d = Assumed diameter of pressure grouted bond zone. This is dependent on soil permeability and is usually 9" - 15" (230mm - 380mm) in open soils where k = 10^{-1} to 10^{-2}, and 6" - 8" (150mm-200mm) in fine grained soils above k = 10^{-3} - 10^{-4}.

L = Fixed anchor length.

Ø = Soil angle of internal friction

b) In cohesive soils:

$T_{ult} = T_s + T_e + T_u$

where,

1) Shaft Adhesion T_s

$= d_s \times f_s \times C_u\text{(Shaft)} \times l_s$

Area x Adhesion X Soil Cohesion Factor

2) End Bearing T_e

$= \frac{\pi}{4}(d_u 2 - d_s{}^2) \times N_c \times c_u\text{(end)} + \sigma'e)$

Area of End X (Bearing Capacity X Soil Cohesion + The Effective Stress Perpendicular to the End)

3) Underream T_u

$= \pi\, d_u \times f_u \times C_{u\text{(underream)}} \times L_u$

Area X Efficiency Parameter X Soil Cohesion of Underream

f_s = Adhesion Factor
(0.3 to 0.6 dependent upon the type and quality of the clay, etc.)

f_u = Efficiency Factor
(0.75 to 0.95 for disturbance caused by underream tool and technique)

N_c = End Bearing Factor
(6 to 13 dependent on depth but more usually between 6 and 9. Where lower values are used a component E may be added equal to the effective stress perpendicular to the end due to surcharge soil)

e' = Effective stress perpendicular to end of cone (See pages 21 and 22 in Chapter 1)

c) In Rock:

$$T_{ult} = L \quad d_s \; f_c \text{ (or } f_r \text{ whichever is less)}$$

(See pages 24 and 25 in Chapter 1)

or, in the absence of data or material and rock strengths typical values for bond stress in various types of rock have been suggested by the Post Tensioning Institute Committee on Rock and Soil Anchors. See Table III in Chapter 1.

Then the anchor capacity can be assumed as follows:

$$T_{ult} = \quad d \quad L \quad n$$

where, d = Drilled hole diameter in bond zone

L = Bond length

n = Bond stress assumed between grout and rock face.

Provided that the tendon layout is designed to evenly distribute the anchor stress -- a basketed and banded strand tendon is ideal in this respect, then fixed anchor length for straight shafts may be determined from this fairly simple empirical approach which takes into account grout and rock strength.

8. Calculate tendon steel requirements based on anchor design load = 50% tendon g.u.t.s. (except for special test anchors).

ANCHOR TEST PROCEDURES

Anchor test loading procedures are recommended in two forms. These differentiate between pre-contract or pre-production test anchors and the tests carried out on production anchors that will be incorporated into the work.

PRE-CONTRACT TESTS

1. Use hydraulic center hole jacking equipment capable of stressing all elements of the anchor tendon simultaneously to 1.5 times maximum test load. Jack to be calibrated as a unit with pump and gauge and to be accurate to within $\pm$ 2%. Pump shall be fitted with automatic device to enable pressure and thus jack load to be maintained at constant level.

2. Apply initial stress to the anchor equivalent to 10% of its design load to center the jacking equipment, remove tendon slack, seat all bearing plates and stress components and to ensure that pull wedges (if used) are properly engaged.

3. Mount tendon extension measuring instruments on an independent frame so that only tendon displacements are observed. Movements of the wall or reaction block may also be required, but these should be made by separate gauges. Two measuring gauges should be used for each test with sensitivity of 0.001 inch (0.01 mm). As an additional check on the reference frame, its stability can be monitored optically or by taut wire.

4. Set all measuring devices to zero using 10% of working load (W.L.) initial stress as datum point for stress measurements, and commence cyclic loading.

5. The following test loading cycles shall be performed:
   * - Loads expressed as a percentage of design or working load.

   Cycle a) 10, 20, 40, 50*, 25, 10.
   Cycle b) 10, 25, 50, 75, 100*, 50, 25, 10.
   Cycle c) 10, 25, 50, 75, 100, 125, 150**, 100, 50, 25, 10.
   Cycle d) 10, 25, 50, 75, 100, 125, 150, 175, 200*, 150, 100, 50, 25, 10.
   Cycle e) 10, 50, 100, 150, 200 and to anchor failure or 80% g.u.t.s. of tendon, whichever occurs first. If no failure, return load to zero and record recovery.

---

* Maintain load for 30 minutes minimum

** Maintain load for 24 hours

6. Dial gauge readings should be recorded to nearest 0.001 inch (0.01mm). Readings should be taken upon the application of the load and at 1 minute, 3 minutes and 5 minutes after application of each load increment and at a maximum of 5 minute intervals thereafter if continuous movement is taking place. For 30 minute hold periods, record movement at 5 minute intervals after the initial set at 0, 1, 3, and 5 minutes after load application and for the 4 hour hold record as for 30 minute hold, but then record at 10 minute intervals for the first 2 hours, at 30 minute intervals for the next 2 hours and at hourly intervals for the next 2 hours.

7. No further increment of load should be applied until the rate of tendon displacement has dropped to 0.012 in/hr (0.3mm) or less (0.001"/5 mins [0.03mm]). If the average rate of movement exceeds this figure for 2 hours after the application of any load increment and the rate of movement is not diminishing, then the load should be reduced until the rate of movement is 0.012 in/hr or less (0.001"/5 mins [0.03mm]).

8. If the rate of movement exceeds 0.012 in/hr (0.3mm) for 2 hours and the rate of movement is not diminishing, then the load should be reduced until the stable load can be determined as in 7 above. The load on the anchor should then be increased until continuous movement is recorded and no further increase in load can be attained. This peak load should determine the ultimate capacity of the anchor under test. This verifies design of the anchor and factors of safety.

9. Reduce loads to 10% of W.L. in the specified intervals holding each load level for 5 minutes and recording anchor tendon recovery.

10. Hold 10% W.L. on the anchor at conclusion of stages c) and d) for 1 hour before taking final reading to determine total net anchor rebound and thus elastic and plastic movements.

The test sequence specified above gives data which may be used to produce stress/strain graphs and load-time-displacement curves. These not only verify test anchor performance, but can be used as control graphs giving an envelope of performance against which production anchor performance may be plotted. The time-displacement data are used to calculate creep coefficients and associated limit loads. Figure 24 shows a typical control graph with superimposed production anchor results. On the graph the limit lines of 0.8 X stressing length, and stressing length + 1/2 fixed length are used to indicate anchors acceptable from the stress/strain criteria. Anchors with curves plotted outside these limits need to be carefully investigated before acceptance. For example, if an anchor should hold the test load satisfactorily, but the load/extension curve indicates displacements less than would equate to the elastic movement of 0.8 x the stressing length, then the load is being taken in a potentially active zone and so failure of the total tied wall system could ensue.

Production Tests

Between 5% and 10% of all permanent production anchors should be subject to tests similar to the pre-production tests, but with modified loading cycles to reflect the fact that the anchors will be incorporated into the permanent work.

1. Use hydraulic center hole jacking equipment capable of stressing all elements of the anchor tendon simultaneously to 1.5 times full test load. Jack should be calibrated as a unit with pump and gauge and be accurate to within $\pm$ 2%. Pump should enable pressure and thus jack load to be maintained at constant level.

2. Apply initial stress to the anchor equivalent to 10% of its design load to center the jacking equipment, remove tendon slack, seat all bearing plates and stress components and to ensure pull wedges (if used) are properly engaged.

3. Mount tendon extension measuring instruments on an independent frame so that only tendon displacements are observed. Measurements of movements of the wall or reaction block may also be required, but these should be made by separate gauges. Two measuring gauges should be used for each test with a sensitivity of 0.001 inch (0.01 mm). As an additional check on the reference frame, its stability can be monitored optically or by taut wire.

4. Set all measuring devices to zero using 10% W.L. initial stress as datum point for stress measurements, and commence cyclic loading.

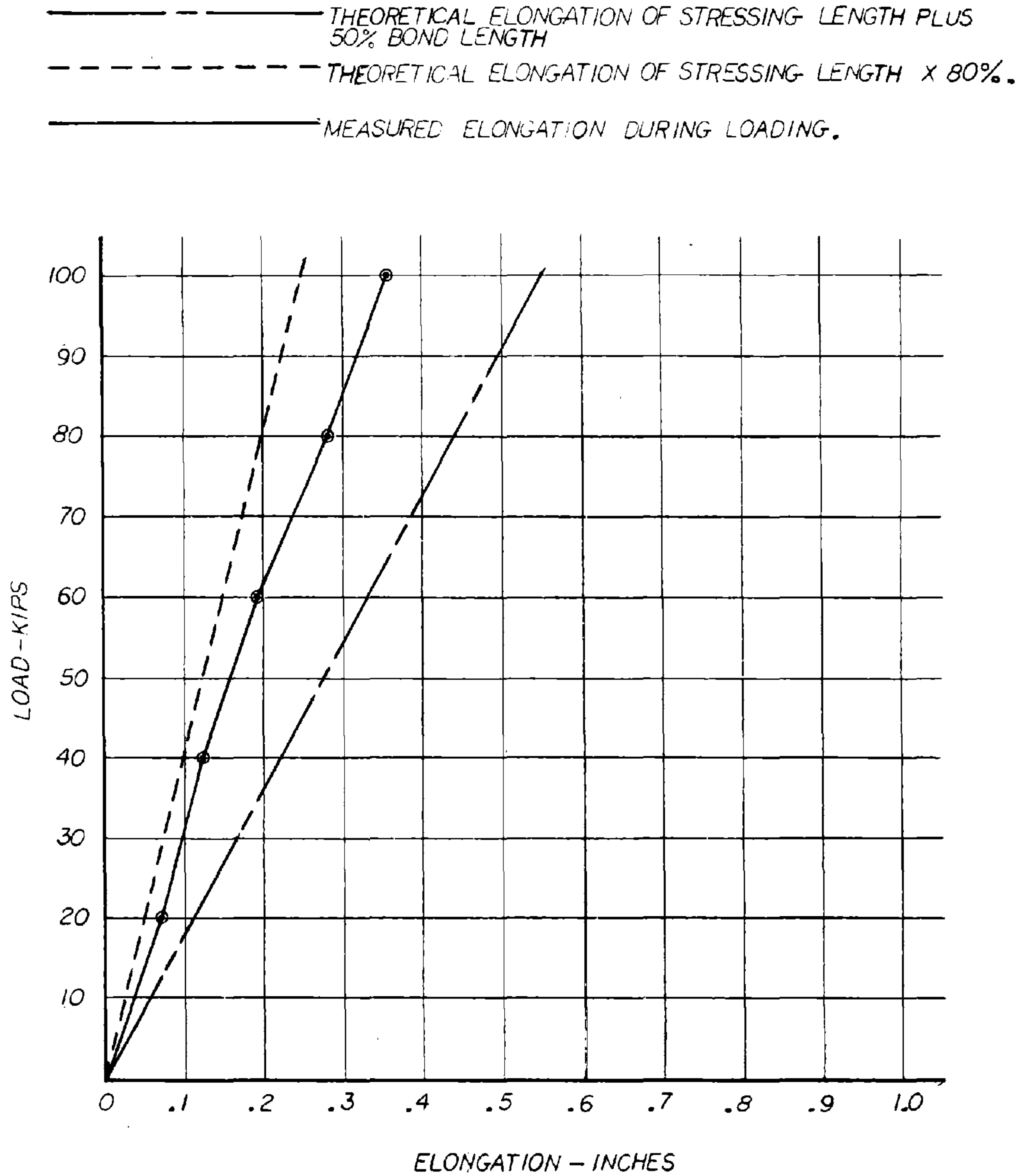


Figure 24. LOAD vs. ELONGATION

5. The following cycles shall be performed:
   * - Loads expressed as a percentage of design or working load.

Cycle a) 10, 25, 50*, 75, 100**, 50, 25, 10.

Cycle b) 10, 25, 50*, 75, 100**, 125, 150†, 100, 50, 25, 10.

---

* Hold for 15 miniutes.
** Hold for 30 minutes.
† Hold for 4 hours

Hold times are minimum and depend upon anchor performance.

After completion of this test, stress and lock-off the anchor in the specified manner at the required lock-off load. Perform lift-off test a minimum of 24 hours later and check for load loss. If this exceeds 10% of the lock-off load, restore load and recheck load loss 24 hours later. If load loss exceeds 10% of lock-off load, again restore load and recheck after a further 24 hours. If load loss is still above 10%, carry out additional tests and investigations to establish whether load loss is continuous, will diminish with time, or whether the anchor should be replaced.

Anchors exhibiting load losses less than 10% of lock-off are normally acceptable providing acceptable creep charaterisitics have been established.

6. Dial gauge readings shall be recorded to nearest 0.001 inch (0.01mm). Readings should be taken upon the application of the load and at 1 minute, 3 minutes and 5 minutes after application of each load increment and at a maximum of 5 minute intervals thereafter if continuous movement is taking place. For 30 minute hold periods, record movements at 5 minute intervals after the initial set at 0,1,3, and 5 minutes after load application and for the 4-hour hold record as for 30-minute hold, but then record at 10 minute intervals for the first 2 hours, and at 30 minute intervals for the next 2 hours.

7. No further increment of load shall be applied until the rate of tendon displacement has dropped to 0.012 ins/hr (0.3mm/hr) or less (0.001 [0.03mm] in 5 mins). If the average rate of movement exceeds this figure for 2 hours after the

application of any load increment and the rate of movement is not diminishing, then the load should be reduced until the rate of movement is 0.012 in. (0.3mm)/hr (SI) or less (0.001 in. [0.03mm]in 5 mins). The load at which continuous movement was noted shall be deemed the ultimate capacity of the anchor and the relevant safety factors shall be applied to determine its working load, and whether anchor replacement or reinforcement will be required.

8. After acceptable load carrying and creep charateristics are indicated, load anchor to 115% of design load and lock-off.

The results obtained from the production anchor tests should be compared with those of the pre-contract tests to enable assessments to be made of acceptable stress-strain performance and of creep characteristics. Control graphs can be used with these and all production anchors. Where possible, allowance should be made for carrying out lift-off checks on selected anchors during the life of the contract and also as part of a long-term monitoring program.

OTHER PRODUCTION ANCHOR CHECKS

The remaining production anchors should be stressed to 133% of W.L. with extension measurements taken at the following loads:

1. 10% W.L., 25% W.L., 50% W.L., 75% W.L., 100% W.L., 133% W.L. The highest load should be maintained for a minimum of 5 minutes and extension readings taken at application of load and then at 1 minute, 3 minutes and 5 minutes. After acceptable creep characteristics are indicated, then reduce the anchor load to 115% of design load and lock-off. Should creep characteristics be suspect, then additional time should be taken to establish the anchor behaviour.

2. The stress/strain measurements for production anchors can be plotted against the control graphs obtained from the other critical tests and acceptability judged accordingly. The time-displacement records will show creep characteristics.

INTERPRETATION OF LOAD TESTS

1. From construction records prepare theoretical stress-strain graphs with limit lines shown thereon that start from graph origin and equate to:

   a) Anchor stressing length X 0.8
   b) Anchor stressing length
   c) Anchor stressing length + 1/2 fixed anchor length

   These lines are the elastic movements expected from the tendons of the stated lengths.

2. Carry out anchor tests in the manner described and use the load-extension data to construct curves which are plotted on the graphs.

3. From the time-extension data, construct curves on semi-log graph paper to evaluate the creep characteristics. Graphs are as described on Pages 32 and 33 in Chapter I.

4. In the stress/strain graphs, where the performance plot lies to the left of line a) investigations should be made to discern where load is being taken in the active soil zone. Where the plot lies to the right of line c) then the anchor will probably be close to the point of failure if not already failed. The maintainance of constant load with apparent mobilization of the majority of total tendon length should be investigated very carefully.

5. In the case of pre-production anchors, test results will be used to give verification of anchor design and perhaps dictate any changes needed. With production anchors, the test results will be compared to the results of the initial tests and to each other.

6. Creep characteristics are ascertained and used to determine limit loads for all anchors and potential load loss with time. This is done by taking the creep coefficient from the time/dis- placement chart for the required load level and multiplying this by log time to indicate long-term creep and thus anticipated load loss.

7. Wherever possible, periodic checks should be made on all permanent anchor systems either by lift-off checks or the inclusion of instrumentation for that purpose. The results of these checks should be plotted against a time base to indicate tendencies of load change in the anchor and thus the structure.

CHAPTER III - DESIGN EXAMPLE

The following is an example of the step by step procedure given in Chapter II using the basic information obtained from the Federal Highway Administration.

In arriving at the design, the writers assumed that the wall system would be a soldier beam and lagging system with a reinforced concrete surface poured against the lagging after the tiebacks were installed, stressed and locked-off. It is beyond the scope of the report to finalize a design of the wall members.

The step by step procedure will be followed at a given cross section, in completing the design for the entire wall. The procedure should be repeated as necessary wherever cross sections vary significantly.

CROSS SECTION AT - STA. 30+04 (See Figure 21)

Data obtained from soils investigation: (Boring Logs pp. 82-86)

$\phi = 30°$
$\gamma = 110$ PCF
$K_a = 1/3$
$K_o = 1 - \sin\phi = 0.5$
$\gamma_e = K_o = (0.5)(110) = 55$ PCF

Neglect Capillary Rise

Step 1. Compute Total Horizontal earth pressure max. case @ STA 30+04

Top of existing ground @ EL 963
Final grade at base of excavation @ EL 930

H = 33 Feet

Total horizontal earth pressure

$= 1/2\ \gamma_e\ K_o H^2$

$= 1/2\ (55)(33^2) = 29{,}948$ Lb/Ft

Say 30,000 Lb/Ft

Note: S. I. Equivalents are on Page 80.

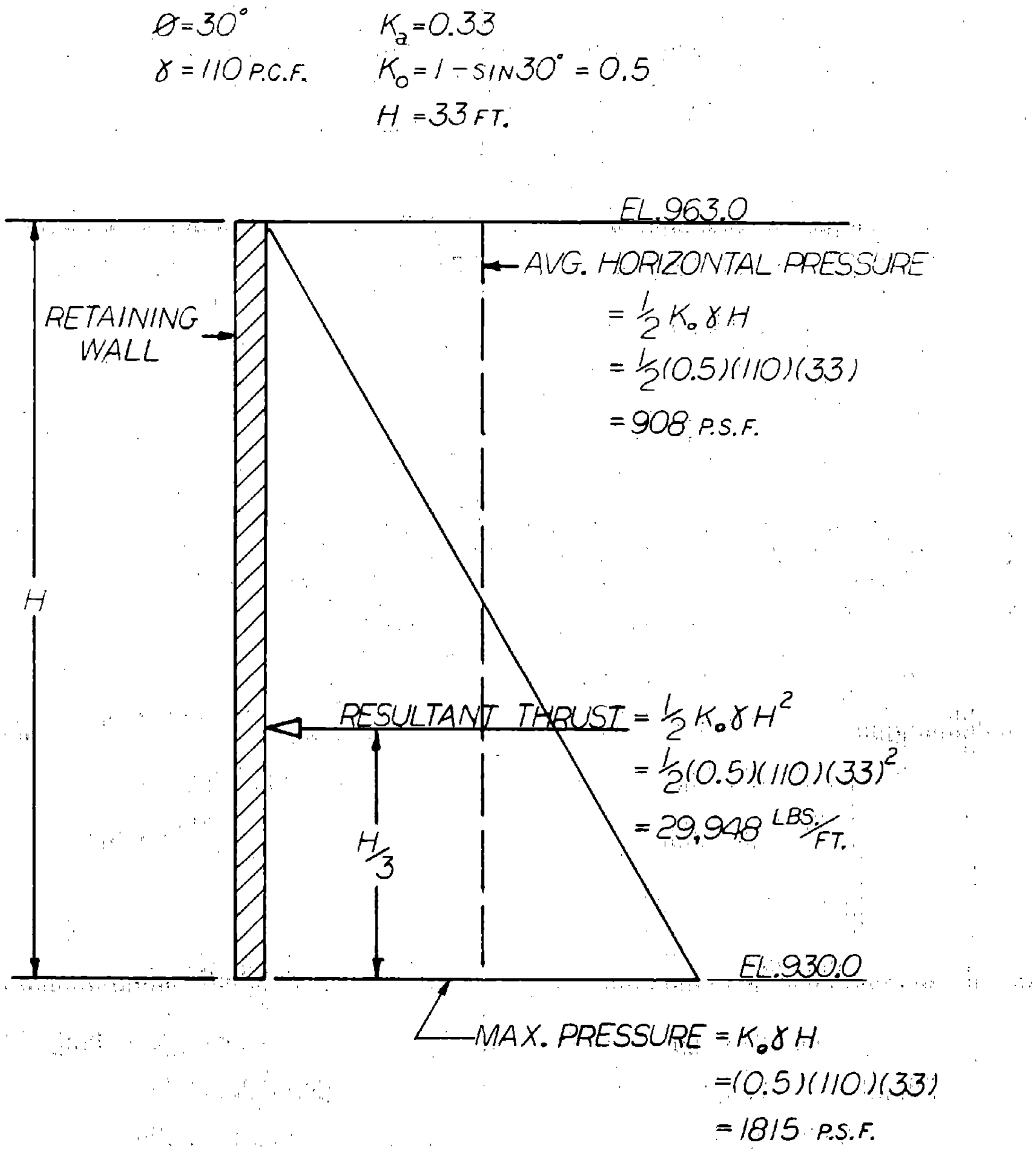


Figure 25. ACTIVE PRESSURES IN DESIGN STUDY

From the information supplied by FHWA no definitive estimation of surcharge loading can be determined, and therefore will be neglected here.

Step 2. Determine tieback vertical spacing

Assume 2 tieback locations, at H/4 from top and bottom of wall. (See Figure 25.) The anchor can now be assumed to take a load component from the top (or bottom) of the wall to mid-height of the wall.

Step 3. Tieback horizontal force:

Assume uniform horizonal pressure distribution behind wall from top to bottom equal to:

$$P_H = 1/2\,\gamma_e H = 1/2\ (55)\ (33) = 908\ \text{PSF}$$

Uniform horizontal pressure on wall = 908 PSF

For tiebacks located at H/4 from top and bottom of wall the reaction at each tieback is

= 1/2 x 908 PSF x 33 ft = 14,982 Lb/Ft of wall length

Say 15,000 lbs/ft

Step 4. Angle of inclination:

Anchor tiebacks in dense micaceous sandy silt. Since this suitable anchoring strata is close to ground surface, a steep ($\geq$ 45°) angle is not needed.

After considering influences from adjacent utilities, foundations, soil strata, and wall draw-down induced by the tieback vertical component, a suitable angle of inclination of 30° the horizontal was chosen.

Step 5. Determine tieback load:

$$\text{Tieback Load} = \frac{15\ \text{K/Ft.}}{\text{COS } 30°} = 17.32\ \text{K/Ft}$$

for both top and bottom rows for the soldier beam and lagging wall system assumed in this example. It is assumed that the soldier beams will be placed on 8 ft. centers since lagging lengths of 8 feet are standard, and practical beam sizes can be used. Therefore, the required tieback design load will be:

$17.32\ K/Ft \times 8\ ft. = 138.6^{K}$, say 140^{K}

A calculation of the tieback vertical component should be made to check its effect on the wall member design:

$140^{K} \times \sin 30° = 70^{K}$

The design of wall members must include this component, particulary to check bearing capacity at the toe of the soldier beams.

Step 6. Determine bond length:

Use safety factor = 2.0

From Step 5, tieback design load = 140,000 lbs (140 kips)

Use empirically derived and tested formula as shown on Page 65 of Chapter II

$T_{ult} = p' \cdot \pi \cdot d \cdot L \cdot \tan \phi$

Apply safety factor, and solve for bond length, L

$T_{ult} = 2 \times 140$ Kips $= 280$ Kips.

Assume effective hole diameter will be 9 inches. This is an average value, a bit small for permeable soils, and a bit large for fine-grained materials. More detailed information is required from the soils investigation to verify this assumption.

Assume tieback bond zone is 25' min. below ground surface and check after tieback design is finalized.

$p' = 2 \times 30 = 60$ psi (See Page 65 in Chapter II)

$$L = \frac{2.0 \times 140{,}000}{50\ (\text{psi}) \times \pi \times 9''\ (\text{dia}) \times \tan 30°} = 343 \text{ ins.}$$

or $L = 28.6$ Feet

Use bond length L. = 30.0 Ft.

Step 7. Determine failure plane and tieback free length:

Using ϕ = 30°, assume failure plane beginning 5 feet below bottom of final grade at base of excavation and extending upward at 45° + ϕ/2 = 60°.

Failure plane begins at EL 925.0
Tieback in upper row at EL 954.75
Tieback in lower row at EL 938.25

Using law of sines,

Upper tieback length to failure plane
= 29.75' x sin 30°

= 14.875' say 15'.

Add 5' penetration beyond failure plane
= 15' + 5' = 20'.

Use free length at upper row = 20 feet

Lower tieback length to failure plane
= 13.25 x sin 30°

= 6.625 feet

Add 5' penetration beyond failure plane

6.625' + 5' = 11.625', but add extra to give minimum stress length (Page 23 Chapter I).

Use free length at lower row = 20 feet.

NOTE - From Figure 26 it can be seen that in a free length of 20 feet and a bond length of 30 feet, the depth of overburden to the mid-point of the bond zone is:

$$8.25' + \left(20' + \frac{30'}{2}\right) \times \sin 30°$$

$$8.25' + 35 \sin 30' = 25.75',$$

Therefore, our assumption of 25 feet of overburden in step 6 is valid. If the actual overburden depth varied greatly from the assumed depth, a new bond length calculation could be performed.

Step 8. Check effect of construction sequencing. At this point, insufficient information is available to perform this check. Additional information on the type of wall to to be constructed and its design is required.

Step 9. Calculate tendon steel required for a design load of 140 Kips.

Assuming 0.6 inch diameter, 270 ksi strand will be used, an anchor tendon comprised of 5 strands is selected.

$$\frac{140{,}000 \text{ lbs} \times 2 \text{ (S.F.)}}{270{,}000} = 1.04 \text{ in}^2 \text{ required}$$

As steel area of 0.6" Ø strand = 0.215 in^2,

$$\frac{1.04 \text{ in}^2}{0.215 \text{ in}^2} = 4.84 \text{ strands}$$

So use 5 strands.

S.I. Equivalents

1 ft	=	0.3048m	1m	=	3.28 ft.
1 lb	=	4.448N	1N	=	0.2248 lbs.
1 psi	=	$0.00689 N/mm^2$	$1N/mm^2$	=	145 psi
1 inch	=	25.4mm	1mm	=	0.03937 ins.
1 in^2	=	$645.16 mm^2$	$1mm^2$	=	0.00155 in^2
1 Kg	=	9.806N	1N	=	0.10198 Kg
			1 pcf	=	$0.157 kN/m^3$

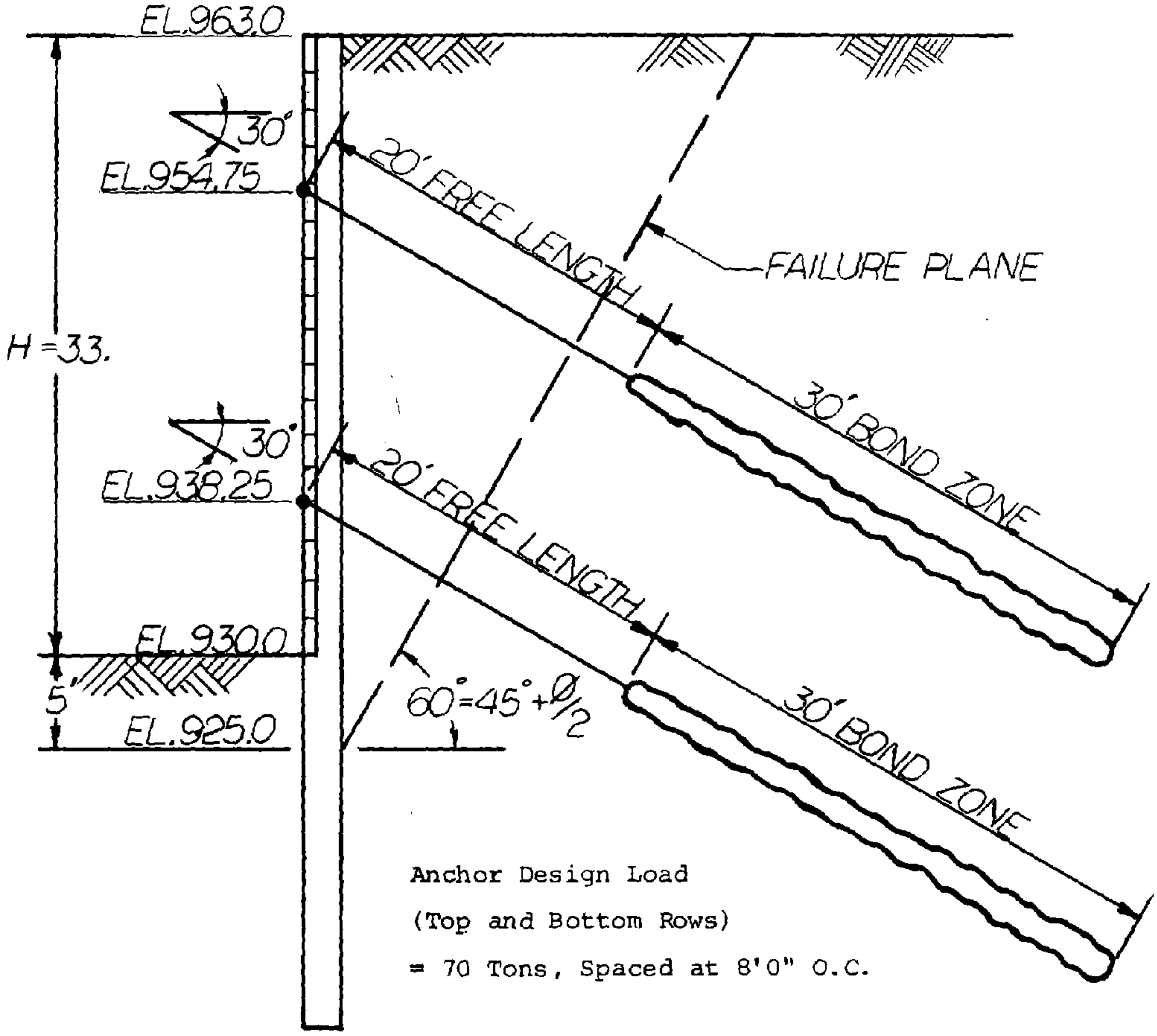


Figure 26. ANCHOR DESIGN LOAD

DOT 490

DEPARTMENT OF TRANSPORTATION

OFFICE OF MATERIALS AND TEST, FOREST PARK, GEORGIA
SOILS ENGINEERING AND GEOLOGY BRANCH

BRIDGE SUBSURFACE INVESTIGATION

PROJECT I-75-2 (41) 256 COUNTY FULTON DATE 1-10-74

LOCATION RET. WALL "E-1" BORING NO. E1-4

BENT NO. ______ FOOTING 118'RT. STA. 32+02 EXISTING ℄ / 65'RT. STA. 31+76 ℄ PROP. NBL GROUND ELEV. 957.7

PROPOSED FOOTING ELEV. ______ PARTY CHIEF SIMMONS

ELEV.	BORING LOG	SAMPLE	BLOW	UNIFIED	W	γ	Gs	C	Ø	BC	LL	PI	% 200	% CLAY
(Ft.)	GR. EL. 7													
		1u												
950														
		2u												
	MED. DENSE MLTC.													
	MICAS. SANDY SILT	3u												
940		4s	18											
		5s	20											
		6s	18											
		7s	21											
930		8s	28											
		9s	25											
	DENSE SAME													
		10s	27											
920														
GWT		11s	19											
		12s	22											
910	MED. DENSE MLTC.													
	MICAS. SANDY SILT	13s	20											
		14s	22											
900														
		15s	24											
	SLI. WEATH. TO													
890	HARD ROCK													

PRACT. REFUSAL

DOT 490

DEPARTMENT OF TRANSPORTATION

OFFICE OF MATERIALS AND TEST, FOREST PARK, GEORGIA
SOILS ENGINEERING AND GEOLOGY BRANCH

BRIDGE SUBSURFACE INVESTIGATION

PROJECT I-75-2 (41) 256 COUNTY FULTON DATE 1-10-74

LOCATION RET. WALL "E-1" BORING NO. E1-5

BENT NO. ____ FOOTING 118' RT. STA. 33+00 EXISTING ℄ / 75' RT. STA. 32+71 L PROP. NBL GROUND ELEV. 951.9

PROPOSED FOOTING ELEV. ____ PARTY CHIEF SIMMONS

ELEV.	BORING LOG	SAMPLE	BLOW	UNIFIED	W	γ	Gs	C.	Ø	BC	LL	PI	% 200	% CLAY
(FT)														
	GR. EL.													
950														
	MED. DENSE MLTC.	1u												
940	MICAS. SANDY SILT	2u												
		3s	18											
		4s	18											
	DENSE SAME	5s	27											
	MED. DENSE MLTC.	6s	21											
930	MICAS. SANDY SILT													
		7s	25											
	DENSE SAME													
		8s	25											
920														
	MED. DENSE MLTC.	9s	15											
	MICAS. SANDY SILT	10s	22											
910														
	DENSE SAME	11s	33											
	END DRILLING													
900														

DOT 490

DEPARTMENT OF TRANSPORTATION

OFFICE OF MATERIALS AND TEST, FOREST PARK, GEORGIA
SOILS ENGINEERING AND GEOLOGY BRANCH

BRIDGE SUBSURFACE INVESTIGATION

PROJECT I-75-2(41)256 COUNTY FULTON DATE 1-9-74

LOCATION RET. WALL "E-1" BORING NO. E1-6

BENT NO. ______ FOOTING 110' RT. STA. 33+95 EXISTING ℄ / 77' RT. STA. 33+72 ℄ PROP. NBL GROUND ELEV. 946.0

PROPOSED FOOTING ELEV. ______ PARTY CHIEF SIMMONS

ELEV. (Ft)	BORING LOG	SAMPLE	BLOW	UNIFIED	W	γ	Gs	C.	Ø	BC	LL	PI	% 200	% CLAY
	GR. EL. ?													
940	LOOSE MLTC. MICAS. SANDY SILT	1s	9											
		2s	10											
	MED. DENSE MLTC. MICAS. SANDY SILT	3s	14											
		4s	12											
930		5s	12											
		6s	17											
		7s	16											
920		8s	20											
		9s	17											
910		10s	14											
		11s	14											
900		12s	16											
		13s	20											
890	DENSE SAME	14s	45											
		15s	49											
880		16s	49											

END DRILLING

DOT 490

DEPARTMENT OF TRANSPORTATION

OFFICE OF MATERIALS AND TEST, FOREST PARK, GEORGIA
SOILS ENGINEERING AND GEOLOGY BRANCH

BRIDGE SUBSURFACE INVESTIGATION

PROJECT I-75-2 (41) 256 COUNTY FULTON DATE 1-9-74

LOCATION RET. WALL "E-1" BORING NO. E1-7

BENT NO. ______ FOOTING 110' RT. STA. 35+50 EXISTING ℄ 90' RT. STA. 35+35 ℄ PROP. NBL GROUND ELEV. 938.5

PROPOSED FOOTING ELEV. ______ PARTY CHIEF ______

ELEV.	BORING LOG	SAMPLE	BLOW	UNIFIED	W	γ	Gs	C.	Ø	BC	LL	PI	% 200	% CLAY
(5+)	GR. EL.													
	LOOSE MLTC. MICAS SANDY SILT	1s	7											
	MED. DENSE MLTC. MICAS. CLY. SDY. SILT	2s	15											
930		3s	14											
		4s	16											
		5s	14											
		6s	11											
920														
		7s	13											
		8s	19											
910	MED. DENSE MLTC. MICAS. SANDY SILT													
		9s	11											
		10s	14											
900														
		11s	22											
		12s	25											
890														
	VERY DENSE SAME	13s	57											
	SLI. WEATH. TO HARD ROCK													
880														
	PRACT. REFUSAL													

DOT 490

DEPARTMENT OF TRANSPORTATION

OFFICE OF MATERIALS AND RESEARCH, FOREST PARK, GEORGIA
GEOTECHNICAL ENGINEERING

PROJECT I-75-2(41)256 COUNTY FULTON DATE 2/12/79

LOCATION WALL "E-1" BORING NO. E1-8

BENT NO. ______ FOOTING 133' RT. STA. 36+70 EXISTING ℄ 116' RT STA 36+59 PROP NBL GROUND ELEV. 932.07

PROPOSED FOOTING ELEV. ______ PARTY CHIEF SIMMONS

LEV (??)	BORING LOG		BLOW	UNIFIED	γ	W	Gs	% 200	% CLAY	LL	PI	C	ø	
	Gr. El.													
930	Loose Mltc. Micas Sandy Silt													
920														
910														
WT														
900	Med. Dense Same													
890	Dense Same													
880	Very Dense Same													
870	End Boring													

CHAPTER IV

THREE CASE HISTORIES

The following case histories provide detailed information on design data, installation procedures and testing and stressing procedures, as well as types of material and equipment used on three of Nicholson Anchorage Company's* projects completed since 1974.

* - Now Nicholson Construction Company

CASE HISTORY

RETAINING WALL

FOR

ROANOKE MEMORIAL HOSPITAL PARKING GARAGE

ROANOKE, VA

OWNER: ROANOKE MEMORIAL HOSPITAL

ENGINEER: SHERETZ, FRANKLIN, AND SHAFFNER
ROANOKE, VA

GENERAL CONTRACTOR: NELLO L. TEER CO.
DURHAM, NC

The architecture and engineering firm of Sheretz, Franklin, and Shaffner developed the concept for converting a steep hillside into a site for a 4 level parking garage using a permanent tieback retaining wall. As called for in specifications issued by the owner, (using design criteria provided by the engineer) the contractor was to prepare the complete design of the wall, subject to final approval by the engineer.

After a review of final bids, Nello L. Teer Co. was chosen as general contractor. Teer retained the consulting firm of Eason-Coffin Associates, Durham, to design the wall, and selected Nicholson Anchorage Company* to provide design assistance and to install the tiebacks.

GEOLOGIC CONDITIONS

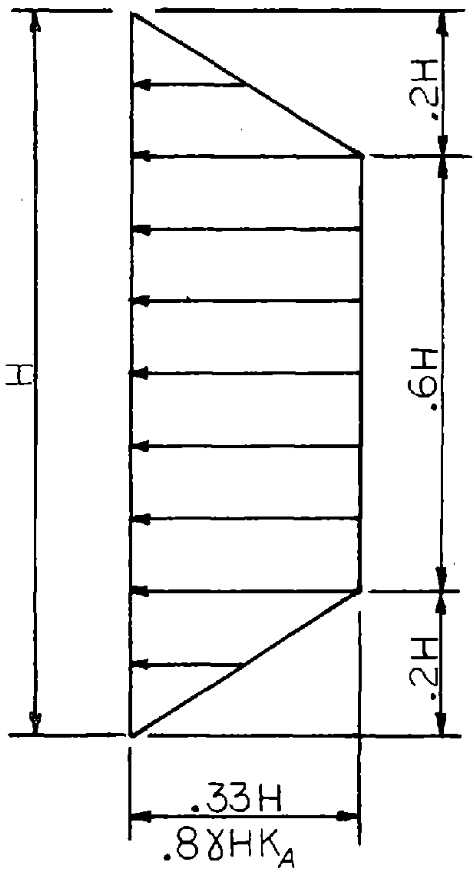
Eleven borings were made at the project site, located in the southeast section of Roanoke on a hillside above a bend in the Roanoke River. Soils at the site included colluvium, residuum, and minor amounts of alluvium. The colluvium, ranging in thickness from 15.7 feet (4.8m) to 46.3 feet (14m), typically consisted of yellow-tan clayey to sandy silt with shale and sandstone fragments common, and some mottling. Residuum ranged from 1.2 feet (0.37m) to 23.5 feet (7.2m) thick and included yellow-tan clayey to sandy silt with highly weathered to fresh shale fragments and occasional mottling. Alluvium encountered was typically 1.2 feet (0.37m) to 2 feet (0.6m) thick and consisted of tan to blue-gray clayey to sandy silt. The test borings indicated that most of the colluvium was relatively tight and of low permeability. Blow counts ranged from 10 to 40 blows/foot. Bedrock was predominately green shale with limey interbeds and zones of weathered limestone and gray dolomite. Copies of typical borings are attached, Pages 95, 96 and 97.

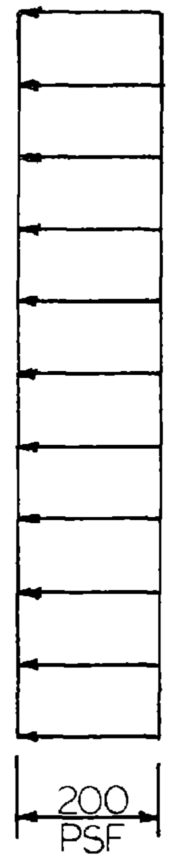
DESIGN CRITERIA

Design data relevant to the tieback system design was specified as follows:

* - Now Nicholson Construction Company

1. Lateral pressure diagrams to be used for design.





Pressure Caused
By Earth

Pressure Caused
By Surcharge

H = Total height of earth behind wall (after filling to finished grade).

NOTE: SOIL PROPERTIES FOR CALCULATING LATERAL EARTH PRESSURE DIAGRAM.

Unit wt. of soil angle of internal friction	γ	=	130 pcf
Angle of shearing resistance in Coulomb's Equation	ϕ	=	30°
Angle of wall friction	δ	=	0°
Coefficient of active earth pressure	K_a	=	.32

2. All components of the wall at every section of the wall shall be designed to resist the total load caused by the combining of the two pressure diagrams shown above, without exceeding the allowable stresses as noted below or as recognized in the various applicable codes and/or references.

3. Applicable codes and references:

   a) Tentative Recommendations for Prestressed Rock and Soil Anchors by Prestressed Concrete Institute.

   b) Building Code Requirements for Reinforced Concrete (ACE 318-77) by American Concrete Institute.

   c) Manual of Steel Construction (Seventh Edition) by American Institute of Steel Construction.

4. All anchors are to be considered as permanent anchors; therefore, all anchors shall be provided with protective corrosion seals over their entire lengths. Assure that sufficient free stressing length is provided so that small movements of the stressing anchor will not result in large changes in load.

   a) The allowable stress of the tendon or tendons shall not exceed 0.6 f_{pu} at the Design Load.

5. Exact type of soil or rock anchor and bond length (socket) to be selected by the contractor to develop the loads applied thereto.

NOTE: 1 pcf = 0.157 kN/m^3

f_{pu} = ultimate strength of prestressing steel

Anchor spacing selected averaged 8 feet (2.4m) c-c horizontally and vertically so that economical soldier beam and lagging size could be used. The angle of inclination for the anchors varied from 20° to 25° from horizontal to keep vertical components and resultant anchor loads practical and for ease of anchor installation. It was determined from inspection of the soil conditions that suitable anchoring strata was close to the existing ground surface.

Resultant anchor design loads varied considerably from 29 kips (129K/N) to 175 kips (778K/N) with most of the tiebacks anchored in the colluvial soil. A few of the bottom row tiebacks had a portion of their bond zones in bedrock.

The tiebacks were designed to have a free-stressing length extending 4 feet 6 inches (1.4m) minimum beyond an assumed failure plane. This failure plane began 2 feet (0.6m) below final grade in front of the wall and extended upward, at a 30° angle vertical to the wall, from the back of the wall. A minimum stressing length of 15 feet (4.6m) was maintained. The anchor bond length was designed by Nicholson Anchorage Company* using the empirical formula:

$$L = \frac{T_{ult}}{p' \times \pi \times d \times \text{TAN } \emptyset}$$

Where T_{ult} = 2 x Design Load
$\emptyset$ = 30° = Angle of Internal Friction
p' = Grout pressure, assumed as 2 psi (0.014 N/mm$_2$) per foot of overburden above the top of the anchor bond zone
d = effective diameter of bond zone (assumed 8" (200mm) since permeability was low and soil was fine grained).

As an example, an anchor with a 150 kip (667KN) design load and 30 feet (9m) of overburden above the bond zone would require a bond length of:

$$L = \frac{2 \times 150{,}000}{2 \times 30 \times \pi \times 8 \times \tan 30°} = 345" \text{ (8763 mm)}$$

or 28.7 feet (8.7m). $\dfrac{2 \times 667\text{KN}}{2 \times 0.014\text{N/mm}^2 \times \pi \times 200 \text{ mm} \times \tan 30}$

This would be rounded to 30 feet (9m). Over the entire wall, anchor bond zones varied from 20 feet (6m) to 40 feet (12m).

The material for the tiebacks consisted of the required number of 0.6 inch (1.5mm) diameter 270 ksi (1862N/mm^2) strands, conforming to ASTM A-416. The bond length of each tieback remained bare with spacer plates located at approximate 8 feet (2.5m) centers to provide a "basket" effect. The stressing length of each strand was greased and polyethylene sheathed. At the top bearing plate, each strand was anchored with a wedge and wedge housing.

RETAINING WALL AND TIEBACK CONSTRUCTION

The final design of the 546 feet (166.5m) long wall consisted of 68 steel double soldier piles at an average spacing of 8 feet (2.5m). The number of ties at each double soldier pile varied from 2 at the 18 feet (5.5m) minimum wall height to 7 at the 58.5 feet (17.8m) maximum height.

To construct the wall, the contractor first drilled caissons to bedrock and placed concrete to 6 feet (1.8m) below the bottom of the wall. Then he set the double soldier piles on top of the new concrete and completed concrete placement to the bottom of the wall. Next, perforated vertical drain pipe was installed adjacent to the double soldier piles and a low strength, high porosity, free draining concrete was poured to fill the remainder of the drilled caissons. This concrete served to hold the beams in place and also as an excellent drain for the soil behind the wall.

After excavating to about 2 feet (0.6m) below each tieback level, the contractor installed timber lagging and Nicholson drilled for and installed the ties, welded bearing plates to each double soldier pile and stressed each tieback. When excavation was completed and all levels of tiebacks were installed, tested and sheathed, the contractor poured a 14 inch (355mm) thick reinforced concrete facing over the lagging and double soldier beams. This concrete facing serves as a structural wall only to span the 8 feet (2.5m) gap between soldier beams. It is attached to the soldier beams by means of Nelson studs welded to the flanges of the beams.

Drilling for the tiebacks was performed using procedures and equipment identical to those described in the case history for the construction of the Seawall in Ft. Pierce, FL. Again, the bond zone of each tiebck was pressure grouted, maintaining an average of 80 psi (0.038N/mm^2) to 100 psi (0.690N/mm^2) pressure on the grout during withdrawal of the casing. The average grout take for the bond zone was about 20 bags of cement, however, in a few holes voids were present, presumably in or near the limestone bedrock, and over 100 bags of cement were required to fill some holes.

Difficulty in drilling occurred in some holes where boulders were encountered. Most of the boulders were penetrated but in a few cases, the angle of inclination was adjusted 2° or 3° and a new hole was drilled to miss the boulder.

The following stressing and testing procedures were used:

> <u>Performance Test</u> - Tiebacks were stressed in load increments of 0.25D, 0.50D, 0.75D, 1.00D, 1.20D and 1.33D where D is the Design Load. After each load increment was applied, the load was reduced to 0.05D prior to increasing the load to the next increment. Each intermediate stress level was maintained and monitored for at least 1 minute and the max. load was maintained and monitored for 24 hours. Performance Tests were run on designated tiebacks selected by the owner's representative to determine performance criteria for the remaining tiebacks. An average of at least 1 Performance Test was made for every 20 tiebacks installed.
>
> <u>Proof Test</u> - Tiebacks were stressed in load increments of 0.25D, 0.50D, 0.75D, 1.00D and 1.20D where D is the Design Load. Each intermediate stress level was maintained and monitored for at least 30 seconds and the maximum load was maintained and monitored for at least 10 minutes; unless longer monitoring times were required based on the results of the Performance Tests. Proof Tests were run on all tiebacks immediately before the specified "lock-off" load was applied. At the completion of each Proof Test for anchors found to be acceptable, the load was reduced to the specified "lock-off" load and secured. The final lock off load for all tiebacks was specified as 0.80 times design load.

Records of tieback elongation were kept for each increment during stressing and testing. Elongation was measured with a dial gauge accurate to 0.001 inch (0.01mm). In all cases, the tiebacks performed satisfactorily during testing and locking off. Copies of typical stressing records and stress-strain graphs are attached, Pages 98-100, 101-103 respectively.

As was previously mentioned, at completion of all tieback installations the entire wall was covered by 14 inches (35.5mm) thick cast-in-place concrete which provided permanent corrosion protection for the entire top anchorage assembly, thus eliminating need for future tieback maintenance.

The wall was completed in 1975 and has been in service since then with no required maintenance.

Location: ROANOKE MEMORIAL HOSPITAL, ROANOKE, VIRGINIA	BORING LOG	Structure: OFFICE & GARAGE	Comm. No. 1073
		Geologist: J.T.C.	Sheet 1 of 3
Contractor: C.C.D.		Engineer: ——	Boring No. (1)
			Date 7 JAN 1974

Misc. Data

- Lnth of hole: 79.7'
- Rock: 36.2'
- Wt of hammer: 140#
- Av fall of ham: 30"
- El of grd. water: ——

Elevation	Depth	Legend	Description of Materials (Type, color & Consistency)	Blows	Penetration	Sample No.	REMARKS
96.3	0.0		TOPSOIL.				
96.1	0.2		Brown to black clayey to sandy SILT, roots, organics and rock fragments.				
			COLLUVIUM.	5	1'	1	Sample 1.5'-2.5'
			Brown to yellow-tan clayey to sandy SILT w/ shale and sandstone fragments.	39	1'	2	Sample 5.0'-6.0'
86.3	10.0						
				25	1'	3	Sample 10.0'-11.0'
							Quartzite block at 14.0'
			More sand in matrix at 15.0'	35	1'	4	Sample 15.0'-16.0'
76.3	20.0		RESIDUUM.				
				21	1'	5	Sample 20.0'-21.0'
			Yellow-tan clayey to sandy SILT w/ very few highly weathered shale fragments.				
				21	1'	6	Sample 25.0'-26.0'
66.3	30.0						

GEOTECHNICS Form 10

Location	ROANOKE MEMORIAL HOSPITAL, ROANOKE, VIRGINIA	BORING LOG	Comm. No. 1073
		Structure OFFICE & GARAGE	Sheet 2 of 3
		Geologist J.T.C.	Boring No. 1
Contractor	C.C.Q.	Engineer ——	Date 7 JAN 1974

Misc. Data: Lnth of hole 79.7' · Rock 36.2' · Wt of hammer 140# · Av fall of ham 30" · El of grd. water ——

Elevation	Depth (Ft)	Legend	Description of Materials (Type, color & Consistency)	Blows	Penetration	Sample No.	REMARKS
66.3	30.0						
				36	1'	7	Sample 30.0'-31.0'
			Highly weathered tan and green SHALE.				
				10	1'	8	Sample 35.0'-36.0'
56.3	40.0						
				51	1'	9	Sample 40.0'-41.0'
52.8	43.5		TOP OF ROCK.				
52.3	44.0		Slightly weathered light green siliceous SHALE.				
			Slightly weathered light gray brecciated DOLOMITE.	Hrr	DLG 3Fr		Calcite healed fractures 43.5'-46.4'
49.9	46.4		Green SHALE w/limy interbeds.				Sporadic vugs, crystallized partings w/dark gray greasy mineralization 48.9'-54.5'.
49.0	47.3						
			Slightly to partially weathered gray brecciated DOLOMITE.	Hrr	DLG		
					7Fr		Core Recovery 43.5'-49.3' 98%
41.8	54.5		Slightly weathered green siliceous SHALE.				
41.3	55.0			Hrr	DLS, 2Fr		Lost drill water 55.0'
			Soft, highly weathered LIMESTONE.				Core Recovery 49.3'-59.3' 84%
39.1	57.2		Slightly weathered green siliceous SHALE.				
37.1	59.2						Sporadic vugs w/crystallization along some partings.
36.3	60.0		Slightly weathered gray brecciated DOLOMITE.				

GEOTECHNICS Form 10

ROANOKE MEMORIAL HOSPITAL

BORING LOG

Location	ROANOKE, VIRGINIA
Contractor	C.C.D.
Structure	OFFICE & GARAGE
Geologist	J.T.C.
Engineer	——
Comm. No.	1073
Sheet	3 of 3
Boring No.	1
Date	8 JAN 1974

Misc. Data: Lnth of hole 79.7'; Rock 36.2'; Wt of hammer 140#; Av fall of ham 30"; El of grd. water ——

Elevation	Depth Ft	Legend	Description of Materials (Type, color & Consistency)	Blows	Penetration	Sample No.	REMARKS
36.3	60.0						
			Slightly weathered gray brecciated DOLOMITE.		[illegible]		Calcite healed fractures 47.3'-62.6'; 70.6'-77.5'
33.7	62.6		Fracturing 62.6'-64.1'				
32.2	64.1		Green SHALE w/ buff limestone interbeds.				Slightly vuggy 65.0'-66.0
							Core Recovery 59.3'-69.5' 98%
25.7	70.6		Light gray brecciated DOLOMITE.		[illegible]		Calcite healed fractures 70.6'-77.5'
24.4	71.9						
23.6	72.7		Limy zone. Light green siliceous SHALE.				Core Recovery 69.5'-79.7' 98%
21.6	74.7		Light gray brecciated DOLOMITE				Vuggy w/ crystallization along partings 75.1'-76.3'
18.8	77.5		Light green SHALE w/ limy interbeds.				
17.6	78.7		Light gray brecciated DOLOMITE w/ calcite healed fractures.				
16.6	79.7						
			BOTTOM OF HOLE.				
			Completed 3:00 P.M. 8 January 1974			W.L. at	completion: 43.2' Hole collapsed when casing pulled.

GEOTECHNICS Form 10

STRESSING DATA

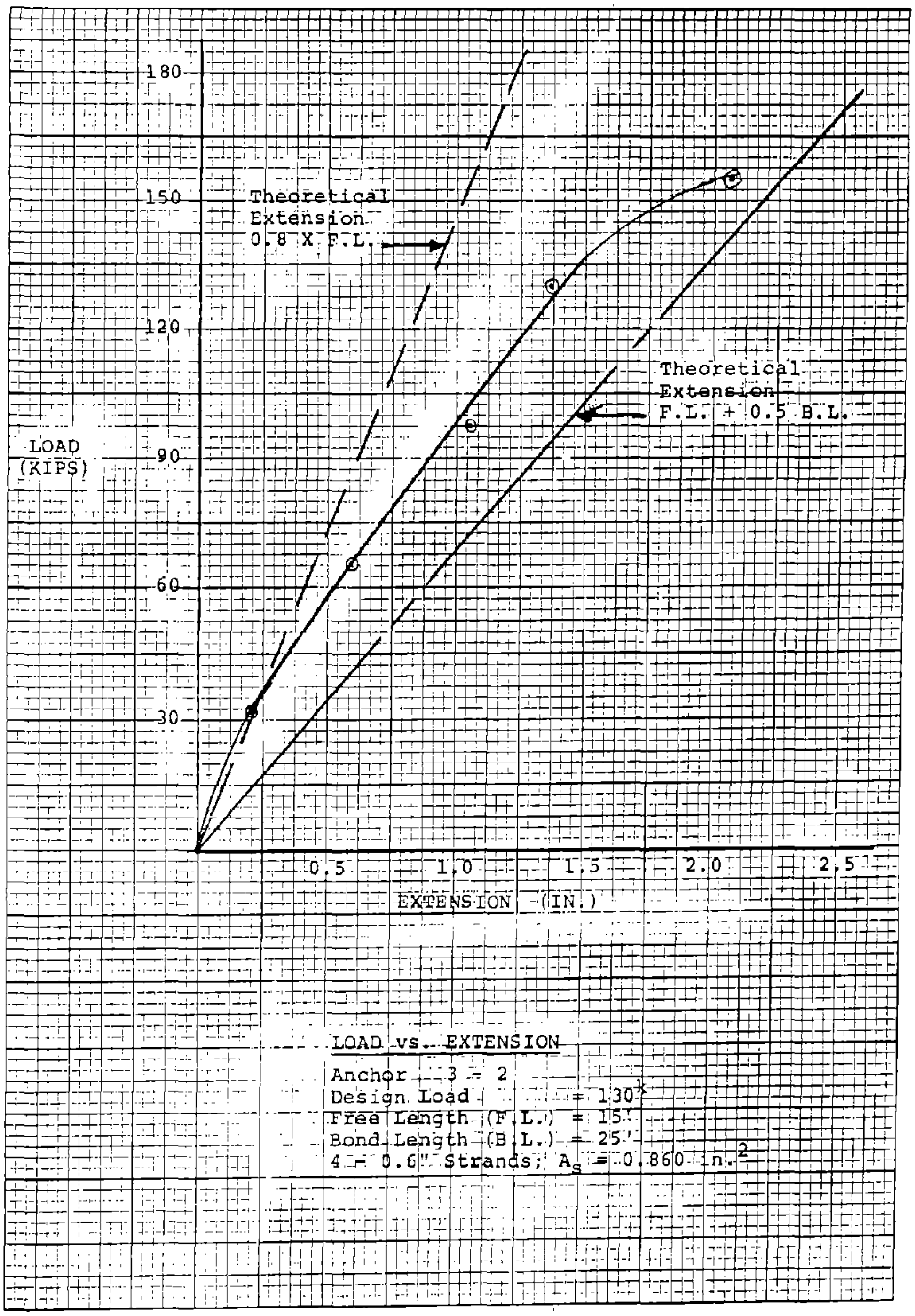
Hole No.	Tend. Inst.	Stress Date	Date Stress	0- .25D	.25- .50D	.50- .75D	.75- 1.00D	1.00- 1.2D	After 24 1.00-1.33D	Total ΔL	F.L.	Ant. ΔL
1-1	5/3	5/9	5/13	.055	.142	.178	.188	.212		.775		
2-1	5/3	5/9	5/13	.102	.323	.434	.357	.352		1.568		
3-1	4/27	5/2	5/4	.088	.372	.347	.484	.374		1.685		
4-1	4/27	5/2	5/3	.068	.063	.264	.259	.211		.869		
5-1	4/27	5/2	5/4	.150	.347	.547	.451	.389		1.884		Lock-Off
6-1	4/22	4/27	5/6	.184	.484	.597	.739	.542		2.546		5/6
7-1	4/14	4/19	5/6	.051	.291	.310	.383	.280		1.315		
8-1	4/14	4/19	5/6	.229	.701	.652	.590	.663		2.835		
9-1	4/5	4/10	6/10	.292	.738	.536	.556	.448		2.600		
10-1	3/17	3/22	5/7	.096	.382	.707	.740	.735	.258	2.918		Perf. Test
11-1	3/16	3/22	5/15	.235	.620	.854	.906	.768	.761	4.144		Perf. Test
12-1	3/16	3/21	3/22	.588	.630	.707	.803	.790		3.518	44'	
13-1	3/17	3/22	3/22	.557	.638	.820	.802	.770		3.587	44'	
14-1	3/17	3/22	3/22	.378	.799	.672	.801	.754		3.404	44'	
15-1	3/16	3/27	3/22	.182	.798	.925	.960	.873		3.738	44'	
16-1	3/17	3/22	3/23	.836	.832	.847	.818	.813		4.146	44'	
17-1	3/16	3/21	3/23	.604	.654	.940	.768	.904		3.870	44'	
18-1	3/17	3/22	3/24	.636	1.596	2.525	3.405	4.351	5.086	5.086	44'	Perf. Test
19-1	3/17	3/22	3/23	1.169	.753	.832	.806	.814		4.374	44'	
20-1	3/17	3/22	3/23	.466	.818	.831	.849	.780		3.744	44'	
21-1	3/18	3/23	3/18	.380	.652	.748	.759	.701		3.240	44'	
22-1	3/18	3/23	3/24	.424	.626	.780	.794	.765		3.389	44'	
23-1	3/18	3/23	3/24	.590	.772	.786	.784	.840		3.772	44'	
24-1	3/22	3/27	3/29	.418	.844	.813	1.005	.578		3.658	44'	
25-1	3/23	3/28	3/29	.399	.931	.996	.916	.606		3.736	44'	
26-1	3/18	3/23	3/25	.767	.933	.939	.849	.671		4.159	44'	
27-1	3/18	3/23	3/25	.339	.346	.557	.380	.383		2.005	43'	
28-1	3/18	3/23	3/29	.474	.422	.551	.453	.395		2.295		
29-1	3/19	3/24	3/29	.255	.353	.340	.430	.347		1.725		
30-1	3/25	3/29	3/29	.346	.384	.538	.477	.448		2.193	48'	
31-1	3/25	3/30	3/30	.456	.774	.878	.817	.740		3.665	48'	@37M

STRESSING DATA

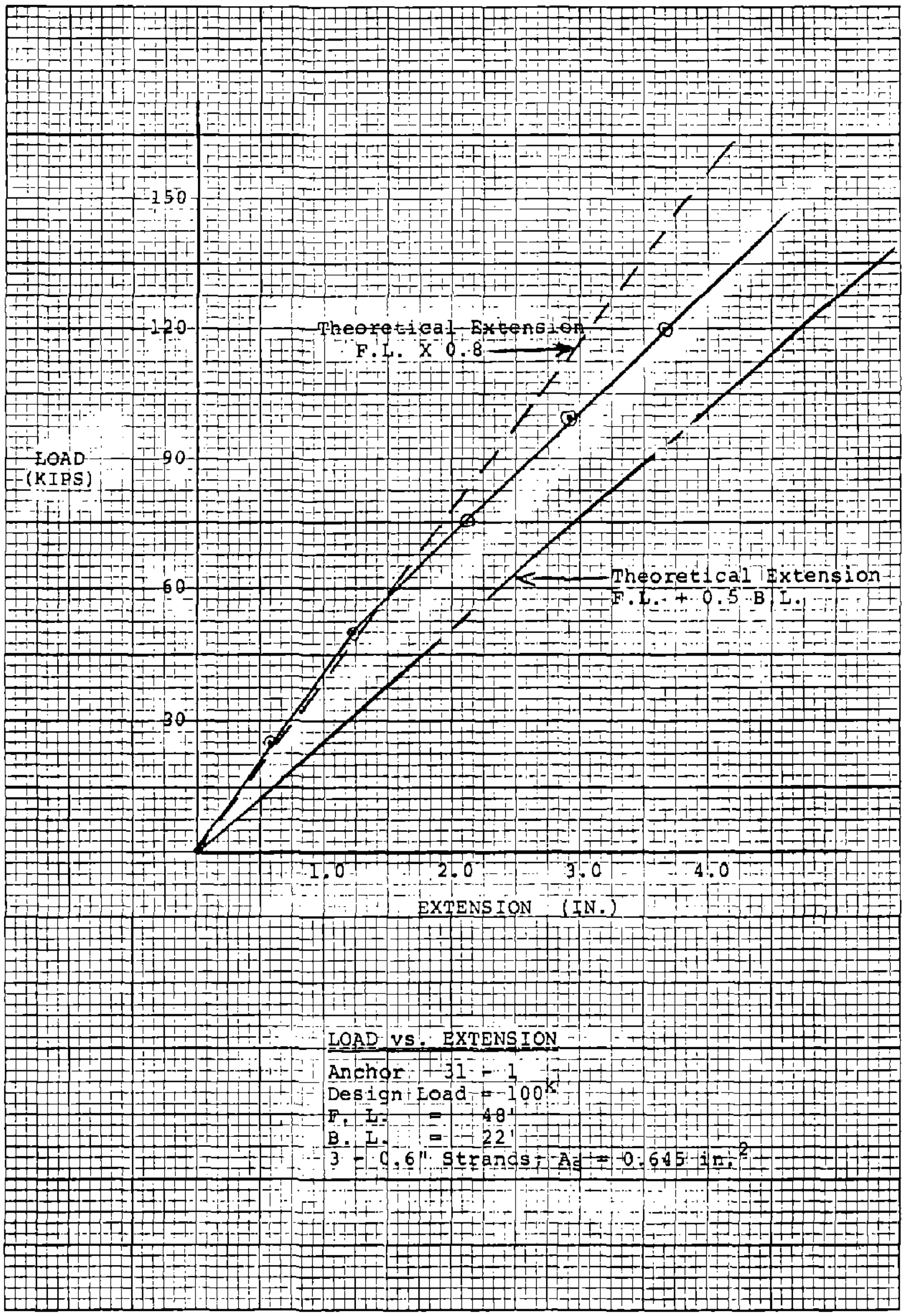
Hole No.	Tend. Inst.	Stress Date	Date Stress	0-.25D	.25-.50D	.50-.75D	.75-1/00D	1.00-1.2D	After 24 1.00-1.33D	Total ΔL	F.L.	Ant. ΔL
1-2	5/10	5/14	5/17	.178	.425	.419	.523	.473		2.018		
2-2	5/10	5/14	5/17	.154	.364	.385	.453	.366		1.722		
3-2	5/10	5/14	5/17	.184	.406	.457	.563	.467		2.077		
4-2	4/27	5/2	4/30	.134	.317	.374	.339	.359		1.503		
5-2	4/27	5/2	4/30	.128	.519	.511	.552	.522		2.192		
6-2	4/27	5/2	4/30	.338	.659	.709	.813	1.058		3.577		
			5/3	.291	.652	.692	.737	.628		3.010	@ 100psi	1.125
7-2	4/19	4/24	4/23	.608	.427	.433	.440	.350		2.258		
8-2	4/19	4/24	4/23	.515	.475	.600	.521	.611		2.722		
9-2	4/6	4/11	4/13	.195	.381	.467	.462	.450		1.955		
10-2	4/5	4/10	4/13	.344	.705	.716	.826	.654		3.245		
11-2	4/5	4/10	4/13	.298	.525	.694	.748	.695		2.960		
12-2	4/12	4/17	4/15	.374	.603	.753	.791	.714		3.235		
13-2	4/2	4/7	4/14	.650	.574	.616	.674	.662		3.176		
14-2	4/2	4/7	4/8	.190	.490	.729	.720	.681	.408	3.295	4 hrs. Perf. Test	
15-2	4/5	4/10	4/8	.378	.641	.561	.762	.636		2.978		
16-2	4/5	4/10	4/12	.458	.558	.713	.731	.713		3.173		
17-2	4/6	4/11	4/12	.252	.533	.708	.792	.690		2.975		
18-2	4/6	4/11	4/12	.395	.646	.639	.710	.668		3.058		
19-2	4/7	4/12	4/12	.503	.635	.740	.752	.755		3.385		
20-2	4/7	4/12	4/12	.450	.577	.700	.688	.650		3.065		
21-2	4/7	4/12	4/12	.408	.722	.792	.804	.697		3.423		
22-2	4/7	4/12	4/12	.513	.690	.689	.742	.675		3.309		
23-2	4/7	4/12	4/12	.615	.775	.689	.799	.715		3.593		
24-2	4/12	4/17	4/15	.495	.600	.693	.751	.629		3.168		
25-2	4/8	4/13	4/12	.525	.487	.657	.716	.610		2.995		
26-2	4/8	4/13	4/18	.451	.634	.720	.760	.667		3.232		
27-2	4/13	4/18	4/19	.337	.598	.680	.788	.693		2.996		
28-2	4/13	4/18	4/19	.475	.483	.464	.598	.618		2.738		
29-2	4/13	4/18	4/19	.420	.538	.610	.668	.622		2.858		
30-2	4/13	4/18	4/19	.238	.610	.725	.712	.591		2.876		
31-2	5/4	5/10	5/13	.410	.726	.831	.718	.690		3.375		
6-2A	5/4	5/10	5/13	.098	.146	.191	.204	.191		.830		

STRESSING DATA

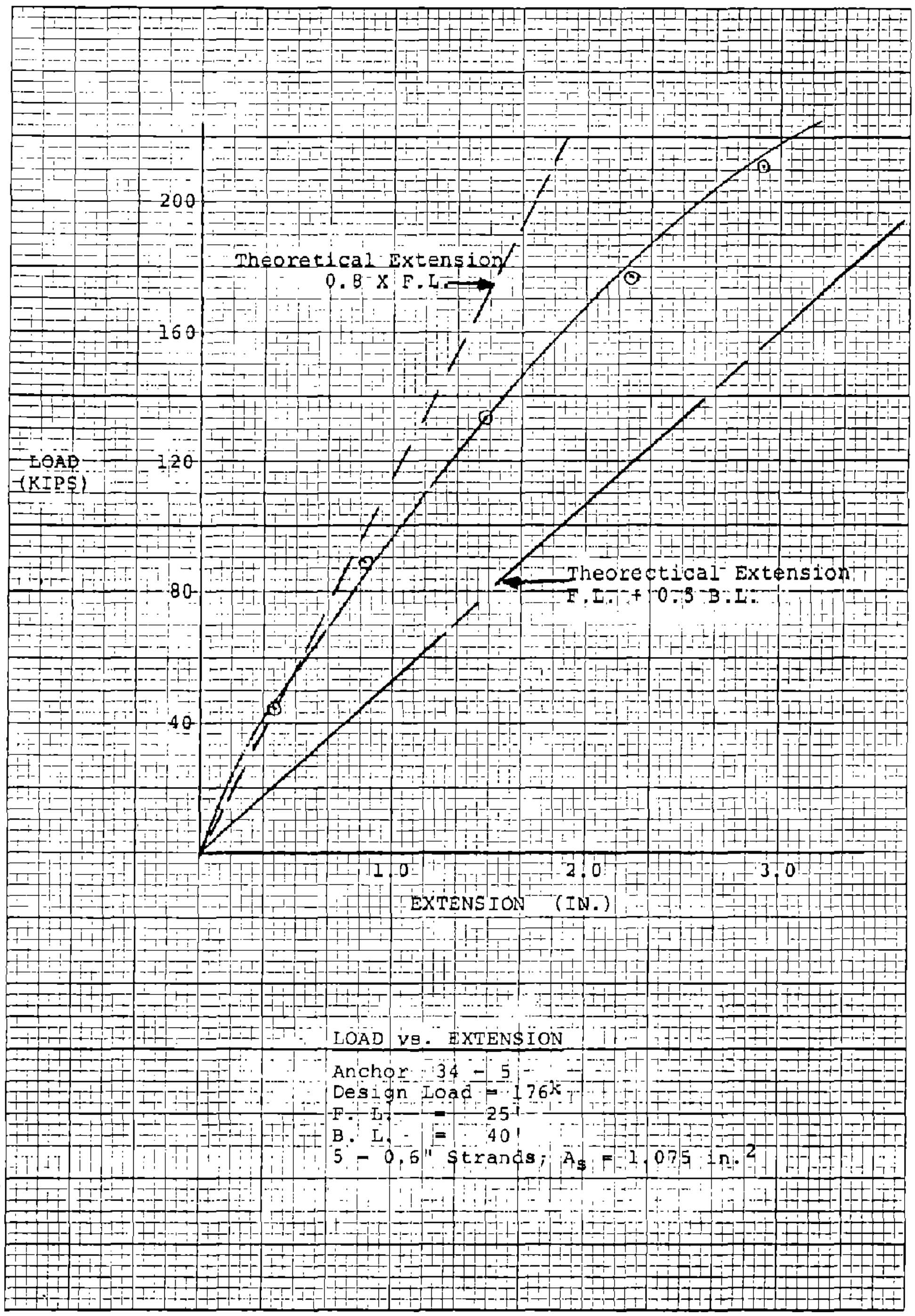
Hole No.	Tend. Inst.	Stress Date	Date Stress	0-.25D	.25-.50D	.50-.75D	.75-1.00D	1.00-1.2D	After 24 1.00-1.33D	Total ΔL	F.L.	Ant. ΔL
9-5	5/13	5/17	5/28	.271	.394	.486	.563	.524	.509	2.247		Perf. Test
10-5	5/14	5/18	5/28	.231	.334	.445	.458	.455		1.923		
11-5	5/13	5/17	6/8	.312	.474	.504	.711	.664		2.664		
12-5	5/11	5/15	6/8	.371	.609	.560	.647	.539		2.726		
13-5	5/11	5/15	6/8	.409	.561	.606	.711	.587		2.874		
14-5	5/11	5/15	6/9	.363	.447	.541	.593	.555		2.489		
15-5	5/11	5/15	5/18	.260	.342	.504	.577	.513		2.251		
16-5	5/11	5/15	5/18	.277	.408	.453	.518	.449		2.105		
17-5	5/12	5/16	5/18	.245	.421	.512	.571	.498		2.247		
18-5	5/12	5/16	5/18	.340	.464	.671	.611	.619		2.705		
19-5	5/12	5/16	6/9	.216	.356	.428	.485	.420	.385 *	2.290	*	Perf. Test
20-5	5/13	5/17	5/18	.250	.424	.432	.475	.422		1.958		
21-5	5/13	5/17	6/9	.329	.430	.465	.490	.431		2.145		
22-5	5/13	5/17	6/9	.305	.411	.484	.530	.432		2.162		
23-5	5/13	5/17	6/9	.262	.359	.475	.514	.379		1.989		
24-5	5/19	5/23	6/7	.275	.466	.494	.514	.460		2.209		
25-5	5/20	5/24	6/7	.380	.480	.499	.501	.490		2.350		
26-5	5/20	5/24	6/7	.290	.424	.450	.486	.453		2.103		
27-5	5/14	5/18	5/19	.237	.373	.412	.491	.423		1.936		
28-5	5/14	5/18	5/14	.210	.396	.437	.499	.374		1.916		
29-5	5/14	5/18	5/19	.204	.357	.383	.495	.376		1.820		
30-5	5/14	5/18	5/19	.263	.368	.426	.507	.439		2.003		
31-5	5/14	5/18	5/19	.302	.496	.540	.687	.598		2.623		
32-5	5/15	5/19	5/19	.240	.460	.530	.595	.440		2.265		
33-5	5/17	5/21	5/20	.306	.582	.677	.770	.704		3.039		
34-5	5/17	5/21	5/20	.327	.521	.632	.754	.664		2.898		
35-5	5/15	5/19	5/19	.393	.629	.710	.855	.708		3.295		
36-5	5/15	5/19	5/19	.294	.624	.637	.746	.521		2.822		
37-5	5/17	5/21	5/20	.314	.463	.511	.649	.563		2.500		
38-5	5/17	5/21	5/20	.298	.474	.556	.667	.553		2.648		
39-5	5/17	5/21	5/20	.400	.566	.643	.768	.666		3.043		
40-5	5/25	5/30	6/3	.429	.435	.536	.685	.631		2.716		
41-5	5/25	5/30	6/3	.222	.462	.591	.720	.590		2.585		
42-5	5/25	5/30	6/1	.368	.418	.589	.735	.686		2.796		
43-5	5/26	5/31	6/1	.392	.435	.638	.643	.652		2.760		
44-5	5/25	5/30	6/1	.313	.541	.644	.754	.715		2.967		
45-5	5/25	5/30	6/1	.455	.540	.393	.673	.702		2.763		
46-5	5/25	5/30	6/4	.349	.467	.532	.658	.878	.703	3.743		Perf. Test



NOTE: 1 KIP = 4.448 kN $\quad$ 1 in.^{2} = 0.000645 m^2
1 in. = 0.0254 m $\quad$ 1 ft. = 0.305 m



LOAD (KIPS)
150
120
90
60
30
1.0
2.0
3.0
4.0
EXTENSION (IN.)
Theoretical Extension F.L. X 0.8
Theoretical Extension F.L. + 0.5 B.L.
LOAD vs. EXTENSION
Anchor 31 - 1
Design Load = 100k
F. L. = 48'
B. L. = 22'
3 - 0.6" Strands; As = 0.645 in.2



LOAD
(KIPS)
200
160
120
80
40
1.0
2.0
3.0
EXTENSION (IN.)
Theoretical Extension
0.8 X F.L.
Theorectical Extension
F.L. + 0.5 B.L.
LOAD vs. EXTENSION
Anchor 34 - 5
Design Load = 176K
F. L. = 25'
B. L. = 40'
5 - 0.6" Strands; As = 1.075 in.2

CASE HISTORY

RETAINING WALL

FOR

MONTEFIORE HOSPITAL PARKING GARAGE

OWNER: ALLEGHENY COUNTY HOSPITAL
DEVELOPMENT AUTHORITY

ENGINEER: RITCHIE ASSOCIATES, INC.
CHESTNUT HILL, MA

GENERAL CONTRACTOR: NAVARRO CORPORATION
PITTSBURGH, PA

The construction of a parking garage for Montefiore Hospital required an excavation up to 40 feet (12cm) deep adjacent to the existing hospital. In order to restrain the existing building and overburden and to allow excavation and construction of the garage to be completed, the engineer designed a permanent retaining wall tied back with prestressed ground anchors.

The design of the wall called for a 2 foot (0.6m) diameter tangent caisson wall, 400 feet (122m) long, with one to two rows of tiebacks, depending on height of excavation. A double channel wale encased in concrete was used to transfer the tieback loads to the wall.

Soil conditions at the site were fairly consistent for the entire wall length. At the typical 40 foot (12m) excavation, the top 15 feet (4.6m) was fill material consisting of silty and sandy clay, with some shale, gravel, and stone fragments. The next 10 feet (3m) consisted of a weathered shale formation and fractured sand and siltstones. Underlying the shale for- mation was medium to hard brown and gray clay shale. This clay and shale is part of the notorious Pittsburgh red bed formation which has many slickenside seams and is extremely prone to sliding when disturbed.

The wall caissons were founded on sandstone underlying the clay shale. The bond zone for most of the anchors was in the clay shale with some of the anchors in the bottom row founded in the sandstone.

The lateral design forces on the wall were determined by the engineer and these were shown on the contract drawings as horizontal tieback design loads. The point of application of the tiebacks was also determined by the engineer so that design of the caisson wall could be completed. A failure surface rising at a 45° angle from the bottom of excavation at the back of the wall was assumed.

After investigating the site soil and rock conditions, Nicholson Anchorage Company determined that the clay shale layer was suitable to develop the anchor loads as shown by the engineer. The anchors were spaced on 4 feet (1.2m) centers horizontally and, where two rows were required, the typical vertical spacing was 18 feet (5.5m). The method of construction of the wall with a wide flange beam placed full length every 2 feet (0.61m) gave great structural strength in the vertical direction. To take advantage of this strength and save on the number of anchors and depth of wale, a large relatively small horizontal spacing was used.

Using the concepts outlined in Tasks A and B, an angle of inclination varying from 15° to 30° was selected. The unbonded stressing length extended a minimum of 5 feet (1.5m) beyond the 45° failure surface rising from the bottom of the excavation. This length varied from 15 feet (4.6) to 25 feet (7.6m). A minimum 15 feet (4.6m) stressing length was maintained for the bottom row of anchors.

The bond length varied depending on required anchor load from 20 feet (6m) minimum for a 50 kip anchor to 50 feet (15m) for the 200 kip maximum anchor design load. The longest anchor was 75 feet (22.8m) and the shortest 45 feet (13.7m) total length.

The procedure for calculating the bond length used the following equation:

$$W_{ult} = T_{ult} \times d \times L \times \pi$$

where,

W_{ult} = Desired ultimate capacity of anchor or 200% of anchor design load
d = Drill hole diameter
L = Bond Length
T_{ult} = Assumed ultimate bond stress between rock (or soil) and anchor grout = 50psi (.34N/mm^2).

For the clay shale encountered in this project, a ultimate bond stress of 50 psi (0.34N/mm^2) was assumed. Using this stress, the bond length for a 200 kip (890 KN) design load and a 5 inch (12.7mm) diameter drill hole is calculated as follows:

$$L = \frac{W_{ult}}{T_{ult} \times d \times \pi}$$

or $$L = \frac{2\ (\text{S.F.}) \times 200{,}000}{50\ \text{p.s.i.} \times 5'' \times \pi}$$

$$\frac{2\ (\text{S.F.}) \times 890\ \text{kN}}{0.34\text{N/mm}^2 \times 127\ \text{mm} \times \pi}$$

or $L = 509.3'' = 42.4'$ (12.9m)

The complete tabulation for each of the tiebacks is shown in Table I at the back of this report.

The engineer had specified underreaming the bottom of the drill hole as required to provide the required anchor forces, but Nicholson Anchorage Company , using the above equation, concluded that underreaming was not required. This was proven by the 200% test anchor as will be described later in this report.

The post tensioning system selected consisted of the required quantity of 0.6 inch (1.5mm) diameter strands conforming to ASTM A-416 with an ultimate strength of 270 ksi ($1862N/mm^2$). A copy of the material test certifications is attached to the back of this report. The bond length remained bare with strand spreaders wired at about 7 feet (2m) centers to obtain a "basketing" effect. The stressing length of each strand was greased and polyvinylchloride sheathed to provide corrosion protection and to prevent grout from bonding to the stressing length steel.

The entire tieback hole length was filled with grout in one stage with a grout mix consisting of 5 gallons (18.9L) of potable water per bag of Type I Portland cement.

Installation of the ground anchors did not begin until all 199 of the 24 inch (60mm) diameter caissons had been constructed.

The tangent caisson wall was constructed by drilling the 24 inch (60.0mm) diameter hole, then placing the required reinforcement, which consisted of a wide flange structural shape. Concrete was then placed to fill the hole. Then Nicholson Anchorage Company* drilled and installed the tiebacks on 4 feet (1.2m) centers at the tangent point between two adjacent caissons. Also at this time, the steel double-channel wale and concrete encasement were placed, leaving a pocket in the concrete at the tieback locations to allow for stressing after the wale concrete had reached sufficient strength.

After anchor lock-off, the contractor continued the excavation either to its final grade or to 5 feet (1.5m) below the elevation of the second row of ties.

Installation and stressing of the second row proceeded the same as the first row, and finally the contractor continued the excavation to final grade.

In addition to the above stages of construction along the vertical height of the wall, construction was also accomplished in stages along the length of the wall to limit excavated areas not supported by tiebacks to approximately 30 feet (9m).

Each of the tiebacks was constructed using Nicholson Anchorage Company's specially designed and built track mounted drill rigs. Drilling of holes was accomplished with tri-cone roller bits with water as the flushing medium. Holes were temporarly cased when required until the tendon and grout had been placed.

Some problems were encountered during the installation of the first 20 or so anchors. The problems were caused when a fractured rock layer was encountered with some voids as much as 2 feet (0.6m) thick. This resulted in extremely slow and difficult conditions for drilling, anchor tendon installation, and grouting.

The solution to the problem required significant drilling, pre-grouting, re-drilling and re-grouting the holes where voids or significant fracturing was observed. One side effect of the additional grouting was that it undoubtedly strengthened the foundation of the existing building and also improved the stability of the hillside. Over 500 C.F. (14 cubic meters) of excess grout was used in solving this problem for the first few holes. As work progressed, however, voids and fractured rock were not observed.

As mentioned earlier, anchor stressing proceeded as soon as the wale concrete had achieved sufficient strength. Each of the anchors was proof tested to 1.25 times design load before being locked off at 1.15 times design load. The proof load was held for 5 minutes before locking off. Anchor elongation was measured accurate to within .062" (1.5mm). Copies of typical stressing records are attached herewith.

In addition, two anchors were selected to be tested to 2.0 times design load to demonstrate the adequacy of the anchor design. The anchors selected to be tested had design loads of 143 and 200 kips (636KN - 890KN). The procedure used to test the anchors was as follows:

1. Load anchor in 2 equal increments to 50% of design load, then reduce the load in the same increments to zero, measuring anchor extension at each increment.

2. Load anchor in 3 equal increments to 75% of design load, then reduce the load in the same increments to zero, measuring anchor extension at each increment.

3. Load anchor in 4 equal increments to 100% of design load, then reduce the load in the same increments to zero, measuring anchor extension at each increment.

4. Load anchor in 6 equal increments to 150% of design load, then reduce the load in the same increments to zero, measuring anchor extension at each increment.

5. Load anchor in 8 equal increments to 200% of design load, measuring anchor extension at each increment. Hold test load on anchor for 30 minutes minimum, recording anchor extension at 5 minute intervals. Then reduce load back to zero in the same increments, again measuring anchor extension at each increment.

6. Upon successful completion of test, the anchor was stressed and locked off at 115% of design load similar to other production anchor stressing procedures.

7. Anchor extension readings were measured with dial gauges accurate to 0.001 inch (.001mm) mounted to the back of the jack stressing plate.

A copy of the data obtained for these two test anchors which demonstrated the validity of the anchor design procedures is attached to the back of this report.

Stressing of the anchors was mainly performed using a 150 ton (1209 kN) center hole hydraulic jack stressing jack when all anchor strands were stressed simultaneously. A few lightly loaded anchors were stressed using a single strand stressing jack having a capacity of 30 tons (267 kN), and the test anchors were stressed using a pair of 100 ton (890 kN) capacity jacks connected to the anchor by a bridging beam.

Each of the tiebacks successfully held proof test loads and all behaved elastically during stressing, as verified by anchor elongation readings. Also, a few of the anchors were lifted off after a few days and all lift-off checks showed a loss of stress within 5% of the initial lock- off load.

The engineer specified that ground movement near the wall be monitored during and after anchor stressing. To do this, two slope indicator investigation wells were installed directly behind the wall. Inclinometer surveys carried out after stressing show a wall movement towards the hospital, indicating a positive response to the anchors (but also indicating a proportional decrease in anchor stress).

Since its completion in 1976, the wall has performed as intended with no remedial work necessary to date.

TIE-BACK ANCHOR TABULATION

ANCHOR NUMBER	*ANGLE TO HORIZ.	MIN. FREE LENGTH	EST. ANCHOR LENGTH (BOND)	ESTIMATED LENGTH (AVG.)	ANCHOR LOAD (KIPS)	PROOF LOAD (KIPS)	NUMBER OF STRANDS
1	20 Deg.	25 Ft.	50 Ft.	75 Ft.	143	179	5
2	20 "	25 "	50 "	75 "	143	179	5
3	20 "	25 "	50 "	75 "	143	179	5
4	20 "	25 "	50 "	75 "	143	179	5
5	20 "	25 "	40 "	65 "	143	179	5
6	20 "	25 "	40 "	65 "	143	179	5
7	20 "	25 "	40 "	65 "	143	179	5
8	20 "	25 "	40 "	65 "	143	179	5
9	20 "	25 "	40 "	65 "	143	179	5
10	20 "	25 "	40 "	65 "	143	179	5
11	20 "	25 "	40 "	65 "	143	179	5
12	20 "	25 "	40 "	65 "	143	179	5
13	20 "	25 "	40 "	65 "	143	179	5
14	20 "	25 "	40 "	65 "	143	179	5
15	20 "	25 "	40 "	65 "	143	179	5
16	20 "	25 "	40 "	65 "	143	179	5
17	20 "	25 "	40 "	65 "	143	179	5
18	20 "	25 "	40 "	65 "	143	179	5
19	20 "	25 "	40 "	65 "	143	179	5
20	20 "	25 "	40 "	65 "	143	179	5
21	20 "	25 "	40 "	65 "	143	179	5
22	25 "	25 "	40 "	65 "	132	165	4
23	25 "	25 "	40 "	65 "	132	165	4
24	25 "	25 "	40 "	65 "	132	165	4
25	25 "	25 "	40 "	65 "	132	165	4
26	25 "	25 "	40 "	65 "	132	165	4
27	25 "	25 "	40 "	65 "	132	165	4
28	25 "	25 "	40 "	65 "	132	165	4
29	25 "	25 "	40 "	65 "	132	165	4
30	20 "	25 "	40 "	65 "	128	160	4
31	15 "	25 "	40 "	65 "	124	155	4
32	15 "	25 "	40 "	65 "	124	155	4
33	15 "	25 "	40 "	65 "	124	155	4
34	15 "	25 "	40 "	65 "	124	155	4
35	15 "	25 "	40 "	65 "	124	155	4
36	15 "	25 "	40 "	65 "	124	155	4
37	15 "	25 "	40 "	65 "	124	155	4
38	15 "	25 "	40 "	65 "	124	155	4
39	25 "	25 "	40 "	65 "	132	165	4
40	25 "	25 "	40 "	65 "	132	165	4

*Vary the angle to horizontal of adjacent anchors by 2^{o} to 4^{o} in order to separate bond portion of adjacent anchors.

Note: 1' = 0.3 meters
1K = 0.5 metric tons

ANCHOR NUMBER	*ANGLE TO HORIZ.	MIN. FREE LENGTH	EST. ANCHOR LENGTH (BOND)	ESTIMATED LENGTH (AVG.)	ANCHOR LOAD (KIPS)	PROOF LOAD (KIPS)	NUMBER OF STRANDS
41	25 Deg.	25 Ft.	40 Ft.	65 Ft.	132	165	4
42	20 "	25 "	30 "	55 "	94	118	3
3	20 "	25 "	30 "	55 "	94	118	3
44	20 "	25 "	30 "	55 "	94	118	3
45	20 "	25 "	30 "	55 "	94	118	3
46	20 "	25 "	30 "	55 "	94	118	3
47	20 "	25 "	30 "	55 "	94	118	3
48	20 "	25 "	30 "	55 "	94	118	3
49	20 "	25 "	30 "	55 "	68	85	3
50	20 "	25 "	30 "	55 "	68	85	2
51	20 "	25 "	30 "	55 "	68	85	2
52	20 "	25 "	30 "	55 "	68	85	2
53	30 "	25 "	30 "	55 "	74	93	2
54	30 "	25 "	30 "	55 "	74	93	2
55	20 "	25 "	30 "	55 "	68	85	2
56	20 "	25 "	30 "	55 "	68	85	2
57	20 "	25 "	30 "	55 "	68	85	2
58	20 "	25 "	30 "	55 "	68	85	2
59	30 "	15 "	20 "	35 "	50	63	2
60	30 "	15 "	20 "	35 "	50	63	2
61	30 "	15 "	20 "	35 "	50	63	2
62	30 "	15 "	20 "	35 "	50	63	2
63	30 "	15 "	20 "	35 "	50	63	2
64	30 "	15 "	20 "	35 "	50	63	2
65	30 "	15 "	20 "	35 "	50	63	2
66	30 "	15 "	20 "	35 "	50	63	2
67	30 "	15 "	20 "	35 "	50	63	2
68	30 "	15 "	20 "	35 "	50	63	2
69	30 "	15 "	20 "	35 "	50	63	2
70	30 "	15 "	20 "	35 "	50	63	2
71	30 "	15 "	20 "	35 "	50	63	2
72	30 "	15 "	20 "	35 "	50	63	2
73	30 "	15 "	20 "	35 "	50	63	2
74	20 "	15 "	50 "	65 "	200	250	6
75	20 "	15 "	50 "	65 "	200	250	6
76	20 "	15 "	50 "	65 "	200	250	6
77	20 "	15 "	50 "	65 "	200	250	6
78	20 "	15 "	50 "	65 "	200	250	6
79	20 "	15 "	50 "	65 "	200	250	6
80	20 "	15 "	50 "	65 "	200	250	6
81	20 "	15 "	50 "	65 "	200	250	6
82	20 "	15 "	50 "	65 "	200	250	6
83	20 "	15 "	50 "	65 "	200	250	6
4	20 "	15 "	50 "	65 "	200	250	6
85	20 "	15 "	50 "	65 "	200	250	6

*Vary the angle to horizontal of adjacent anchors by 2° to 4° in order to separate bond portion of adjacent anchors.

Note: 1 ft = 0.3 meters
1 kip = 0.5 metric tons

ANCHOR NUMBER	*ANGLE TO HORIZ.	MIN. FREE LENGTH	EST. ANCHOR LENGTH (BOND)	ESTIMATED LENGTH (AVG.)	ANCHOR LOAD (KIPS)	PROOF LOAD (KIPS)	NUMBER OF STRANDS
86	20 Deg.	15 Ft.	50 Ft.	65 Ft.	200	250	6
87	20 "	15 "	50 "	65 "	200	250	6
88	20 "	15 "	50 "	65 "	200	250	6
89	20 "	15 "	50 "	65 "	200	250	6
90	20 "	15 "	50 "	65 "	200	250	6
91	20 "	15 "	50 "	65 "	200	250	6
92	20 "	15 "	50 "	65 "	200	250	6
93	20 "	15 "	50 "	65 "	200	250	6
94	20 "	15 "	50 "	65 "	200	250	6
95	20 "	15 "	50 "	65 "	149	186	5
96	20 "	15 "	50 "	65 "	149	186	5
97	20 "	15 "	50 "	65 "	149	186	5
98	20 "	15 "	50 "	65 "	149	186	5
99	20 "	15 "	50 "	65 "	149	186	5
100	20 "	15 "	50 "	65 "	149	186	5
101	20 "	15 "	50 "	65 "	149	186	5
102	20 "	15 "	50 "	65 "	149	186	5
103	20 "	15 "	50 "	65 "	149	186	5
104	20 "	15 "	50 "	65 "	149	186	5
105	20 "	15 "	50 "	65 "	149	186	5
106	20 "	15 "	50 "	65 "	149	186	5
107	20 "	15 "	50 "	65 "	149	186	5
108	20 "	15 "	50 "	65 "	149	186	5
109	20 "	15 "	50 "	65 "	149	186	5
110	20 "	15 "	50 "	65 "	149	186	5
111	20 "	15 "	50 "	65 "	149	186	5
112	20 "	15 "	50 "	65 "	149	186	5
113	20 "	15 "	50 "	65 "	149	186	5
114	20 "	15 "	50 "	65 "	149	186	5

*Vary the angle to horizontal of adjacent anchors by 2° to 4° in order to separate bond portion of adjacent anchors.

Revised 5-20-75

Note: 1' = 0.3 meters
1K = 0.5 metric tons

TEST RESULTS

Test Anchor No. 1 Anchor No. 8 Tested Sept. 11, 1975

Time	Load (Tons)	Dial Gauge Reading	Rule Gauge Reading	Anchor Extension	Remarks
11:20	-0-	0.100	0.00	-0-	
11:21	17.87	0.556	0.45	.456	
11:23	35.75	0.822	0.82	.822	50% x W.L.
11:27	17.87	0.710	0.60	.610	
11:29	-0-	0.400	0.30	.300	
11:32	-0-	0.300	0.30	.300	Gauge Re-set
11:33	17.87	0.517	0.51	.517	
11:34	35.75	0.842	0.84	.842	
11:35	53.62	1.205	1.20	1.205	75% x W.L.
11:40	53.62	1.205	1.20	1.205	
11:41	35.75	1.020	1.01	1.020	
11:42	17.87	0.670	0.67	.670	
11:43	-0-	0.342	0.34	.342	
11:44	17.87	0.565	0.57	.565	
11:45	35.75	0.888	0.88	.888	
11:46	53.62	1.222	1.22	1.222	
11:48	71.50	1.581	1.58	1.581	
11:53	71.50	1.585	1.58	1.585	
12.32	71.50	1.585	1.58	1.585	
12:33	35.75	1.081	1.08	1.081	
12:35	17.87	0.708	.71	.708	

NOTE: 1 ton force = 2000 lb f = 8.896 kN

SHEET 2

Time	Load (Tons)	Dial Gauge Reading	Rule Gauge Reading	Anchor Extension	Remarks
12:36	-0-	0.362	.36	.362	
12:37	-0-	0.362	.36	.362	
12:38	17.87	0.581	.58	.581	
12:39	35.75	0.917	0.91	.917	
12:40	53.62	1.263	1.26	1.263	
12:41	71.50	1.587	1.58	1.587	
12:43	89.37	1.921	1.92	1.921	
12:45	107.25	2.342	2.34	2.342	150% x W.L.
12:50	107.25	2.342	2.34	2.342	
12:51	71.50	1.897	1.90	1.897	
12:52	35.75	1.154	1.18	1.154	
12:53	17.87	0.759	0.76	0.759	
12:54	-0-	0.410	0.41	0.410	
12:57	-0-	0.410	0.41	.410	
12:58	17.87	0.623	0.63	.623	
12:59	35.75	0.972	0.97	.972	
13:00	53.62	1.313	1.31	1.313	
13:01	71.50	1.665	1.66	1.665	
13:02	89.37	1.972	1.97	1.972	
13:03	107.25	2.345	2.34	2.345	
13:04	125.12	2.700	2.70	2.700	

SHEET 3

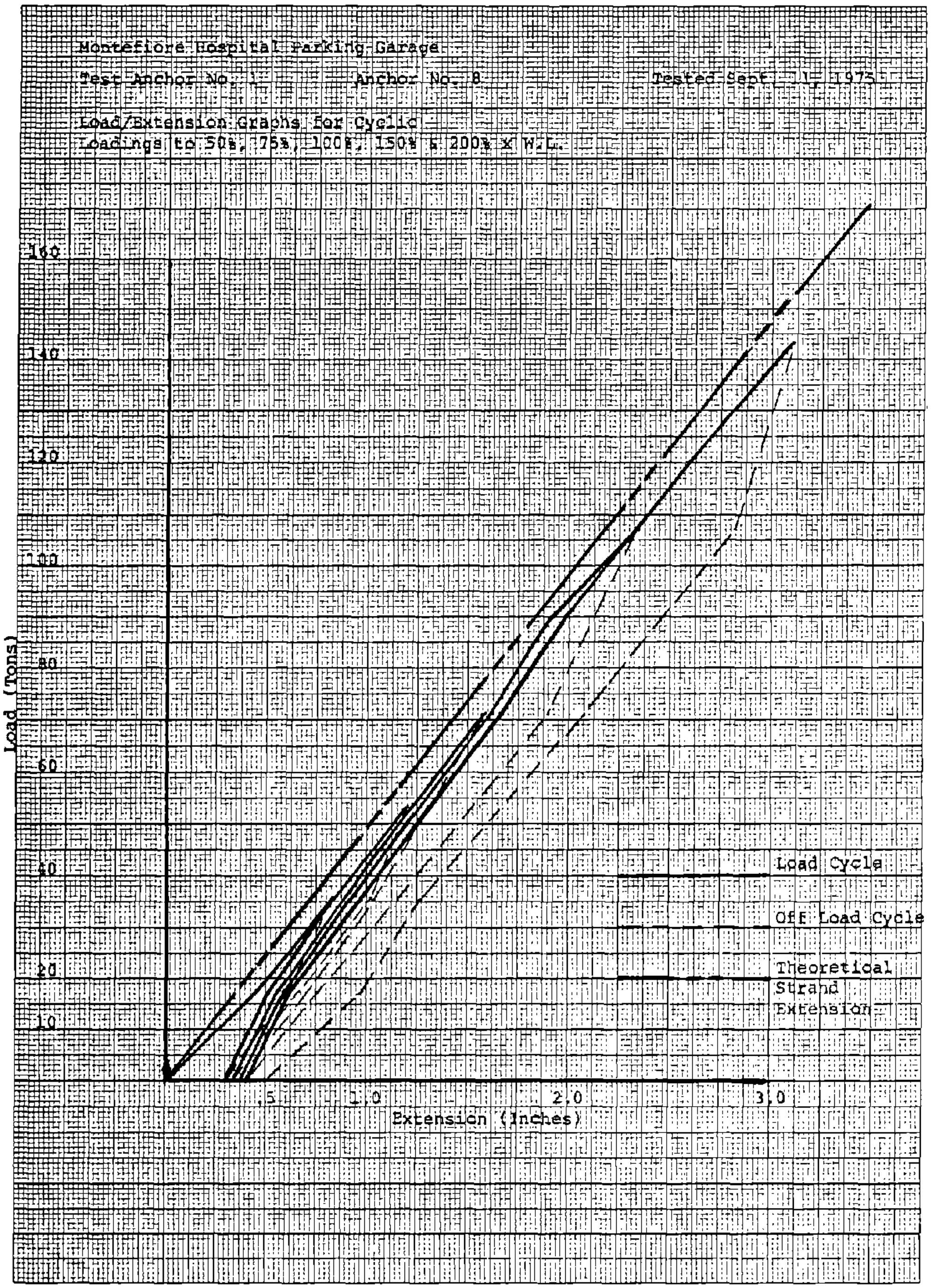
Time	Load (Tons)	Dial Gauge Reading	Rule Gauge Reading	Anchor Extension	Remarks
13:05	143.00	3.110	3.10	3.110	200% x W.L.
13:10	143.00	3.117	3.11	3.117	
13:25	143.00	3.122	3.12	3.122	
13:35	143.00	3.130	3.13	3.130	
13:40	143.00	3.130	3.13	3.130	
13:41	107.25	2.819	2.81	2.819	
13:43	71.50	2.082	2.08	2.082	
13:44	35.75	1.265	1.27	1.265	
13:45	17.87	0.978	0.95	.978	
13:46	-0-	.500	0.50	.500	
13:50	-0-	.500	0.50	.500	

TEST COMPLETED

ANCHOR SUBSEQUENTLY RE-STRESSED AND LOCKED

OFF AT W.L. x 115%, i.e., 82.225 TONS.

SEE SEPARATE STRESSING RECORD SHEET.



NOTE: 1 ton = 8.896 kN
1 in. = 25.4 mm

TEST RESULTS

Test Anchor No.2 Anchor No.85 Tested Nov.5 & 6, 1975

TIME	LOAD (TONS)	DIAL GAUGE 1	DIAL GAUGE 2	ANCHOR EXTENSION	REMARKS
Nov.5,1975					
14:30	-0-	0.00	0.00	-0-	
14:31	25	0.15	0.18	0.165	
14:34	25	0.15	0.18	0.165	
14:35	50	0.28	0.34	0.31	W.L. x 50%
14:40	50	0.28	0.34	0.31	
14:41	25	0.225	0.25	0.237	
14:45	25	0.225	0.25	0.237	
14:46	-0-	0.00	0.00	0.00	
14:50	-0-	0.00	0.00	0.00	
14:51	25	0.15	0.28	0.215	
14:53	25	0.15	0.28	0.215	
14:54	58	0.33	0.39	0.36	
15:00	58	0.33	0.39	0.36	
15:02	75	0.535	0.61	0.572	
15:07	75	0.535	0.61	0.572	
15:09	100	0.806	0.875	0.84	Working Load
15:20	100	0.810	0.875	0.842	
15:30	100	0.810	0.876	0.843	
15:35	100	0.810	0.876	0.843	
15:37	50	0.505	0.546	0.525	
15:39	50	0.505	0.546	0.525	
15:40	25	0.33	0.36	0.345	
15:42	25	0.33	0.36	0.345	
15:45	-0-	0.00	0.00	0.00	
16:00	-0-	0.00	0.00	0.00	
Nov.6, 1975	-0-	0.00	0.00	0.00	
08:51	25	0.141	0.161	0.151	
08:55	25	0.141	0.161	0.151	
08:56	50	0.319	0.342	0.33	W.L. x 50%
09:00	50	0.319	0.342	0.33	
09:01	25	0.227	0.245	0.236	
09:04	25	0.227	0.245	0.236	
09:05	-0-	0.00	0.00	0.00	

TEST RESULTS

Sheet 2

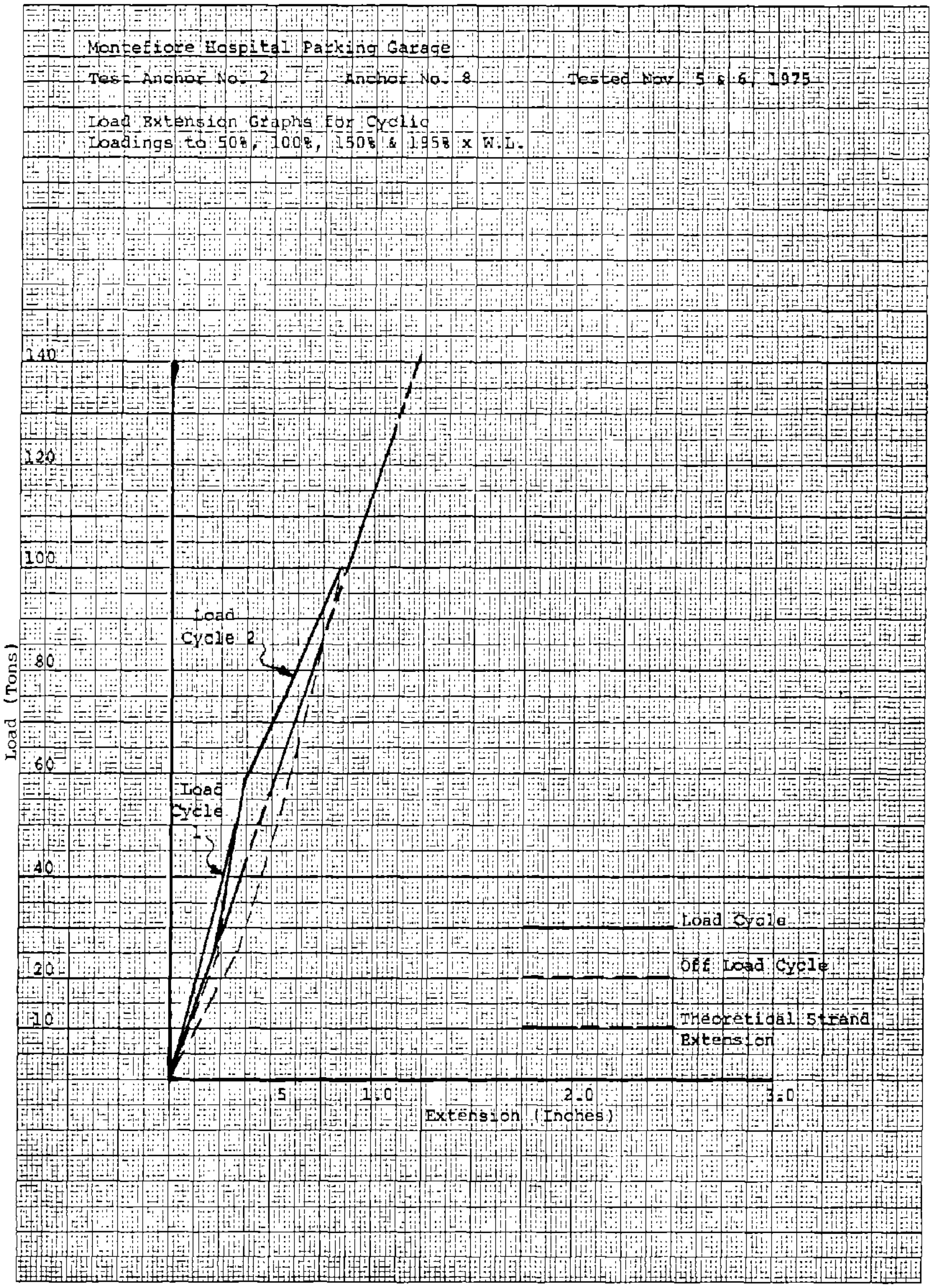
TIME	LOAD (TONS)	DIAL GAUGE 1	DIAL GAUGE 2	ANCHOR EXTENSION	REMARKS
09.06	-0-	0.00	0.00	-0-	
09.08	25	0.154	0.172	0.163	
09.10	25	0.154	0.172	0.163	
09.11	50	0.312	0.336	0.324	
09.15	50	0.312	0.336	0.324	
09.16	75	0.552	0.577	0.564	
09.20	75	0.552	0.577	0.564	
09.21	100	0.774	0.803	0.788	Working Load
09.25	100	0.779	0.813	0.796	
09.30	100	0.777	0.811	0.794	
09.45	100	0.776	0.807	0.791	
10.00	100	0.775	0.805	0.790	
10.35	100	0.780	0.808	0.794	
10.36	50	0.362	0.384	0.373	
10.38	50	0.362	0.384	0.373	
10.39	25	0.267	0.281	0.274	
10.40	25	0.267	0.281	0.274	
10.41	-0-	0.005	0.00	0.002	
10.42	-0-	0.005	0.00	0.002	
10.43	25	0.171	0.182	0.176	
10.45	25	0.171	0.182	0.176	
10.46	50	0.334	0.350	0.342	
10.47	50	0.334	0.350	0.342	
10.48	75	0.554	0.565	0.559	
10.50	75	0.554	0.565	0.559	
10.52	100	0.783	0.800	0.791	
10.55	100	0.783	0.800	0.791	
10.56	125	1.053	1.071	1.062	
11.00	125	1.053	1.072	1.062	
11.05	150	1.238	1.250	1.244	150% x W.L.
11.10	150	1.238	1.250	1.244	
11.15	150	1.238	1.250	1.244	
11.16	100	1.060	1.072	1.066	
11.18	100	1.060	1.072	1.066	
11.19	50	0.511	0.503	0.507	
11.20	50	0.511	0.503	0.507	

TEST RESULTS

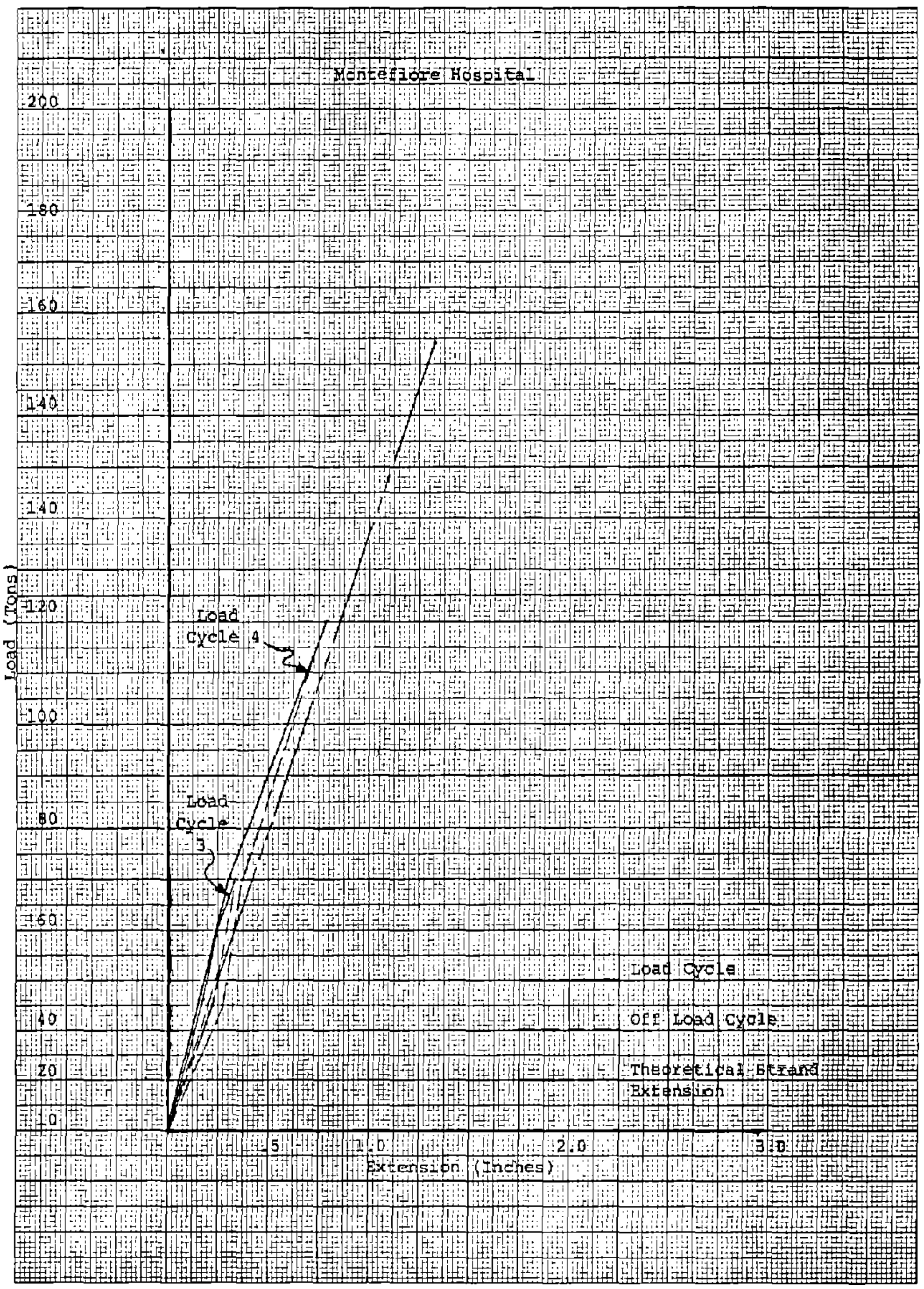
Sheet 3

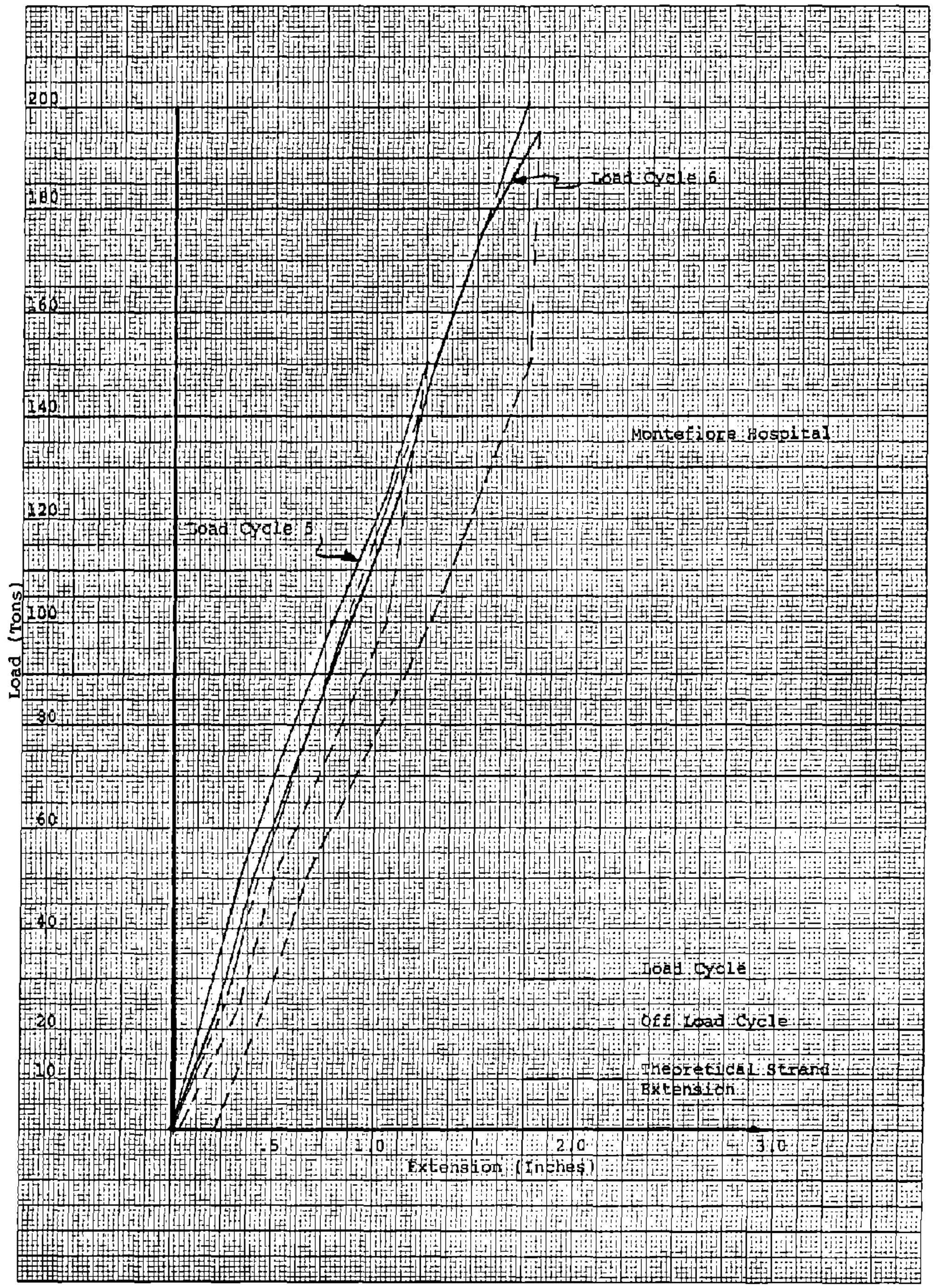
TIME	LOAD (TONS)	DIAL GAUGE 1	DIAL GAUGE 2	ANCHOR EXTENSION	REMARKS
11.21	25	0.35	0.343	0.346	
11.22	25	0.35	0.343	0.346	
11.23	-0-	0.055	0.042	0.048	Slight Residual curvature in stress head plates
11.30	-0-	0.055	0.020	0.037	
12.50	-0-	0.034	0.035	0.0345	
12.51	25	0.222	0.239	0.230	
12.54	25	0.222	0.239	0.230	
12.55	50	0.386	0.405	0.395	
12.58	50	0.386	0.405	0.395	
12.59	75	0.625	0.654	0.639	
13.04	75	0.625	0.654	0.639	
13.05	100	0.864	0.881	0.872	
13.09	100	0.864	0.881	0.872	
13.10	125	1.113	1.135	1.124	
13.14	125	1.113	1.135	1.124	
13.15	150	1.272	1.300	1.286	
13.19	150	1.272	1.300	1.286	
13.20	175	1.511	1.542	1.526	
13.25	175	1.511	1.542	1.526	
13.26	195	1.787	1.825	1.806	195% x W.L.
13.35	195	1.785	1.824	1.804	Load could not be exceeded due to excessive curvature in stress head plate
13.55	193	1.781	1.821	1.801	
14.15	193	1.779	1.819	1.799	
14.30	192	1.774	1.815	1.794	
14.31	150	1.725	1.797	1.761	
14.33	150	1.725	1.797	1.761	
14.34	100	1.283	1.290	1.286	
14.36	100	1.283	1.290	1.286	
14.37	50	0.685	0.664	0.674	
14.40	50	0.685	0.664	0.674	
14.41	25	0.494	0.463	0.478	
14.43	25	0.494	0.463	0.478	
14.44	-0-	0.190	0.188	0.189	*
15.00	-0-	0.185	0.178	0.181	*

*Residual curvature in stress head plates = 0.10" measured from ℄ of anchor to dial gauge tips.



NOTE: 1 ton = 8.896 kN
1 in. = 25.4 mm


Montefiore Hospital
Load (Tons)
200
180
160
140
140
120
100
80
60
40
20
10
Load Cycle 4
Load Cycle 3
Load Cycle
Off Load Cycle
Theoretical Strand Extension
.5
1.0
2.0
3.0
Extension (Inches)



Load (Tons)
200
180
160
140
120
100
80
60
40
20
10
.5
1.0
2.0
3.0
Extension (Inches)
Load Cycle 6
Load Cycle 5
Montefiore Hospital
Load Cycle
Off Load Cycle
Theoretical Strand Extension

NICHOLSON ANCHORAGE COMPANY
P. O. Box 98, Bridgeville, PA 15017

STRESSING DATA SHEET

Date Sept. 11 19 75

Site Montefiore Hospital Parking Garage, Pittsburgh, PA Job No. 4011

Main Contractor: Navarro Corporation

Anchor No. 1,2,3,6 & 7 No. of Strands 5 Test Load 125% x W.L.

Working Load 71-1/2 Tons Tested To: 89-1/2 Tons Locked Off 115% x W.L.

Load Increments 25%, 50%, 75%, 100% 125% x W.L. Free Length 28 ft.

Anchor No.	1st Inct.	2nd Inct.	3rd Inct.	4th Inct.	5th Inct.	Ext. at LockOff	Remarks
1	.54"	1.00"	1.46"	1.94"	2.38"	2.36"	
2	.40"	.77"	1.27"	1.71"	2.33"	2.32"	
3	.32"	.78"	1.23"	1.69"	2.18"	2.16"	
6	.49"	.89"	1.30"	1.70"	2.10"	2.07"	
7	.40"	.83"	1.26"	1.67"	2.10"	2.07"	

at test load: Average Elong. 2.22 Inches

at test load: Anticipated Elong. 2.24" Inches

Strand Nos.

STRESSED BY ____________

MAIN CONT. REP. ____________

NOTE: 1 ton = 8.896 kN
1 in. = 25.4 mm

NICHOLSON ANCHORAGE COMPANY
P. O. Box 98, Bridgeville, PA 15017

STRESSING DATA SHEET

Date Sept. 11 19 75

Site Monteffore Hospital Parking Garage, Pittsburgh, PA Job No. 4011

Main Contractor: Navarro Corporation

Anchor No. 8 No. of Strands 6 Test Load 200% x W.L.

Working Load 71-1/2 Tons Tested To: 143 Tons Locked Off 115% x W.L.

Load Increments 25%, 50%, 75% 100%, 125% x W.L. Free Length 28 ft.

Anchor No.	1st Inct.	2nd Inct.	3rd Inct.	4th Inct.	5th Inct.	Ext. at LockOff	Remarks
8	.23"	.57"	.92"	1.29"	1.50"	1.50"	See Separate Graph for
							Test Results

at working load { Average Elong. -- Inches
Anticipated Elong. 1.30 Inches

STRESSED BY ______

MAIN CONT. REP. ______

Strand Nos.

CASE HISTORY

CONSTRUCTION OF SEA WALL

AT THE MARINE BIOLOGICAL RESEARCH CENTER

FORT PIERCE, FLORIDA

OWNER: HARBOR BRANCH FOUNDATION

ENGINEER: CLARKE AND RAPUANO, INC.
NEW YORK, NY

GENERAL CONTRACTOR: CANDLER - RUSCHE, INC.
WIXOM, MI

In November of 1973, a request for bids was issued for the construction of this 2500 feet (762m) seawall. At that time, the design system of the seawall consisted of steel sheet piling anchored to a concrete deadman 40 feet (12m) to 50 feet (15m) behind the wall by 1-3/4 inches (4.4mm) and 2-1/4 inches (5.7mm) diameter steel tie rods spaced at 9 feet (2.7m) centers. Because of the aggressive marine environmental conditions, the tie rods were to be shop coated with coal tar epoxy and double wrapped with a fiberglass reinforced layered bituminous kraft paper to prevent abrading the coating during backfilling operations. After the rods were placed, excavation in front of the bulkhead and backfilling behind it could proceed.

The engineer allowed an alternate sand anchor system for anchoring the sheet piling, replacing the steel tie rods and concrete deadman. The sand anchorage system was to be designed, tested, and constructed by an approved ground anchor specialist.

Candler - Rusche, Inc. was awarded the general contract for this work and subcontracted Nicholson Anchorage Company* to design, test, and construct the alternate sand anchor system. The system chosen for the construction of this seawall consisted of prestressed, post-tensioned soil anchors, anchored to a W18X70 wale attached to the top of the sheet pile wall. A sketch showing the details of this system is attached to the back of this report

SOIL CONDITIONS

Test holes, located at strategic points on the site, were drilled using drilling mud to prevent caving. At regular intervals and/or strata changes, the drilling tools were removed and the material sampled with a 1.5 inch (3.8mm) I.D., 2 inch (5mm) O.D standard split barrel sampler, driven with a 140 pound (63kg) hammer falling 30 inches (76mm). The standard penetration resistance of the soil was determined by the number of hammer blows required to drive the sampler 1 foot (0.3m). Some of the pertinent soils information is attached to the back of this report.

DESIGN DATA

The required design forces to be restrained by the sand anchors were determined by a simple substitution for the forces provided by the originally designed tie rods. Two types of anchors were used: Type "A" anchors were substituted for the 1-3/4 inch (44mm) diameter tie rods to provide a horizontal force of 4.6 kips (1.09 KN) per foot of seawall, and the 2-1/4 inch (5.7mm) diameter tie rods were replaced with Type "B" anchors, providing 7.5 kips (33.3KN) horizontally per foot of seawall. The anchor spacings were increased to take advantage of the high strength prestressing steel used.

Using the design procedures outlined in Tasks A and B, an angle of inclination of 40° to the horizontal was chosen, and spacings selected were 21 feet (6m) average for Type "A" and 15 feet (4.5m) average for Type "B" anchors. Therefore, for Type "A" anchors the design load was:

$$\frac{4.6\ \text{k/ft}}{\text{Cos } 40°} \times 21" = 126 \text{ kips } (560 \text{ KN}) \text{ or } 63 \text{ tons.}$$

Likewise, for Type "B" anchors, the design load was:

$$\frac{7.5\ \text{k/ft}}{\text{Cos } 40°} \times 15' = 146 \text{ kips } (649 \text{ KN}) \text{ or } 73 \text{ tons.}$$

ANCHOR AND GROUT MATERIALS SELECTION

The anchor tendon system selected for use on this job consisted of tendons fabricated from five (5) 0.6 inch (1.5mm) diameter strands conforming to ASTM A-416 with an ultimate strength of 270 ksi (1862 N/mm^2). The bond length remained bare while in the 30 feet (9m) stressing length the strands were greased and individually sheathed with polyvinylchloride. Because of the corrosive marine environment, a steel pipe was placed over the top 20 feet (6m) of each anchor to provide triple corrosion protection (grout, grease and sheathing, and steel pipe). To provide permanent corrosion protection for the anchor system, the top anchorage portion of each anchor was completely encased in concrete at the completion of the project.

The grout mix selected for use on this project consisted of 5-1/2 gallons (20.71) of potable water per sack of Portland Type II cement. The Type II cement was chosen, because of its sulphate resisting characteristics, to provide additional corrosion protection.

ANCHOR CONSTRUCTION

Installation of the sand anchors began in June of 1974. The installation procedure consisted of drilling holes with a Nicholson Anchorage Company custom-built, crawler-mounted hydraulic rotary drill using tri-cone roller bits, with water as the flushing medium. Drilling and installing an anchor was not initiated until the steel sheet piling had been driven at least 40 feet (12m) in advance of the anchor. Five (5") inch (12.7cm) flush jointed steel casing with drill rods inside the casing was used to drill holes the full length of the anchor. After the casing was flushed clean with water, it was tremied full of neat cement grout through the drill rods. The drill rods were then removed and the anchor tendon inserted. Then, the casing was reconnected to the rotary drill head and the anchor pressure grouting began. As grout pressure increased and casing rotation speed decreased, indicating grout take refusal, the casing was slowly withdrawn. This pressure grouting procedure continued over the full length of the anchor bond zone. The stressing length portion of the anchor was kept full of grout but did not require pressure grouting. After the casing was fully withdrawn and grouting completed, a 20 feet (6m) long section of 3 feet (7.6mm) steel pipe was placed in the upper portion of the hole to provide additional corrosion protection for the anchor.

During the pressure grouting, records were kept of grout take and pressure developed for each anchor. Grout take averaged ten (10) bags of cement per anchor at 100 psi (0.69N/mm^2) grouting pressure.

In all, 107 Type "B" and 51 Type "A" anchors were installed using these construction techniques on this project.

STRESSING AND TESTING PROCEDURES

Nicholson Anchorage Company proposed and the engineer approved the following testing and stressing procedures:

1. Test the first two Type "A" and the first two Type "B" anchors installed to 150% of design load.

2. Test load shall be applied in 5 increments and held at 150% of design load for 1/2 hour minimum, noting any anchor movement with respect to the sheet pile wall at each increment.

3. After acceptance of the test, the anchor shall be locked off at 115% of design load.

4. After the initial test anchors, 10% of the remaining anchors will be tested as noted above.

5. All other production anchors will be stressed in 3 increments to 115% of design load and locked off. Records of anchor elongation shall be kept at each increment.

Equipment used for testing and stressing the anchors consisted of a 30 ton (27 tonnes) single strand jack. Each of the 5 strands of the anchor tendon was stressed individually to the desired load. The 15% overstress applied to each anchor was to compensate for prestress losses. (Wedge seating, creep, steel relaxation, etc.)

Sixteen (16) test anchors wre installed and stressed to either 110 tons (99.7tonnes) for Type "B" or 94 tons (85tonnes) for Type "A" anchors. Of the 16 anchors tested, only the initial test of a Type "B" tendon failed to hold the desired load for this test period. This was because backfilling behind the sheeting was not yet complete and excessive movement of sheeting was noted. The test was stopped to 100 tons (90.7tonnes) instead of 110 tons (99.7tonnes), but the testing procedure was modified to hold the 110 ton (99.7tonnes) load overnight. At the conclusion of this test, no anchor movement had occured and the test was deemed acceptable. In all other test and production anchor stressing, desired loads were obtained and held. Results from several of the tests are included at the end of the report.

Production anchor stressing was performed in 2 stages. In the first stage, anchors were locked off at 25 tons (22.7tonnes) and after backfilling behind the wall was completed, final stressing and lock off at 115% of design load was performed.

Stressing data was recorded for each anchor on forms as shown on Page 92. A review of the data obtained from each anchor showed that each behaved elastically during stressing and the proper steel elongation was obtained.

After completion of the backfilling and stressing operations, it was necessary to adjust the load on some of the anchors to move the sheet pile wall back into alignment before pouring the concrete capping beam, covering the top anchorage assembly and wale.

Installation and stressing of the 158 anchors was completed in September of 1974. Since that time the seawall has been in use and has performed satisfactorily.

NICHOLSON ANCHORAGE COMPANY
P. O. Box 98, Bridgeville, PA 15017

STRESSING DATA SHEET

Date __________ 19___

Site ____ Job No. ______

Main Contractor: ____

Anchor No. ____________ No. of Strands ________ Test Load __________

Working Load __________ Tested To: ____________ Locked Off __________

Load Increments _________________ Free Length ____________________

Strand	Strand at Zero	1st Inct.	2nd Inct.	3rd Inct.	4th Inct.	Total Elong.	Remarks
1							
2							
3							
4							
5							
6							
7							
8							
9							
10							
11							
12							

Average Elong. ___________ Inches

Anticipated Elong. ________ Inches

STRESSED BY ____________________

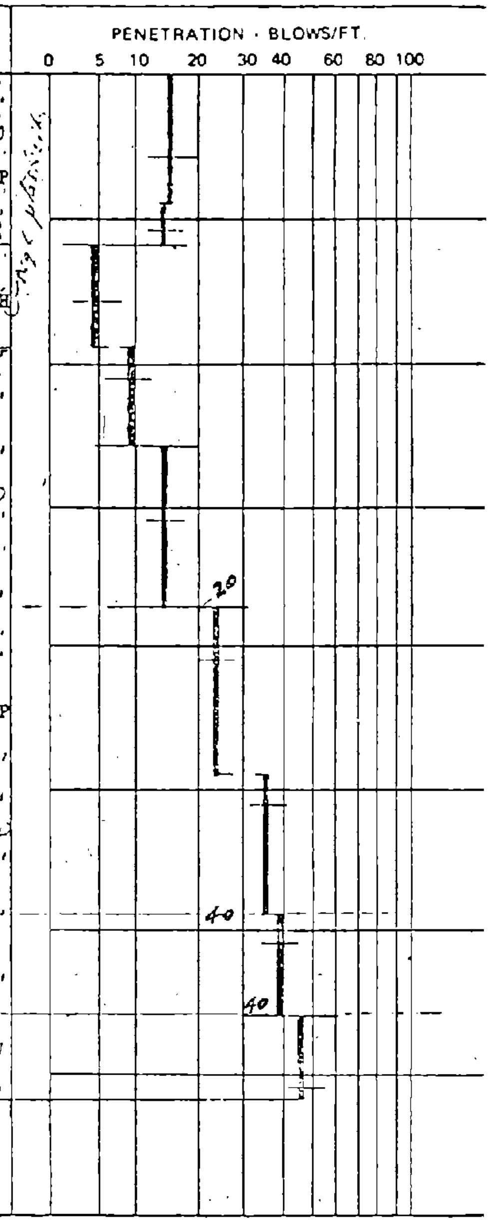
MAIN CONT. REP. ________________

Strand Nos.

SOIL BORING LOG

TEST HOLE 7-1 /-01   JOB ORDER NO. 3587-A-1

DEPTH (Ft.)	ELEV. (Ft.)	SOIL DESCRIPTION	
0 – 6.0		MED., GRAY SHELL-SAND MIX. W/COQUINA	
		FIRM, MEDIUM, GRAY SHELL-SAND MIXTURE WITH COQUINA	GP
6.0		GWT 7-30-73	
6.0 – 9.5		LOOSE, MEDIUM, GRAY SILT WITH SHELL-SAND MIXTURE AND ROOTS	MH
9.5 – 13.0		LOOSE, MEDIUM, GRAY SHELL-SAND MIXTURE WITH SILT	
13.0 – 18.5		FIRM, MEDIUM, GRAY SHELL-SAND MIXTURE	
18.5 – 25.0		WITH COQUINA	GP
25.0 – 29.5		COMPACT, FINE GRAY SHELL-SAND MIXTURE WITH COQUINA	
29.5 – 33.0			
33.0 – 36.0		VERY COMPACT, FINE SHELL-SAND MIXTURE	
36.0		BORING TERMINATED @ 36'	



Remarks:

Penetration number of blows required of 140 lb. hammer falling 30 in. to drive 2 in. O. D. split spoon sampler one foot.

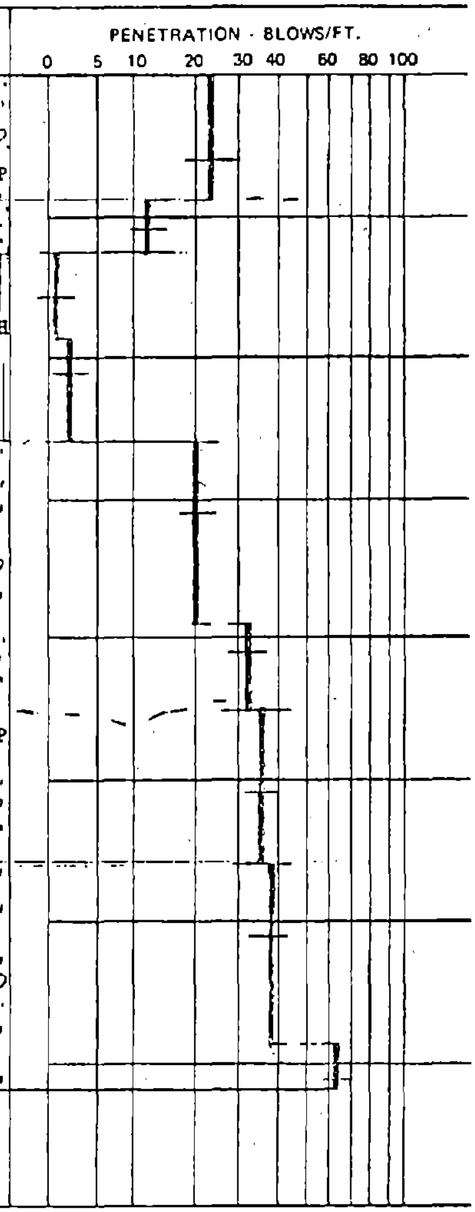
Page 3

PENINSULA ENGINEERING & TESTING COMPANY

SOIL BORING LOG

TEST HOLE /-#2 JOB ORDER NO. 3587-A-1

Depth (ft)	ELEV. (Ft.)	SOIL DESCRIPTION		PENETRATION - BLOWS/FT.
0 – 2.5		MEDIUM, GRAY SHELL-SAND MIXTURE WITH COQUINA		
2.5 – 6.5	4.5	FIRM, MEDIUM, GRAY SHELL-SAND MIXTURE WITH COQUINA AND TRACES OF BROWN CLAY	GP	
	6.0	~~0.0 6.0~~ GWT 7-30-73		
6.5 – 13.0	6.5	SOFT, GRAY SILT WITH SHELL-SAND MIXTURE -6	MH	
13.0 – 19.5	13.0	FIRM, MEDIUM, GRAY SHELL-SAND MIXTURE -14		
19.5 – 27.5		COMPACT, FINE, GRAY SHELL-SAND MIXTURE		
		WITH COQUINA PEBBLES	GP	
27.5 – 34.5	28.0	COMPACT, FINE GRAY SHELL-SAND MIXTURE		
34.5 – 36.0	34	VERY COMPACT, FINE, GRAY SHELL-SAND MIXTURE WITH COQUINA		
		BORING TERMINATED @ 36'		



Remarks:

Penetration number of blows required of 140 lb. hammer falling 30 in. to drive 2 in. O. D. split spoon sampler one foot.

Page 4

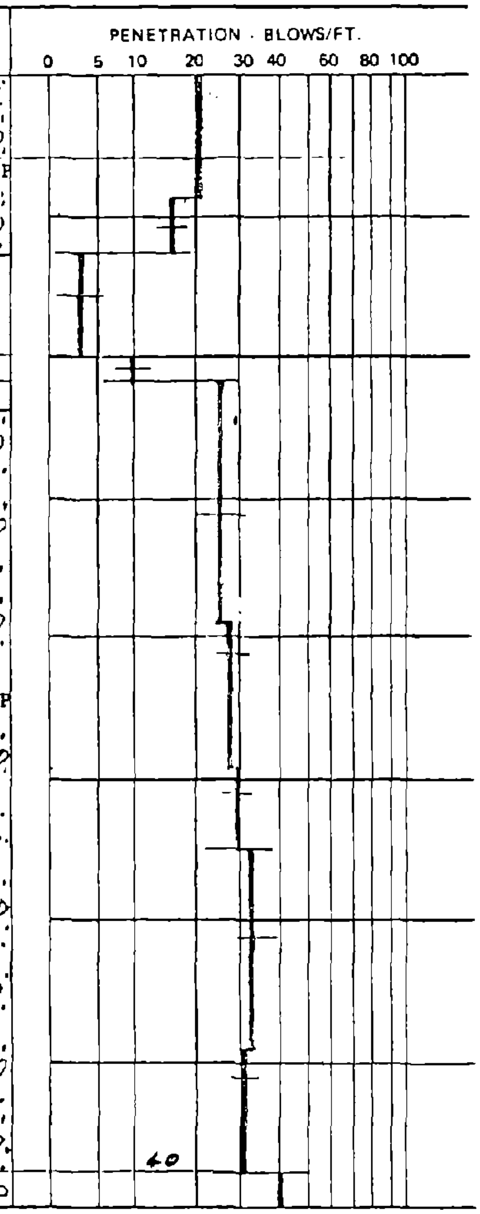
PENINSULA ENGINEERING & TESTING COMPANY

SOIL BORING LOG

TEST HOLE /- #3 JOB ORDER NO. 3587-A-1

Depth (ft)	ELEV. (Ft.)	SOIL DESCRIPTION	
0		MEDIUM, GRAY SHELL-SAND MIXTURE	
2.5 – 3		FIRM, MEDIUM, GRAY SHELL-SAND MIXTURE	GP
6.5	6.5	GWT 7-30-73	
		SOFT, GRAY SILT WITH FINE ROOTS AND SHELL-SAND MIXTURE	MH
10.0	10.0	LOOSE, MED. GRAY SHELL-SAND MIX. WITH SILT	GP
11		COQUINA	
12		LOOSE, MEDIUM, GRAY SHELL-SAND MIXTURE WITH SILT	
15.0		FIRM, MEDIUM, GRAY SHELL-SAND MIXTURE	
		WITH COQUINA PEBBLES	GP
25.0		FIRM, MEDIUM, GRAY SHELL-SAND MIXTURE	
27.5		COMPACT, MEDIUM, GRAY SHELL-SAND MIXTURE WITH COQUINA PEBBLES	
		WITH COQUINA PEBBLES	
39	39	COMPACT, FINE GRAY SHELL-SAND MIX.	

PENETRATION - BLOWS/FT.



Remarks:

Penetration number of blows required of 140 lb. hammer falling 30 in. to drive 2 in. O. D. split spoon sampler one foot.

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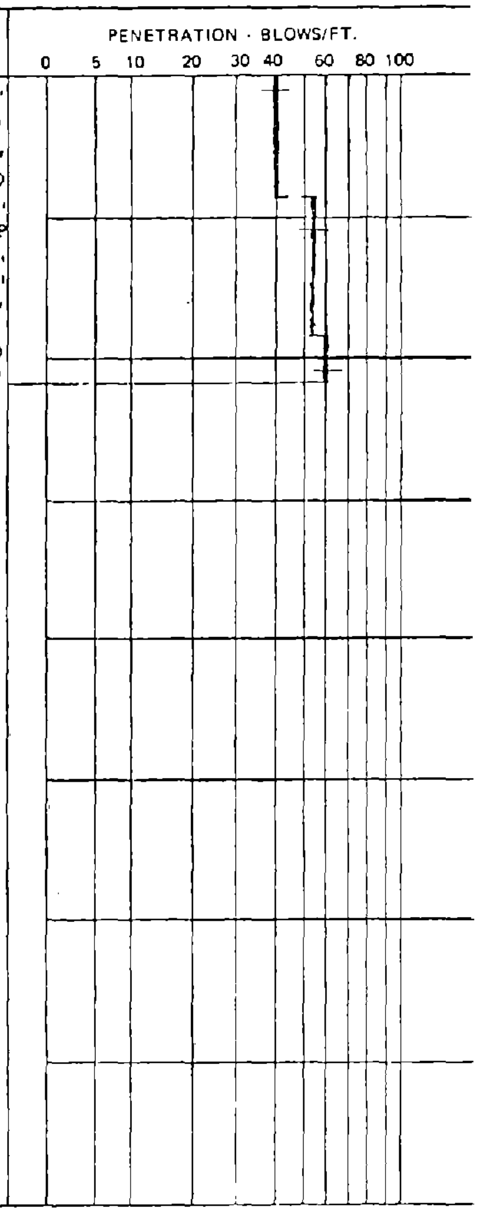
PENINSULA ENGINEERING & TESTING COMPANY

SOIL BORING LOG

TEST HOLE /-Ø3 Con't JOB ORDER NO. 3587-A-1

Depth	ELEV. (Ft.)	SOIL DESCRIPTION	
40.0 – .5		COMPACT, FINE GRAY SHELL-SAND MIXTURE	
45.0 – 47.5		VERY COMPACT, FINE, GRAY SHELL-SAND MIXTURE	GP
50.0		VERY COMPACT, FINE, GRAY SHELL-SAND MIXTURE WITH COQUINA PEBBLES	
52.5		BORING TERMINATED @ 51'	



Remarks:

Penetration number of blows required of 140 lb. hammer falling 30 in. to drive 2 in. O. D. split spoon sampler one foot.

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PENINSULA ENGINEERING & TESTING COMPANY

NICHOLSON ANCHORAGE COMPANY - P.O. BOX 308, BRIDGEVILLE, PA. 15017

TEST

STRESSING DATA SHEET

DATE July 30 1974

Site: Lindport - FT. PIERCE - Fla. Job.# 3126

Main Contractor: Cardlee - Rusche

Anchor No. 96 No. of Strands 5 Test Load 110 Tons

Working Load Tested To: Locked Off 84 Tons

Load Increments Free Length

		25 Tons	50 Tons	75 Tons	100 Tons	110 Tons	
Strand	Strand at Zero	1st. Inct.	2nd. Inct.	3rd. Inct.	4th. Inct.	~~Total~~ 5th ~~Elong~~ Inct	Total Elong Remarks
1	—	14 5/8	16 1/16	17 3/8	18	18 5/8	4"
2	—	14 3/4	16 5/16	17 1/2	18 1/4	18 3/4	4"
3	—	14 13/16	16 3/16	17 3/8	18 5/16	18 5/8	3 13/16"
4	—	15	16 1/2	17 3/4	18 9/16	19 1/16	4 1/16"
5							
6	AVG	14.797	16.2656	17.5	18.281	18.7656	4"
7							
8	wale	–	5/8	9/16	3/16	3/16	-1 9/16
9							
10							
11							
12							

Average Elong. ______ Inches

Anticipated Elong. ______ Inches

STRESSED BY Bill [signature]

MAIN CONT. REP. ______

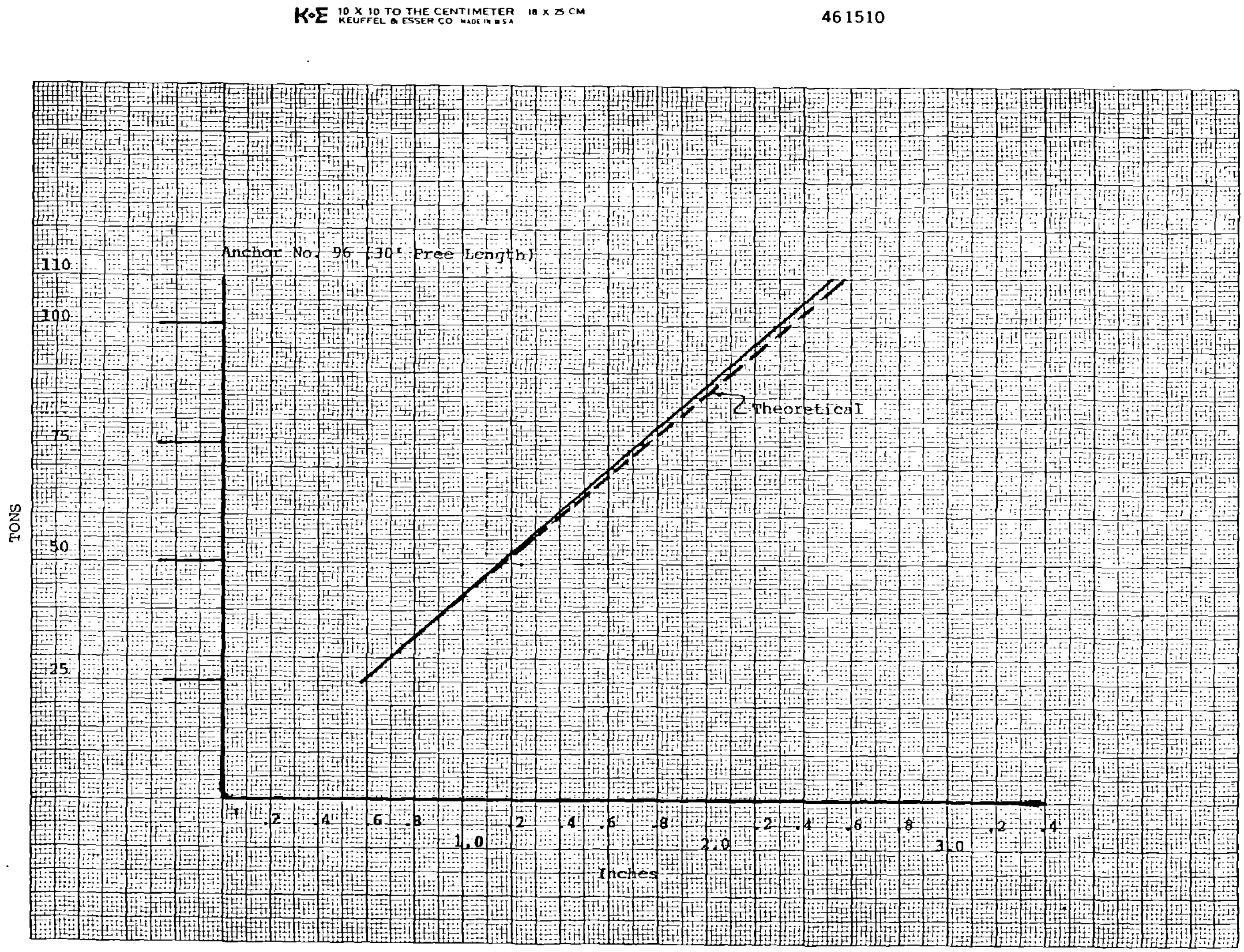
STRAND NOS.

1 3

2 4

Sheet pile move $1\frac{9}{16}$"

NOTE: 1 ton = 8.896 kN
1 in. = 25.4 mm



NOTE: 1 ton = 8.896 kN
1 in. = 25.4 mm

NICHOLSON ANCHORAGE COMPANY - P.O. BOX 308, BRIDGEVILLE, PA. 15017

TEST
STRESSING DATA SHEET

DATE July 29 1974

Site: Linkport - Ft. Pierce Fla. Job. # 3126

Main Contractor: Cardllee Rusche

Anchor No. 105 No. of Strands 5 Test Load

Working Load Tested To: 110 TONS Locked Off

Load Increments Free Length

Strand	Strand at Zero	1st. Inct. 25 TONS	2nd. Inct. 50 TONS	3rd. Inct. 75 TONS	4th. Inct. 100 TONS	~~Total Elong.~~ 5th Inct. 110 TONS	TOTAL ELONG. Remarks
1	—	14 5/8	15 13/16	17 1/8	18 1/4	18 7/8	4 1/4
2	—	14 3/4	16	17 5/16	18 3/8	19	4 1/4
3	—	14 13/16	16 1/8	17 3/16	18 7/16	18 7/8	4 1/16
4	—	15	16 9/16	17 5/8	18 3/4	19 1/16	4 1/16
5							
6	AVG.	14.797	16.0625	17.3125	18.4531	18.9531	4 1/8
7							
8	wale	-	3/8	3/8	3/8	1/8	-1 1/4
9							
10							
11							
12							

Average Elong.__________ Inches

Anticipated Elong.__________ Inches

STRESSED BY Bill Nugra

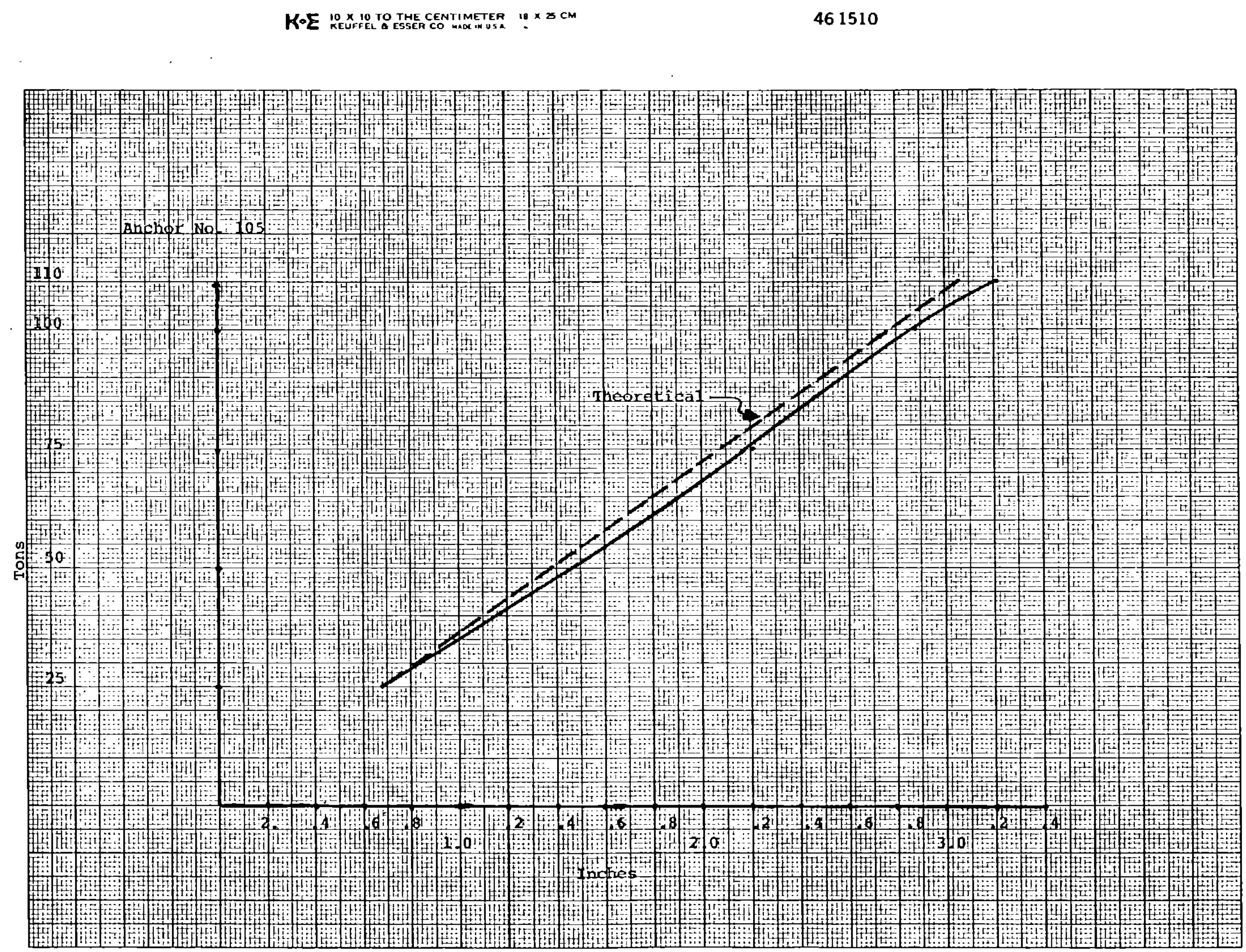
MAIN CONT. REP.

STRAND NOS.

1	3
2	4

Sheets pulled 1 1/4"

NOTE: 1 ton = 8.896 kN
1 in. = 25.4 mm



NOTE: 1 ton = 8.896 kN
1 in. = 25.4 mm

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