PERMANENT GROUND ANCHORS

Stump Design Criteria

U.S. Department
of Transportation

**Federal Highway
Administration**

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Washington, D.C. 20590

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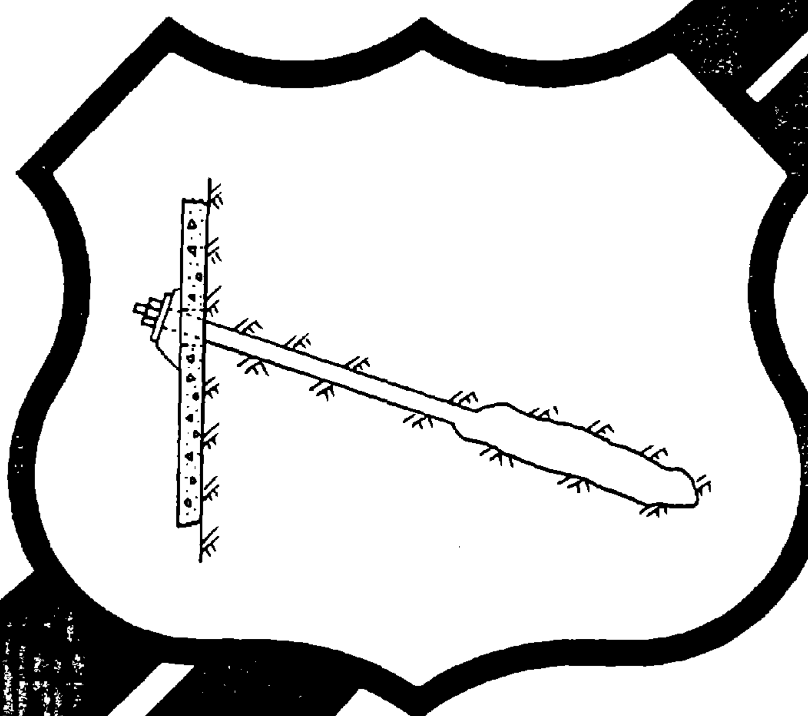

FOREWORD

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
Research and Development
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NOTICE

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16. Abstract

This document summarizes the main design methods used by the principal investigatiors in the design of permanent ground anchors, including basic concepts, design criteria, and analytical techniques. The application of these design methods are illustrated in the step-by-step design of a permanent ground anchor system for an example problem.

Companion reports on the design of permanent ground anchors are:
FHWA/RD-81/150 Soletanche Design Criteria
FHWA/RD-81/151 Nicholson Design Criteria

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CONVERSION OF SI UNITS TO ENGLISH

Metric and SI units are used throughout this report. English equivalents are shown below:

Metric		SI		English
Length				
1 mm	=	1 mm	=	0.039 in.
1 cm	=	1 cm	=	0.39 in.
1 m	=	1 m	=	39.37 in.
1 m	=	1 m	=	3.28 ft.
Force				
1 kg	=	9.8066 N	=	2.205 lb.
1 tonne	=	9.8066 kN	=	2205 lb.
Pressure				
1000 kg/m^2	=	9.8 kPa	=	204.82 psf
---		1 Pa	=	0.02088 psf
---		1 Pa	=	0.000145 psi
Unit Weight (γ)				
1 t/m^3	=	9.8 kN/m^3	=	62.5 pcf
Subgrade Modulus				
1 $t/m^2/m$	=	9.8 kPa/m	=	62.5 psf/ft.
		1 Pa/m	=	0.00637 psf/ft.

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

LIST OF FIGURES

LIST OF FIGURES (Cont.)

LIST OF FIGURES (Cont.)

LIST OF TABLES

LIST OF SYMBOLS

C'_R	Theoretical value of cohesion corresponding to ϕ'_R.
C_{UR}	Theoretical value of shear strength from undrained tests in saturated cohesive soils.
d	Thickness of a soil layer.
e_a	Active horizontal earth pressure.
e_h	Earth pressure magnitude due to horizontal backfill.
$e_{h\beta}$	Earth pressure magnitude due to backfill inclined at angle β.
$e_{ah,q}$	Active earth pressure due to a surface load, q.
$e_{oh,q}$	At rest pressure due to a surface load, q.
Δ_{eh}, $\Delta e_{h\beta}$	Reduction in active and increased active earth pressures due to cohesion.
e_{rh}	Earth pressure on the right hand side of wall.
e_{lh}	Earth pressure on the left hand side of wall.
E	Coefficient of compressibility.
E_e	Modulus of elasticity of tendon.
$\Delta E_{ah,q}$	Total active force due to a surface load, q.
F	Factor of safety of the anchored structure against slippage - ratio of the shear strength of the ground to the shear stress.
F_e	Cross sectional area of the tendon.
g	Acceleration due to gravity ($\sim$ 10 ms^{-2}).
i	Hydraulic gradient.
k	Permeability of the soil.
K_a	Coefficient of active earth pressure.
K_{ah}	Coefficient of active earth pressure for horizontal ground surface.
$K_{ah,\beta}$	Coefficient of active earth pressure for a ground surface sloping at an angle, β.
K_{eh}	Coefficient for an increased active earth pressure and horizontal backfill.
K_{oh}	Coefficient of earth pressure at rest due to a horizontal backfill.
K_{ph}	Coefficient of passive earth pressure for a horizontal backfill.

l	Anchor length - distance between the anchor head and the anchor foot.
l_f	Free anchor-length.
l_{fr}	Calculated free anchor length.
l_v	Fixed anchor length.
Δl	Movement of the anchor head with respect to a fixed point.
Δl_{bl}	Plastic deformation of the anchor.
Δl_e	Elastic elongation of the anchor.
Δl_r	Calculated elastic elongation of the tendon.
Δl_k	Movement of the anchor plate in an axial direction (piston stroke).
p	Uniformly distributed live surface load.
q	Pressure applied at the ground surface.
R	Frictional force.
S	Factor of safety of an anchor - ratio of the load bearing capacity of the anchor to the working load.
ΔS	Movement of the anchor plate.
t_o	Theoretical depth of embedment of sheet pile.
Δt	Observation time for a load increment during a test.
V	Anchor force
V_A	Initial anchor load for a test.
V_G	Working load of the anchor.
V_O	Stressing load of the anchor.
V_P	Test load on an anchor.
V_S	Load on an anchor when the stress in the tendon is $\sigma_{2.0}$ (nominal value of the yield stress).
V_U	Load bearing capacity of the anchor.
V_V	Limiting load of the fixed anchor
V_Z	Tendon failure load.

w	Seepage force.
w_H	Water pressure due to ground water on the back of wall.
w_V	Counter water pressure on the front of wall.
X_A	Distance from the wall to the inside boundary of a strip load.
X_E	Distance from the wall to the outside boundary of a strip load.
Z	A given depth.
Z_{GW}	Depth to ground water.
Z_o	Depth at which the pressure on wall reduces to zero.
Z_q	Depth of effect of a strip load on the wall.
Z_S	Depth to ground surface in front of wall.
Z_W	Depth to free water surface.
α_1	Angle of inclination of unloading curve.
α_2	Angle of inclination of loading curve.
β	Angle of the backfill surface with respect to the horizontal.
β_Z	Nominal value of tensile strength of tendon.
γ	Unit weight of the soil = ρg.
γ_w	Unit weight of water.
δ	Angle of wall friction.
η	Factor of safety.
ρ	Bulk density of moist earth.
ν	Factor of safety against piping.
σ_G	Working stress in the tendon.
σ_O	Stress in the tendon at time equal to zero.
σ_P	Test stress in the tendon.
$\sigma_{2.0}$	Nominal value of the yield stress (0.2% permanent elongation)
ϕ'_R	Theoretical value of the angle of internal friction in cohesive and non-cohesive soils.

INTRODUCTION

Ground anchors may be used advantageously in any situation where massive weight is required to maintain stability of a structure and may be replaced in an efficient manner by tensile forces transmitted to the ground. Typical applications include retaining walls, uplift or horizontally loaded structures. Tensile stresses that occur in tent type (folded) roof structures can be taken by permanent anchors. The use of anchors has important advantages not only with regard to the replaced structural weight but, because external bracing is eliminated, they allow the unrestricted operation of large excavation and construction machines resulting in more rapid construction. They also provide new techniques, such as in the improvement of slopes.

Although ground anchors are a fairly recent development they are an essential structural element in the overall stability of the wall-anchor-soil system. The interrelated load-deformational behaviour of the wall, anchor, and soil components of this system is very complicated and to date no exact mathematical analysis has been devised. In addition, there are problems of long term behaviour of the grouted zone and of corrosion. Since failure of the wall-anchor-soil system can result in serious damage and threat to human lives, instrumented monitoring of such systems is strongly encouraged.

Standards and Codes of Practice, based on experience and long term observations of permanent anchor installations,have been developed in various countries, and specify design, construction, and monitoring procedures. The historical development of permanent anchors relates mainly to protection against corrosion, transmission of anchor forces into the ground, the form of the tension members and grouting methods.

This report summarizes the design methods and analyses used by Stump Bohr, AG in the design of permanent ground anchor systems. The application of the methods has been illustrated in an example problem.

SOIL CONDITIONS

Information needed for design of a permanent ground anchor system includes:

- Description and specifications from the construction project
- Information about the geotechnical properties of the soil
- Information about all other boundary conditions and requirements which the anchorage must fulfill.

The scope of the investigations, planning and design work depends on the type and magnitude of the project and the degree of risk connected with it.

The feasibility of an economical and reliable design for ground anchors depends on existing soil properties. Preliminary soils investigations involve location of the soil strata and their properties, as well as the ground water conditions. This refers not only to the region of the wall excavation, but also to the region where the anchors will be placed and grouted. In addition to the Standard Penetration Test, in cohesive or silty soils, laboratory tests on undisturbed samples are recommended to determine the angle of internal friction, cohesion, moisture content, Atterberg limits, deformation modulus and grain size distribution.

Depth and Spacing of Borings

In the investigation of foundation soil conditions, it can be assumed that in most cases, soils of more uniform bearing value will be encountered deeper down rather than in the upper layers. In general, the deeper layers supply the supporting forces for the structure; the upper layers produce active pressures. The evaluation of passive pressures requires fewer borings than the elevation of active pressures. Figure 1 will serve as a guide for boring locations.

Main borings, along the alignment of the proposed wall, are drilled to a depth equal to twice the difference in the elevations of the ground surfaces or until they encounter a known geological stratum. Spacing of borings along the proposed alignment is 50 m.

Intermediate borings of first order are drilled, after the results of the main borings are known, to twice the difference in the elevations of the ground surfaces or to a depth at which the known uniform soil layer, identified by the main borings, is encountered. Spacing of borings is 50 m.

Intermediate borings of second order are drilled only when there is a considerable change in the upper layers. Normally they are located at 50 m spacing as shown, but the spacing should be reduced if the subsoil conditions require it. The boring depth depends on the results of the preceding borings.

Soil Properties for Preliminary Design (see Table 1)

In the absence of other information, loose deposits are to be assumed for undisturbed sandy soils. Except in older geological stratifications, medium dense compaction is to be expected only after compaction by vibration or tamping. The values for gravelly sand are the same as for sand. The density given for coarse gravel is a rough average value. The actual density depends on the type of rock.

The angle of internal friction ϕ'_R and the cohesion C'_R for cohesive soils are rough average values for calculating the final stability (consolidated state = final strength). If soft to stiff clay and silty clay layers of considerable depth will receive a surcharge such as backfill, structures, etc., the influence of pore pressure is to be considered in the determination

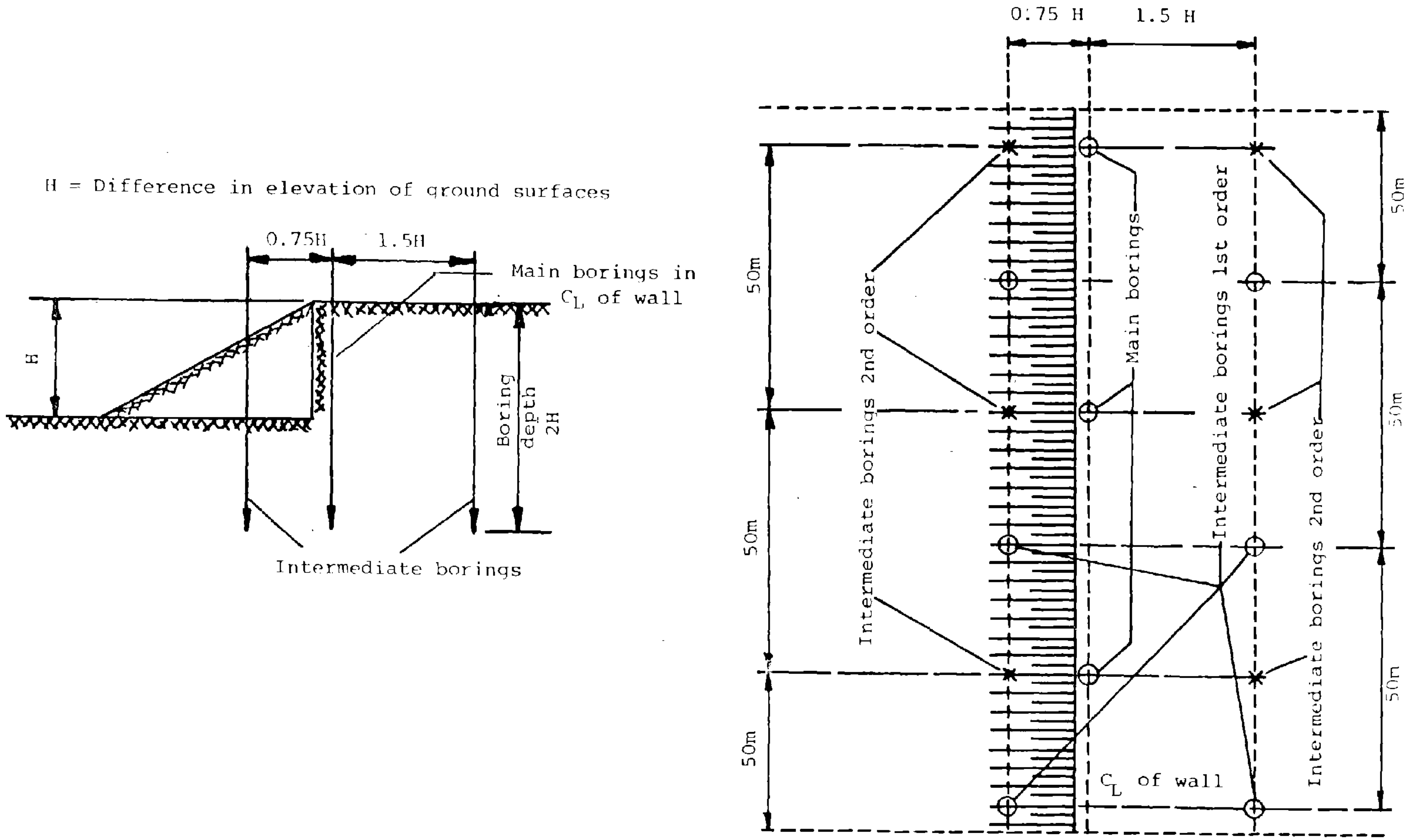


Figure 1. Layout of borings.

of the active earth pressure (initial strength). In some cases, the initial strength may also be considered in the determination of the passive earth pressure.

Soil Properties for Final Design

The final design should, of course, be prepared using the soil properties determined by actual laboratory tests. With non-cohesive soils, it is recommended that the density be determined by field tests.

The utilization of laboratory testing should be waived only for unimportant structures. The use of average soil values given in Table 1 can generally not be recommended for cohesive soils.

The cohesion of cohesive soil may be included in the calculation as long as the soil in-situ is undisturbed and does not become soft and pulpy when kneaded. The wall, however, must be able to withstand the smallest permissible design earth pressure corresponding to the value K_a = 0.15 - 0.20.

For permanent rock anchors or in the case of anchors whose fixed anchor part extends into rock the following conditions must be clarified beforehand:

a) the external form of the rock mass and the depth of overburden.

b) the rock geology and the primary state of stress

c) the jointing system (position, dip and strike, material inclusions etc.), particular attention must be paid to large joints parallel to the slope.

d) the stratification with its host of fissures and joints, the extent of jointing (i.e. if there is complete interconnection) etc.

e) strength properties of the rock mass and of individual rock specimens

f) the presence of water in the joints, its quality, quantity and the pressure conditions.

Rock Properties

In the determination of the rock properties one must proceed by considering existing interfaces between strata. Basically one must distinguish among three kinds of failure mechanisms:

a) slip surfaces running continuously through material existing between beds of rock (such material may be the result of filling and/or weathering)

b) slip surfaces running parallel to discontinuous joints

c) stepwise slip surfaces partly through filled material and partly along joints.

Table 1. Estimated values for preliminary calculations.

Type of Soil	Density: Above water γ	Density: Sub-merged γ'	Long Term Strength: Angle of Internal Friction ϕ'_R	Long Term Strength: Cohesion c'_R	Immediate Strength: Shear Strength c_{uR}	Coefficient of Compressibility E
	kN/m^3	kN/m^3	degrees	kN/m^2	kN/m^2	MN/m^2
Non-Cohesive Soils						
Sand, loose, round	18	10	30	-	-	20 - 50
Sand, loose, angular	18	10	32.5	-	-	40 - 80
Sand, medium dense, round	19	11	32.5	-	-	50 - 100
Sand, medium dense, angular	19	11	35	-	-	80 - 150
Gravel without sand	16	10	37.5	-	-	100 - 200
Coarse gravel, sharp edged	18	11	40	-	-	150 - 300
Cohesive Soils	(Empirical Values for Undisturbed Samples from the North German Area)					
Clay, semi-firm	19	9	25	25	50 - 100	5 - 10
Clay, difficult to knead, stiff	18	8	20	20	25 - 50	2.5 - 5
Clay, easy to knead soft	17	7	17.5	10	10 - 25	1 - 2.5
Boulder clay, solid	22	12	30	25	200 - 700	30 - 100
Loam, semi-firm	21	11	27.5	10	50 - 100	5 - 20
Loam, soft	19	9	27.5	-	10 - 25	4 - 8
Silt	18	8	27.5	-	10 - 50	3 - 10
Soft, org. slightly clayey sea silt	17	7	20	10	10 - 25	2 - 5
Soft, very org. strongly clayey sea silt	14	4	15	15	10 - 20	0.5 - 3
Peat	11	1	15	5	-	0.4 - 1
Peat under moderate initial loading	13	3	15	10	-	0.8 - 2

Note:

$1\ kN/m^2 = 20.870\ lb/ft^2$
$1\ MN/m^2 = 20.870\ lb/ft^2$
$1\ kN/m^3 = 6.363\ lb/ft^3$

The estimation of rock pressure on the assumption of a stepwise failure surface can be dispensed with, as computationally a smaller total load is obtained than when a continuous failure surface is assumed. If the distance between joints is small with a highly continuous joint system such that in comparison to the sliding body the dimensions of the jointed elements are small, it may be necessary to perform a stability analysis which considers the material as a soil in order to estimate the rock pressure. The shear strength of the material is then estimated according to the following section.

For a failure surface running completely in filled material between layers, generally the shear strength of the filled material must be taken. When possible, samples are extracted from the filled material and corresponding soil mechanics investigations are performed. If there is insufficient material in the samples to carry out strength tests then at least the grain size distribution should be determined. The angle of internal friction may then be estimated from empirical relationships between ϕ' and material classification as follows:

$\phi' = 30°$ for sandy filling material
$\phi' = 20°$ for silty-sand filling material
$\phi' = 10°$ for clayey-silt filling material

As a rule, in this case, the additional component of cohesive strength may be neglected. Further, for filled material composed of pure clay or smeary silty-clay mixtures or with unconsolidated material under high pore pressures it may be necessary to assume that $\phi' = 0$. Only if it can safely be ascertained that there is no soft material filling the joints (and also that such material cannot be formed during the construction time) is it allowable to estimate the rock pressure on the basis of the residual shear strength of the fissured rock mass.

Chemistry of the Ground Water

In unfamiliar regions it is very important to determine the chemical properties of the ground water and its effects, especially with respect to cement attack. The chemical analysis of water of natural origin comprises the following tests

a) pH-value
b) smell
c) potassium permanganate used in mg $KMnO_4/l$
d) total hardness in mval/l or °d
e) carbonate hardness in mval/l or °d
f) non-carbonate hardness in mval/l or °d
g) magnesium in mg Mg^{2+}/l
h) ammonium in mg NH_4^+/l
i) sulphate in mg SO_4^{2-}/l

j) chloride in mg Cl^-/1
k) lime - dissolving carbonic acid in mg CO_2/1 (determined using Heyer's marble test)

The aggressivity of the ground water is judged according to the limits given in Table 2. The limits apply to standing and weakly flowing water of plentiful supply, which attacks immediately and for which the attacking effect is not diminished by the reaction with the concrete. To assess the water, the highest degree of aggressiveness given in the table is used even if it is obtained for only one of the values in the rows 1 to 5.

Table 2. Limits for assessing the aggressiveness of ground waters.

	Test	Aggressivity		
		weak	strong	very strong
1	pH-value	6.5 to 5.5	5.5 to 4.5	below 4.5
2	lime-dissolving carbonic acid (CO_2) in mg/l determined by Heyer's marble test	15 to 30	30 to 60	more than 60
3	Ammonium (NH_4^+) in mg/l	15 to 30	30 to 60	more than 60
4	Magnesium (Mg^{2+}) in mg/l	100 to 300	300 - 1500	more than 1500
5	Sulphate SO_4^{2-}) in mg/l	200 to 600	600 - 3000	more than 3000

(In certain cases it is also recommended to conduct laboratory tests to determine the degree of aggressivity of the soil.)

If two or more values lie in the upper quarter of a range (with the pH-value in the lower quarter) then the degree of aggressiveness is increased one grade. This increase is not valid for sea water.

In certain circumstances a greater aggressivity must be allowed for at higher temperatures or pressures or if the concrete is subject, in addition, to mechanical abrasion due to swiftly flowing or agitated water. The degree of aggressiveness decreases at lower water temperatures, if only small amounts of water are present and the water is practically still, so that the aggressive constitutents can only be renewed slowly as for example in soils of low permeability (coefficient of permeability, $k < 10^{-5}$m/s).

ENVIRONMENTAL CONDITIONS

Special Requirements for Permanent Anchors

In order to increase their service life, it is necessary to require more stringent specifications for permanent anchors than for temporary anchors. These more stringent specifications relate particularly to:

- Provision for supplementing the anchorage by means of new anchors.

- Monitoring by means of displacement measurements in the structure and long-term checks of the anchors. The movements of the structure are to be checked periodically in accordance with Table 3 by means of geodetic or other suitable measurements. In the design, critical deformations (e.g. danger of collapse) are to be specified which, if exceeded, will necessitate stricter monitoring or structural measures. Some anchors are to be designated as check anchors and observed periodically.

- Lower working load than with temporary anchors. Increased safety factors are to be adhered to when dimensioning permanent anchors.

- Corrosion protection. As there is only very little information available about the suitability and long-term performance of the present-day corrosion protection agents, particularly for soil anchors, it is recommended to provide the facility for at least a random check of the state of the whole anchor (e.g. by removing it).

Table 3. Special requirements for permanent anchors.

Requirement	
- Monitoring of the deformations	recommended
- Check anchors for load measurement	about 5% of all anchors

Additional Technical and Legal Requirements

When planning the anchorage of structures, the following technical details are to be fixed or agreed upon, in addition to the selection of the anchor and its dimensioning:

- Pre-stressing load, V_o
- Permissible settling or uplift deformation and stability, particularly of adjacent structures and roads
- Distance of the anchors from utility service lines in the ground
- The effects of grouting on such lines and structures
- The removal of material (e.g. by means of drilling)
- Limit conditions if anchor fails
- Tolerances for the manufacture of the anchors, if this is of importance for the project

In addition, some of the legal problems to be kept in mind are the following:

- The consequences of the effects on adjacent plots of land
- The necessary contractual agreements regarding stressed or unstressed anchorage parts in adjacent plots of land
- The recording of the locations of permanent anchors with local authorities.

Potential Wall Movements with Tied-Back Excavation Walls

1. Based upon previous experience with anchored excavation walls, especially in the case of long excavations in cohesive soils, wall movements cannot with certainty be ruled out even if the walls and their anchored parts are designed (and prestressed) for the condition of increased active or at rest pressure. Of decisive importance here is the movement of the body of soil, which, as with a cofferdam, is enclosed by the retaining wall and the force transmitting constructional elements. The prestressing of the anchors produces body stresses, which tend to prevent the earth mass from straining laterally. However, movement of the body of soil is unaffected. In addition, too high a prestress can lead to a substantial lateral deformation and to especially large settlements behind the anchoring zone.

2. Basically the wall movements arise from the following causes:

   a) movement of the entire system including the anchorage zone.

b) shear deformation of the soil mass and the underlying ground

c) horizontal displacement caused by compression of the soil beneath the base of the excavation.

The settlement and tilting may be estimated from a settlement calculation. The heave of the base of the excavation due to unloading influences the components of deformation a) and c).

3. For cohesionless soils the wall deformations due to the "cofferdam effect" lie in the millimeter range, and are thus generally of no damaging consequence to adjacent buildings. For cohesive soils, on the other hand, depending upon the conditions and the plastic behaviour, substantially larger deformations are possible, particularly for deep, long or wide excavations.

4. If a corresponding investigation shows that a wall constructed to normal engineering practice is likely to develop inadmissible deformations, then the following measures must be adopted.

   a) an increased length of anchor must be used

   b) the construction of the excavation and the structure in a stepwise fashion.

5. Irrespective of the remedial measures given in item 4, in the vicinity of other buildings it is always useful not to let the anchors terminate in one plane. By staggering the lengths of the anchors the danger of an abrupt settlement behind the enclosed soil mass is generally avoided. Instead, a fairly uniform settlement can be expected.

6. If large wall deformations cannot be ruled out, it is recommended, for tied-back walls, from the very start to measure the horizontal and vertical displacements of the top of the wall, so that protective measures may be taken in good time. For excavations in soft clay soils and in the vicinity of structures anchor force measurements and settlement measurements in the surrounding areas are advisable.

7. Recommended permissible settlements and movements are given in Table 4.

STRUCTURAL CHARACTERISTICS OF THE RETAINING WALL

Sheet Pile Walls

Not all commercially available sheet pile forms are suitable for use in tied-back walls for excavations. Of the rolled-steel profiles the following are preferred.

Table 4. Recommended permissible settlements and movements

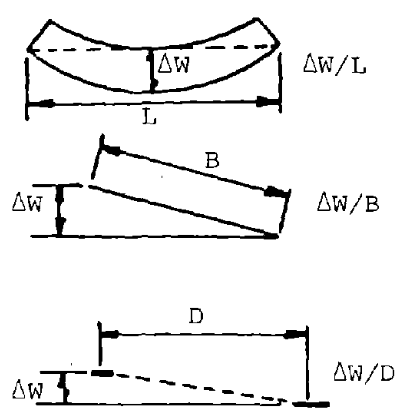
Structural Type	Soil			
	Sand (Rapid consolidation)		Clay (Slow consolidation)	
Structure consits of	Non-uniform*	Total (cm)	Non-uniform*	Total (cm)
a) precast concrete elements	0.0005	6	0.0007	8
b)				
c)				
d)				
Structure				
a) statically determinate	0.003	10	0.003	10
b)				
c)				
d) stiff, with massive foundations				
1) up to 20 m high	0.005	20	0.005	20
2) higher than 20 m	0.002	10	0.002	10
Crane running with mobile bridge	0.0015	--	0.0015	--

* Definition of non-uniform settlement:

a) flexure

b) tipping

c) change of angles



a) the Hoesch sheet pile with the interlocking part, consisting of a knob and a hook, in the flange (Figure 2) (Compare U.S. types PZ27 or PZ32).

b) the Larssen sheet pile with the interlocking part, consisting of two identical hooks, laying in the neutral axis (Figure 2) (compare U.S. type PDA27).

In recent years bulkheads have come on the market which are especially designed for excavation walls. These are similar to lightweight cold-rolled wide sections and are hooked together instead of having the interlock unit (Figure 3).

The material specifications for sheet piling are usually for the steel types. The higher quality steels are given preference if they are to be used over and over again or if they are to be subjected to heavy ramming. It must not be overlooked that, for the full utilization of the permissible stresses, the deflection of the sheet pile walls increased linearly with the quality of the steel.

As a rule, sheet piling is rammed into place. In an effort to try to avoid the disadvantages connected with ramming, e.g. noise emissions to the neighborhood or damage to the pile caps under heavy ramming, two further techniques have been developed: bringing into position with the aid of vibration using a so-called vibro-pile hammer and pile-driving with the English "Pilemaster". Whereas the vibration technique has shown itself, according to past experience, to be suited to cohesionless, water bearing soils, driving seems to be especially advantageous for saturated cohesive soils. An advantage of both methods is that if ramming becomes difficult the direction of the force can be changed as often as one likes and in this way, in less difficult cases, by moving the sheet pile up and down obstacles can be overcome.

In the design of anchored sheet pile wall systems attention should be paid to the need to be able to transmit the necessary compressive, tension or shear forces for the calculated bending action. This requirement is especially important in the case of Larssen-piles. The friction in the interlock is only sufficient to carry the shear forces occurring during ramming if the soil is of the gravel, sand or coarse silt types and if the shear stressing is light. To be sure, the friction in the interlock is often increased due to crusting up and bending out of shape, which occurs when the sheet piles are rammed into the ground and when they are bent by the action of applied loads. In addition, when two piles are rammed together it is possible that every second interlock can be pressed together in the works after fabrication.

When ramming in clayey soils and under high shear stresses there is no guarantee that the interlock can carry the shear stresses in the case of Larssen sheet pile elements. Under these circumstances, it is necessary to weld at least every second interlock. Only when every second sheet wall

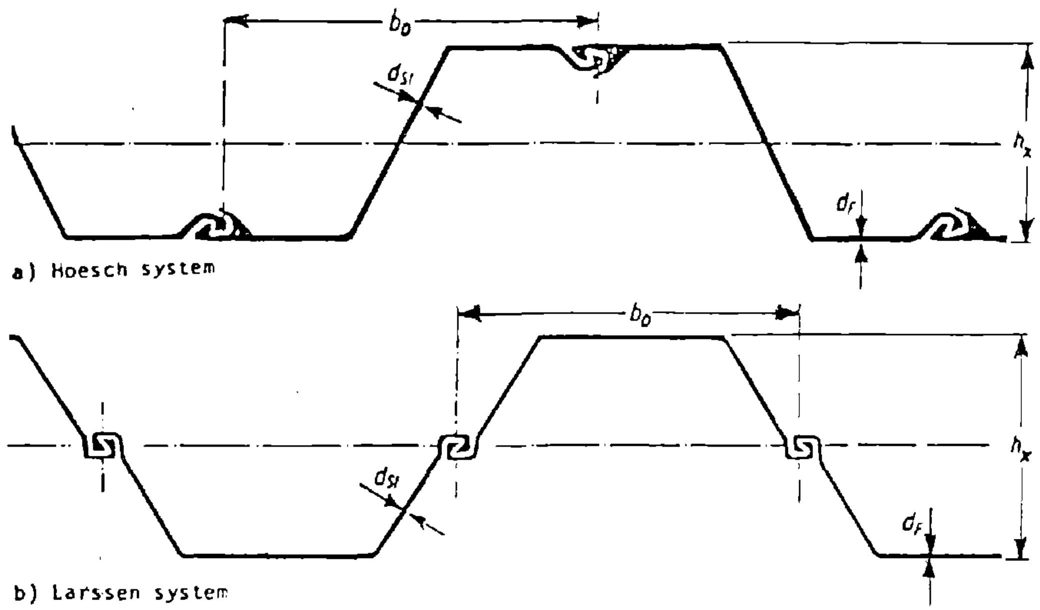


Figure 2. Hot-rolled sheet wall profiles.

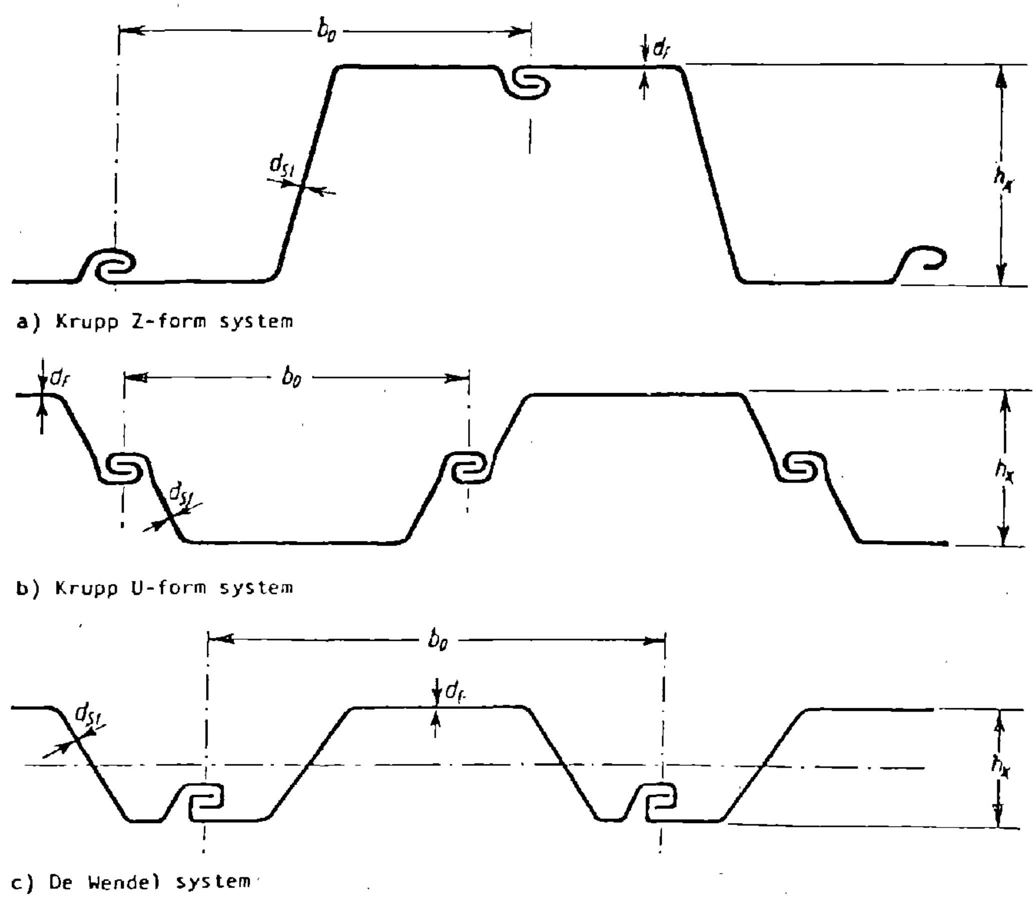


Figure 3. Cold-rolled sheet wall profiles.

section is supported may shoring and anchors be placed directly against the sheet piling without the introduction of a waling. Generally, only I-sections (rolled steel joists), double channel sections and reinforced concrete beams are used as waling. RC-beams have the advantage that they easily accommodate the wave form of the wall and any unevenness due to the ramming action. For steel girders the unevenness due to ramming is compensated by the use of steel strips, welded plates, steel wedges or concrete. Reinforced concrete walings are usually connected to the sheet piling by means of welded stirrups.

Sheet pile wall construction is relatively expensive at greater depths and, in addition, not very adaptable. Buried pipes must be shifted; obstacles in the ground can cause the sheets to be wrenched apart at the joints during driving. Thus sheet piling is normally only appropriate when water has to be held back or if the ground water table cannot or may not be lowered.

Anchored Berlin Type (H-piles and lagging) Retaining Walls

If sheet piling is not required, i.e. if the ground water present can be dealt with in another way or can be kept away from the excavation, then generally Berlin type retaining walls are preferred. They consist of vertical pile-beam elements about 1.0 to 3.0 metres apart and horizontal planks spanning the gap between the piles. Their many-sided application and their excellent adaptability to local situations have resulted in a variety of modifications. Today, under this type of retaining wall all excavation support walls are classified which consist of vertical beams with horizontal lining (planks), irrespective of the material used or the constructional process.

The following types serve as the most important vertical beam elements:

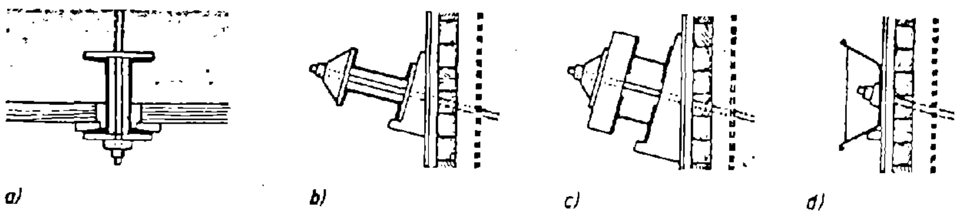
a) driven or vibro-rammed steel piles,

b) steel piles placed in boreholes,

c) reinforced concrete bored piles or unreinforced concrete bored piles with permanent casing.

I-section rolled steel joists, which are relatively flexible normal to the axis of the web and tend to deviate from the given line, are only suitable for use in ground free from obstacles, e.g. in pure sand, if they are driven into position. For hard ground conditions heavier I-sections are preferable.

Different considerations apply to piles placed in boreholes. First and foremost one tries to manage with as small a hole as possible. This approach leads to the use of stocky sections, in which width and thickness are of about equal size. Double channel sections joined together can also be very useful (Figure 4), since the anchors can be placed at the axis of symmetry of the two sections.



a) Pile consisting of 2 channel beams

b) Waling consisting of 2 channel beams

c) Waling consisting of 2 H-beams

d) Waling consisting of sheet pile elements

Figure 4. Force transmission with anchored Berlin type retaining walls.

The introduction of the pile-beam elements into boreholes of diameter 60-80 cm is particularly advantageous if the vibration and noise caused by ramming has to be avoided or if hard layers are present which make ramming unfeasible.

As a rule in cohesionless soils casings are employed; it may be possible, however, to supprt the wall of the borehole using a thixotropic fluid. Casing is usually not required in cohesive soils. The space between the pile and the wall is filled with lean mixed concrete mortar, sand with some fines, pure sand or any other compactable material.

For excavations in the immediate vicinity of buildings or in the underpinning of buildings it may be necessary to dispense with the use of piles placed in boreholes and opt for bored-piles (diameter 60-120 cm) instead, e.g. if large vertical forces must be carried, or if the piles due to lack of headroom have to be introduced in small sections.

In built-up areas, in which one must reckon on there being buried pipes, etc. in the ground, it is usual to dig pits to a depth of 1.5 to 2.0 m before starting pile-driving or boring operations. Thus one tries to avoid unwelcome surprises, sources of danger and damage claims. One only has to think of buried high voltage electric cables or gas mains.

The usual spacing of the pile-beams is 2.00 to 2.50 m. The planks must be continuously introduced as the excavation work proceeds. The allowable height of unlined wall must be determined according to soil mechanics principles and should not exceed 1.50 m in cohesionless soils. There are many ways of lining the exposed ground between the pile elements:

a) wooden planks

b) square wooden beams

c) round wooden beams

d) used railway sleepers

e) canal planks

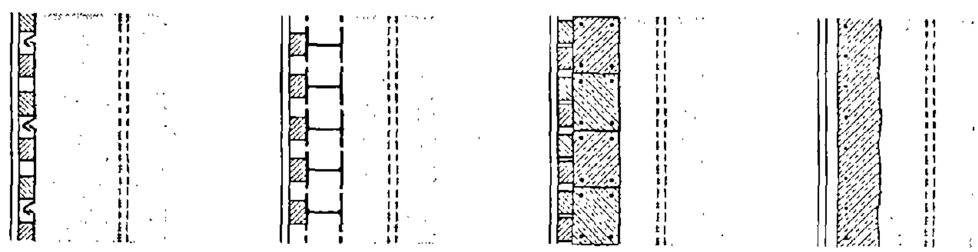
f) light H-steel sections

g) precast reinforced concrete elements

h) encased reinforced concrete

i) colcrete

Some of these are illustrated in Figure 5.



a) with canal planks

b) with H-beams

c) with precast elements

d) with reinforced cast in situ concrete

Figure 5. Berlin type retaining walls with steel or reinforced concrete.

For permanent installation the choices g) and h) are the most suitable.

As a rule, in the case of anchored Berlin type retaining walls, bending resistant main waling is required which transmits the forces from the piles to the anchors. The anchors are often spaced according to their load carrying capacity and the specified minimum spacing without consideration of the distances between the piles.

The main waler must be designed accordingly in this case. In this respect, it is better to arrange the anchors in pairs, thereby achieving a more uniform loading of the walers. A possible choice of wales is rolled steel joists (I-section).

Due to their many advantages anchored Berlin type retaining walls are popular for the support of the sites of excavations and larger trenches for tunnels. Essentially the following points are in their favor:

1. Berlin type retaining walls are exceptionally adaptable if pipes, pits, foundations, old parts of buildings or other hindrances make the construction of a conventional sheet pile wall difficult.

2. With the large number of variants possible this kind of retaining wall can be applied in all types of soil.

3. By leaving out a pile from the row in the wall, excavation can, locally, be systematically extended to take a dewatering facility.

4. If need be the timber sheeting can be inserted in places behind the inside flange to gain working space.

5. By the choice of corresponding sections greater excavation steps can be made.

6. It is possible sometimes to retrieve constructional elements.

7. This method of construction is very economical.

Anchored Diaphragm Walls

In recent years there has been increasing use of slurry trench or diaphragm walls, with betonite suspension acting as the slurry. The constructional process is usually as follows:

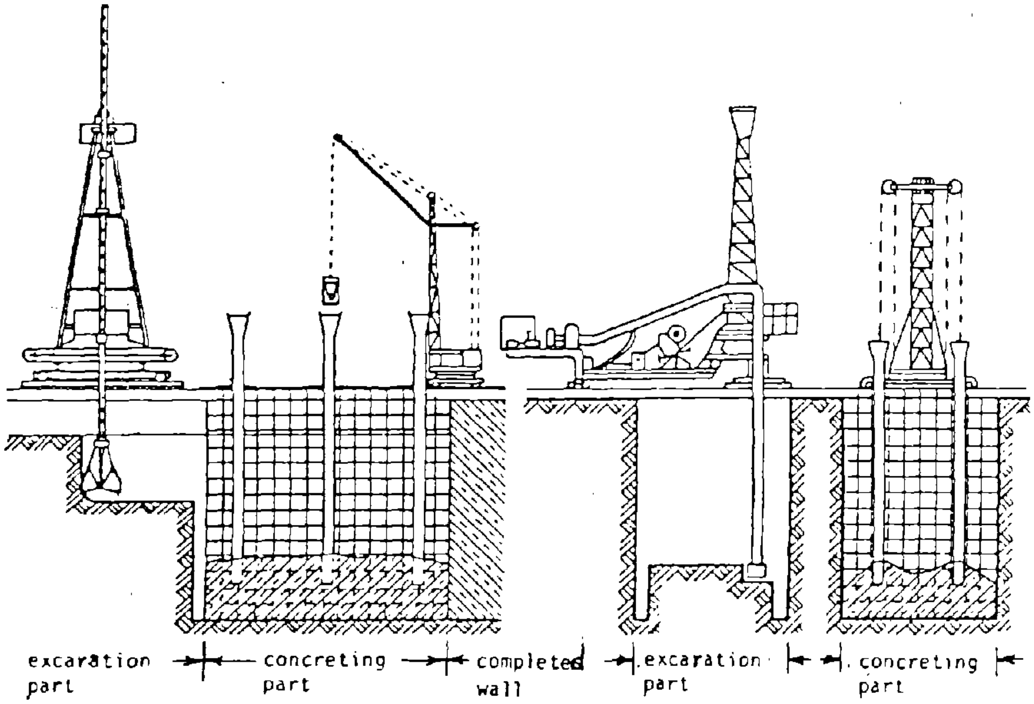
a) construction of guide walls

b) excavation of the soil while simultaneously filling the trench with bentonite slurry to give support to the sides

c) introduction of the reinforcing cage and the concrete

The guide walls are about 0.50 to 1.50 m high. They are made mainly 15 to 25 cm thick out of cast in situ concrete or prefabricated reinforced concrete elements which are fixed end to end and stiffened at the ground level. They support the ground in the depth in which the hydrostatic pressure of the bentonite slurry is ineffective or insufficient and they protect the top edge of the trench when the excavating equipment is lowered and taken out again. They also serve to guide the equipment, the reinforcement and the board segments when being lowered into the trench. The width of the trench is between

0.40 and 1.20 m. To excavate the soil many special machines have been developed. The most common are the cable suspended clamshell grab and the rigidly guided slot-cutting grab bucket. If the soil conditions are suitable the reverse circulation boring technique may be preferable (Figure 6). Figure 7 illustrates the construction sequence.



a) excavation with grab; continuous trench.

b) excavating using reverse circulation method, trench segments.

Figure 6. Methods of construction of slurry trench walls.

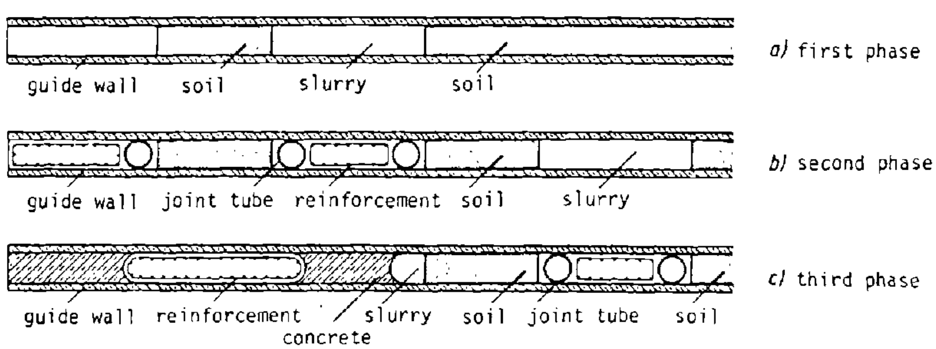


Figure 7. Construction of a diaphragm wall in its various phases.

If the grabbing force of the machine is too small to break through the soil it may be expedient to bore uncased holes before the excavation work is started. These are placed at a distance apart equal to that between the teeth of the open clamshell grab. In this way the soil between the holes can be easily grabbed by the jaws of the bucket. Besides, the holes act as a good guide for the grab bucket. Rock or rock-like soil layers must be broken up either with heavy drop chisels or percussion drilling, or using closely spaced core drilling. If the machine comes up against individual obstacles, then, with increasing depth, the slurry trench may deviate somewhat from the vertical.

Bentonite is used to support the sides of the excavation. The slurry is fed into the trench continuously with the working operations, so that as far as is possible the level of the suspension is flush with the top of the guide walls. Since the trench is always filled with slurry, the excavated material is a mixture of soil and slurry. The slurry is then separated from the waste material in settling ponds or in agitated troughs and with the addition of fresh slurry is reused. The diaphragm wall should normally be constructed immediately after the excavation of the trench, so that the supporting action of the slurry is only relied upon for a short time.

Diaphragm walls to support the sides of excavations are reinforced according to the requirements of static analysis. The reinforcement is lowered into the slurry-filled hole. The concrete is placed in position by means of the tremie method. The fresh concrete pushes the concrete already placed upwards so that the slurry is continuously displaced and voids are avoided. The individual concrete sections are meshed together by a suitable use of tubes, steel beams or expanded metal reinforcement. Joint tape is routinely used with success for sealing joints between pours.

In calculating the safety factor of a slurry trench system the hydrostatic pressure due to the slurry counteracts the earth pressure and the ground water pressure. For short stretches of trenching the arching action and redistribution of earth pressures may be taken into account.

The use of walers for diaphragm walls is not standard procedure. The anchors are generally placed right up against the wall. Narrow trench sections are each held by one support in the middle, wide sections by two supports one each near to the joints. A special reinforcement designed for the purpose provides for the necessary lateral distribution of the load. In addition, through dowelling the sections together a transfer effect from one section to another is achieved.

The tremie method is not the only one for constructing a reinforced concrete wall in a trench. Precast reinforced concrete elements have also been used with success; likewise, the placement of steel sheet piling has been carried out in this way in cases where a disturbance to the environment had to be avoided, such as would be caused by ramming.

In comparison to anchored sheet pile walls or Berlin type retaining walls, anchored diaphragm walls offer the following benefits:

1. The method is independent of ground conditions; it may be used in soft cohesive soils as well as in sand, gravel and coarse gravel beds.

2. Strata which are too hard to penetrate by driving can be excavated and blocks of rock, rubble and other hindrances can be overcome.

3. The depth of embedment of the walls is practically unlimited.

4. Diaphragm walls can be constructed in the immediate vicinity of existing buildings; existing foundations which are at a higher elevation than the excavation level do not have to be underpinned.

5. A distortion of the ground e.g. in driving, or a loosening of the ground when exposing the wall does not occur; disturbance to the ground is avoided.

6. The deformations of the walls of the trench during digging operations and the deformations of the completed concrete wall when the excavation is carried out are very small. Diaphragm walls are thus very suitable if neighboring structures are very sensitive to settlements.

7. There are no vibrations caused by the construction of diaphragm walls, so that there are no unwanted seismic emissions to the neighborhood.

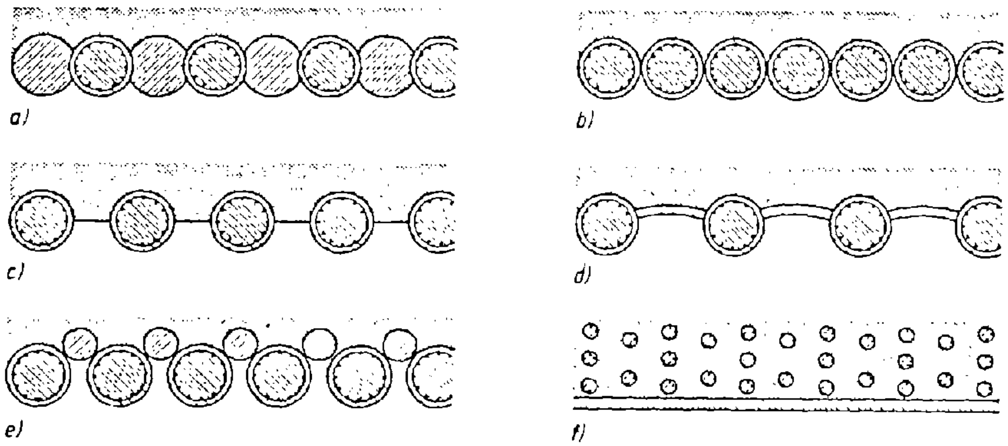
8. There is practically no noise produced by this construction method; the level of noise is less than with normal road traffic.

Despite the advantages diaphragm walls are only employed to a limited extent. This is due to the relatively high costs involved. As a rule, this method is economic and competitive compared to other methods only if the diaphragm wall does not serve as a temporary support, but is incorporated as a structural member into the final structure, e.g. as retaining wall, tunnel lining or basement wall.

Anchored Bored Pile Walls

Besides diaphragm walls, bored pile walls of cast in place concrete may also be used to retain the sides of excavations. The conditions under which the two methods are favorable are essentially the same. Just like single bored piles, walls of bored piles can also be constructed with or without casings. Uncased boreholes in soils with too little cohesion to stand up can be stabilized with a slurry in the same way as with slurry trenches. Since, however, a strong arching action is present in the vicinity of circular boreholes the same high demands are not made on the properties of the slurry as in the case of slurry trenches. Thus bentonite suspensions of lower concentration are adequate.

The standard pile diameters are in the range 0.60 to 1.20 m. Various pile arrangements are shown in Figure 8.



a) overlapping piles

b) touching piles

c) unlined gaps between piles

d) piles with arched elements

e) piles of different thicknesses

f) a wall in which the piles are separated, but the whole is protected with a curtain wall.

Figure 8. Various pile arrangements for bored pile walls.

It is not possible as is the case with diaphragm walls to place the reinforcement for 'hidden' horizontal beams. Thus brackets must be affixed and reinforced concrete or steel beams must be employed as waling. If the distance between anchors is small it is usual to do without waling and to position the anchor between two piles. Anchors could also be located in the axis of the pile, whereby the reinforcement in the piles must be arranged accordingly.

Bored pile walls have the same advantages over sheet pile walls and Berlin type retaining walls that diaphragm walls do. Compared to the latter, however, they have the disadvantage that the greater number of joints is unfavorable with regard to the watertightness of the wall. Bored pile walls are also affected more by obstacles in the ground than the technique of diaphragm walls. The danger then arises that the pile will wander out of the vertical line so that the wall is not sealed off against water. For the same bending moments the piles require more space than the equivalent diaphragm wall. On the other, there are occasions when the bored pile wall is a better solution than the diaphragm wall:

1. The diameter of the borehole is much less than the minimum size of the sections of a diaphragm wall. The stresses in the ground the slurry supported boreholes are more favorable than for sections of slurry trench.

2. For cased boreholes the equilibrium of the soil is hardly affected at all.

3. The depth of embedment of individual piles can be staggered without greatly influencing the passive resistance in front of the wall. It is also possible and useful to construct some piles deeper than the rest, if they are subjected to greater normal forces.

4. The transmission of forces at the foot of a pile wall is more reliable and, as a rule, involves smaller settlements than at the foot of a diaphragm wall, where a mixture of soil and slurry forms due to the excavation process. During concreting this softened zone is not always removed.

5. A bored pile wall can be constructed with a given inclination, such that the top of the wall is near to a structure and the bottom underneath the structure, so that the trouble of a time-consuming underpinning is saved.

Bored pile walls are economic and competitive only, as in the case of diaphragm walls, if they can be incorporated as a load bearing member into the final structure. In all other cases their use is justified only by special local site conditions.

Underpinning Walls

Of late anchored underpinning walls, also called element walls, have also served as retaining walls for excavations.

In the following the typical constructional procedure of an anchored element wall is described.

As soon as one row of elements has been anchored and prestressed the excavation is carried to the depth of the next row of elements, whereby close to the wall itself a suitable soil slope is maintained (Figure 9, a & b). Then with the excavator the slope is removed in front of two or three alternate elements and the new elements (formed of in situ reinforced concrete) are installed, (Figure 9, c & d).

Afterwards, the intermediate elements are completed (Figure 9, d & e). The horizontal and vertical bond is achieved by overlapping steel reinforcements. After the whole row of elements has been placed in position the boreholes are drilled, the anchors installed, grouted and finally prestressed (Figure 9f). This procedure is repeated for the next row of elements.

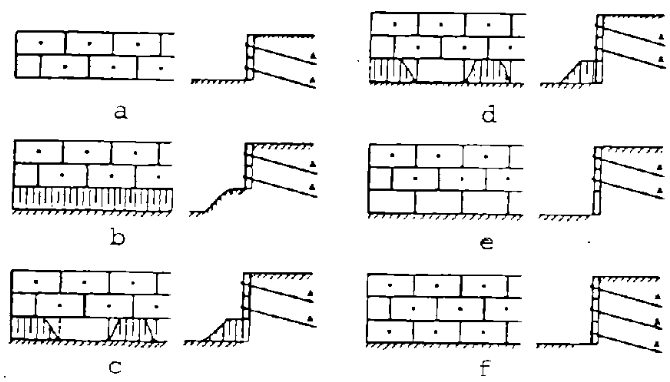


Figure 9. Typical constructional procedure for an anchored element wall.

The advantage of quickness of this method of construction derives from the fact that before the actual excavation work no driving or drilling work with special machines must be carried out. The progress of work in sufficiently large excavations depends upon the number of elements that can be installed daily. This method is especially advantageous in cases, in which the element wall not only serves the purpose of temporary support, but may be incorporated into the final structure. Technical and economic advantages are obtained when large rocks are present, e.g. amongst slope debris, which otherwise impede progress. With the element wall method such hindrances can be overcome quite simply by blasting in the excavation. Anchored element walls are also easily made in cohesionless soils or below the ground water table.

The height of the elements is between 1.0 and 2.0 m, the length between 3.0 and 4.0 m and the thickness 40-60 cm. It is an advantage in the design of the walls if at least one anchor is allocated to each element.

In contrast to other types of retaining wall there is no depth of embedment in the underlying ground. Only a strip foundation is constructed beneath the wall to act as a base support.

PHYSICAL CHARACTERISTICS OF ANCHORS

In this section definitions and nomonclature relating to anchors will be presented along with a physical description of anchors in common use.

Definitions and Nomenclature

Anchors are units which transmit forces into the soil or rock by means of tendons, and are composed of three main parts (refer to Figure 10):

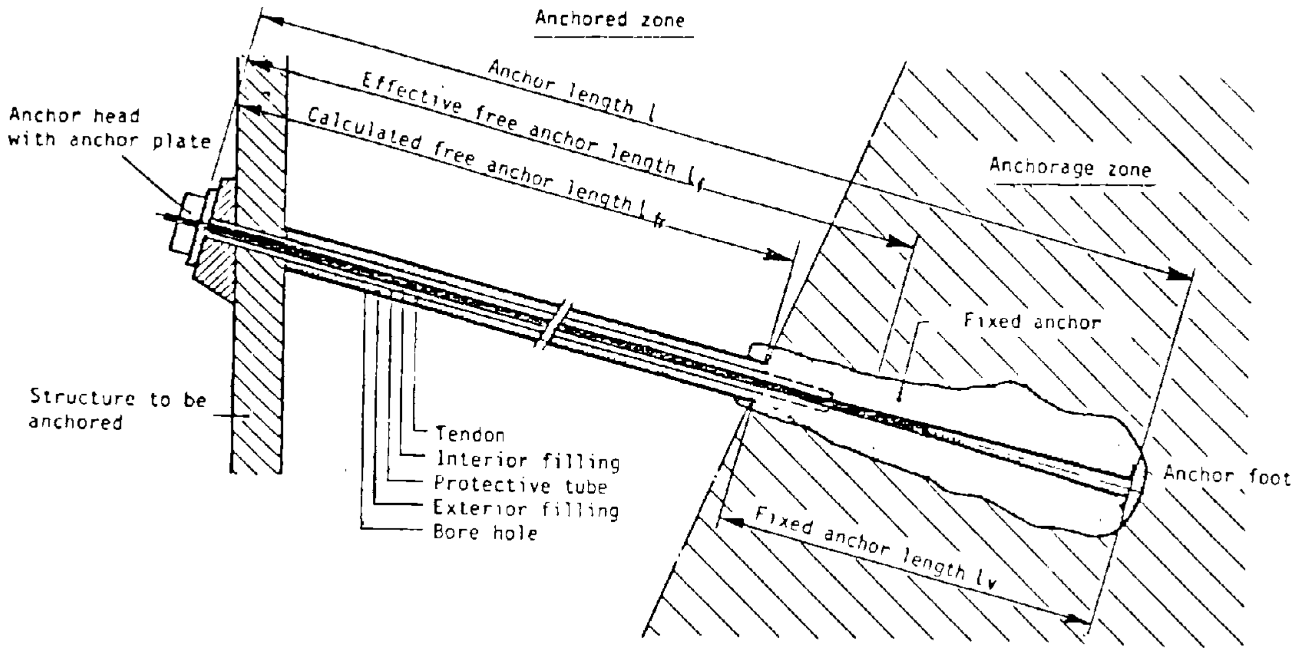


Figure 10. Anchor construction.

- Anchor head: This fastens the anchor to the anchored structure.
- Tendon: This transmits the force from the anchor head to the fixed anchor.
- Fixed anchor: Transmits the forces of the tendon into the ground.
- Anchor foot: The end point of the anchor in the ground or rock.
- Anchor length, l: The distance between the anchor foot and anchor head.
- Fixed anchor length, l_v: The length over which the force is transmitted into the foundation.

For anchors with a free anchor length the following definitions also apply:

- Effective free anchor length, l_f: Length over which the tendon can expand freely when stressed (free steel length)

- Calculated free anchor length, l_{fr}: The length between the anchor head and the point at which the anchor enters the anchorage zone resulting from the static and soil mechanics calculations.

Types of Anchors

The following basic characteristcs are used for a more precise designation of anchor types:

a) Type of anchorage zone

A distinction is made between soil anchors and rock anchors, depending on whether their fixed anchors (see Figure 10) are fixed in soil or rock.

b) Type of application

Long-term or permanent anchors are those which have to fulfill their function throughout the service life of the structure and thus require special design and monitoring.

Test anchors are specially designed and conventionally placed anchors which undergo extensive tests in order to obtain basic information to aid in the selection of the anchor and the dimensioning of the fixed anchor.

Check anchors are anchors in or beside the structure used for long-term observations.

c) Type of anchor fixing in the bore hole

Fixing in the bore hole by means of expansion of the bonding element (e.g. expandable anchors, rotary plate anchors).

Fixing in the bore hole by embedding in a binder (e.g. grouting anchors, mortar anchors, synthetic resin adhesive anchors).

d) Type of action

Fully-bonding anchors are those whose total anchor length is in contact with the surrounding ground. This type of anchor is only suitable as a dead anchor.

Anchors with free anchor length are those which are fixed or bonded to the ground only over the fixed anchor length (see Figure 10).

e) Type of stressing of the tendon

Pre-stressed anchors are those in which only small changes in the prestressing force within certain limits are possible and permissible as a result of the working loads.

The applied load, V_O, relative to the load-bearing capacity, V_U, at time t = 0

$$0.5 < \frac{V_O}{V_U} < 0.75$$

Tension anchors are those in which initially (t = 0) only a part of the possible and permissible stressing force has been applied and quite considerable changes of the tensile force and strain are possible and permissible.

The applied load, V_O, relative to the load-bearing capacity, V_U, at time t = 0

$$0.25 < \frac{V_O}{V_U} < 0.5$$

Dead anchors are those in which initially (t = 0) no or only a very small stressing force has been applied.

$$0 < \frac{V_O}{V_U} < 0.25$$

The anchor designation is given by stating its maximum test load, V_P, and its construction.

Tendon Characteristics

Three types of tendons are in common use:

a) Threaded rods of diameters

ϕ = 26.5 mm (1.04 inches)
ϕ = 32 mm (1.26 inches)
ϕ = 36 mm (1.42 inches)

Steel quality is normally 835/1030 or 1080/1230 N/mm^2 (121/149 or 157/178 ksi) (indicates yield stress/ultimate tensile strength).

These conditions produce tendon failure loads, V_Z, from 568 to 1252 kN (127.7 to 281.4 kips).

b) Composed of strands with cross-sectional areas of 93 mm^2 (diameter $\approx$ 0.5 inches) and 146 mm^2 (diameter $\approx$ 0.6 inches). Ultimate tensile strengths range from 1570 to 1765 N/mm^2 (228 to 256 ksi).

Designs can range from two strands of 93 mm^2 to 19 strands of 146 mm^2 producing tendon failure loads, V_z, from 329 to 4910 kN (74.0 to 1103 kips).

c) Wire cables of diameter 7 mm (area = 38 mm^2) and ultimate tensile strength 1670 N/mm^2. Designs utilize from 1 to 52 strands producing tendon failure loads from 64 to 3340 kN (14.4 to 750 kips).

Anchor Heads

Anchor heads can be of the following types:

- cannot be post-tensioned or load tested.
- can be load tested, but not post-tensioned.
- can be load tested and post-tensioned, but tension cannot be released (or relaxed).

Fixed Anchor Length

Two types of grouted or fixed anchors are in common use:

Mono anchors: force is applied at the tensile end of the grouted length so that the grouted length of the anchor is all in tension.

Duplex anchors: force is applied at the bottom end of the grouted length so that the grouted length of the anchor is in compression.

Corrosion Protection

a) Threaded rods

free length:
1. cement grouting + Polyethylene tube
2. Epoxy resin coating on sand-blast-finished surface + Polyethylene tube
3. corrosion protection filling + Polyethylene tube

4. Polyethylene shrink-on tube + Polyethylene tube

fixed anchor length: basically a double corrosion protection is necessary

1. Epoxy resin coating + cement grouting

2. cement grouting in corrugated PVC pipe at the factory + cement grouting

3. corrosion protection filling in steel tube (Duplex anchor) + cement grouting

4. Polyethylene shrink-on tube + cement grouting

b) Strands 0.5"/0.6" or wire cables Ø 7 mm

free length:

1. cement grouting + Polyethylene tube

2. corrosion protecting filling + Polyethylene tube

3. individually greased and plastic coated strands + cement grouting + Polyethylene tubes

fixed anchor length: basically a double corrosion protection necessary

1. cement grouting in corrugated PVC pipe at the factory + cement grouting

2. corrosion protection filling in steel tube (Duplex anchor) + cement grouting

c) Corrosion protection at anchor head

1. Concreting of the complete anchor head - no checking possible afterwards

2. Protection with galvanized covers filled with grease and anti-corrosive material - checking possible at any time

3. Special insulation of the anchor head with the structure to prevent a positive displacement in potential of the anchor and a reduction of the danger of corrosion due to the formation of an electric cell system.

Access to Anchors

The anchors must be designed such that it is possible to bore and insert them into the borehole. Cable and stranded anchors require less space, since they are not so stiff and can be easily bent. The smallest spacing between the anchor heads depends upon the size of the tensioning jack. It is about 50 cm.

The length of the fixed-zone should be at least 2.0 m. For several anchor positions it is recommended that all the anchor fixed-zones do not terminate in one line, so that the formation of cracks between the anchorage body and the ground behind it is avoided. Depending on the working loads of the anchors, stabilization of the fixed-zone with between 3.5 and 5.0 m of of overburden is necessary. For smaller overburdens there is danger of total ground rupture.

If the purpose of the anchors is to increase the stability of the wall-anchor-soil system, then it is advisable to place the anchors in the lower part of the wall. The optimum inclination of the anchors is -20° with respect to the horizontal in homogeneous soils. If layers of different shear strengths are present it is economical to design the slope and length of the anchors so that the bonding zone lies in the densest soil layer with the highest shear resistance. Thereby a greater ultimate load resistance of the fixed-zone is achieved, allowing heavier anchor units to be installed, which increases efficiency.

In order to avoid excessive deformations it is important that the lowest row of anchors is constructed sufficiently deep into the less deformable layers below the excavation level. In this case greater anchor inclintions (up to -40°) may be used.

The optimum anchor distribution depends on the choice of the wall construction. Vertical load carrying elements (e.g. bored piles) must be distributed horizontally so as to take advantage of the optimum load capacity of the anchors. Sometimes it is advantageous, irrespective of the bearing capacity of the soil, to install smaller anchor units, which can be uniformly distributed along the retaining wall. This is especially true in the case of element (underpinning) walls, as well as for all low walls.

Water-tightness of Bore Holes in Rock

Bore holes in rock into which grouted anchors are to be fitted shall be checked for water-tightness by means of water tests before the anchors are installed.

The water loss is to be measured at a suitable test, pressure, and should not exceed 1 Lugeon (1 Lugeon = 1 liter per meter per minute at a pressure of 1MPa in a measuring period of 10 minutes). If this condition is not met, then special measures shall be taken, e.g. grouting.

A bore hole is also grouted and redrilled if water discharges are found in adjacent bore holes during the water test or if water flows through the bore hole itself.

Deviations are permissible if the tightness of the bore hole is not of major importance for the anchor system used.

LONG-TERM BEHAVIOR OF ANCHORED WALLS

The long-term behaviour of anchors can be monitored exactly by measuring the anchor force and with the aid of parallel built-in extensometers. The long-term behaviour of the complete "wall-anchor-soil" system, on the other hand, requires more extensive measurements, which are described in the following section. Previous experiences could indeed be considered as individual case studies. However, they do not provide a general basis for interpretation. For instance, we know of no test results which show how far blasting and external vibrations influence the ultimate load capacity of anchors. Other external influence should, in cases of doubt, be investigated by means of in-situ testing.

Monitoring

The basic behaviour of the wall-anchor-ground system can be predicted. Static and mechanical calculations can only be considered as approximations for the following reasons:

1. The assumed soil coefficients used in the calculations are the results of more or less representative soil tests, in many cases only estimates or values from tables.

   With soil conditions which are not clear or with soft, very cohesive soils one can be sure that the assumed values for cohesion or deformation modulus used in the calculations may deviate from the actual values.

2. Another reason that the calculations supply only approximate solutions lies in the calculation method itself.

   With deep anchored walls, the behavior and stress of a system can no longer be predicted based on simple assumptions (earth pressure according to Coulomb, wall top deformation as a result of earth pressure activation). Based on measurements made at various construction sites, extensive, complicated sets of rules were established for the design of such systems (1), (2). On the other hand, based on extensive soil mechanics observations, others have tried to explain and account for the measured results using new, more complex methods of calculation.

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(1) Gaibl, A., and Ranke, A., Load of Rigid Sheet Pile Walls, published by W. Ernst & Son, Berlin, 1973

(2) Jenne, G., Earth Pressure. Concrete Calendar 1973, part II, pp 89-129, published by W. Ernst & Son, Berlin, 1973.

In contrast to conventional calculation methods for solid structure designs, determining factors for the safety of anchored foundations include not only load capacity limits of the wall-anchor-ground system, but also deformation limits.

3. The entire system depends heavily on anchor behavior. The behavior of individual anchors with respect to load capacity of the retention zone can be investigated experimentally provided that sufficient time is available. The overall group effect of the anchors cannot normally be evaluated, even though larger deformations often result.

The failure of the chosen "wall-anchor-ground" system can result in serious damage and threat to human lives. It is for these reasons that the necessity of monitoring of such structures is strongly recommended.

In most cases, therefore (Figure 11), the monitoring systems consisting of geodetic measurements of terrain surface and wall top, deformation measurement of wall and earth, and anchor force measurement, provide solid evidence of the behavior of the system chosen.

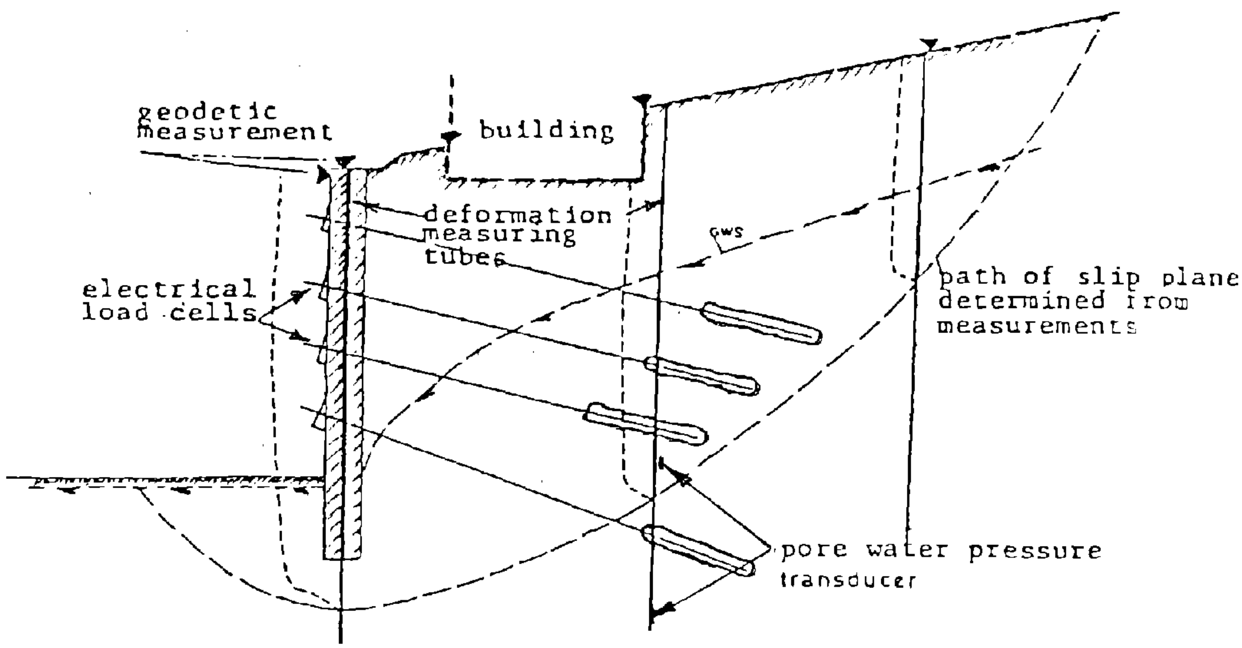

Figure 11. Recommended Arrangement of Excavation Monitoring System.

Limiting Conditions

Limiting conditions provide an indication of the usefulness of data from measurements for the evaluation of the behavior of the chosen wall-anchor-ground system. The definition of a limiting condition must first be stated, however: a limiting condition is defined as the state of a structure for which any qualitative changes in the ground of the structure itself renders it incapable of properly fulfilling its function.

In soil mechanics it is customary to make appropriate calculations to insure against the attainment of a limiting condition. In the design of an anchored wall, conventional computational methods are used to insure safety for the following limiting conditions:

1. The wall structure or the anchor may be overstressed by earth pressure, which could result in failure of the structure or the anchor. Earth pressure must therefore be determined, and all structural system components dimensioned accordingly (Figure 12a). This risk is discernible through measurement of anchor force and structural deformation.

2. With insufficient anchoring depth in the soil, the wall may shift resulting in twisting of the structure (Figure 12b). Earth resistance in front of the wall must be calculated and correlated with the calculated supporting force of the structure in the ground. This dange is perceptible through measurement of structure deformation. Earth pressure can in any case be activated only by a deformation.

3. Failure may result under the wall foundation due to vertical loads (Figure 12c). There must be proof that the vertical wall components are carried satisfactorily by the wall foundation. Geodetic measurement of the wall top can signal this danger promptly.

4. The wall can tilt forward because the anchor is too short and the retention zone of the anchor is in the region which slips with the wall (Figure 12d). The inner stability of the system must be calculated. Ground deformations occur before this condition is reached and are measurable.

5. Ground slipping may occur even with the anchored wall (Figure 12e). The rigidity of the entire structure must be monitored. This is a classic example of how ground deformation measurement is used to signal such danger.

6. Even if a failure in the wall-anchor-ground system does not occur, deformation in the surrounding ground may be so large that the proper functioning of adjacent structures or the current project are threatened (Figure 12f). Deformation measurements of surface, structures and ground provide extensive data in this case as well.

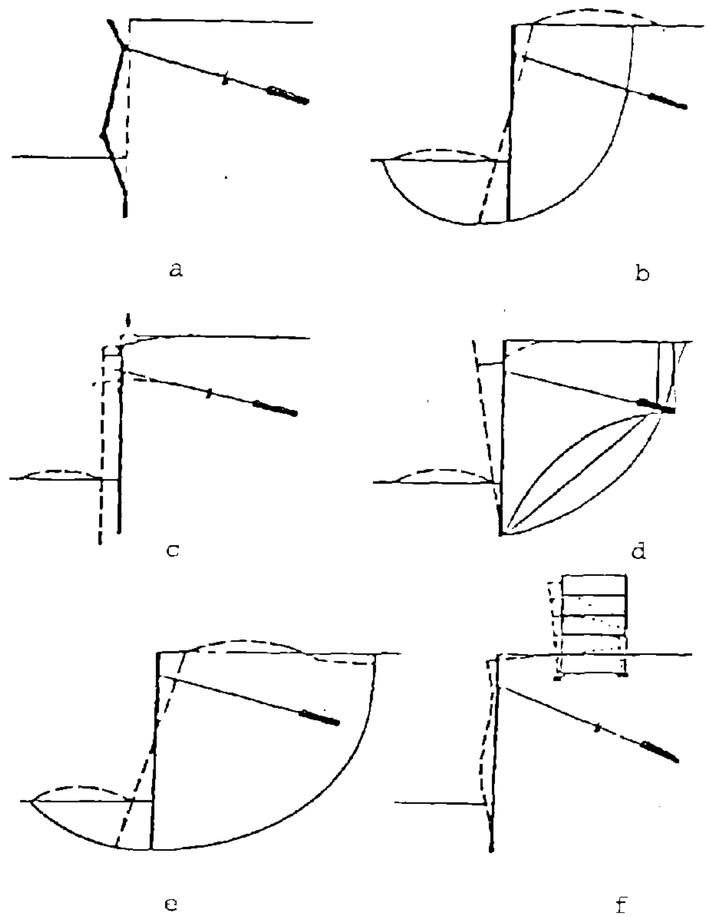

Figure 12. Limiting conditions for anchored walls.

Larger deformations occurring in the structure indicate the presence of yield hinges; occurring in the ground they indicate plastic zones.

These deformations, as long as they remain within certain limits, are not always cause for alarm. The material is self-healing, and stress is shifted to the neighboring zones, whose deformation resistance is then activated. This is analogous to the principle of arching effect and to the ultimate load method, as well as to the method of finite elements developed for soil mechanics calculations.

With large differences in measured values, the reciprocal behavior of measured anchor forces and deformation determines the type of limiting condition possible. For example: the difference between limiting conditions in Figure 12a and Figure 12d lies in the increase in anchor force in the first case. Also, measured anchor forces provide information concerning the long term behavior of anchors and their group effects.

Provisions for Monitoring

The decision whether or not to use the described monitoring system for anchored structures, and to what extent, depends on several factors. As a rule, the system is recommended for the following cases:

a) for anchored walls more than 11.0 m deep in granular soils and longer than 25.0 m.

b) for anchored walls more than 7.0 m deep in cohesive soils and longer than 18.0 m.

c) for slopes in which rigidity is the determining factor in the design

d) where water table difference in and outside of the excavation exceed 5.0 m.

e) where difficulties with respect to load capacity of the retention zone are suspected, or where vague soil conditions exist.

f) where deformations or anchor failure could cause economic damage or pose a threat to human lives.

The arrangement of the monitoring system should be proposed by the project engineer and decided upon by the field superintendent in due time before beginning the work. The monitoring system project as well as co-ordination of measurements should be done by an expert in this field. Specially developed computer programs can be used for the interpretation of measured values and evaluation of the accuracy of assumptions concerning soil mechanics (3). It is important to bear in mind the time factor when preparing the measuring equipment and carrying out any preliminary operations (drilling holes for slope indicators placement, etc.). Practice shows that in most cases a monitoring system is only installed after visible deformations occur, making possible only relative statements concerning the condition of ground movements.

Measurement should be made:

a) before beginning the work

b) at each stage of excavation

c) at least once about three weeks after the final excavation stage

d) for permanent anchoring, four (4) times per year during the first year following the final excavation stage, twice yearly during the next two years, and once yearly in subsequent years.

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(3) Foundation Works pocket-book, vol. I, 2nd edition, published by W. Ernst & Son, Berlin, 1966.

e) several times as needed for establishing structural deformations or increasing/decreasing anchor prestressing forces.

Monitoring the corrosion protection of anchors is of great importance for permanently anchored structures. Such monitoring can be accomplished if:

a) reference anchors are installed

(Reference anchors are additional anchors which are installed in the same manner, under the same conditions, including soil conditions, and with the same corrosion protection. These anchors must be installed in such a way that the entire length of the tension member can be removed.)

b) anchor systems are used where monitoring is possible by removal of the tension member.

(The anchoring system must provide for the removal of the whole tension member, as in the Duplex system.)

PRELIMINARY ANALYSIS AND DESIGN

The project engineer must have comprehensive knowledge of material technology, soil mechanics and rock mechanics for the analysis and dimensioning of anchors.

The basic factors used in the analysis and dimensioning processes are:

- The planning basics
- The material properties and the permissible loading and deformations of the individual structures and of the foundation.

The analysis and dimensioning of the anchors shall take into account their specific task. This includes:

- Determining the working load, V_G
- Determining the calculated free anchor length, l_{fr}
- Selecting the stressing load, V_O

Also to be taken into account are:

- The possibilities of movement of the structure
- The rigidity of the structure
- The compatibility of the movements of the anchors and the anchored zone

- In the case of soil anchors, the behavior especially of clayey ground during pre-stressing

- In the case of rock anchors, anisotropy, inhomogeneity and discontinuities of the rock

The following loads are to be taken into account:

- Dead weight of the ground and of the retaining construction
- Water pressure
- Permanent loads (e.g. building)
- Working loads and live loads
- Any dynamic loads

The stability of the structure and the foundation is to be proven also for the relevant construction stages.

The calculations are to be documented and are to be dated and signed.

Soil Anchors

The working load, V_G, is determined as a function of the tasks the anchor is to fulfill. Thus, the necessary anchor force, V, for a given safety factor F can be determined using stability calculations. The working load V_G, is determined from the sum of the calculated earth and water pressures. When calculating the water pressures, all water levels are to be examined.

In the following two cases the procedure to be adopted is this:

For stabilizing hillsides: the working load of the anchor is determined using stability calculations whereby, on the slipping body, only the forces of anchors cut within the calculated free anchor length, i.e. in the anchored zone shown in Figure 10, may be taken into account.

For single and multi-anchored walls: This standard for calculating the working load requires that the earth pressure from the dead weight of the ground and from a uniformly distributed live load, p, is uniformly distributed over the excavation depth, h (rectangular distribution in accordance with (4), see Figure 13). The water pressure is not affected by this.

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(4) Terzaghi, K., and Peck, R.B., Soil Mechanics in Engineering Practice, John Wiloy & Sons, Inc., New York, 1967.

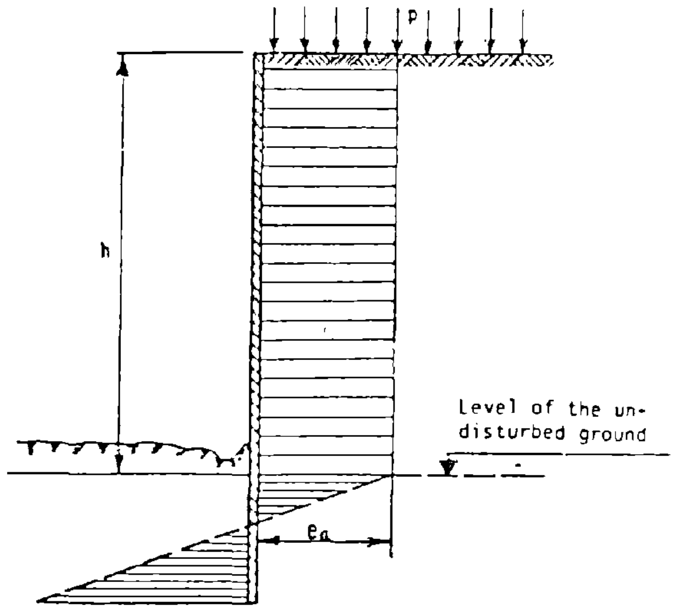


Figure 13. Earth pressure of the dead weight of the ground and distributed live load (rectangular distribution) in the case of singly and multianchored walls.

The earth pressure resulting from linear loads and individual loads is also unformly distributed over the depth of the foundation.

The reliability of attaining the computed passive earth resistance when stressing the anchors must also be considered.

In special cases, the earth pressure can be determined using a different soil mechanics method. In addition, large linear and concentrated loads must be specially allowed for.

For instance the SI System is given by:

$$e_a = 1.3\ (0.5K_a\ \rho\ g\ h + K_a\ p) \quad \ldots\ldots\ldots\ldots \quad (1)$$

Where:

e_a : Active horizontal earth pressure

K_a : Coefficient of active earth pressure

ρ : Bulk density of the moist earth

g : Acceleration ($\sim$ 10 ms^{-2})

h : Assumed depth of excavation

p : Uniformly distributed live load

γ : Unit weight

The dimensioning of the calculated free anchor length, l_{fr}, depends on the purposes of the anchored zone:

- Increase of the safety factor F against slipping
- Establishment of a stable "prestressed" body in anchored walls, etc.
- Use of an adequate mass for stability against uplift.

The calculated free anchor length is determined by the stability conditions. In the case of slip areas in the anchored zone (see Figure 10) the anchor forces, V, in kN/m^1 are to be taken into account. The anchor forces, V, are to be fixed by the project engineer in each case depending on the conditions.

In this case, the upper limit values are as follows:

$$V \leq V_S$$

$$V \leq V_V$$

$$V \leq F\ V_G$$

where

$V_S = F_e\ \sigma_{2.0}$ Load when the stress in the tendon is $\sigma_{2.0}$ (nominal value of the yield stress)

V_V = Limiting load of the fixed anchor

$V_G = F_e\ \sigma_G$: Calculated working load

F_e = Cross-sectional area of the tendon

In the case of slip areas through the anchorage zone (see Figure 10) no anchor forces may be introduced into the stability calculation.

The fixed anchor shall be capable of reliably transmitting the forces of the tendon into the ground up to the failure point of the former.

The anchor system provided and the installation methods are generally to be tested for suitability in the ground in question using test anchors.

If there are no test anchors or no practical experience has been gained in comparable ground, the estimated values given in Table 5 can be used.

Table 5. The load-bearing capacity and fixed anchor length in the ground.

Type of ground	Load-bearing capacity, V_u		Fixed anchor length l_v
	Unconsolidated deposit	Consolidated deposit	
Sandy gravel	up to 600 kN	up to 1000 kN	4... 7 m
Silty sand	up to 400 kN	up to 600 kN	4... 7 m

Note: 1 kN = 0.2248 lb (force)
1 m = 3.279 ft

Rock Anchors

The behavior of the rock anchor with regard to its direction of pull is not only dependent on the strength of the rock, but also on the stratification, fissuring, etc. Rock stabilizing work using anchors whose test load, V_p, is under 200 kN is generally dimensioned not on the basis of static calculations, but on experience. For these anchors the calculations required in this section can be dispensed with.

The working load, V_G, is selected so that the part of the rock in question remains stable.

For stabilizing hillsides: Same procedure as for soil anchors; sliding surfaces are, however, determined mostly by the fissuring and stratification.

For cavities (tunnels and caverns): anchor forces and anchor lengths cannot be determined independently of each other. They both have to be selected beforehand and then tested in successive calculation steps and matched to each other so that the rock around the cavity remains stable and no impermissible deformations occur.

The calculation of the free anchor length, l_{fr}, is the same as that in the case of soil anchors unless it is determined together with the anchor force.

The anchorage in the rock shall be capable of reliably transmitting the forces of the tendon into the rock up to the failure of the former.

The anchor system provided and the installation method are generally to be tested for their suitability in the given rock by means of test anchors.

If there are no test anchors or no practical experience in comparable rock available, the estimated values given in Table 6 can be used.

Table 6. Load-bearing capacity and fixed anchor length in the rock.

Type of rock	Load bearing capacity, V_u		Fixed anchor length, l_v
	High degree of fissuring	Low degree of fissuring	
Granite, gneiss, basalt, hard limestones and hard dolomites	up to 2000 kN	up to 4000 kN	4 ... 7 m
Soft limestones, soft dolomites, hard sandstones	up to 1200 kN	up to 2000 kN	4 ... 7 m

Safety Factors

The classes of anchors are fixed in accordance with the degree of risk and service life and are used in selecting the required safety factor. They are classified according to the magnitude of the consequences which would occur if the anchor fails. Anchor classes are given in Table 7.

Table 7. Definition of anchor classes.

Degrees of risk	Class: Temporary	Class: Permanent
Anchors whose failure would have few serious consequences and would not endanger public safety and order	1	4
Anchors whose failure would have quite serious consequences, but would not endanger public safety and order	2	5
Anchors whose failure would have serious consequences and would probably endanger public safety and order	3	6

The anchor safety factor, S, is the ratio of load-bearing capacity, V_U, to the working load, V_G:

$$S = \frac{V_U}{V_G}$$

The following relationships between anchor forces are to be adhered to:

Temporary Anchors	Permanent Anchors
$V_p \leq 0.95 \ V_S$	$V_p \leq 0.95 \ V_S$
$V_p \geq 1.15 \ V_G$	$V_p \geq 1.40 \ V_G$
$V_G \leq \frac{1}{S} \ V_U$	$V_G \leq \frac{1}{S} \ V_U$
$V_O \leq 0.75 \ V_U$	$V_O \leq 0.75 \ V_U$

where:

$V_p = F_e \ \sigma_p$: Test load during anchor test and stressing test

$V_S = F_e \ \sigma_{2.0}$: Load when stress in the tendon is $\sigma_{2.0}$ (nominal value of yield stress)

$V_G = F_e \ \sigma_G$: Calculated working load

$V_U \begin{cases} \leq V_Z \\ \leq V_V \end{cases}$: Load-bearing capacity of the anchor, the smaller of the two values V_Z and V_V

$V_Z = F_e \ \beta_Z$: Failure load in tendon (nominal value)

V_V : Limiting load of the fixed anchor

The safety factor of the anchored structure against slipping, F, is the ratio of the shear strength of the ground to the shear stress.

The project engineer shall fix the safety factor against slipping in each individual case.

The safety factors F given in Table 8 are the usual ones for retaining walls in soil and only apply to calculation methods in which instability of the sliding body would occur when the safety factor F = 1. They only apply for the area of influence of the structure.

Values of S depending on anchor class are given in Table 8.

Table 8. Safety factors, S and F, as a function of anchor class.

Anchor Classes	S	F
1 and 4	1.6	1.4
2 and 5	1.8	1.4
3 and 6	2.0	1.5

DETAILED ANALYSIS OF WALL-SOIL-ANCHOR SYSTEM

The static analysis of an anchored retaining wall consists of the following steps:

1. Calculation of the cross-sectional dimensions of the wall

    a) Determination of the coefficient of active earth pressure and the passive earth resistance for the soil layers present.

    b) Determination of the effective earth pressure and water pressure, possibly taking into account the influence of ground water flow.

    c) Assumption of a pressure redistribution diagram and calculation of the required depth of embedment.

    d) Calculation of the shear forces and bending moments in the wall.

    e) Estimation of the necessary anchor forces.

2. Calculation of the stability with respect to sliding along deep-seated bedding planes or in weaker layers.

3. Calculation of the overall factor of safety with respect to sliding for the whole system.

4. Check on the stability with respect to hydraulic conditions (piping etc.).

The extent to which all calculations, except 1a), must be repeated for all stages of construction or constructional circumstances depends on the ability of the design engineer to judge the behavior of the structure.

In any case, however, the condition at the deepest level of excavation and at the end of construction must be investigated. Separate investigations for case 4 are generally only required in special cases.

The retaining wall acts as a cantilevel, a singly or multiply supported beam. Its lower end is elastically embedded or due to passive earth resistance or an underlying rock layer is held horizontally or fixed completely.

Earth Pressure Coefficients

The following earth pressure coefficients are determined separately for each soil layer.

A) Active earth pressure

the coefficient of active earth pressure for a horizontal ground surface, K_{ah}, is given by

$$K_{ah} = \frac{\cos^2 \phi}{\left[1 + \sqrt{\frac{\sin \phi \sin(\phi + \delta)}{\cos \delta}}\right]} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where

ϕ = the angle of internal friction for the soil layer

δ = wall friction angle

the coefficient of active earth pressure for a sloping ground surface, $K_{ah,\ \beta,}$ is given by

$$K_{ah,\ \beta} = \frac{\cos^2 \phi}{\left[1 + \sqrt{\sin(\phi - \beta)\ \frac{\sin(\phi + \delta)}{\cos \delta \cos \beta}}\right]^2} \quad \ldots\ldots\ldots\ldots (3)$$

where

β = the angle of the surface of the backfill with respect to the horizontal

The wall friction angle, δ, is positive if the soil moves downwards relative to the wall. The angle δ depends essentially on

1) the shear strength of the soil

2) the surface roughness of the wall

3) the way the wall is introduced into the ground

4) the relative movement between the ground and the wall

For bulkheads, sheet pile walls and cast in-situ concrete walls, generally, it may be assumed that

$\delta = + \frac{2}{3} \phi'$ and for slurry trench walls $\delta = + \frac{1}{2} \phi'$, provided the vertical forces are properly transmitted to the underlaying layers. If it cannot be demonstreated that $\Sigma V = 0$ then a smaller or even a negative value of δ must be taken. However, the absolute values of δ cannot exceed $\frac{2}{3} \phi'$ and $\frac{1}{2} \phi'$, respectively, in the calculation.

B. Earth pressure at rest

The coefficient of earth pressure at rest, K_{oh}, is given by

$$K_{oh} = 1 - \sin \phi \qquad (4)$$

C. "Increased" active earth pressure

The coefficients for an increased active earth pressure, K_{eh}, are obtained from a combination of active and at rest earth pressure

$$K_{eh} = (1 - k)\, K_{ah} + k\, K_{oh} \qquad (5)$$

with $0 \leq k \leq 1$

D. Passive earth pressure

The coefficient of passive earth pressure for a horizontal surface, K_{ph}, is given by

$$K_{ph} = \frac{\cos^2 \phi}{\left[1 - \sqrt{\sin \phi \, \frac{\sin(\phi - \delta_p)}{\cos \delta_p}} \right]^2} \qquad (6)$$

Normally, to calculate the passive earth resistance it is assumed that the bottom of the excavation is horizontal. The sign of the wall friction angle is negative. For $\delta \leq -30°$ the K_{ph} - values may be more critical for a non-planar slip surface with $\delta_p = -\phi$. Both cases are investigated, the smallest K_{ph} - value applying.

The earth pressure magnitudes, e_h, at the upper and lower interfaces of each layer are determined by multiplying the self-weight of the soil by the earth pressure coefficients. They are reduced for active and "increased" earth pressure, due to the cohesive component, by the amounts

$$\Delta e_{h,\beta} = -2c' \sqrt{K_{h,\beta}}\ \cos\delta\ (1 - k) \quad \ldots \quad (7)$$

and

$$\Delta e_h = -2c' \sqrt{K_h}\ \cos\delta\ (1 - k) \quad \ldots \quad (8)$$

respectively,

The reduction applies, however, only in so far as the resultant earth pressure due to self-weight; cohesion and distributed loads do not become negative.

E. Stepped slope surface

In the case of a stepped slope behind the wall and several soil layers, the earth pressure is calculated such that for all sections of the ground surface, lines of earth pressure distribution are constructed from the point of intersection with the wall of the line drawn horizontally from the given section.. Starting with the first section each successive section is taken in order (Figure 14).

The procedure can also be used, in special cases, for the calculation of the passive earth resistance.

F. Water pressures

The water pressure due to ground water on the back of the wall at a depth z is given by

$$w_H + 1.0\ (z - z_{GW}) \quad \ldots \quad (9)$$

Refer to figure 15 for definition of symbols.

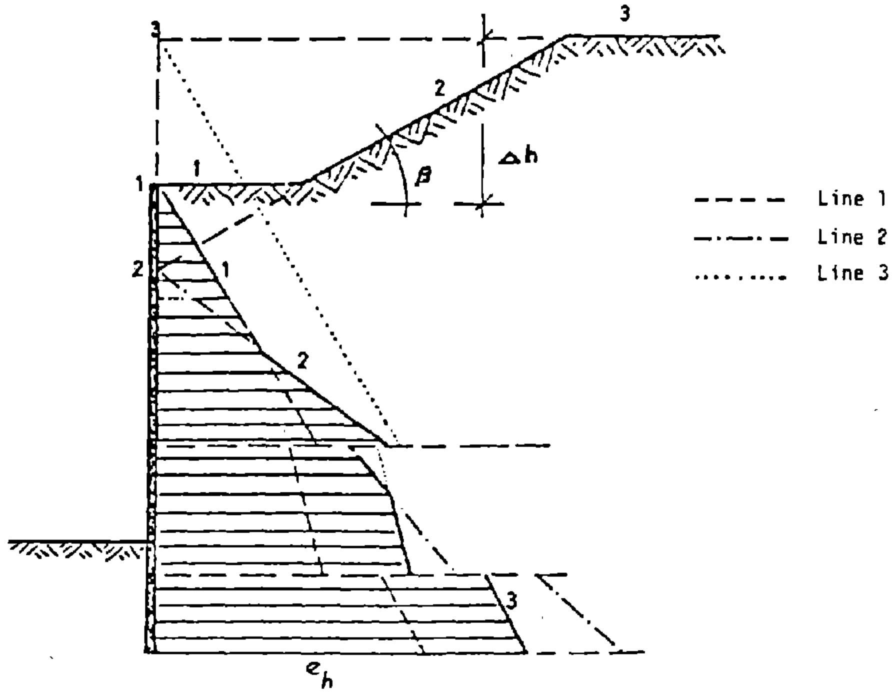


Figure 14. Earth pressure due to self-weight.

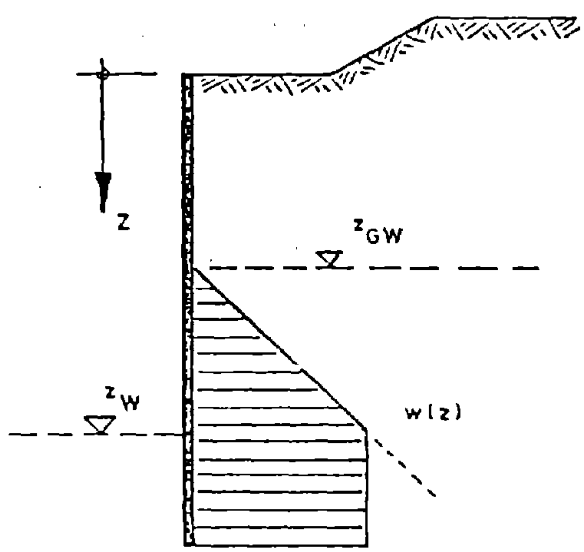


Figure 15. Water pressure w in the case of pervious layers.

The counter water pressure on the front of the wall is

$$w_V + -1.0\ (z - z_W) \quad (10)$$

so that the net water pressure acting on the wall is

$$w(z) = w_H + w_V \quad (11)$$

These expressions are valid only in the absence of ground water seepage.

G. Seepage

In general, the forces acting on a soil grain are its weight and the buoyancy force. If a ground water flow is present there is in addition a seepage force, w, which is a body force acting in the direction of flow

$$w = i\ \gamma_w \quad (12)$$

in which i is the hydraulic gradient and γ_w the unit weight of water.

If the seepage force acts in the vertical direction the unit weight of soil γ is modified by the amount $\Delta\gamma = + i\ \gamma_w$. For downwards flow γ^1 is increased; for upwards flow it is decreased. Likewise, the pressure losses connected with the flow of water can also be handled by correcting the water's unit weight, i.e. $\Delta\gamma_w = i\ \gamma_w$. The unit weight γ_w is reduced for downward flow and increased for upward flow.

For homogeneous soil conditions the hydraulic gradient along a flow line is constant.

If layers are present with different coefficients of permeability the hydraulic gradient in the m-th layer is found as follows:

$$i_m = \frac{\Delta h}{k_m \sum_{i=1}^{n} \left(\frac{d_i}{k_i}\right)} \quad (13)$$

in which Δh is the driving head (i.e. the difference between the water levels on either side of the wall), d_i is the thickness of the i-th layer (assuming flow is normal to the layer) and k_i is the corresponding coefficient of permeability (Figure 16).

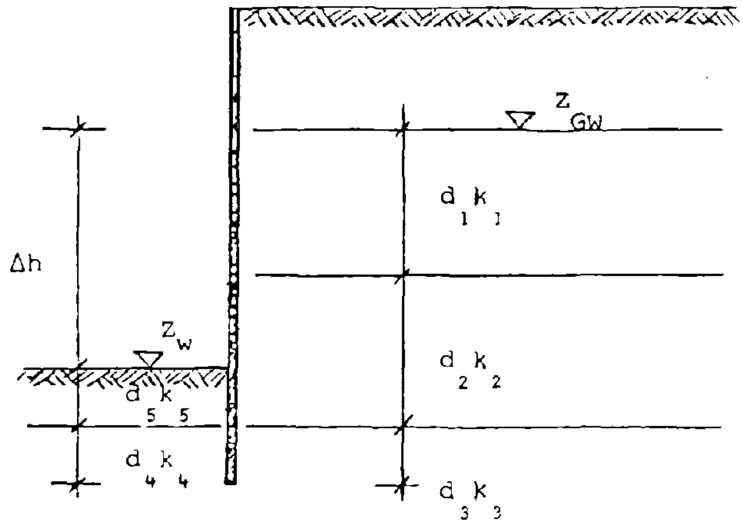
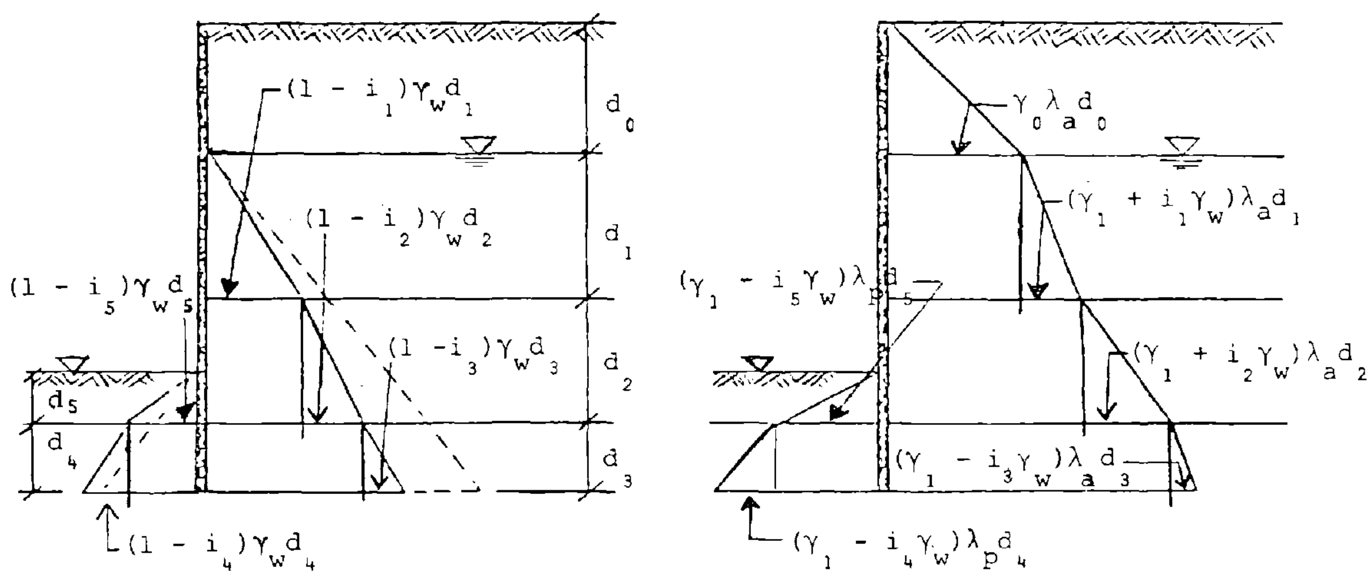


Figure 16. Flow through layers of differing permeabilities.

The distribution of water and earth pressures acting on the wall under the influence of seepage pressures is explained with the aid of Figure 17.



λ = appropriate earth pressure coefficent

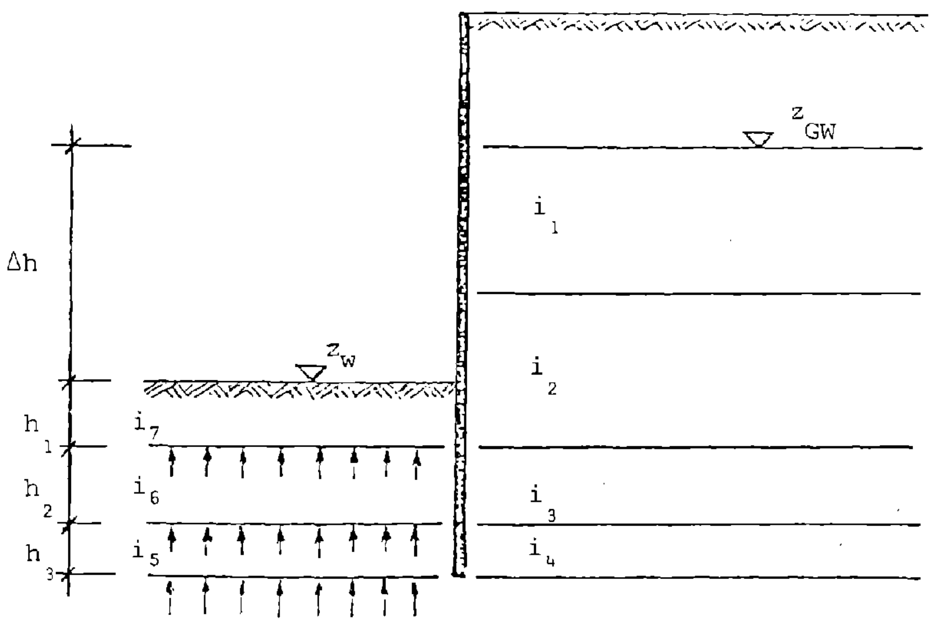
a) water pressure force

b) seepage force influence

Figure 17. Distribution of water and earth pressures with seepage pressures.

If the seepage force acts together with the buoyancy force the condition may arise, whereby the grain matrix in the flow region attains a weightless condition. The corresponding hydraulic gradient is called the critical hydraulic gradient. Under these circumstances the "quicksand" condition may occur, which causes a loosening and "piping" in the soil layers located under the ground water table.

The factor of safety against piping, ν, which is defined as the ratio of the intergranular pressure without seepage to the intergranular pressure for the seepage condition, is checked for every layer interface in the area of the embedded part of the wall, i.e. on the excavation side of the wall. Referring to Figure 18 the potential difference is calculated for each interface. The seepage force is assumed to act vertically, opposing the overburden force. Soil cohesion is not considered in this analysis.



$$\nu_1 = \frac{h_1 \gamma_1}{p_1} \qquad p_1 = h_1 + i_7$$

$$\nu_2 = \frac{h_1 \gamma_1 + h_2 \gamma_1}{p_2} \qquad p_2 = p_1 + h_2 i_6$$

$$\nu_3 = \frac{h_1 \gamma_1 + h_1 \gamma_1 + h_1 \gamma_1}{p_3} \qquad p_3 = p_2 + h_3 i_5$$

Figure 18. Illustration of computations for factor of safety against piping.

Earth Pressure Due to Traffic and Other Applied Surface Loads

Traffic loads and the weight of buildings are represented by strip loads at a distance between x_A and x_E from the wall. They act at a depth z_q below or above the top of the wall.

The following assumptions are made with respect to the earth pressure on retaining walls due to surface loads.

A) Active earth pressure

The influence of the load in terms of earth pressure begins at a depth z_ϕ determined by the line drawn at an angle ϕ from the nearer edge of the load through the various soil layers, its direction changing at layer interfaces as ϕ changes. The lower limiting depth, z_θ, is given by a line inclined at $\theta = 45° + \phi/2$ respectively in each layer, starting from the same point at the inner edge of the surface load, unless the load is of limited extent, in which case it starts at the far edge.

1) strip load (Figure 19a)

For homogeneous ground the total active force per unit length of wall, $\Delta E_{ah,q}$, due to the surface load $Q = q\,(x_E - x_A)$ is

$$\Delta E_{ah,q} = Q \frac{\sin(\theta - \phi)}{\cos(\theta - \phi - \delta)} \cos \delta \quad \ldots\ldots\ldots\ldots \quad (14)$$

For layered ground the arithmetic mean of the values of ϕ, δ and ϕ are taken for the soil between the depths z_ϕ and z_θ. For the assumption of a rectangular pressure distribution the earth pressure, $e_{ah,q}$, between points z_ϕ and z_θ is

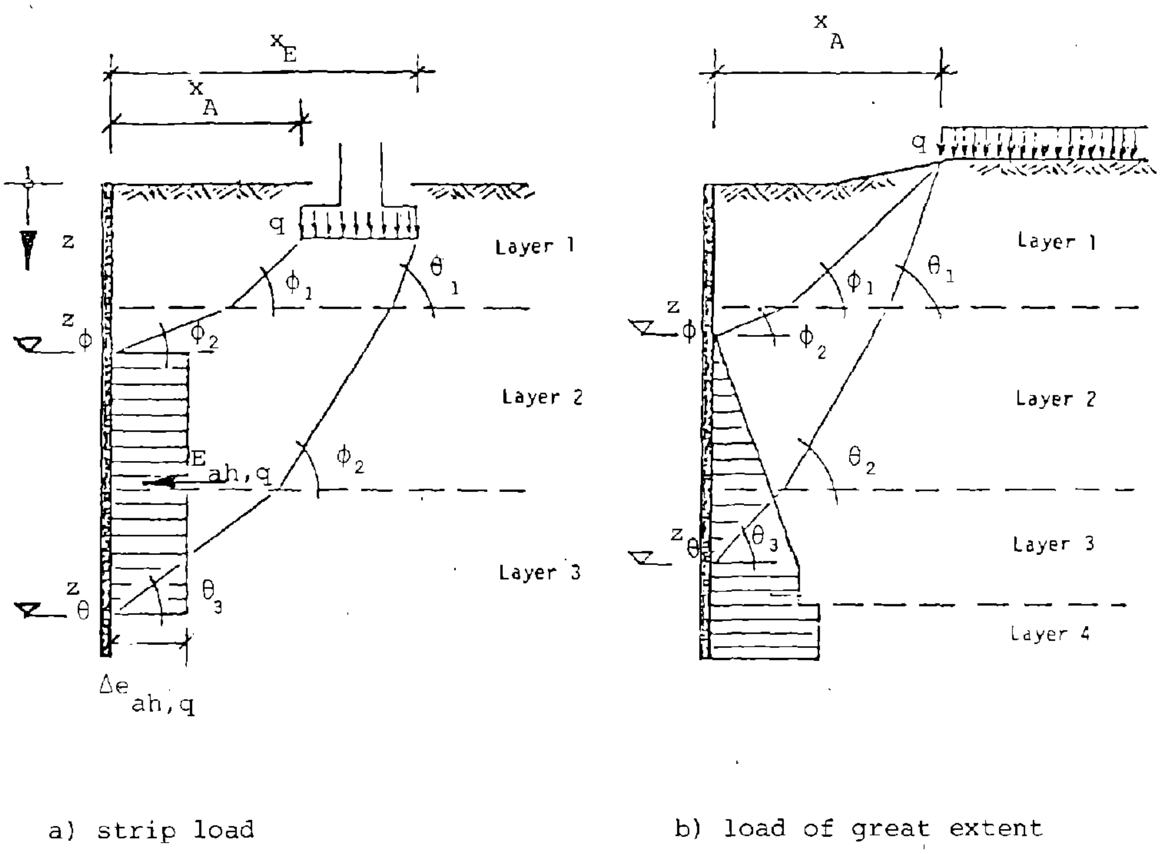
$$e_{ah,q} = \Delta E_{ah,q}/(z_\theta - z_\phi) \quad \ldots\ldots\ldots\ldots \quad (15)$$

2) distributed surface load of great extent (Figure 19b)

The earth pressure due to surface applied pressure q reaches the full value below a depth z_θ equal to

$$e_{ah,q} = q\,K_{ah} \quad \ldots\ldots\ldots\ldots \quad (16)$$

whereby K_{ah} is the active earth pressure coefficient (horizontal terrain) of the corresponding layers. Between z_ϕ and z_θ the earth pressure due to q increases linearly from 0 to the full value.



a) strip load b) load of great extent

Figure 19. Earth pressure distribution due to traffic and other applied surface loads.

3) Line load acting normal to the excavation wall.

It is proposed that a line load P normal to the wall should be treated as a strip load of width 2a, whereby $a = \frac{x_A + x_E}{2}$ is the mean distance from the wall.

$$e_{ah,P} = \frac{P}{x_A + x_E} K_{ah} \qquad (17)$$

Since this assumption is only justified for short load sections, a longer line load should be considered to be composed of several shorter parts.

B) Earth pressure at rest (Figure 20)

1) strip load

The earth pressure coordinates are estimated, using elastic half space theory for cohesionless soil and normally consolidated cohesive soil with the concentration factor n = 4, as follows:

$$e_{oh,q} = \frac{q}{4} (\sin^3 \beta_2 - \sin^3 \beta_1) \quad . . . . (18)$$

and for overconsolidated cohesive soils with the concentration factor n = 3:

$$e_{oh,q} = \frac{q}{\pi} (\beta_2 - \beta_1 + \sin\beta_1 \cos\beta_1 - \sin\beta_2 \cos\beta_2) \quad . . . (19)$$

The angles β_1 and β_2 are defined in Figure 20.

The earth pressure begins to act at the depth of the line load, or for negative values of z_q at the top of the wall.

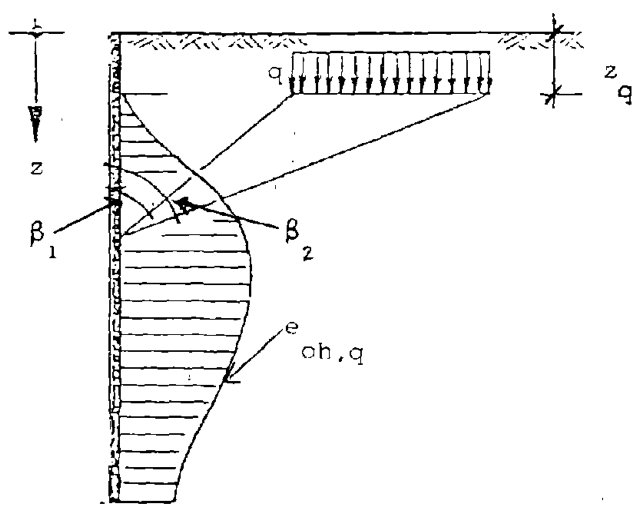


Figure 20. At rest earth pressure distribution due to traffic and other applied surface loads.

2) distributed surface load of great extent

The earth pressure coordinates, $e_{oh,q}$, for distributed loading of great extent (x_E approaches infinity) are determined with the use of the coefficient of earth pressure at rest, K_{oh}, i.e.:

$$e_{oh,q} = qK_{oh} \quad . . . . (20)$$

The distribution of the earth pressure is analogous to Figure 19b.

3. line load acting normal to the excavation wall

For a concentrated load P applied at a distance x from the wall, the horizontal earth pressure distributed, $e_{oh,P}$, for N = 4 is:

$$e_{oh,P} = \frac{2x^2\bar{z}^2}{2\pi(x^2 + \bar{z}^2)^3} P \quad \ldots\ldots\ldots\ldots\ldots (21)$$

and for n = 3:

$$e_{oh,p} = \frac{3x^2\bar{z}}{2\pi(x^2 + \bar{z}^2)^{5/2}} P \quad \ldots\ldots\ldots\ldots\ldots (22)$$

in which $\bar{z} = z - z_P$

By integrating in the x - direction one obtains for a line load $p = P/(x_E - x_A)$. For n = 4:

$$e_{oh,p} = \frac{P}{4\pi} \left[\frac{x}{x^2 + \bar{z}^2} - \frac{z\, x\, \bar{z}^2}{(x^2 + \bar{z}^2)^2} + \frac{1}{\bar{z}} \operatorname{atan}\left(\frac{x}{\bar{z}}\right) \right]_{x = x_A}^{x_E} \quad \ldots\ldots\ldots\ldots\ldots (23)$$

and for n = 3:

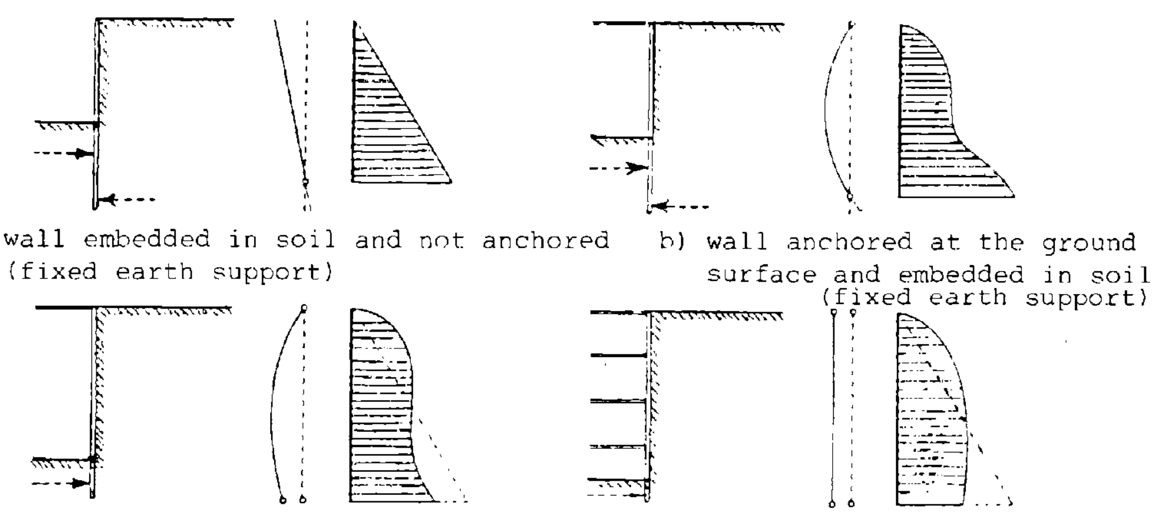
$$e_{oh,p} = \frac{P\bar{z}}{2\pi} \left[\frac{x}{\bar{z}^2(x^2 + \bar{z}^2)^{1/2}} - \frac{x}{(x^2 + \bar{z}^2)^{3/2}} \right]_{x = x_A}^{x_E} \quad (24)$$

"Increased" Active Earth Pressure

The increased active earth pressure for surface loads is composed, analogous to equation (5), of the (1-k)-th earth pressure line for the active earth pressure and the k-th earth pressure line for at rest conditions.

Earth Pressure in Relation to Deformation

The earth pressure acting on sheet pile and excavation walls depends upon the stiffness of the individual systems and may, to a certain extent, be traced back to four basic types, (Figure 21). For unanchored sheet pile walls depending upon embedment alone for stability, a linear earth pressure distribution corresponding to the wall movement is obtained (Figure 21a). A wall supported elastically at the ground surface level and embedded at its lower end can only deform like a simple beam, since there is a point in the wall which does not displace laterally. This point, which acts as a pinned support, is located near to the bottom of the wall (Figure 21b). If the wall is supported at the top and embedded only a small amount at its base or is completely free, then the bottom can displace, rotating about the top end together with a bending action. This combination of deformation modes leads to a greater redistribution of earth pressure (Figure 21c). Multiply-supported walls deform more or less in a uniform manner and the wall moves approximately into a parallel position. The earth pressure distribution thus lies intermediate to the previous pressure diagrams corresponding to the deformation shown in Figure 21d.


a) wall embedded in soil and not anchored (fixed earth support)

b) wall anchored at the ground surface and embedded in soil (fixed earth support)

c) wall with free earth support and anchored at ground surface

d) multiply-supported wall with free earth support condition

Figure 21. Earth pressure in relation to wall deformation.

The magnitude and distribution of the earth pressure on tied-back excavation walls depends on whether the anchor is prestressed or fixed, and if prestressed, what force it is stressed to. The earth pressure, which governs the size of section, does not exhibit the classical form, according to Figure 22 when the anchors are not designed for at least 80% (for active

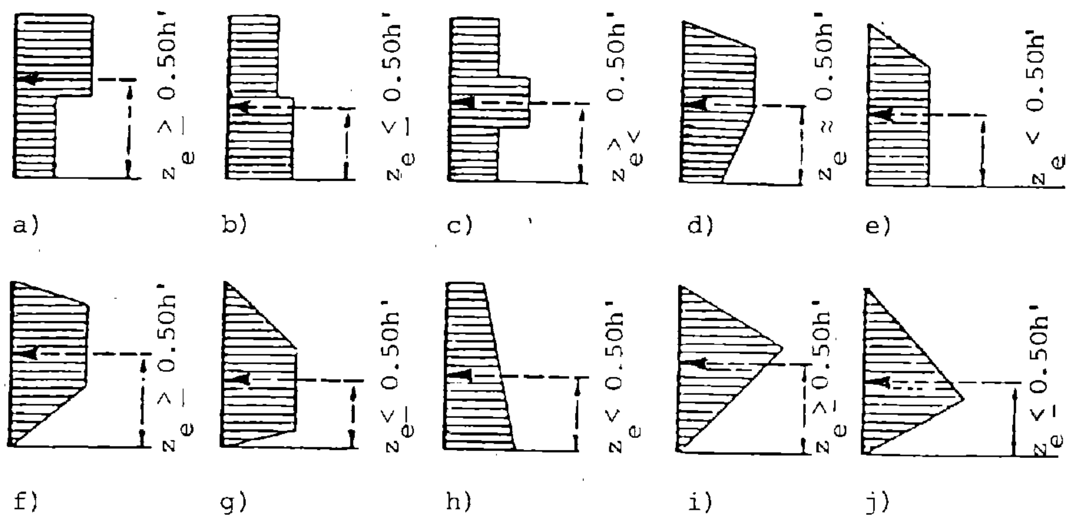


Figure 22. Pressure diagrams for anchored excavation walls (examples).

conditions) or 100% (for increased active pressure conditions) of the calculated forces for each of the constructional stages. If the anchor forces are fixed at substantially lower values the earth pressure distribution is largely determined by the interaction of local factors, such as live load, dead load, soil type, degree of restraint of the bottom of the wall amongst others and cannot be calculated straightforwardly.

Within certain limits, especially in connection with the rigidity of the excavation wall, any desired earth pressure distribution can be brought about by a particular arrangement and prestressing of the anchors.

For instance, if it is required that the distribution be changed considerably towards the top of the wall, e.g. with an earth pressure diagram whose resultant lies in the upper half of the wall, then it is necessary, for multiply anchored walls, to make the upper anchors longer than the lower ones. Otherwise, the anchor lengths are designed with respect to conditions of stability for the deep-seated bedding planes and of overall stability. The effective or redistributed earth pressure is superimposed with the pressures due to surface loads.

Base Restraint and Passive Pressures

For an excavation wall whose bottom is not embedded in bedrock it must be demonstrated that the passive resistance provides a base restraining force of adequate safety factor, η. That is $A_{acting} \leq A_{permissible}$

A_{acting} is given for the following conditions:

- for elastically embedded sheet-pile and cast-in-place concrete walls, A_{acting} is the sum of the positive spring forces (i.e. no tension forces are allowed).

- for sheet-pile and cast-in-place concrete walls considered pinned at the level of the resultant passive force, A_{acting} is the soil reaction or the passive resistance.

- for pile walls free to rotate about the line of the resultant passive force, A_{acting} is given by the soil reaction (passive resistance) reduced by the active earth pressure at the bottom of the wall.

- for fully restrained sheet pile and cast-in-place concrete walls, A_{acting} is given by the shear force at a height z_o corresponding to zero load point, plus the restraining force at the theoretical bottom end (i.e. the force required to maintain equilibrium of horizontal force components) plus the water pressure between z_o and the theoretical bottom end.

- for fully embedded pile walls, A_{acting} is given by the shear force at the level of excavation diminished by the active earth pressure between excavation level and the theoretical bottom end, plus one-half of the restraining force at the theoretical bottom end.

$A_{permissible}$ is given for the following conditions:

- for sheet pile and cast-in-place concrete walls elastically embedded or free to rotate at the base and under the condition of earth pressure at rest, and for pile walls, $A_{permissible}$ is given by the area of the passive earth pressure triangle e_{ph}/η at the actual base of the wall.

- for sheet pile and cast-in-place concrete walls elastically embedded or free to rotate at the base under active and increased active earth pressure and for fully restrained sheet pile and cast-in-place concrete walls, $A_{permissible}$ is given by the area of the earth pressure triangle $e_{rh} = e_{ph}/\eta - e_{ah}$.

  For completely fixed sheet pile and cast-in-place concrete walls the value $A_{permissible}$ according to the appropriate equation is distributed triangularly and superimposed upon the effective earth pressure.

Earth Pressure Redistribution

The active and increased active earth pressures due to self-weight of soil and applied surface loading, as calculated in the previous sections, may be subject to redistribution; see Figure 22 for examples.

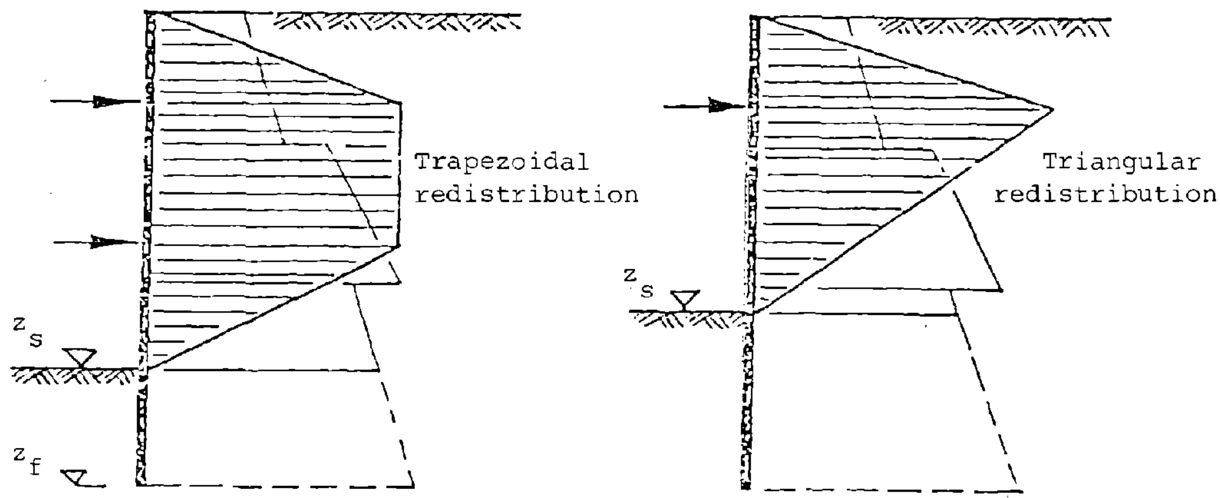
There is no redistribution in the case of

a) unanchored cantilever-type walls, and
b) always when a redistribution is excluded

Also, water pressures are not subject to redistribution

A) <u>Redistribution of active and increased active earth pressures in the case of Berlin-type retaining walls</u>

The earth pressure diagram resulting from self-weight, live load and surface load due to buildings is, down to excavation level, changed to a diagram having the same area, and shaped as shown in Figure 23.



a) doubly supported   b) singly supported

Figure 23. Example of earth pressure redistribution for a Berlin-type retaining wall.

B) <u>Redistribution of active and increased active earth pressure in the case of sheet pile and cast in-situ concrete retaining walls</u>

Below the excavation line a passive resistance (passive earth pressure divided by the safety factor, η) reduces the earth pressure to the value

$$e_{rh} = e_{ah} - e_{ph}/\eta$$

At a depth z_o the pressure reduces to zero.

Figure 24 shows the redistributed earth pressure diagram for this case.

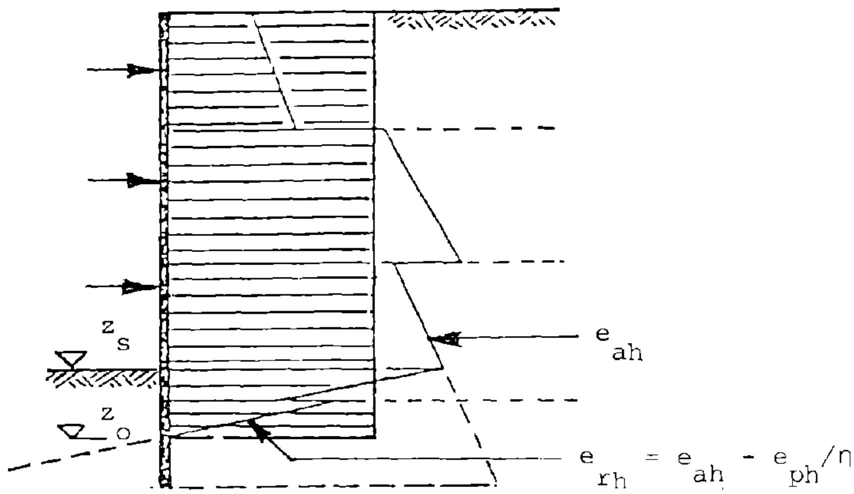
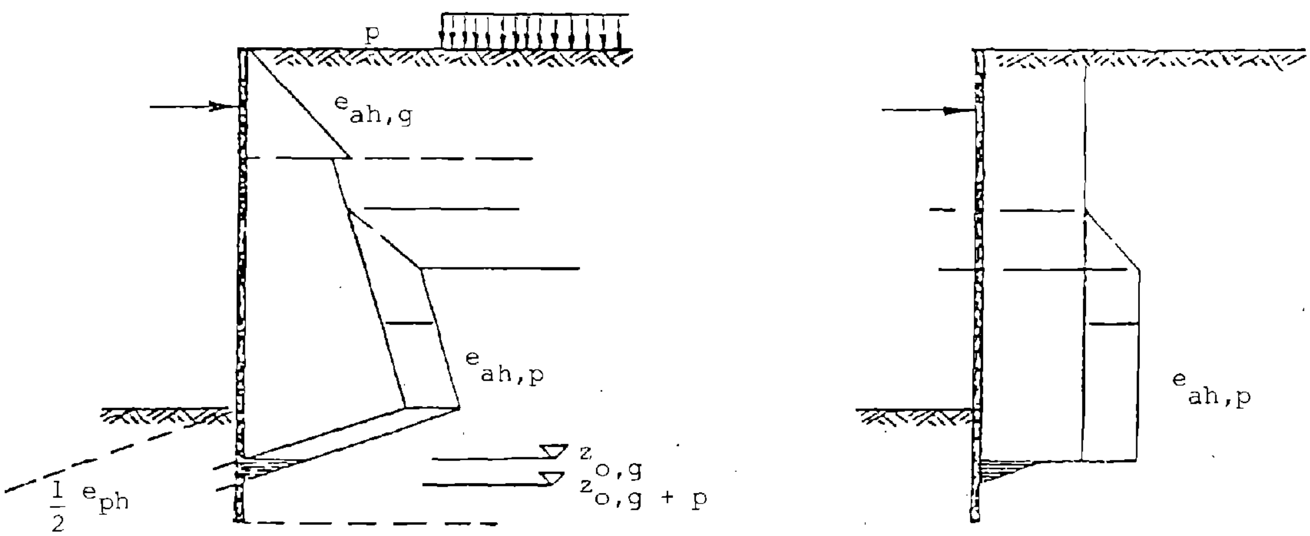


Figure 24. Example of earth pressure redistribution for sheet pile and cast in-situ retaining walls under active earth pressure conditions.

C) <u>Redistribution of active earth pressure due to self-weight of soil in the case of sheet pile and cast in-situ concrete retaining walls</u>

If the earth pressure due to self-weight of soil only is redistributed the method illustrated in Figure 25 is followed.



a) calculated earth pressure b) redistributed earth pressure

Figure 25. Example of the redistribution of earth pressure due to self-weight of soil.

D) Earth pressure at rest in the case of sheet pile and cast in-situ concrete retaining walls

If the wall is supported at two or more positions the earth pressure at rest due to self-weight of soil remains constant from the lowest anchor level to the bottom of the wall (Figure 26). To this reduced pressure diagram the earth pressure due to traffic and the dead weight of buildings is superimposed over the full height of the wall.

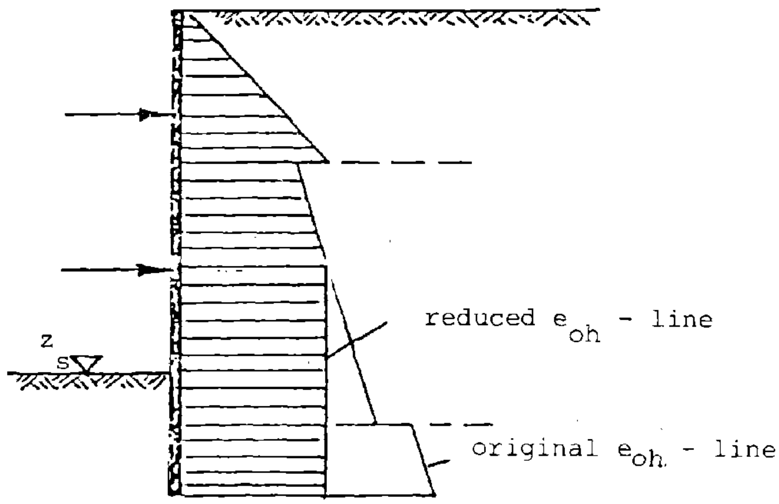


Figure 26. Reduction of the earth pressure at rest.

Statical Analogy of Retained Wall

The static analogy of the anchored wall is that of a continuous beam, which is fixed against lateral displacement (earth anchors) at the support positions and loaded by earth and water pressures. Support movements can be regarded as follows for varying conditions. Concentrated forces or moments, e.g. due to basement floors which are connected to the wall, may act on the wall along its height. The top of the wall can be assumed to be free or pinned or fixed against rotation or completely fixed (no displacement laterally or rotation). For the bottom support condition beneath each excavation level the following assumptions may be applicable:

- The bottom is supported by an elastic spring.

- The bottom restraint according to Blum's theory may be calculated. Beneath the point of zero loading, z_s, (= excavation depth in the case of Berlin type retaining walls, or the position of zero lateral pressure for sheet pile and diphragm walls) a traingular counter-pressure is assumed.

Design of Diaphragm Walls Free at the Top and Fixed at the Bottom (fixed earth support)

Earth resistances are developed on the left and right sides of a diaphragm wall embedded sufficiently to be fixed in the ground, i.e. E_{pl} and E_{pr} in Figure 27. To determine these resistances, the position of the transition from E_{pl} to E_{pr} must be known. An exact analysis is not possible. The analysis for a wall pinned at the top end and fixed at the bottom is usually carried out with sufficient accuracy using Blum's theory.

A) Blum's elastic line theory by graphical procedure

The loading diagram shown on Figure 28 was developed by the methods already given. The diagram is constructed by using the wall, earth, water, and tie rod elevations in unit lengths, the lateral earth pressures in force per unit area and the total lateral loads in force per unit width of wall to any convenient scale as shown. To take into account the increase in the passive pressure of the earth when the piling is driven into undisturbed, compact earth, Blum recommends increasing the passive earth pressure coefficient by a factor up to two. In this case the passive resistance on the left side of the wall is considered to be fully effective down to the theoretical bottom point, and the slope of this portion of the loading diagram is given by $(1.5\ K_{ph} - K_{ah})$. At the bottom point the passive resistance is replaced by a concentrated load, whose magnitude is determined by graphical solution.

The upper support (anchor or bracing) is considered to be a non-yielding, pinned support and the foot of the wall is considered to be completely fixed. The depth of embedment, t_z, is initially assumed and later corrected in the course of the solution.

The total height of the wall (length of beam) is divided into any number of convenient sections which need not be all of equal width. For each area the earth pressure is considered as a total concentrated load acting through the center of gravity, the magnitude of the load being determined by scaling off the mean length of the strip. A vector diagram (E-force polygon) is then drawn as indicated. On a horizontal base line, commencing at the right and moving from left to right, the successive, loads for the sections from the bottom to the top (E_{10} to E_1) of the equivalent beam are laid off end to end. A pole distance, H_E, is chosen to some convenient scale approximately equal to $1/3\ \Sigma\ E_a$, and the remainder of the loads E_{11} to E_{17} are laid off as indicated. The lateral location of the pole, E_1, is chosen at any convenient location.

The moment diagram is then constructed by starting at point 0 and constructing the first segment parallel to ray number 1. The moment diagram is completed by constructing succeeding segments parallel to rays 2 through 17 respectively. The point A is located by extending the first segment to intersect the anchor force. The point B is fixed as the intersection of the moment diagram with the elevation of zero force on the loading diagram. The line extended through A and B is used to locate point C as its intersection with the moment diagram as indicated on Figure 28. The maximum moment at segment 6 is given by the y ordinate (2500 mm) times 10.0 M_p equals 25.0 M_p - m.

The graphical method of constructing the deflection diagram is based on the fact that the area enclosed by the moment diagram can be designated as an "elastic weight" with which the beam is loaded. The deflection curve then bears the same relation to the moment curve as the moment curve does to the pressure diagram. These "elastic weights" F_1, F_2, etc., pass through the centers of gravity of the strips into which the area between the moment curve and the base line is divided. These strips need not be of equal width, and in Figure 28 their boundaries are the lines of action of the loads E_4, E_5, etc. Each F value plotted is just equal to the area of the appropriate strip between the moment diagram and the base line. When drawing the F-force diagram it is convenient to start at the bottom of the beam, i.e. at F_{17}. In order to make the base line of the deflection diagram vertical it is necessary that the last ray (F_{17}) of the F-force diagram be vertical. The zero line of the deflection diagram intersects the anchor line at a distance $\Delta f'$ = 1.40 m from ray 17 extended.

Thus the wall would yield at point A. However, as the wall must be fixed at A (unyielding anchor support) the final closing line must be directed through A such that $\Delta f'$ = 0. Thus the closing line must be displaced at the foot of the wall by an amount Δy either to the left or to the right of C.

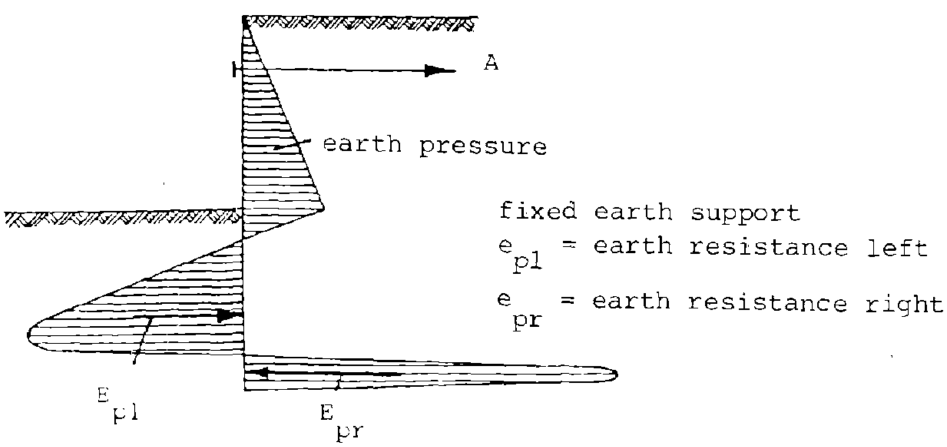


Figure 27. Distribution of earth pressure and passive resistance for a diaphragm wall, top free (restrained only at anchor level) and bottom fixed.

If we designate the side $\overline{AC}$ of the error triangle ACC_o as 1, we obtain its area $F = 1\Delta y/2$. If this area is assumed to act as a load, the necessary deflection $\Delta f' = 0$ results. Thus it follows that

$$\Delta f' \; H_F + \frac{1\Delta y}{2} \; \frac{2}{3} \; 1$$

$$\therefore \quad \Delta y = \Delta f' \; \frac{3H_F}{1^2} \quad \ldots \quad (25)$$

For this case

$$\Delta y = 1.4 \; \frac{3(4)}{(10.7)^2} = 146 \text{ mm}$$

By displacing the point C by this amount one obtains the point C_o. If $\Delta f'$ lies to the left of the base line, ΔY must be the distance to the right of C, and vice versa. In this way the correct position of the closing line and of t_o is obtained. The depth of embedment is

$$t = t_N + 1.2 \; t_o \quad \ldots \quad (26)$$

$$1.12 + 1.2 \; (4.02) = 5.94 \text{ m}$$

The factor 1.2 provides a margin of safety.

By transferring the final line to the E-force diagram one obtains the anchor tensile force $A = 19.30 \; M_p$. The maximum bending moment is obtained from the moment area, i.e.

$$\max M = \max y \; (H_E) \quad \ldots \quad (27)$$

B) Blum's equivalent beam method

The equivalent beam method is a simplification of the graphical solution described in the previous section. Since the moment distribution of the wall, which is free at the top end and fixed by embedment in the ground, shows a zero point near the base, at which - apart from the axial force - only a shear force is effective, the wall can be replaced by two beams hinged together at the point of zero moment. The upper part can be taken as a simply-supported beam, while the lower part acts as a beam pinned at one end and fixed at the other. Since the design of the diaphragm wall is decided by the maximum bending moment it is sufficient simply to design for the upper beam (Figure 29). The point of zero moment, B_o, the lower end of the beam, can be assumed, according to Blum, to lie at the depth of zero loading.

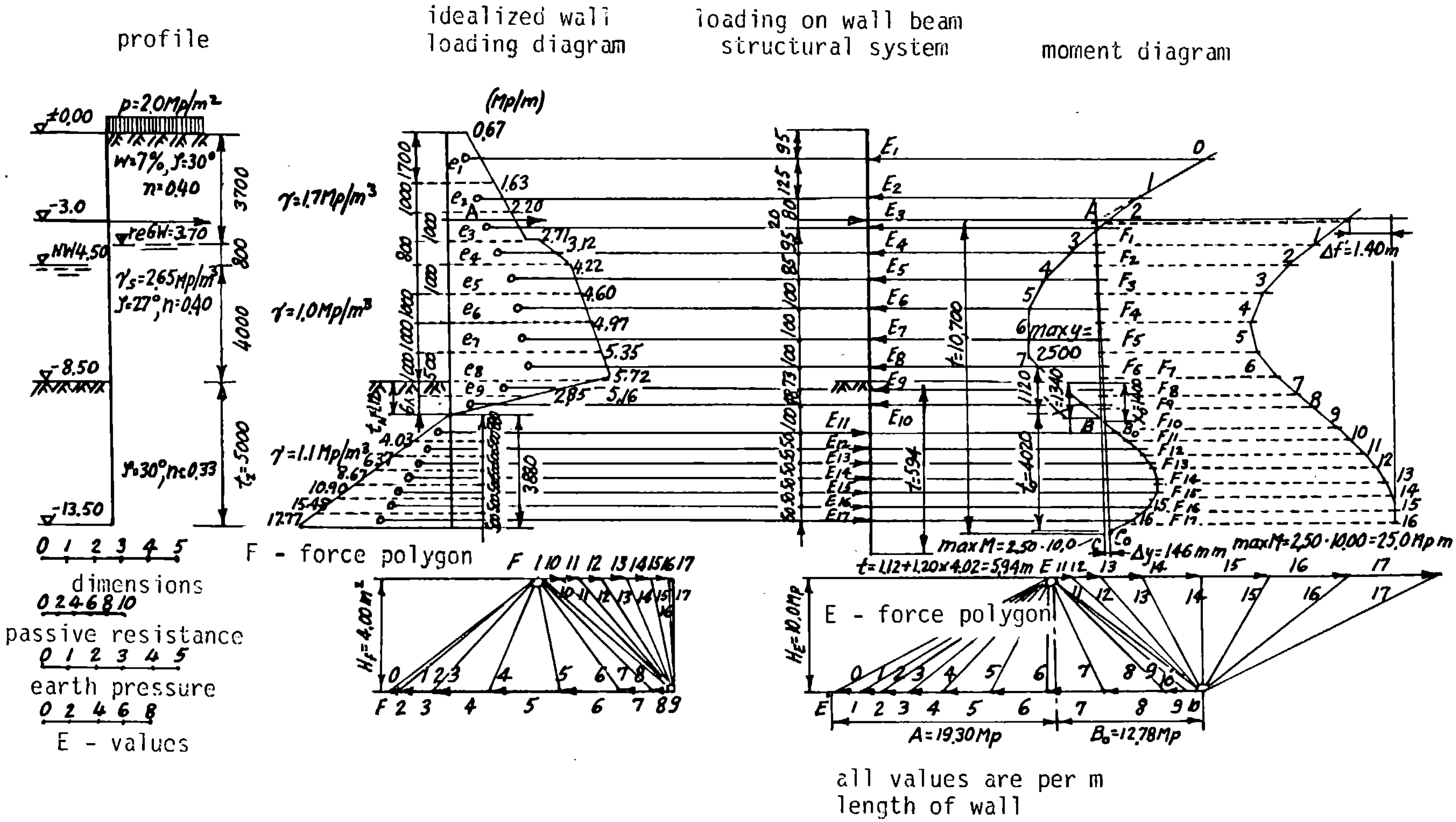


Fig. 28 Design of singly-anchored wall by means of graphical solution for bending moments

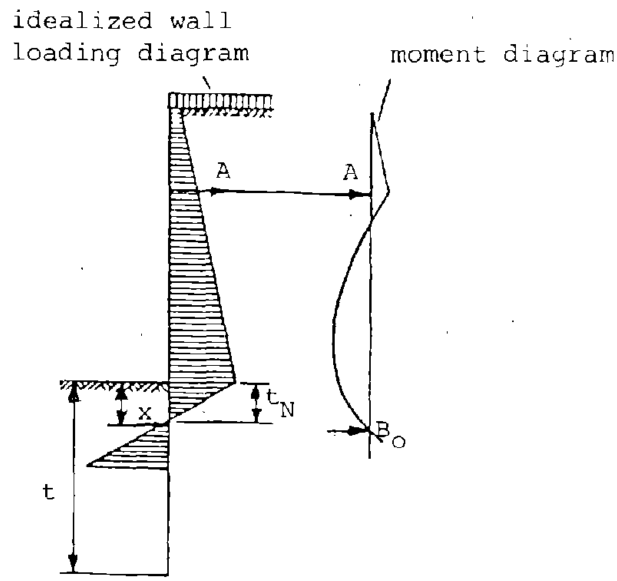


Figure 29. Equivalent beam method.

The beam shown in Figure 29 is then statically determinate with the assumed loading and end conditions.

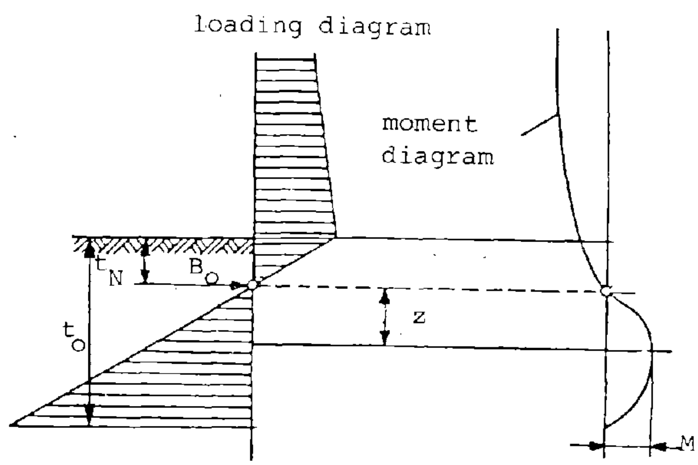


Figure 30. Loading and moment distribution.

The theoretical depth of embedment, t_o, can be determined from the equilibrium conditions for the lower beam (Figure 30). The moment, M, at a distance, z, from B_o amounts to

$$M = B_o z - \gamma(K_{ph} - K_{pa})\frac{z^3}{6}$$

For M = 0 obtain

$$z = \sqrt{\frac{6B_o}{\gamma(K_{ph} - K_{pa})}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (28)$$

and

$$t_o = t_N + \sqrt{\frac{6B_o}{\gamma(K_{ph} - K_{pa})}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (29)$$

Thus, providing a factor of safety, the embedded depth becomes

$$t = t_N + 1.2\sqrt{\frac{6B_o}{\gamma(K_{ph} - K_{ah})}} \quad \ldots\ldots\ldots\ldots\ldots \quad (30)$$

The maximum bending moment is determined at the point of zero shéar force (Q = 0). For alternating layers of soil, especially near the base, the point of zero moment can only be estimated very approximately. In such cases the graphical procedure is preferable.

Walls Acting as Free Earth Supports

If the wall is not fixed at its lower end, the smallest depth of embedment and the largest bending moments are produced. In this case, the wall is embedded only deep enough so that the earth pressure (force), E_a whose action is to rotate the wall clockwise about A, is just kept in equilibrium by the passive earth resistance, E_p (Figure 31). The condition $\Sigma M = 0$ at the anchor level gives directly the necessary depth of embedment, while the condition $\Sigma H = 0$ gives the anchor force. The maximum bending moment (max. M) lies at the position Q = 0.

A) Blum's method

The moment diagram is obtained as for a diaphragm wall fixed at its bottom end (Figure 28). The closing line, however, is drawn from A to be tangent to the moment diagram. The depth of embedment is determined by the point B at which the tangent line touches the moment diagram (Figure 32).

In order to determine the loading diagram the depth of embedment, t_z, is assumed to vary between 0.5 . . . 0.6 h. The pole distance in the force diagram is set at 0.5 ΣE_a.

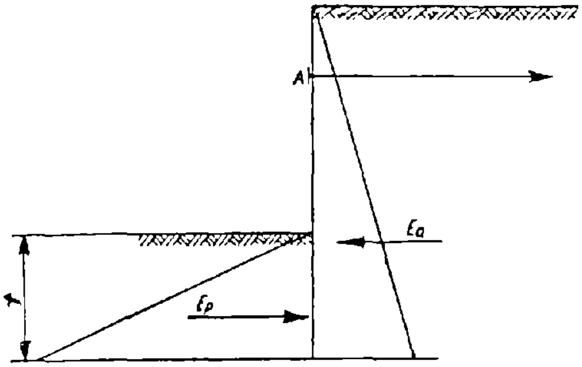


Figure 31. Diaphragm wall free earth support.

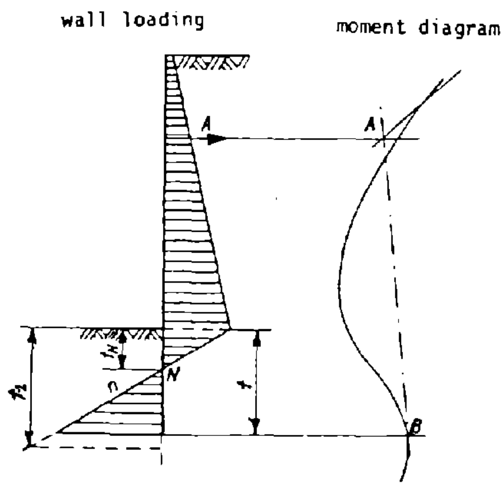


Figure 32. Loading and moment diagram of wall acting as free earth support.

If the point of tangency of the line drawn from A to the moment diagram cannot be determined with any great accuracy, then the depth of embedment cannot be determined (Figure 33).

In this case the depth of embedment is found with the aid of the relation

$$t = t_N + \sqrt{\frac{2B}{n}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (31)$$

If there is no fixity at the bottom end of the wall then the maximum bending moment is greater, the depth of embedment, however, is less than for a pile fixed at its bottom. The choice of a diaphragm wall freely supported at its bottom end is dependent upon firm ground conditions.

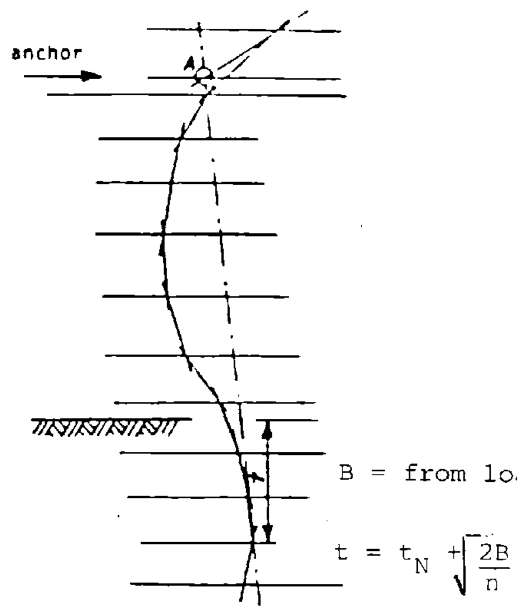


Figure 33. Determination of "t" for uncertain intersection of the closing line.

A Diaphragm Wall Partially Restrained at Its Bottom End

In exceptional cases partial fixity of the wall may be required. If for example the wall thickness is insufficient to permit the simple support condition at its foot or it cannot be made long enough for complete fixity, then a partial embedment condition is chosen.

The analysis procedure is practically the same as for a wall unrestrained at its foot.

Since in this case the depth of embedment is given, then the position of the point C or the value of t_o can be estimated. Thus,

$$t_o = \frac{t - t_N}{1.20} \quad \ldots \quad (32)$$

In investigating a diaphragm wall with partial fixity the moment diagram is drawn using the Blum's method previously described and the estimated depth t_o is inserted. In this way the point C on the moment diagram is established. By drawing the connecting line A-C the values of B, max y etc. are found.

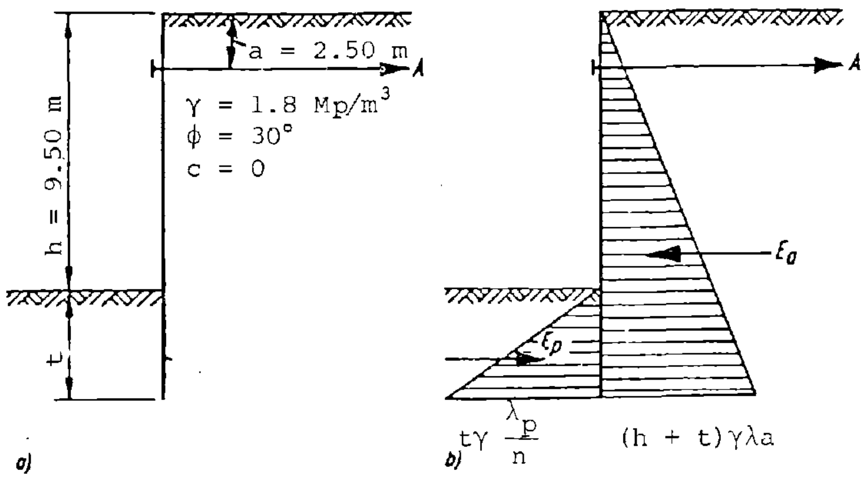


Figure 34. Anchored diaphragm wall with free earth anchor support condition.

Multiply-Anchored Walls

In the case of multiply-anchored walls the same assumptions can be made as described previously. The exact calculation is carried out by varying the depth of embedment until the given boundary conditions are fulfilled, i.e. for complete restraint of the foot (fixed earth support) the bending moment condition applies, and for the free earth support condition the resultant of the passive resistance must equal the support forces in the beam. For the fixed earth support the depth of embedment below the center of moments is increased by 20% of the depth below the point of zero earth pressure.

Calculation of the Anchor Forces

The calculation of the anchor forces, which support the earth and water pressures, is based upon the determination of the shear forces in the continuous beam, whose bottom end fulfills the variously defined support conditions (fixed or free, etc.). The assumed redistribution of the earth pressure, which is induced by the pretensioning in the upper row of anchors, causes the increase in the ultimate value of the active earth pressure.

This situation can be taken care of in two ways:

a) by increasing the effective redistributed earth pressure as described previously.

b) by increasing the calculated shear forces in the beam by a coefficient F in the range 1.0 to 1.3, depending upon the position of the anchors.

In the middle third of the wall height the shear forces can be increased by about one-third.

The calculation of the required anchor forces is made by dividing the shear forces by the cosine of the slope angle of the anchors with respect to the horizontal.

The magnitude of the anchor forces determined in this way should be checked by means of a stability investigation.

Stability of Anchored Slopes and Retaining Walls

In the investigation of the stability of anchor-retained walls it is advantageous to use the method of slices, whereby not only the heterogeneous nature of the soil, but also surchage loads and the influence of individual anchor forces and structural elements can be considered.

The position and shape of the most unfavorable sliding surface is strongly dependent upon the soil constants, pore water pressures and also the external forces, and, except in trivial cases, are not known beforehand. The investigation requires, therefore, a large number of similar calculations, for which an electronic computer and a suitable software package finds good application.

In the general method of slices the potential sliding mass is divided up into vertical strips or slices. In all slices the forces due to self-weight of earth etc., surcharge loading, pore pressure and earthquake effects are determined according to the criteria of the particular method chosen and summed over the whole of the sliding body. Likewise, the influence of external forces, such as horizontal forces, resistances from parts of a struture and anchor forces, can be estimated in the calculation.

All methods of analysis can be applied, which are based upon the method of slices, and for which the safety factor is given by the relationship (5).

$F = \frac{Mr}{Md}$ (resisting moment to driving moment)

or $F = \frac{Tb}{Ta}$ (max. possible shear force to shear force acting)

---

(5) Otta, L., "Stability Analysis of Tied-back Walls, Slopes, Dams and Strip Foundations," Third International Conference on Numerical Methods in Geomechanics, Aachen, 2 - 6, April, 1979.

In the program the safety factor is represented as follows:

$$F = \frac{\Sigma Z + \Sigma DZ}{\Sigma N + \Sigma DN} \quad \ldots\ldots\ldots\ldots \quad (33)$$

$$F = \frac{\sum \frac{Z1 + Z2 + Z3}{ZU} + \Sigma DZ}{\Sigma\ (N1 + N2 + N3) + \Sigma DN} \quad \ldots\ldots\ldots\ldots \quad (34)$$

For the conventional methods of analysis the value of the components of the forces in the slices, as indicated in Figure 35, are given in Table 9.

In the methods due to Bishop (6) and Janbu (7), the following equation is used

$$S = \Sigma N_i + \Sigma DN_i - (\Sigma Z_i + \Sigma DZ_i)/F \quad \ldots\ldots\ldots\ldots \quad (35)$$

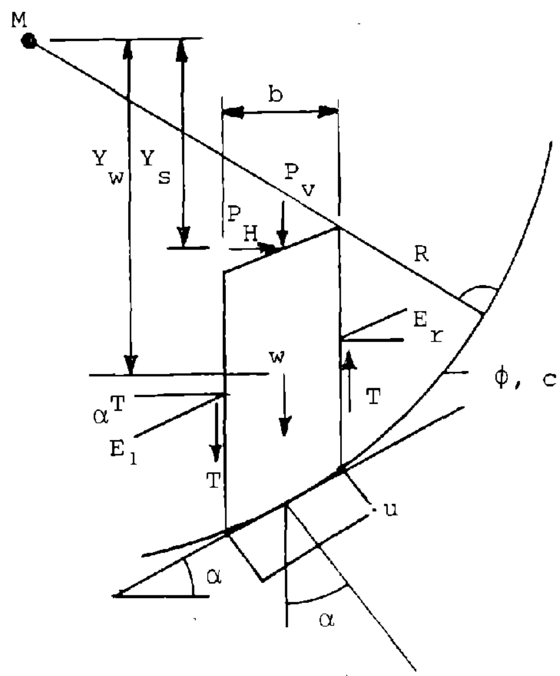


Figure 35. Forces in the slice.

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(6) Bishop, A.W., "The Use of the Slip Circle in the Stability Analysis of Slopes," European Conference on Stability of Earth Slopes, Vol. I, Stockholm, 1954.

(7) Janbu, N., "Application of Composite Slip Surfaces for Stability Analyses," European Conference on Stability of Earth Slopes, Discussion, Vol. III, Stockholm, 1954.

Table 9. Partial components of forces on slices.

Partial component of		Fellenius (1940)	DIN 4084 (1974)	BISHOP (1954)	JANBU without iteration	JANBU (1954)
weight and vertical load	Z1	WS $\cos^2 \alpha$	WS	WS	WS $\tan \phi$	(WS+T) $\tan \phi$
pore pressure	Z2	- u b	- u b	- u b	- u b $\tan \phi$	- u b $\tan \phi$
cohesion	Z3	c b/$\tan \phi$	c b/$\tan \phi$	c b/$\tan \phi$	c b	c b
angle ϕ and α	ZU	$\cos \alpha/\tan \phi$	$\cos \alpha/\tan \phi$ + sin	$\cos \alpha/\tan \phi$ /F + $\sin \alpha$	$(1+\tan \alpha \tan \phi)\cos^2\alpha$	$(1+\tan \alpha \tan \phi/F)\cos^2\alpha$
weight and vertical load	N1	WS $\sin \alpha$	WS $\sin \alpha$	WS $\sin \alpha$	WS $\tan \alpha$	(WS+T) $\tan \alpha$ + E
horizontal water pressure	N2	-WH Ys/R	-WH Ys/R	-WH Ys/R	-WH	-WH
horizontal earth-quake forces	N3	EQH WS YW/R	EQR WS YW/R	EQH WS YW/R	EQH WS	EQH WS

where WS=W + PV + W EQV and EQV, EQH vertical and horizontal earthquake coefficients.

The solution for S = 0 is sought, whereby the value of T_x is used in Janbu's method where

$$T_x = -\tan\alpha^{T_x} \Sigma (N_i - \frac{Zi}{F}) \quad (36)$$

and α^{T_x} is the inclination of the idealized line through the center of gravity of the slice.

The influence of external forces acting on the sliding mass as a whole is shown in Table 10 with the values corresponding to the information in Figure 36.

The safety factor for the system as a whole can be checked using circular or composite slip surfaces and the method due to Krey. For composite surfaces the circular arc can be extended, if one wishes, by straight portions to the ground surface corresponding to the passive (left) or active (right) earth pressure theory, see Figure 37 - 40.

If the center of a circular surface is below the right hand intersection with the ground surface an overhanging failure surface would result. In this case the overhanging portion is replaced by a straight vertical segment - see Figure 37. Of course many center locations are tried in order to obtain the true critical failure surface - see Figure 39.

Table 10. Influence of external forces

Influence of		Fellenius (1940)	DIN 4084 (1974)	BISHOP (1954)	JANBU without iteration	JANBU (1954)
Horizontal loads	DZ	-	-	-	-	-
	DN	$\Sigma(Hi\ Yhi/R)$	$\Sigma(Hi\ Yhi/R)$	$\Sigma(Hi\ Yhi/R)$	Σ Hi	Σ Hi
Structural elements (e.g. piles)	DZ	Σ Si	Σ Si	Σ Si	-	-
	DN	-	-	-	Si	Si
Anchors according to e.g. (2)	DZ	$\Sigma\ ANCO_i$	$\Sigma\ ANCO_i$	$\Sigma\ ANCO_i/F$	-	-
	DN	$\Sigma\ ANSI_i$	$\Sigma\ ANSI_i$	$\Sigma\ ANSI_i$	-	-
Anchors according to e.g. (1)	DZ	$\Sigma\ (ANCO_i + ANSI_i)$	$\Sigma\ (ANCO_i + ANSI_i)$	$\Sigma\ (ANCO_i/F + ANSI_i)$	-	-
	DN	-	-	-	-	-
Anchors for method Janbu	DZ	-	-	-	$\sum \frac{-A_i \sin(NA)_i \tan\phi_i}{ZU_i}$	
	DN	-	-	-	$\Sigma-(A_i(\cos(NA)_i + \sin(NA)_i \tan\alpha_i$	

where $ANCO_i = A_i \cos\psi_i \tan\phi_i$ and $ANSI_i = A_i \sin\psi_i$.

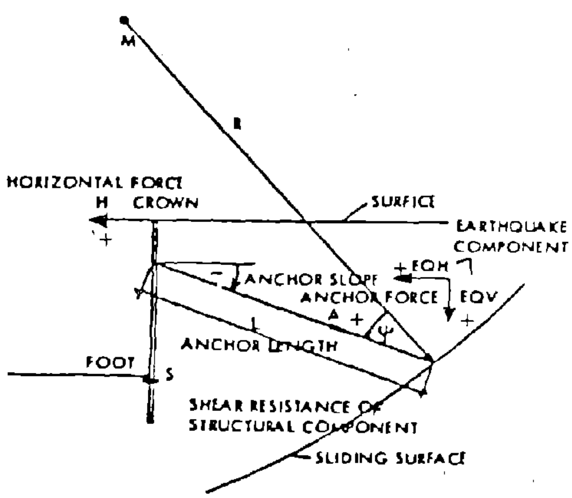

Figure 36. Geometrical description and positive direction of forces acting.

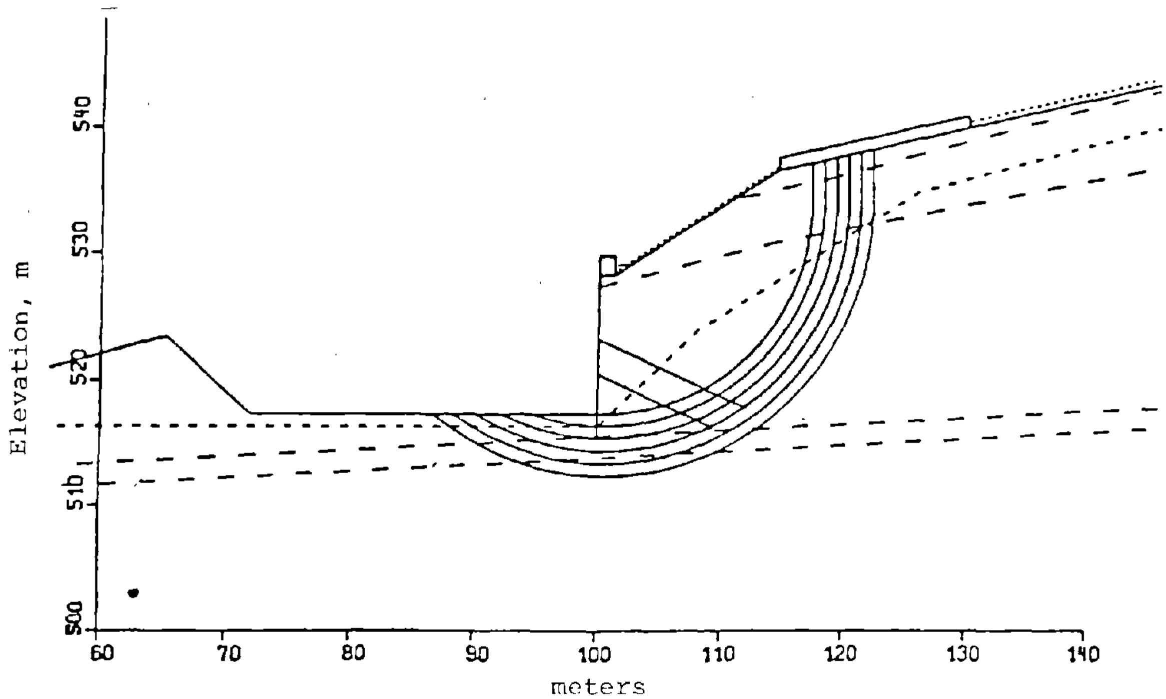

Figure 37. Stability analysis of the system as a whole for one center point; all investigated circular sliding surfaces are shown.

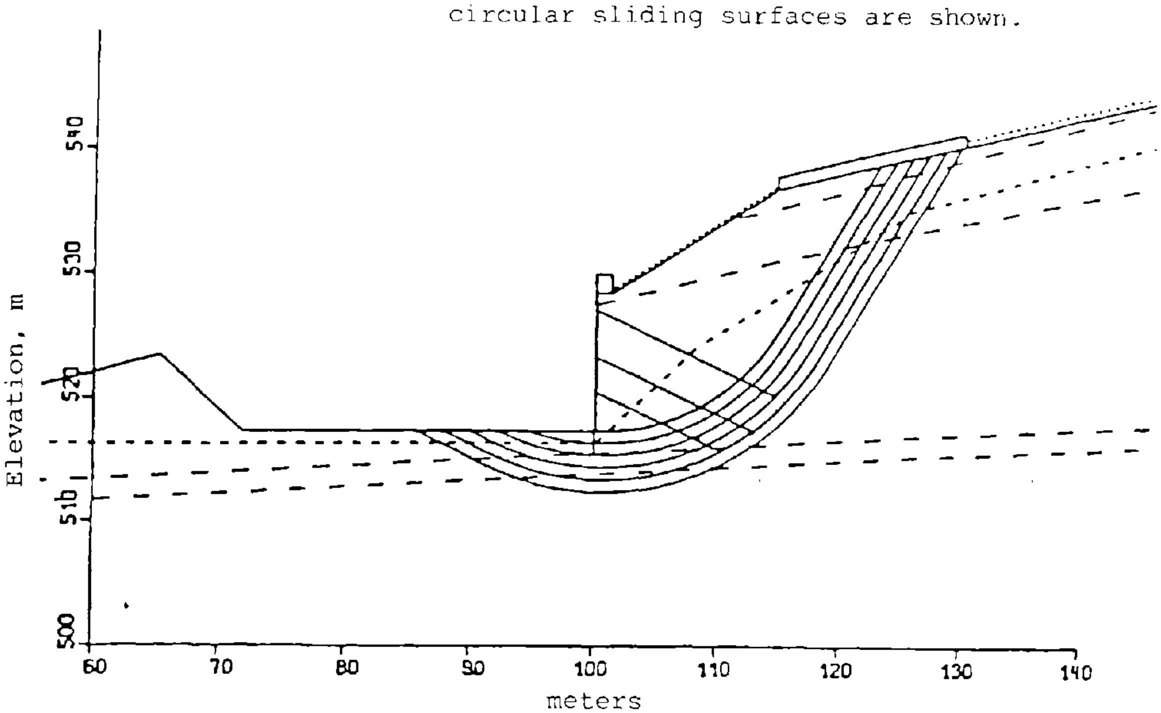

Figure 38. Stability analysis of the system as a whole for one center point; all investigated composite sliding surfaces are shown.

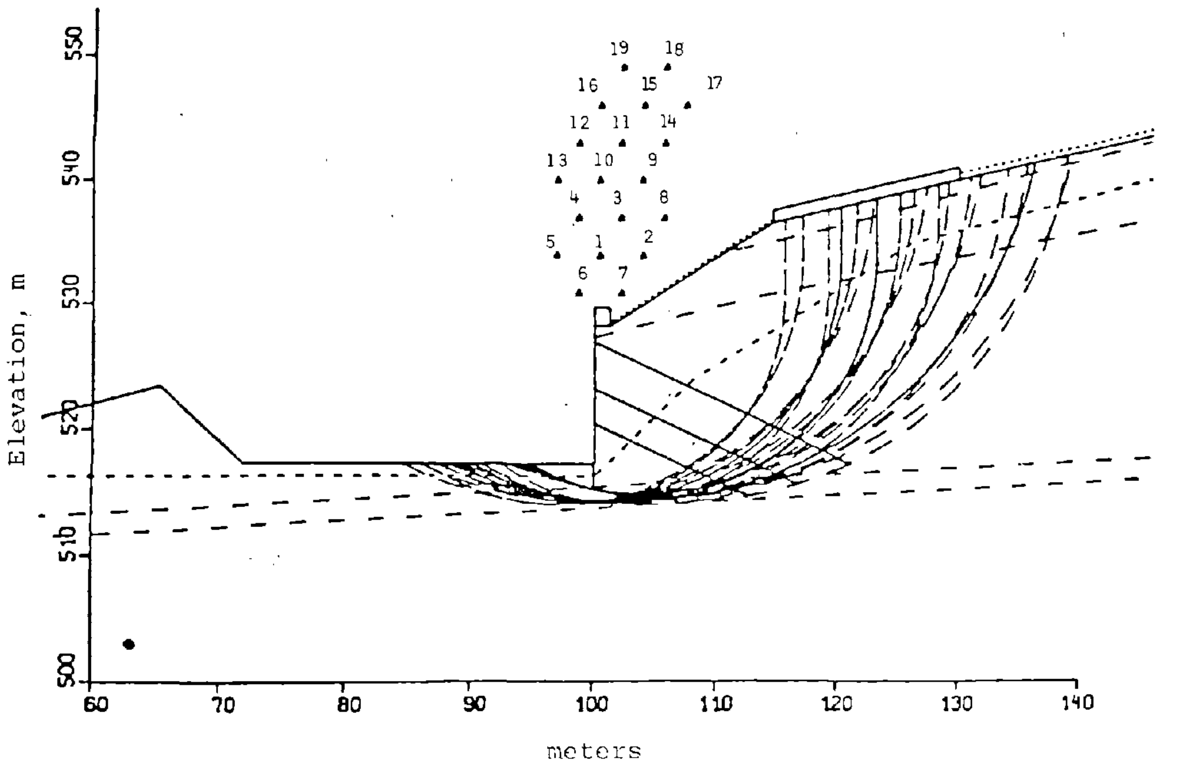

Figure 39. Automatic search with circular sliding surfaces; only the critical sliding surfaces are shown.

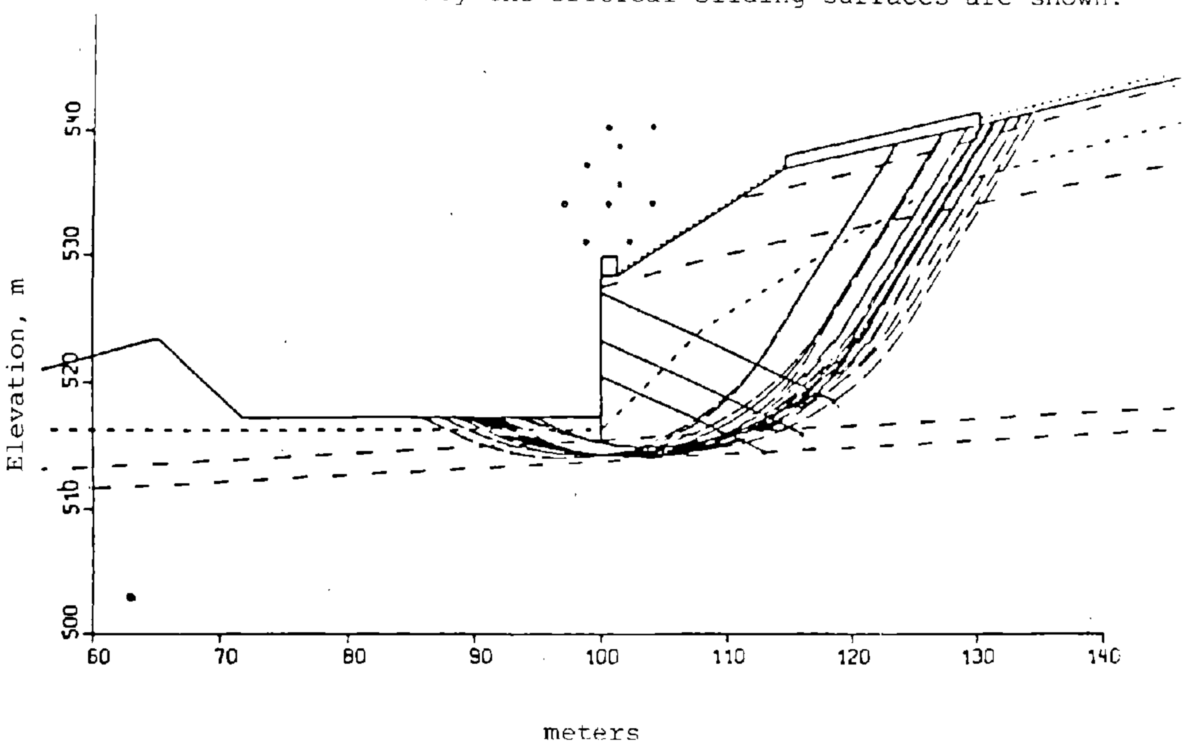

Figure 40. Automatic search with composite sliding surfaces; only the critical sliding surfaces are shown.

Safety Factor for Deep-Lying Slip Zone Using Janbu's Method

It is usual in the case of tied-back walls to check the safety of the wall-anchor-system in deep-lying slip zones, whereby slip surfaces are chosen which lie mainly within the anchored zone. Starting from a point A (Figure 41) beneath the foot of the wall (approximately the point on the fully embedded length of wall where the shear force equals zero) the slip surface passes through the slip zone under investigation up to where it intersects a chosen anchor B.

The failure surface is extended from a point A with a left-oriented passive pressure line to the ground surface at L and likewise at B with a right-oriented active pressure line (point R).

The necessary anchor length at the given depth of anchor (whereby the grouted length is not taken into account) is estimated by letting the anchor reach to the slip surface, for which the computed safety factor (with the potential influence of the anchor) is more than adequate. This procedure is repeated for the remaining rows of anchors. The starting point for the deep-lying slip zones can also be varied (Figure 42). The results of these investigations are the estimated anchor lengths and the accompanying slip surfaces.

The following procedure is suggested for tied-back walls:

1. On the basis of earth pressure theory and Blum's theory the wall is analysed as a continuous beam to estimate the support forces for all anchor depths.

2. Local factor of safety for deep-lying slip zones may be investigated using Janbu's method.

3. The overall safety factor is estimated. For 'soft' layers sandwiched between or adjacent to stiffer layers, if the most unfavorable sliding surface is close-by, an additional composite sliding surface is investigated, which runs through the soft layers.

4. For the case of investigated sliding surfaces for which the specified safety factor is not reached, the anchor forces are increased and if necessary the calculations (2) and (3) repeated.

5. The greater of the anchor lengths calculated in (2) and (3) above, added to the grouted lengths, are regarded as the required lengths for the construction.

   In the automatic stability analysis in (2) and (3) above, the computation of the anchor forces in (1) is also indirectly checked. The critical slip surface corresponding to active earth pressure conditions almost certainly lies very close to one of the slip surfaces investigated and also for this case it must be verified that the specified safety factor has been attained.

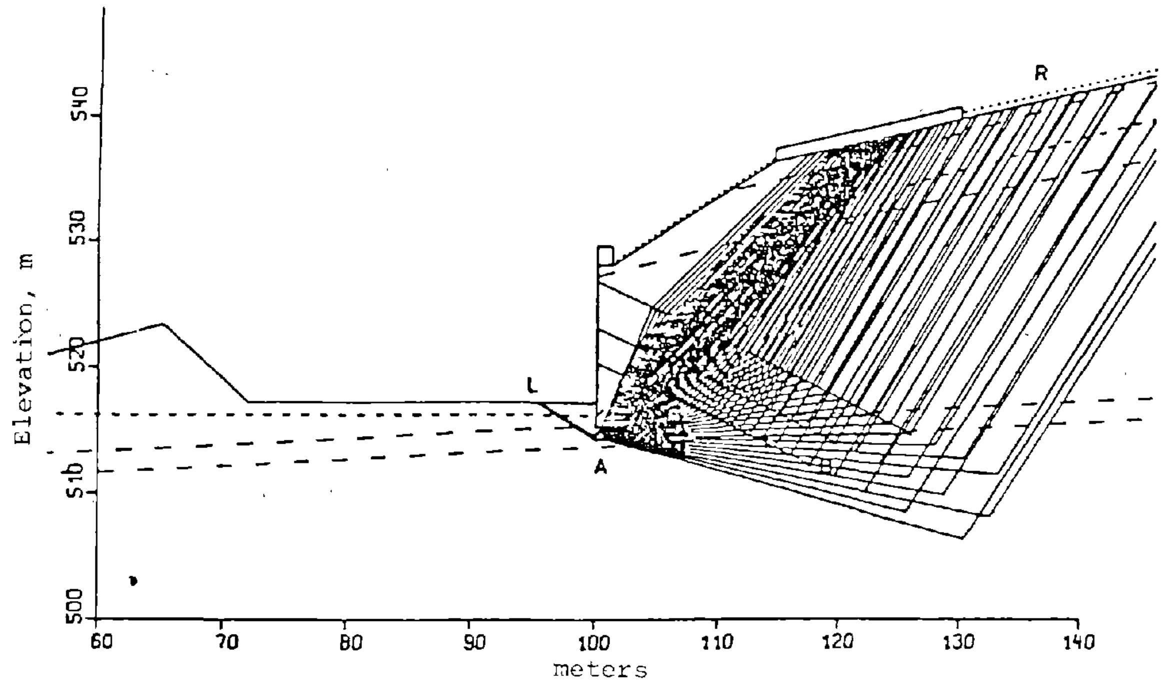

Figure 41. Stability analysis for a deep-lying slip zone for a given point, all investigated sliding surfaces are shown.

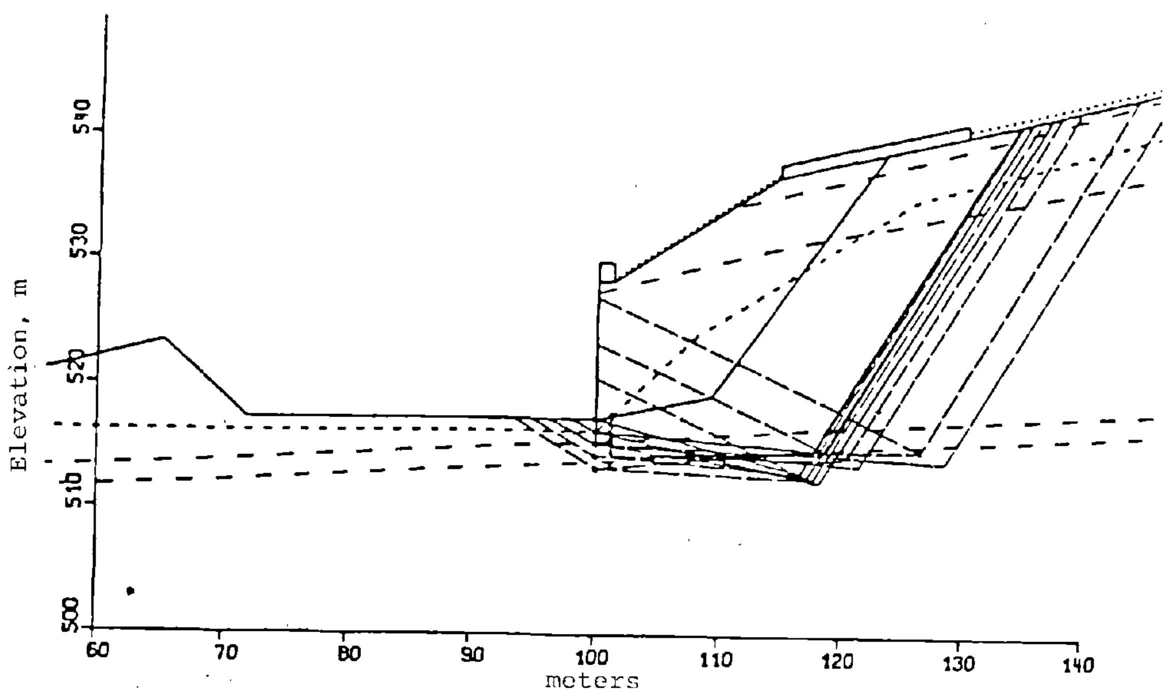

Figure 42. Stability analysis for a deep-lying slip zone for several points, only the critical sliding surfaces are shown.

The investigation of the safety factor in deep-seated slip zones cannot, in general, replace the investigation of the complete system: wall-anchor-soil - and vice versa. In layered ground it is very difficult beforehand to know which of these stability analyses is going to be the deciding one for estimating the required anchor forces and anchor lengths for the individual rows of anchors.

TEST ANCHORS

Detailed tests are carried out on test anchors, in order to establish fundamental information about the dimensioning of the anchorage body and, where necessary, the selection of the anchor. This mainly concerns proof of the necessary safety of the fixed anchor. For this, the tendon of the test anchor is to be adequately dimensioned.

Geotechnical investigations should be made in the ground formation area representative of the test anchor site.

The manufacture of the anchors and the excavation of the tests shall be supervised by the project engineer. An accurate drilling and injection record is to be kept about each test in order to be able to compare the test anchors with other anchors which have not undergone any tests. If the deviations are too large, it may be necessary to construct additional test anchors.

Number of Test Anchors

The number of test anchors in any one project depends on the magnitude of the project, the heterogeneity of the ground, the degree of risk and the experience which has been gained in earlier projects carried out under similar conditions.

If there have been no similar projects previously in a soil or rock area with the same geotechnical conditions, called "foundation area", then the number of test anchors is recommended in Table 11.

Cure Time for Grout

The anchor test should not be carried out until the bonding agent has attained the required strength:

- In the case of cement grouting and cement mortar, generally a week after the grouting at the earliest.

- In the case of synthetic resin mortar, after it has been established by tests that it has set hard; for this purpose the samples should be stored at the same temperatures as prevailing in the area surrounding the fixed anchor body.

Table 11. Recommended number of test anchors.

Number of anchors whose fixed anchors lie in the foundation area	Number of test anchors per foundation area for anchor classes in accordance with		
	Class 1	Class 2 and 4	Class 3, 5, 6
up to 20	None	None	3
over 20	1% of the number, but at least 3	1.5% of the number, but at least 3	2% of the number, but at least 3

Measurement Accuracy - Test Anchors

During the anchor test the axial movements of the anchor head, the movements of the anchor plate and the anchor force in the tendon are to be measured with the following accuracies:

- Δl, movements of the anchor head with respect to the fixed point and of the anchor plate in an axial direction (Δl_k):

  absolute accuracy 2% of Δl_r, the calculated elastic elongation of the tendon

  relative accuracy 0.5% of Δl_r

- Δs, movements of the anchor plates (deformation of the foundation or the anchor head support):

- absolute accuracy 2% of Δl_r

- relative accuracy 0.5% of Δl_r

- Anchor force in the tendon (behind the anchor head):

  absolute accuracy 3% of V_p

  relative accuracy 0.5% of V_p

The movements shall be determined from a fixed point. The effects of temperature are to be taken into account. The force measuring instruments are to be calibrated periodically as recommended by the manufacturer.

Performance of the Test

The anchor test is an aid in selecting and dimensioning the anchor. It forms a basis for assessing and accepting the anchor work. The tendon of the test anchor can be reinforced when determining the limit load V_V although the other characteristics of the anchor must be retained in the assessment.

During the anchor test the test anchor is tensioned in steps and its load/deformation curve recorded. After the test the anchor is removed for inspection if possible.

The anchor test is carried out as follows (see Figures 43 & 44).

1. An initial load $V_A = (0.1 \ldots 0.2)\ V_p$ is selected. V_p is the test load.

2. The range between V_A and V_p is divided into 6 to 10 approximately equal steps, ΔV. The absolute accuracy of the measuring device (calibration) shall be better than 3% V_p.

3. A fixed point is established for the measurement of Δl, the movement of the anchor head relative to a fixed point. The movement, Δl, is composed of two parts, Δl_e, the elastic deformation, and, Δl_{bl}, the plastic deformation. The absolute accuracy of the Δl measurement shall be better than 2% of Δl_r, the calculated elastic elongation of the anchor under load V_p. The movement of this point should be less than 0.5% of Δl_r.

4. The loading and measuring program is then carried out up to an agreed maximum load. At each load step observations are made either of the load decrease with the deformation remaining constant (see Figure 43), or of the deformation increase with the load remaining constant (see Figure 44).

5. The observation times, Δt, for rock and various types of earth are given in Table 12.

6. After each step the load is reduced to V_A and the deformation Δl_{bl} is measured.

7. Each time the load is reapplied the intermediate points of the load/deformation curve are to be recorded.

Evaluation and Assessment of Anchors with Test Load $V_p > 200$ kN

The evaluation is used to determine the limit load, V_V, the free anchor length, l_f, and the plastic deformation, Δl_{bl}.

Table 12. Observation times, Δt.

Foundation	Time Δt
A Rock and cohesionless ground	At least 5 minutes
B Slightly cohesive ground and over-consolidated clays	At least 15 minutes
C Clays and clayey silts in the normally consolidated condition	Several hours to days

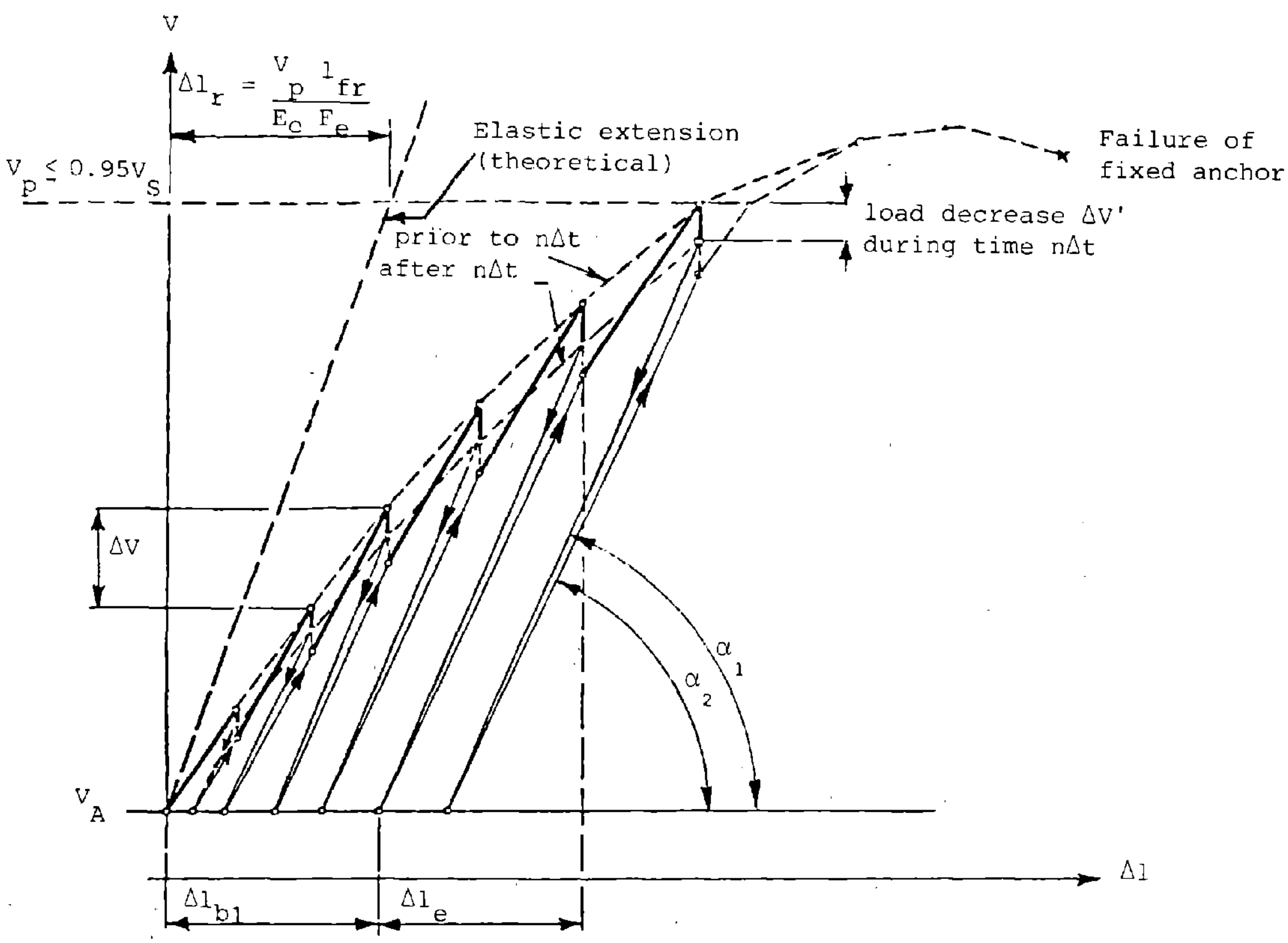

Figure 43. Anchor test with constant deformation increments.

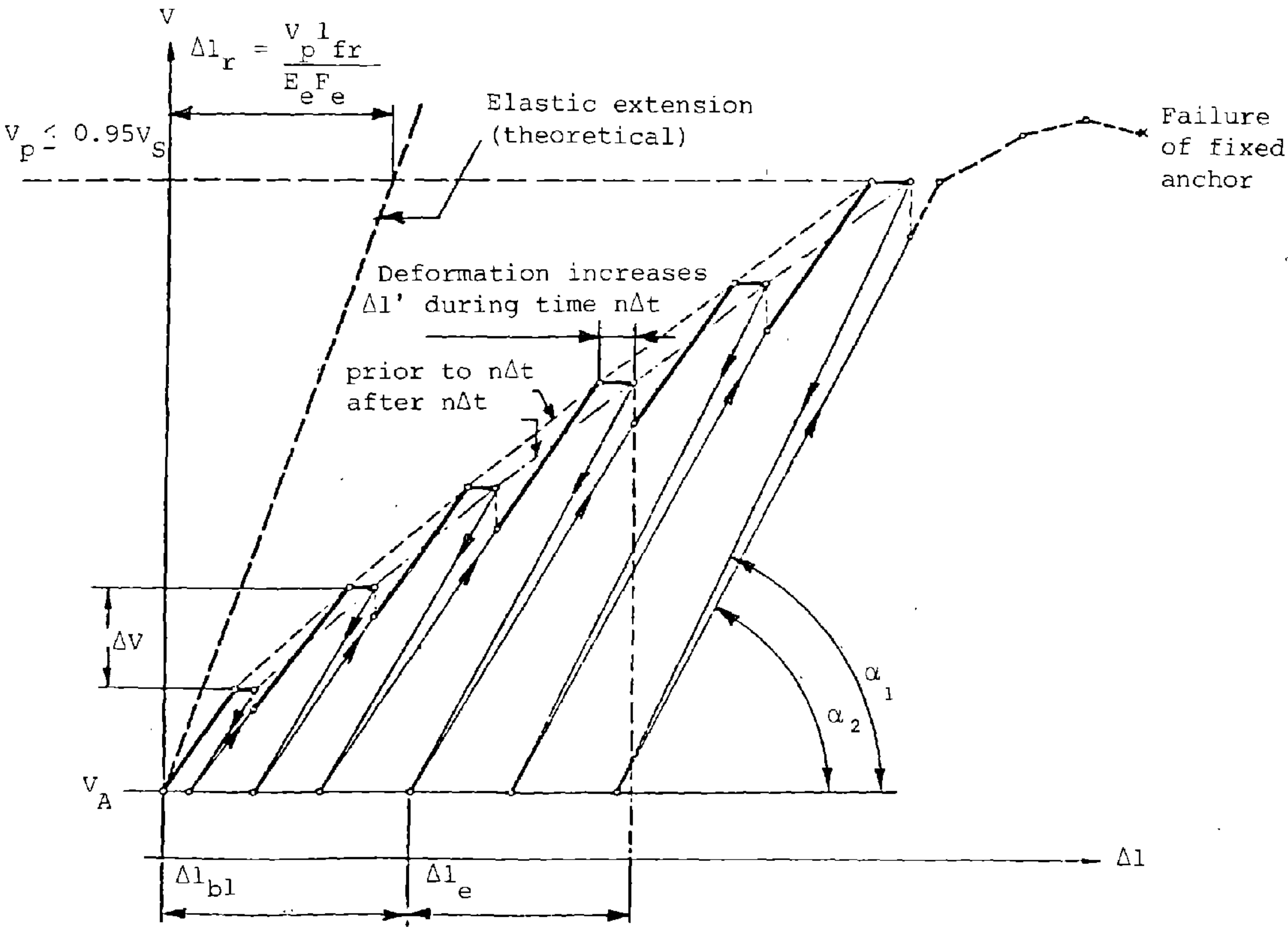

Figure 44. Anchor test with constant load increments.

A. Determining the limit load, V_V

The limit load is the maximum load at which the following two conditions are still satisfied:

1) The change of deformation or load should not exceed the limit values given in Table 13 (condition 1)

If condition as shown in Table 13 (1a) is not satisfied, then the observation time is to be increased to $3\Delta t$. Also, if condition (1b) is not satisfied, then the observation time is to be increased to $10\Delta t$.

Table 13. Limit values - deformation increase/load loss.

Condition	Observation time (according to 5)	Limit values Deformation increase Δl (A)	Load loss $\Delta V'$ (B)
(1a)	0 . . . Δt	max. 2% of Δl_r	max. 2% of V_p
(1b)	Δt . . . $3\Delta t$	max. 1% of Δl_r	max. 1% of V_p
(1c)	$3\Delta t$ . . . $10\Delta t$	max. 1% of l_r	max. 1% of V_p

(A) if the load is kept constant during the observation time
(B) if the deformation is kept constant during the observation time

2) The following inclination ratio must be within the indicated limit: (condition 2)

$$\frac{tg\alpha_2}{tg\alpha_1} \geq 0.90$$

where (see Figure 43)

α_1 = Angle of inclination of the unloading curve

α_2 = Angle of inclination of the reloading curve

B) Determining the effective free anchor length, l_f.

The effective free length of the anchor results from the straight line A' - X (see Figure 45)

$$l_f = \frac{\Delta l_e\ (X)\ F_e}{V\ (X) - V_A - R}\ E_e \quad . . . . (37)$$

Where:

F_e = Cross-sectional area of the tendon

E_e = Modulus of elasticity of the tendon

Δl_e (X) = Elastic deformation of the tendon under load V (X)

V_A = Initial load

R = Frictional force (distance A - A')

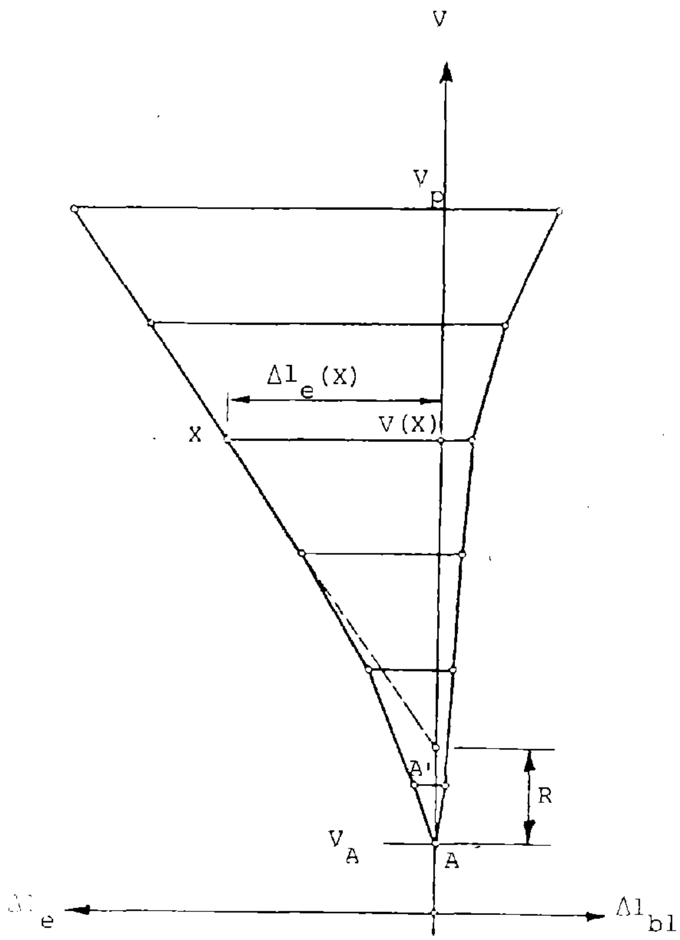

Figure 45. Diagram of the elastic and plastic deformations.

The effective free length, l_f, shall lie between the following limits up to the limit load $\bar{V}_V$: (condition 3)

$$l_f \geq 0.9\ l_{fr}$$

$$l_f \leq l_{fr} + k\ l_v$$

Where

k = 0.5 in systems where the force is introduced into the anchorage body along the anchorage length l_v by tendon

k = 1.1 in systems where the force is introduced in the anchor end block.

C. Determining the plastic deformation

The plastic deformation, Δl_{bl}, is determined in accordance with Figure 45. Its permissible value for the stressing test is to be fixed jointly by the project engineer and the contractor on the basis of the anchor tests and is to be entered in the stressing record (condition 4).

Evaluation and Assessment of Anchors Using Test Load $V_p \leq 200$ kN (45 kips).

For anchors with $V_p \leq 200$ kN the evaluation can generally be simplified. During the test the yield stress of the tendon should not be exceeded, but if possible load should be great enough to cause failure of the fixed anchor.

The mean value of the ratio of inclination $tg\alpha_2/tg\alpha_1$ should be greater than or equal to 0.80 over at least three loading cycles.

When the test load V_p is reached, the plastic deformations should not exceed the following values after the anchor plate deformation has been subtracted:

- Expansion-shell anchor: $\Delta l_{bl} = 18$ mm
- Mortar and synthetic resin adhesive anchors: $\Delta l_{bl} = 3$ mm

If the free length l_f is predetermined by the design, the calculation according to equation (37) can be dispensed with.

The measured values of the anchor tests are to be kept in records, as part of the important construction documents.

The accuracy requirements as previously stated apply to the anchor tests.

Depending on the soil or rock conditions, long-term anchor tests may be necessary in order to determine the long-term behavior. The measuring devices and programs are to be selected by the project engineer in each case depending on the requirements.

STRESSING TEST

The stressing test is used to assess the anchor work. The basis for it is generally formed by the results of the tests described under TEST ANCHORS. If no anchor tests were carried out, then the required basic data are to be determined from comprehensive stressing tests using a test load $V_p = 0.95\ V_S$. V_S is that tendon load which produces the nominal yield stress in the tendon.

All anchors with test load > 200 kN are subject to a simple stressing test. In addition, a limited number of the anchors are to undergo the comprehensive stressing test. During the comprehensive stressing test the anchors is tensioned in steps. The behavior of the anchor is observed during the intermediate steps, the unloading periods and at the test load V_p. The load/deformation curve is recorded until the test load V_p is reached.

When assessing the load/deformation curve the deformation of the stressing device is to be taken into account.

During the simple stressing test the behavior of the anchor at the test load V_p is observed for a certain time.

The test is limited to random samples in the case of anchors of a test load $V_p \leq 200$ kN.

<u>Performance of the Comprehensive Stressing Test</u>

The comprehensive stressing test is performed as follows (see Figures 46 & 47).

1. The minimum number of anchors to be subjected to the comprehensive stressing test is determined in accordance with Table 14.

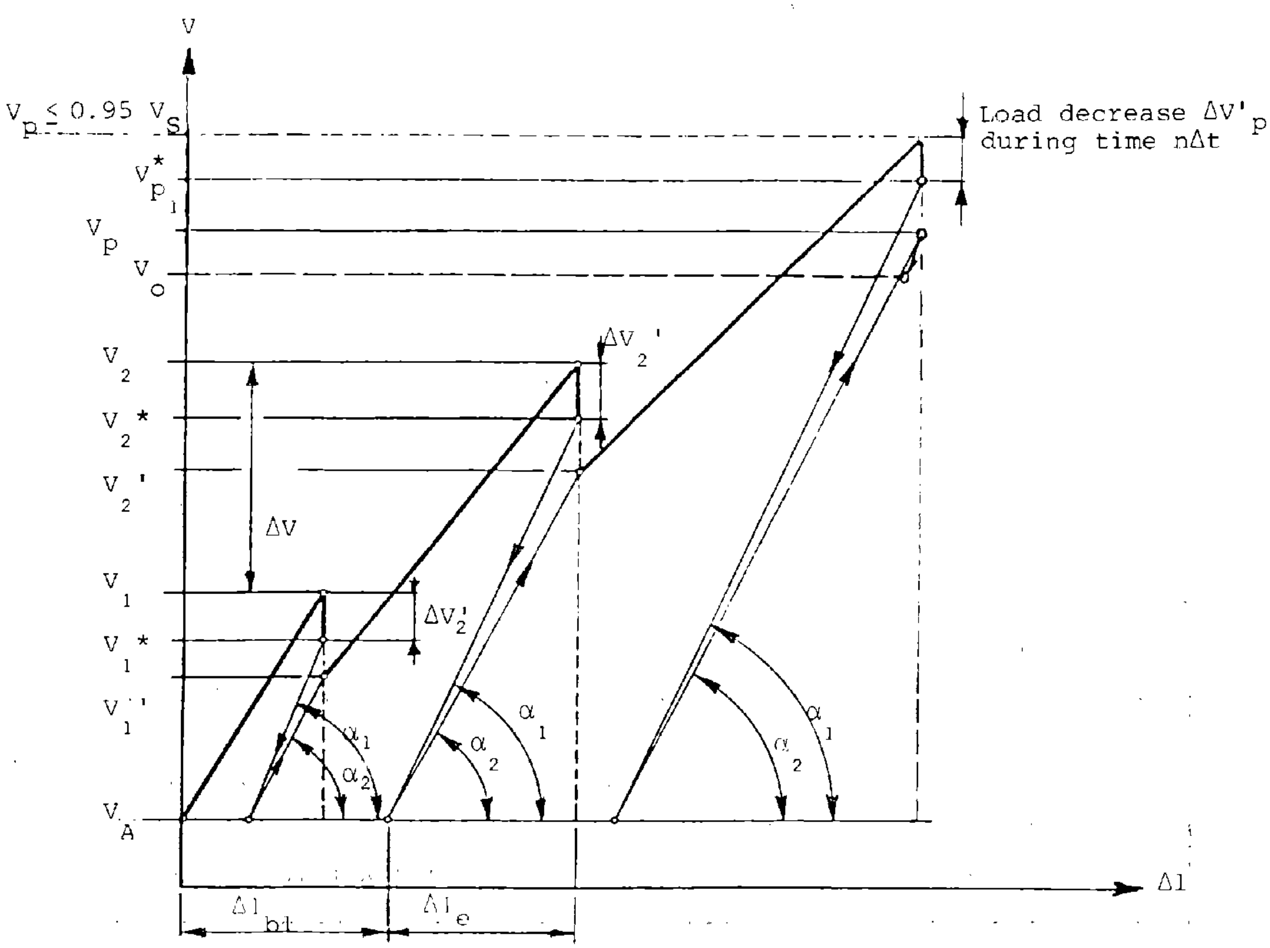

Figure 46. Comprehensive stressing test with constant deformation increments.

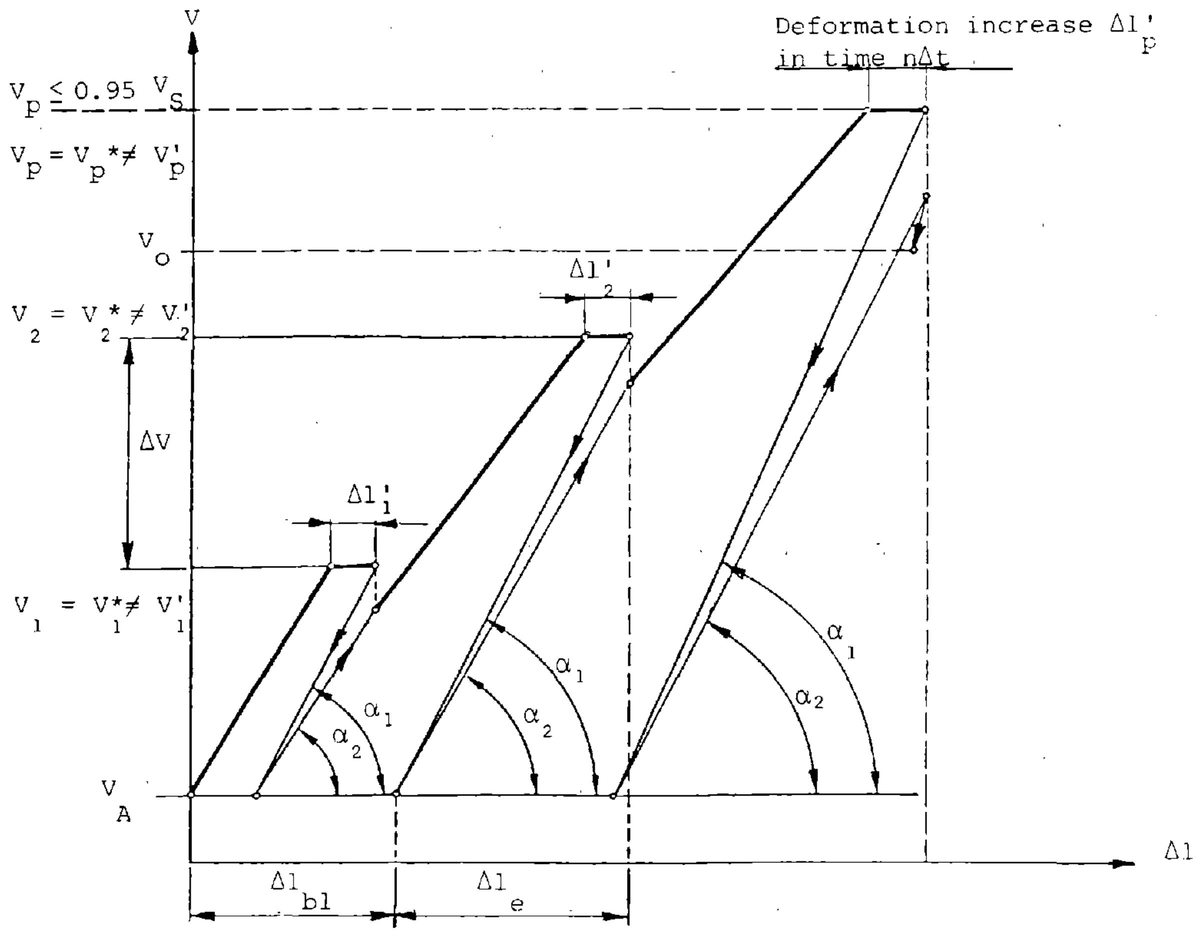

Figure 47. Comprehensive stressing test with constant load increments.

Table 14. Minimum number of all anchors in percent to be subjected to the comprehensive stressing test.

Test load V_p	Anchor classes 1, 2, 4, 5	Anchor classes 3 and 6
> 200 kN	3% but at least 2	6% but at least 4
≤ 200 kN	5% but at least 12	

2. An initial load $V_A \simeq (0.1 \ldots 0.2)\ V_p$ is selected.

3. The load range between V_A and V_p is divided up into three approximately equal steps ΔV.

4. The load-deformation curve is taken to be linear during the applications of ΔV.

5. In each step the load V or the deformation Δl (or the piston stroke Δl_k) is kept constant during the observation time $n\Delta t$ and the deformation increase $\Delta l'$ or the load decrease $\Delta V'$ is observed (for observation time $n\Delta t$, see Table 12) n can assume the value 1, 3 or 10.

6. After each step the load is reduced to V_A and the particular deformation Δl_{bl} measured.

7. When the load is reapplied the intermediate points of the load-deformation curve are to be recorded.

8. After the last step V_p the anchor is fixed at the stressing load, V_O. If necessary, it can be temporarily completely unloaded.

Assessment of the Comprehensive Stressing Test

The following four conditions corresponding to those given under "Evaluation and assessment of anchors with test load $V_p > 200$ kN", are to be satisfied:

Condition (1): After the observation times $n\Delta t$ have lapsed, the change in the deformation or the load should not exceed the limit values given in Table 13.

Condition (2): The inclination ratios $tg\alpha_2/tg\alpha_1 \geq 0.90$ or 0.80.

Condition (3): The free anchor length l_f shall be within the limits stated for condition 3 under
ment of anchors with test load $V_p > 200$ kN).

Condition (4): The plastic deformation Δl_{bl} shall be smaller than the limit value fixed on the basis of the anchor tests (Evaluation and assessment of anchors with test load $V_p > 200$ kN).

Performance of the Simple Stressing Test

The simple stressing test is performed as follows (see Figures 48 & 49).

1. An initial load $V_A \simeq (0.1 \ldots 0.2)\ V_p$ is selected.

2. The anchor is loaded up to the test load V_p.

3. The value of Δl is measured for V_A and V_p.

4. The test load V_p or the deformation Δl (or the piston stroke Δl_k) are maintained during the observation time $n\Delta t$ and the deformation increase $\Delta l'_p$ or the load decrease $\Delta V'_p$ are observed (for observation time $n\Delta t$ see Table 12. The value of n can assume 1, 3 or 10 (see Table 13).

5. In the case of soil anchors the load is reduced to V_A and Δl_{bl} is checked.

   In the case of rock anchors, it is not necessary to reduce the load to V_A. Condition (4) in the following section is then also dispensed with.

6. After the last step, V_p, the anchor is fixed at V_O. If necessary, it can, temporarily be fully unloaded.

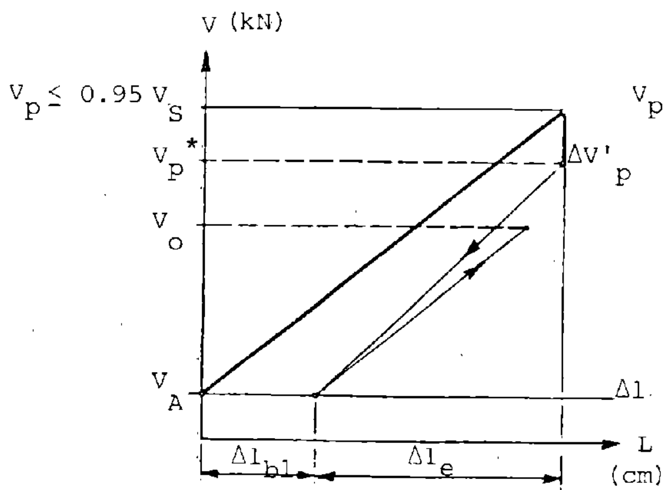

Figure 48. Simple stressing test with constant deformation.

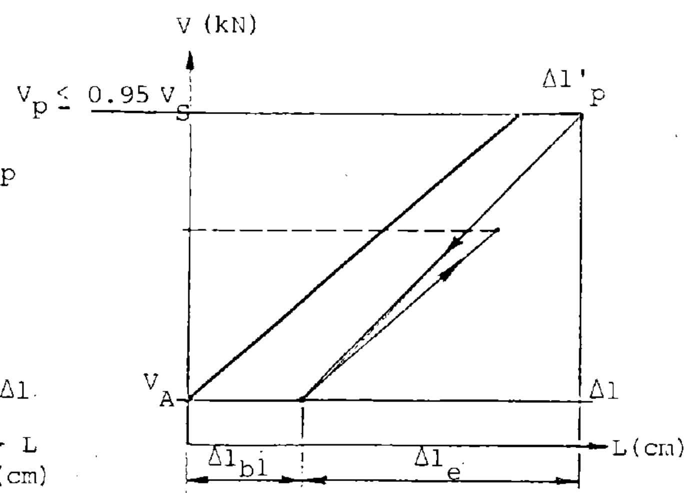

Figure 49. Simple stressing test with constant load.

Assessing the Simple Stressing Test

The following conditions which correspond to the appropriate conditions given under (Evaluation and assessment of anchors with test load $V_p > 200$ kN) are to be satisfied:

Condition (1): After the observation time $n\Delta t$ has elapsed the change in the deformation or the load should not exceed the limit values given under "Evaluation and assessment of anchors with test load $V_p > 200$ kN".

Condition (3): The free anchor length l_f shall lie within the limits as stated.

In the calculation given under "Evaluation and assessment of anchors with test load $V_p > 200$ kN" a mean friction force R can be assumed on the basis of the anchor tests or the comprehensive stressing test.

Condition (4): The plastic deformation Δl_{bl} shall be less than the limit value fixed on the basis of the anchor tests.

Stressing Test Records

The measured values of the simple and comprehensive stressing tests are to be logged in stressing records, and retained as part of the Important Construction Records.

The same accuracy requirements given under "Measurement accuracy-test anchors" is required for the comprehensive stressing test.

For the simple stressing test the measuring accuracy is fixed by the project engineer and the contractor on the basis of the comprehensive stressing tests.

If the conditions require it, the following records are to be drawn up:

- Prior to the start of the anchorage work, an all-party approved report of the condition of the surrounding land, including buildings, streets, ducts, springs, etc.

- After the anchorage work has been completed, an all-party approved acceptance report on the result of the final check.

While the anchorage work is being carried out, the chief construction engineer makes periodic checks to establish any displacements of the ground.

If the ground conditions deviate from the accepted conditions, the contractor must report this immediately to the project engineer.

The contractor keeps continuous records of his drilling, grouting and stressing work.

SAMPLE DESIGN OF ELEMENT WALL

Introduction

The following example problem illustrates a general design investigation for a permanently tied-back wall in connection with the widening of the existing highway I-75-2 (41) at Fulton County, Georgia. The highway section is designated as wall "E-1" at the Penthouse-Motel in the accompanying documents.

To accomplish this task the following documents were supplied by the client:

1. Description of the soil survey boreholes EI-4 to EI-8
2. Results of the laboratory investigations
3. Plan of the area
4. Longitudinal-section
5. Cross-section

The soil profile is shown on Figure 50. The tests on the selected specimens gave the soil parameters summarized below:

Either:	angle of internal friction	$\phi = 25°$
	cohesion	$c = 14.65$ kN/m2 (300 psf)
	unit weight	$\gamma = 17.6$ kN/m3 (110 pcf)
	coefficient of sliding friction	$\mu = 0.45$
or:	effective angle of internal friction	$\phi = 35°$
	effective cohesion	$c = 0$
	effective unit weight	$\gamma = 18.4$ kN/m3 (115 pcf)

These soil parameters apply to the upper layer, which in the given section is about 11.0 to 19.0 m thick, and is composed of sand and silty sand. Under this layer is homogeneous rock of gneiss origin of the Georgia-Piedmont region.

This basically sound rock was given the following parameters in the stability calculations:

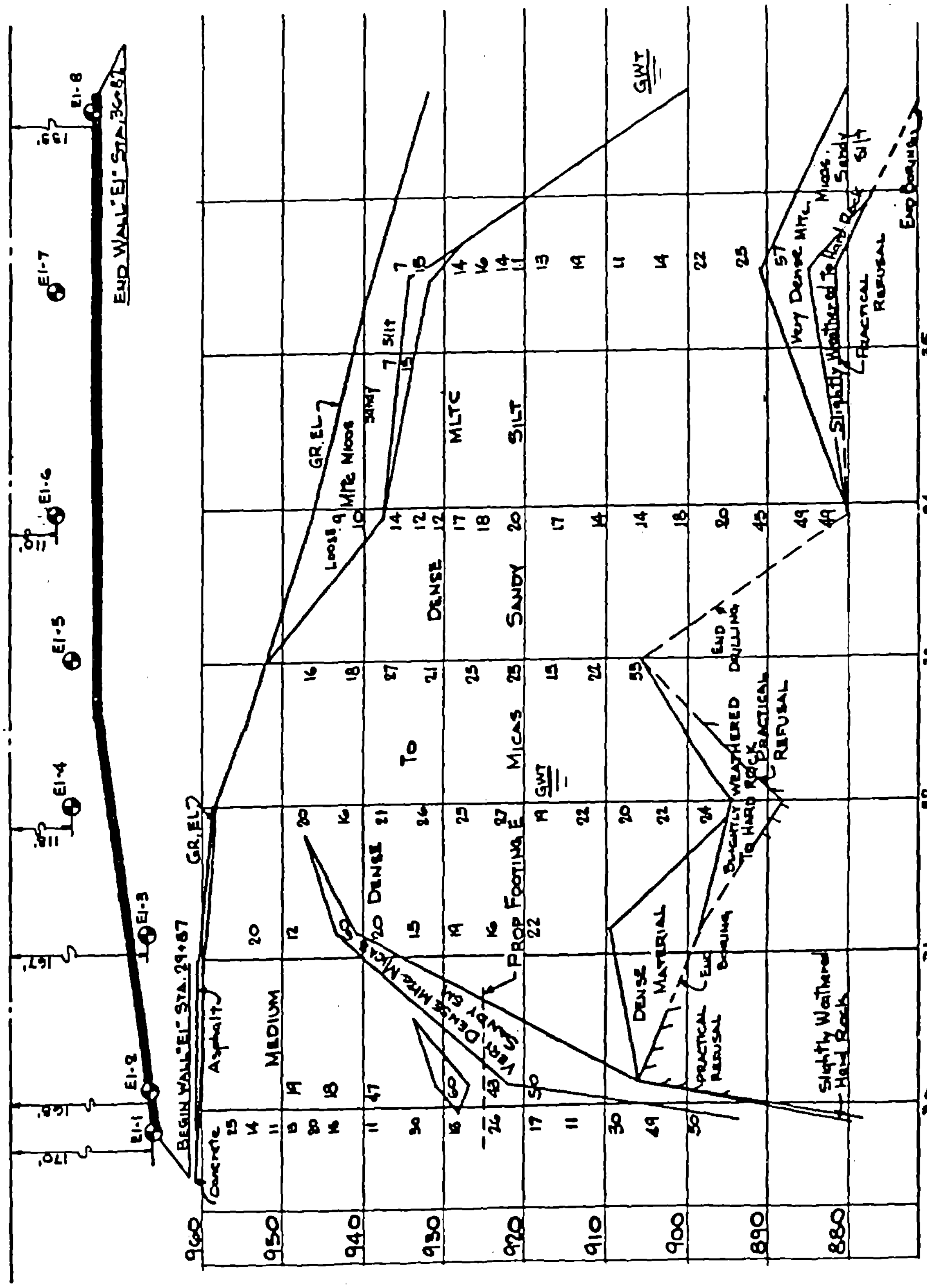

Figure 50. Soil profile.

angle of internal friction $\phi = 40°$

cohesion $c = 500$ kN/m2

unit weight $\gamma = 26.0$ kN/m3

For the upper layer the effective parameters were chosen, as they give somewhat higher earth pressures and thus higher anchor forces, the results lying on the conservative side.

Construction Project

The existing I-75 has to be widened, which necessitates cutting into the slopes in the direction of the Penthouse-Motel. The original project was for a concrete wall supported on piles.

As a variant solution we propose a permanently anchored element wall, which offers various advantages:

- the work can commence without previous large-scale excavation work
- since after placing a row of elements they are immediately anchored, there is no danger of instability as there was for the slope which was an integral part of the original project
- the expensive piling work is no longer needed
- the road running above the proposed wall can be kept open at least for one way traffic during the construction period.

Analysis

Five sections were chosen for calculating the anchor forces, the stability of the ground being investigated in four of them.

a) the anchor force must be able to resist the earth pressure both during and at the end of construction.

b) with the inclusion of the anchor forces in the stability investigations the safety factor must be at least as great as the specified value. If this is not the case the anchor forces must be increased such that this condition is fulfilled.

As mentioned above the earth pressure was calculated using the "effective" soil properties. The earth pressure diagram based on Coulomb's analysis is changed to be of rectangular distribution, the maximum force being thereby increased by 20%. This is done at every constructional stage, whereby in calculating the earth pressure the depth of excavation taken is at the bottom of the next row of elements to be installed underneath. The actual anchor force for the constructional stage is assumed to be calculated from the earth pressure acting at the momentary depth plus 60% of the earth pressure acting on the lower row of elements yet to be installed. It is assumed that the remaining 40% of the earth pressure in the lower row of elements, which will be mobilized when the excavation reaches that level and theoretically should be resisted by the overlying anchors, is in fact directly balanced by arching action in the ground. For the end of construction stage the actual earth pressure acting on each element is calculated simply on the basis of element height to wall height ratio. The design anchor force is always taken to be the greater value comparing the constructional stage to the final stage. Table 15 shows computer output for the static calculations of section S-1.

Remark: The determination of the bottom of the wall was made on the basis of the sections provided for the intersection levels of the wall-top of highway for the future I-75 highway and from this height 1.10 m was deducted (50 cm for the highway construction and 60 cm for the unanchored elements forming the foot of the wall).

The stability analysis was carried out on the basis of the anchor forces thus determined. Firstly Krey's analysis was used with circular slip surfaces and then Janbu's analysis was used to check the stability considering slip in deep-lying joint planes.

According to the Swiss Standard SIA(8) - 191 "Ground Anchors" the minimum safety factor using Krey's method for buildings whose damage potential is in the category 6 (our case) is $\eta \geq 1.5$. This value was not attained in any of the four sections employing anchor forces calculated from the earth pressure diagrams. Thus in the second run the anchor forces had to be increased. Subsequently the deep-seated failure mode (Janbu's analysis) was investigated. For this case a value of $\eta > 1.2$ was required, which was achieved in all four sections.

Design

The anchor lengths were obtained from both calculations (on attaining the prescribed safety factor) by extending the anchors to those slip surfaces where the factor of safety required it. Added to these lengths is an amount due to the fixed anchor part, which equalled 5.0 m for Krey's analysis and 3.0 m for Janbu's analysis.

Sample computer output for Krey's analysis for section S-1 is shown in Table 16, and for Janbu's analysis of section S-1 in Table 17. The remainder of the computer output for Krey's and Janbu's analysis at section S-1 is included in Appendices A and B respectively. Figure 51 shows a computer generated plot of the results of Krey's analysis for section S-1 and the computer generated plot of the results of Janbu's analysis of section S-1 are shown in Figure 52.

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(8) Schweizerischer Ingenieur - and Architekten-Verein (Swiss Society of Civil Engineers and Architects).

Table 15. Sample computer output for static calculations.

a) Summary of input data.

TOP OF WALL	YW = 293.16 M	WHEN EXCAVATING FOR THE NEXT ROW OF ELEMENTS AN EARTH PRESSURE IS DEVELOPED, A CERTAIN PERCENTAGE OF WHICH (FG) MUST, DURING THE CONSTRUCTION PHASE, BE CARRIED BY THE LOWEST ROW OF ANCHORS.
FINAL DEPTH OF EXCAVATION	YE = 282.36 M	
SURFACE SLOPE	BETA = 2.98 DEGREES	
WALL INCLINATION	ALPHA = 0.00 DEGREES	
HEIGHT OF SLOPE	V = 0.00 M	
UNIFORM SURFACE LOADING	Q = 1.00 kN/M2	
MIN. ADMISSIBLE E.P. COEFFICIENT	KA* = 0.20	
PERCENT VALUE FOR ARCHING EFFECT ACTIVE EARTH PRESSURE	FG = 60.00%	

SOIL PARAMETERS

LAYER NO.	GROUND SURFACE (M ABOVE M,S,L)	LAYER THICKNESS M	PHI DEGREES	DELTA DEGREES	COHESION kN/M2	GAMMA kN/M3	KA'	KA
1	293.16	10.80	35.00	0.00	0.00	1.84	.2786	.2786

Table 15. Sample computer output for static calculations (continued).

b) Calculation of earth pressure normal to wall

LEVEL M	1. WITH CALCULATED KA' COORDINATE kN/M2	COHESION REDUCTION kN/M2	EARTH PRESSURE kN/M'	2. WITH MIN. KA* COORDINATE kN/M2	EARTH PRESSURE kN/M'	LAYER THICKNESS PARALLEL TO WALL M	RELEVANT EARTH PRESSURE kN/M'
293.16	.28	.00		.20			
282.36	55.65	.00	361.80	39.94	259.70	10.80	361.80

EFFECTIVE TOTAL EARTH PRESSURE (kN/M') = 361.80
EFFECTIVE HEIGHT OF WALL (M) = 10.80
REDISTRIBUTED EARTH PRESSURE (kN/M2) = 33.50

REMARK: THE COORDINATES OF THE EARTH PRESSURE DIAGRAM (COLUMNS 2 AND 5) DO NOT ACCOUNT FOR THE INCREASE OF 20% NECESSARY DUE TO THE REDISTRIBUTION OF EARTH PRESSURE. THIS IS, HOWEVER, INCLUDED IN THE CALCULATED VALUES (COLUMNS 4 AND 6)
KA' - EARTH PRESSURE COEFFICIENT FOR CALCULATING (NORMAL) EARTH PRESSURE
KA - EARTH PRESSURE COEFFICIENT FOR CALCULATING THE COHESIONAL REDUCTION

Table 15. Sample computer output for static calculations (continued).

c) Calculated anchor forces

ANCHOR POSITION NO.	EFFECTIVE EXCAVATION LEVEL M	ANCHOR FORCE NORMAL TO WALL		ANCHOR FORCE INCLINED			MAX. ANCHOR
		DURING CONSTRUCTION kN/M'	END OF CONSTRUCTION kN/M'	ANCHOR SLOPE DEGREES	DURING CONSTRUCTION KN/M'	END OF CONSTRUCTION kN/M'	ANCHOR INCLINED kN/M'
1	289.76	29.20	56.95	15.00	30.24	58.96	58.96
2	288.06	43.43	56.95	15.00	44.96	58.96	58.96
3	286.36	57.65	56.95	15.00	59.69	58.96	59.69
4	284.66	71.88	56.95	15.00	74.41	58.96	74.41
5	282.96	86.10	56.95	15.00	89.14	58.96	89.14
6	282.36	69.01	56.95	20.00	73.44	60.60	73.44
7	282.36	20.10	20.10	20.00	21.39	21.39	21.39

Table 15. Sample computer output for static calculations (continued).

d) Levels at which anchors act.

ELEMENT ROW NUMBER BEGINNING AT TOP	LEVEL OF TOP OF ELEMENT	ELEMENT HEIGHT PARALLEL TO WALL	ELEMENT HEIGHT PROJECTED ONTO VERTICAL PLANE	LEVEL AT WHICH ANCHORS ACT
NO.	M	M	M	M
1	293.16	1.70	1.70	292.31
2	291.46	1.70	1.70	290.61
3	289.76	1.70	1.70	288.91
4	288.06	1.70	1.70	287.21
5	286.36	1.70	1.70	285.51
6	284.66	1.70	1.70	283.81
7	282.96	.60	.60	282.66

Table 16. Sample computer output for Krey's analysis of Section S-1.

a) Input control and summary of data

```
.Plot 15=1
.TEXTS DESIGN CRITERIA FOR PERMANENT ANCHORS, TASK C
.TEXTA SECTION S1 MARCH 1980 STUMP/VIBROFLOTATION
.SOIL 1 40 50 2.6 X=500 100 500 Y=211.26 274.32 274.32
.SOIL 2 35 0  1.84 X=500 500 Y=900 900
.SURFACE LOAD  1 1 100.02 118.3
.SURFACE LOAD  2 0.5 118.3 500
.ANCHOR  1 100.02 292.31 -15 8.15
.ANCHOR  2 100.02 290.61 -15 8.15
.ANCHOR  3 100.01 288.91 -15 8.15
.ANCHOR  4 100.01 287.21 -15 8.15
.ANCHOR  5 100    285.51 -15 9.62
.ANCHOR  6 100    283.81 -20 7.89
.SURFACE X=500 34.47 34.49 98.9 98.92 BASE=100 TOP=100.02 500
.Y = 290.32 290.32 285 282.96 282.36 282.36 293.16 314.12
.STAND 2 1.5 (CONTROL PARAMETER AND REQD FS)
.STPAR 26=2   8=8   17=25   19=25   11=100.01   12=289.76   13=100   14=274.32
.AUTOM
.END
```

Table 16. Sample computer or output for Krey's analysis of Section S-1 (continued).

b) Summary of slope geometry and soil parameters.

DATA:

COORDINATES OF TOP OF SOIL STRATA

SOIL NO.	NO. PTS.	COORDINATES		
1	3	X = -500.00	100.00	500.00
		Y = 211.26	274.32	274.32
2	2	X = -500.00	500.00	
		Y = 900.00	900.00	

GROUND SURFACE

NO.	NO. PTS.	COORDINATES								
1	9	X = -999.00	-500.00	34.47	34.49	98.90	98.92	100.00	100.02	500.00
		Y = -999.00	290.32	290.32	285.00	282.96	282.36	282.36	293.16	314.12

SOIL CONSTANTS:

LAYER NO.	FI DEGREES	C T/M2	GAMMA T/M3
1	40.00	50.00	2.60
2	35.00	.00	1.84

POREWATER PRESSURE
NO POREWATER PRESSURE FOUND IN AREA INVESTIGATED

Table 16. Sample computer or output for Krey's analysis of Section S-1 (continued).

Table 16. Sample computer output for Krey's analysis of Section S-1 (continued).

c) Surcharge and external forces

SURCHARGE:

MOBILE SURFACE SURCHARGE

NO.	LOAD (T/M2)	X-LEFT	X-RIGHT
1	1.00	100.02	118.30
2	.50	118.30	500.00

POTENTIAL EXTERNAL FORCES
CUT ANCHORS

ANCHOR LEVEL	POINT OF INSERTION		INCLINATION (DEGREES)	ANCHOR FORCE (T)	POSSIBLE LENGTH (M) WITHOUT FIXED PART
1	X = 100.02	Y = 292.31	-15.00	8.15	
2	X = 100.02	Y = 290.61	-15.00	8.15	
3	X = 100.01	Y = 288.91	-15.00	8.15	
4	X = 100.01	Y = 287.21	-15.00	8.15	
5	X = 100.01	Y = 285.51	-15.00	9.02	
6	X = 100.00	Y = 283.82	-20.00	7.89	

Table 16. Sample computer output for Krey's analysis of Section S-1 (continued).

d) Results of calculations according to DIN-Krey for center point No. 1

SAFETY FACTOR WITHOUT ANCHOR INFLUENCE AND SHEARING RESISTANCE OF STRUCTURAL PARTS "

SAFETY FACTOR WITH SHEARING RESISTANCE OF STRUCTURAL PARTS AND ANCHOR EFFECTS USING CALCULATED ANCHOR LENGTHS L in M FOR THE ANCHOR LEVELS:

NO. CYCLES	RADIUS	SYMBOL	WITHOUT ETA	WITH ETA	DENOMINATOR	6 ETA	AND L	5 ETA	AND L	4 ETA	AND L	3 ETA	AND L	2 ETA	AND L	1 ETA
1	10.96		1.93		12.55											
2	11.88		1.84		19.10											
3	12.81		1.75		27.42											
4	13.73		1.71		36.50											
5	14.66		1.67		46.78											
6	15.58		1.64		58.23											
7	16.51		1.62		70.85											
8	17.43		1.60		85.41											
9	18.36		UNREAL SLIP SURFACE: XA = 94.90 YA = 283.09 XB = 98.92 YB = 282.39													
9	18.36	F	1.58		100.39											
10	19.36		1.71		115.21											
11	20.36		1.86		129.44											
12	21.36		2.04		143.21											
13	22.36		2.24		156.64											
14	23.36		2.44		170.05											
15	24.36		2.66		183.06											
16	25.36		2.90		195.85											
17	26.36		3.16		208.49											
18	27.36		6.11		221.19											

FOR THE REMAINDER USE APPROPRIATE SLIP LINE

NO.	CENTERPOINT X(M)	CENTERPOINT Y(M)	RADIUS R	X(L)	Y(L)	X(A)	Y(A)	X(B)	Y(B)	X(R)	Y(R)	SAFETY FACTOR ETA(DIN-KREY)	SLIP LINES	TOTAL COMPT. TIME IN SECS.
1-10	100.00	300.72	19.36	91.80	283.18	91.80	283.18	118.19	294.11	118.19	294.11	1.71	18	136.00

Table 16. Sample computer output for Krey's analysis of Section S-1 (continued).

e) Summary of results

FOR CENTER POINTS 1 TO, 24 THE INVESTIGATION HAS DETERMINED
FACTORS OF SAFETY ACCORDING TO DIN-KREY FOR APPROPRIATE SLIP PLANES
ETA = 1.58 FOR THE FOLLOWING SLIP PLANES:

NO.	CENTERPOINT X(M)	Y(M)	RADIUS R	POINTS OF INTERSECTION: L, A, B AND R X(L)	Y(L)	X(A)	Y(A)	X(B)	Y(B)	X(R)	Y(R)	SAFETY FACTOR ETA(DIN-KREY)	SLIP LINES	TOTAL COMPT. TIME IN SECS.
21-9	97.00	292.06	11.15	90.18	283.24	90.18	283.24	108.15	292.06	108.15	293.59	1.58	346	701.00

FOR CENTER POINTS 1 TO, 24 THE INVESTIGATION HAS DETERMINED
FACTORS OF SAFETY ACCORDING TO DIN-KREY FOR APPROPRIATE SLIP PLANES
ETA = .60 FOR THE FOLLOWING SLIP PLANES:

NO.	CENTERPOINT X(M)	Y(M)	RADIUS R	POINTS OF INTERSECTION: L, A, B AND R X(L)	Y(L)	X(A)	Y(A)	X(B)	Y(B)	X(R)	Y(R)	SAFETY FACTOR ETA(DIN-KREY)	SLIP LINES	TOTAL COMPT. TIME IN SECS.
24-1	99.00	288.60	1.54	100.01	287.42	100.01	297.42	100.54	288.60	100.54	293.19	.60	346	701.00

COMPUTED NECESSARY ANCHOR LENGTHS WITHOUT FIXED ZONE

FOR ANCHOR LEVELS	6	5	4	3	2	1
LENGTH IN M	4.33	5.93	7.35	6.92	8.13	9.17
FOUND IN SLIP PLANE NO.:	19-10	4-9	12-9	5-8	5-8	5-8

THE ABOVE SAFETY FACTOR ACCORDING TO DIN-KREY ETA = 1.50 WITH THE
POTENTIAL INFLUENCE OF STRUCTURAL PARTS AND ANCHORS WAS NOT ATTAINED
IN THE FOLLOWING 7 INVESTIGATED SLIP PLANES

CENTER POINT NO.:	SLIP PLANE NO.:	SAFETY FACTOR ETA ACCORDING TO DIN-KREY
8	5	1.48
14	6.	1.49
15	1	.93
16	1	.83
18	1	1.06
22	1	.90
24	1	.60

Table 17. Sample computer output for Janbu's analysis of Section S-1.

a) Input control and summary of data

```
.Plot 15=1
.TEXTS DESIGN CRITERIA FOR PERMANENT ANCHORS, TASK C
.TEXTA SECTION S1 MARCH 1980 STUMP/VIBROFLOTATION
.SOIL 1 40 50 2.6 X=500 100 500 Y=211.26 274.32 274.32
.SOIL 2 35 0  1.84 X=500 500 Y=900 900
.SURFACE LOAD  1 1 100.02 118.3
.SURFACE LOAD  2 0.5 118.3 500
.ANCHOR  1 100.02 292.31 -15 8.15
.ANCHOR  2 100.02 290.61 -15 8.15
.ANCHOR  3 100.01 288.91 -15 8.15
.ANCHOR  4 100.01 287.21 -15 8.15
.ANCHOR  5 100    285.51 -15 9.62
.ANCHOR  6 100    283.81 -20 7.89
.SURFACE X=500 34.47 34.49 98.9 98.92 BASE=100 TOP=100.02 500
.Y = 290.32 290.32 285 282.96 282.36 282.36 293.16 314.12
.STAND 5 1.20 (CONTROL PARAMETER AND REQD FS)
.STPAR 38 = 1   39 = 6   26 = 2
.TIGLE
.END
```

Table 17. Sample computer output for Janbu's analysis of Section S-1 (continued).

b) Summary of slope geometry and soil parameters

```
DATA:

COORDINATES OF TOP OF SOIL STRATA
SOIL NO.     NO. PTS.    COORDINATES
   1            3        X = -500.00   100.00   500.00
                         Y =  211.26   274.32   274.32
   2            2        X = -500.00   500.00
                         Y =  900.00   900.00

GROUND SURFACE
  NO.        NO. PTS.    COORDINATES
   1            9        X = -999.00  -500.00    34.47    34.49    98.90    98.92   100.00   100.02   500.00
                         Y = -999.00   290.32   290.32   285.00   282.96   282.36   282.36   293.16   314.12

SOIL CONSTANTS:
LAYER     FI        C        GAMMA
 NO.    DEGREES    T/M2      T/M3

  1      40.00    50.00      2.60
  2      35.00      .00      1.84

POREWATER PRESSURE
NO POREWATER PRESSURE FOUND IN AREA INVESTIGATED
```

Table 17. Sample computer output for Janbu's analysis of Section S-1 (continued).

c) Surcharge and external forces

SURCHARGE:

MOBILE SURFACE SURCHARGE

NO.	LOAD (T/M2)	X-LEFT	X-RIGHT
1	1.00	100.02	118.30
2	.50	118.30	500.00

POTENTIAL EXTERNAL FORCES
CUT ANCHORS

ANCHOR LEVEL	POINT OF INSERTION		INCLINATION(DEGREES)	ANCHOR FORCE(T)	POSSIBLE LENGTH (M) WITHOUT FIXED PART
1	X = 100.02	Y = 292.31	-15.00	8.15	
2	X = 100.02	Y = 290.61	-15.00	8.15	
3	X = 100.01	Y = 288.91	-15.00	8.15	
4	X = 100.01	Y = 287.21	-15.00	8.15	
5	X = 100.01	Y = 285.51	-15.00	9.02	
6	X = 100.00	Y = 283.82	-20.00	7.89	

Table 17. Sample computer output for Janbu's analysis of Section S-1 (continued).

d) Results of calculations for null-point 100.00, 282.31 for the 6 anchor

SAFETY FACTOR WITHOUT ANCHOR INFLUENCE AND SHEARING RESISTANCE OF STRUCTURAL PARTS		WITHOUT	WITH	"	SAFETY FACTOR WITH SHEARING RESISTANCE OF STRUCTURAL PARTS AND ANCHOR EFFECTS USING CALCULATED ANCHOR LENGTHS L in M FOR THE ANCHOR LEVELS:											
					6	AND	5	AND	4	AND	3	AND	2	AND	1	
LINE NO.	THETA	ETA	ETA	DENOMINATOR	ETA	L	ETA	L	ETA	L	ETA	L	ETA	L	ETA	L
1	-10.00	1.96		134.75												
2	-8.00	1.69		132.20												
3	-6.00	1.51		130.43												
4	-4.00	1.37		129.11												
5	-2.00	1.27		128.09												
6	.00	1.19		127.28	1.28	4.41										
7	2.00	1.12		126.62	1.21	4.02										

DETERMINATION OF REQ'D ANCHOR LENGTH L = 4.88 M FOR THE ANCHOR LEVEL NO. 6

NULL-POINT NO.	X(M)	Y(M)	THETA DEGREES	POINTS OF INTERSECTION: L, A, B AND R X(L)	Y(L)	X(A)	Y(A)	X(B)	Y(B)	X(R)	Y(B)	SAFETY FACTOR SLIP ETA JANBU	SLIP LINES	TOTAL COMPT. TIME IN SECS.
1-6	100.00	282.31	.00	99.90	282.36	100.00	282.31	104.15	282.31	110.22	293.69	1.28	7	122.00

Table 17. Sample output for Janbu's analysis of Section S-1 (continued).

e) Summary of results

FOR NULL-POINTS 1 TO, 6 THE INVESTIGATION HAS DETERMINED
FACTORS OF SAFETY ACCORDING TO JANBU FOR APPROPRIATE SLIP PLANES
ETA = 1.25 FOR THE FOLLOWING SLIP PLANES:

NULL POINT NO.	X(M)	Y(M)	THETA DEGREES	POINTS OF INTERSECTION: L, A, B AND R X(L)	Y(L)	X(A)	Y(A)	X(B)	Y(B)	X(R)	Y(B)	SAFETY FACTOR SLIP ETA JANBU	SLIP LINES	TOTAL COMPT. TIME IN SECS.
3-54	100.00	280.31	56.00	95.66	283.06	100.00	280.31	106.86	290.48	108.53	293.61	1.25	339	757.00

COMPUTER NECESSARY ANCHOR LENGTHS WITHOUT FIXED ZONE

FOR ANCHOR LEVELS	6	5	4	3	2	1
LENGTH IN M	6.47	7.59	8.08	8.48	8.68	8.72
FOUND IN SLIP PLANE NO.:	3-12	4-25	2-27	3-37	2-42	2-49

THE ABOVE SAFETY FACTOR ACCORDING TO JANBU ETA = 1.20 WITH THE
POTENTIAL INFLUENCE OF STRUCTURAL PARTS AND ANCHORS WAS ATTAINED
IN ALL 339 INVESTIGATED SLIP SURFACES

20.00 18.00 20.00 169.50 .00 .00 .00 227.50

The calculated results are summarized in Table 18.

The final wall elevation layout and cross section are shown in Figure 53. To make more efficient use of the anchors, for the second row of anchors in section S3 two anchors were installed for every three elements. This is permissible from the standpoint of resisting the earth pressure. Also in section S3 stability considerations would allow smaller forces, but for practical reasons at least every second element has to be anchored. Assuming that the type of anchor rod is a Dywidag double-ribbed deformed steel bar u.t.s. 835/1030,Ø 26.5 mm with a risk grading for the anchor S = 2.0 the working load amounts to 289.6 kN and the test load equals 405.4 kN. The elastic limit of the steel is 835 N/mm2 and its tensile strength is 1030 N/mm2.

Steel grade III (elastic limit 0.2%, 460 N/mm2, tensile strength 560 N/mm2) is used for the reinforcement of the elements. The normal size of element is 1.70 m x 3.55 m.

The unanchored foot of the wall is 60 cm high and 110 cm wide. The row of elements directly in contact with it has the function of height compensation (its height is variable). The top of this row is horizontal.

The top of the wall runs parallel to the ground surface. Horizontal closure elements with vertical steps could also have been chosen. However, this would either mean having small slopes above the wall or the wall sticking out above the ground surface. In our project we have not fixed these small details - it is really a question of architectural preference.

Beneath the element wall a reliably functioning drainage system must be installed. Also at distances of 7.10 m a vertical drainage system must be built for the individual phases of construction (ϕ approx. 10 cm, embedded in gravel or permeable concrete), which is connected to the foot of the wall and runs from the front of the wall into a horizontal collecting system.

It must be possible to flush out the whole system. If the drainage system fails the water level could rise behind the wall leading to additional hydrostatic pressures. Further, the stability conditions could make a change for the worse. Thus the greatest care must be given to the handling of this question.

In constructing the element wall the following constructional steps must be followed:

a) Before starting to excavate for the next (lower-lying) row of elements the complete upper row must be anchored in place.

b) The excavation must be carried out in a staggered manner, i.e. no two adjacent elements must be excavated in one piece.

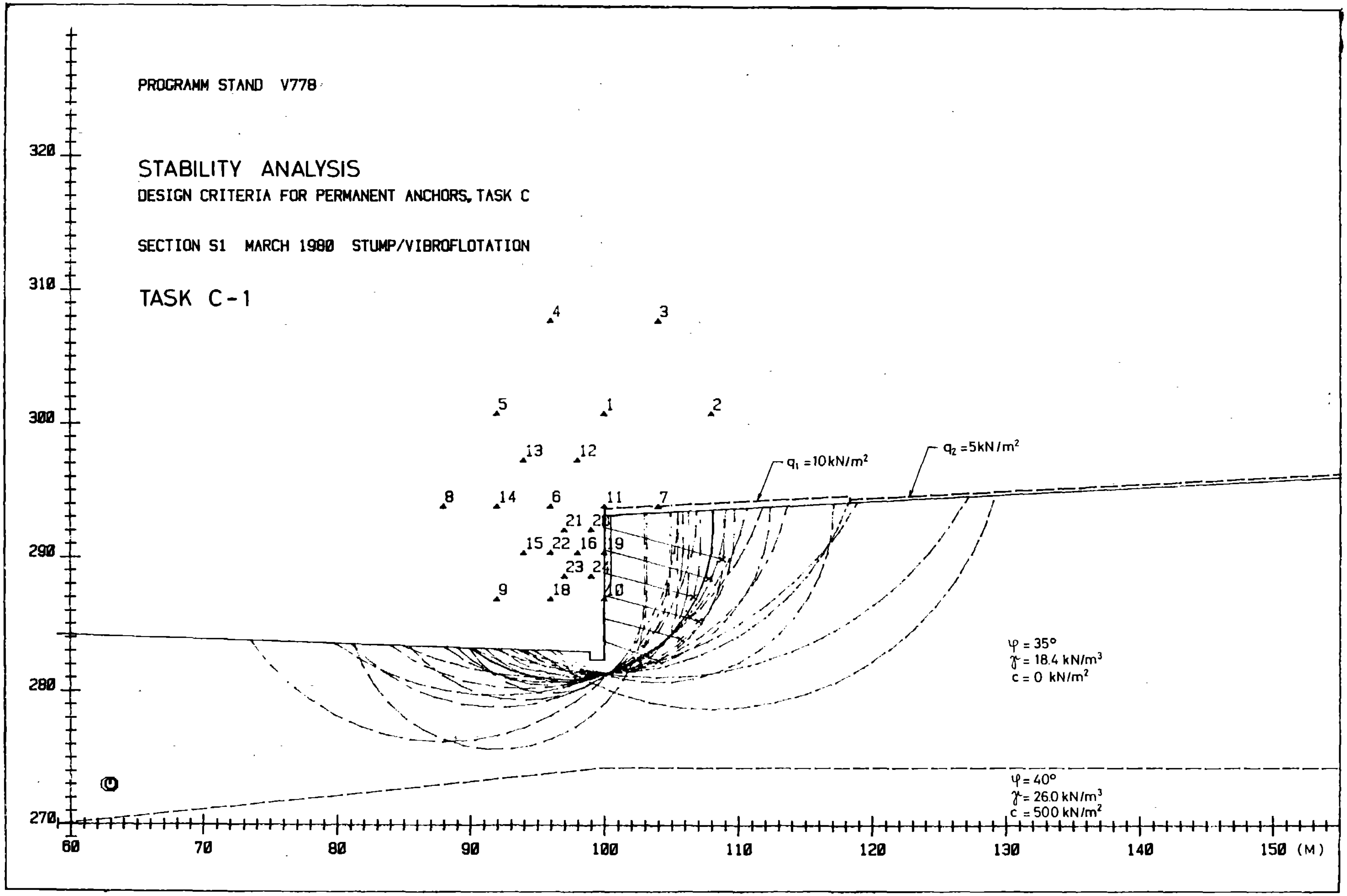

Figure 51. Cross section at S-1 for Krey's analysis.

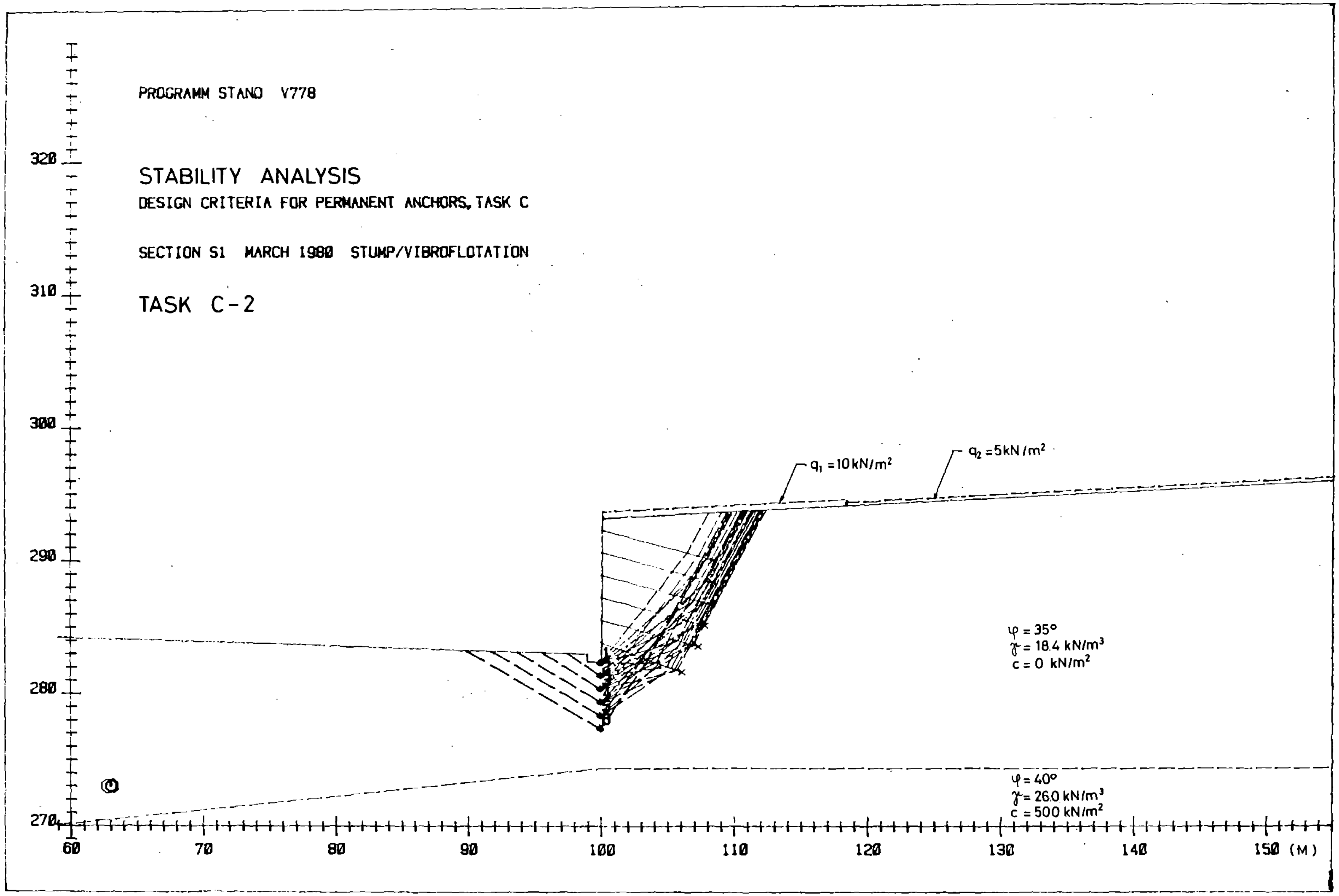

Figure 52. Cross section at S-1 for Janbu's analysis.

Table 18. Summary of calculated results.

	Anchor Position	Anchor Force (from earth pressure diagram) kN/m'	S.F. for Krey's Analysis	Increased Anchor Force kN/m'	S.F. for Krey's Analysis	S.F. for Janbu's Analysis	Free Anchor Length (Krey) m	Free Anchor Length (Janbu) m	Design Anchor Length m
Section S1	1	63,4	1,47 < 1,50	81,5	1,58 > 1,50	1,25 > 1,20	9,17	8,72	14.5
	2	63,4		81,5			8,13	8,68	13,5
	3	66,7		81,5			6,92	8,48	12,5
	4	81,5		81,5			7,35	8,05	12,5
	5	96,2		96,2			5,93	7,59	11,0
	6	78,9		78,9			4,33	6,47	9,5
Sect.S2	1	54,2	1,44 < 1,50	81,5	1,54 > 1,50	1,29 > 1,20	7,09	6,97	12,5
	2	54,2		81,5			6,08	7,48	12,0
	3	66,7		81,5			6,56	7,22	12,0
	4	81,5		81,5			5,63	6,89	11,0
	5	67,5		67,5			3,66	5,42	9,0
Sect.S3	1	45,0	1,42 < 1,50	70,0	1,66 > 1,50	1,36 > 1,20	5,3	5,37	11,0
	2	52,0		70,0			5,64	6,13	11,0
	3	66,7		70,0			5,26	5,76	10,5
	4	56,0		56,0			2,98	4,86	8,0
Section S4	1	37,3	1,37 < 1,50	50,0	1,91 > 1,50	1,79 > 1,20	3,52	3,14	9,0
	2	33,1		50,0			2,51	3,11	8,0

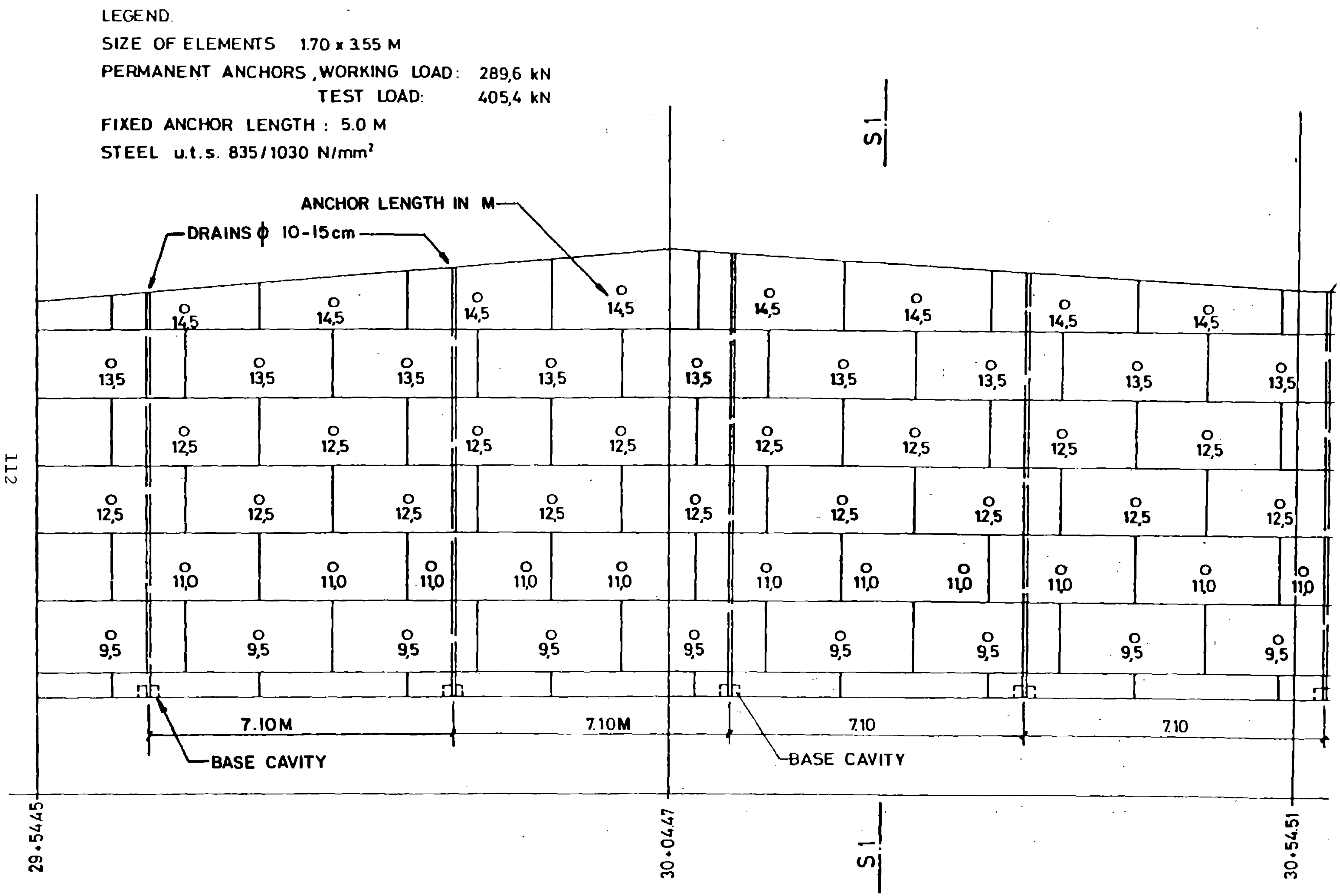

Figure 53. Wall layout and cross section.

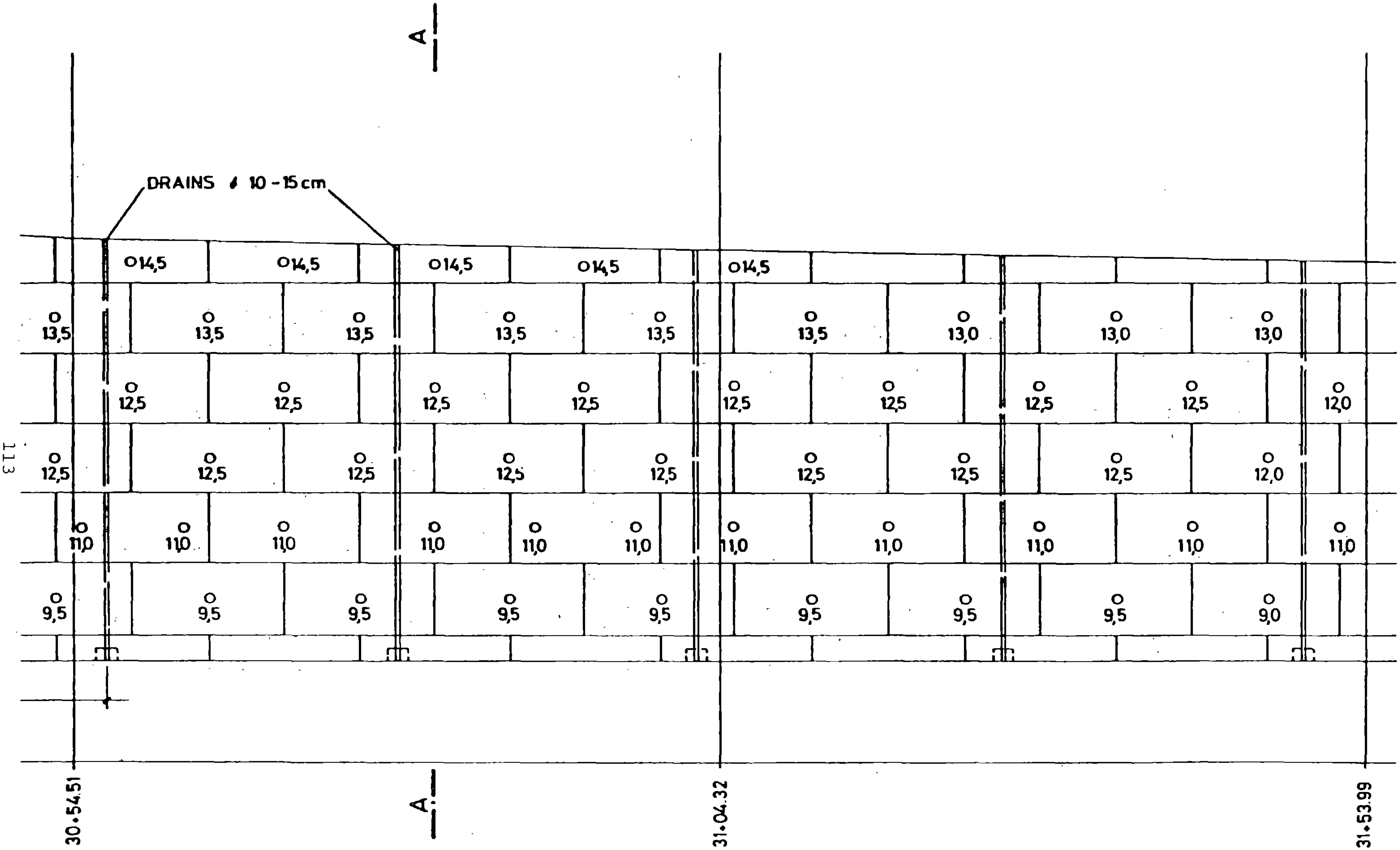

Figure 53. Wall layout and cross section (continued).

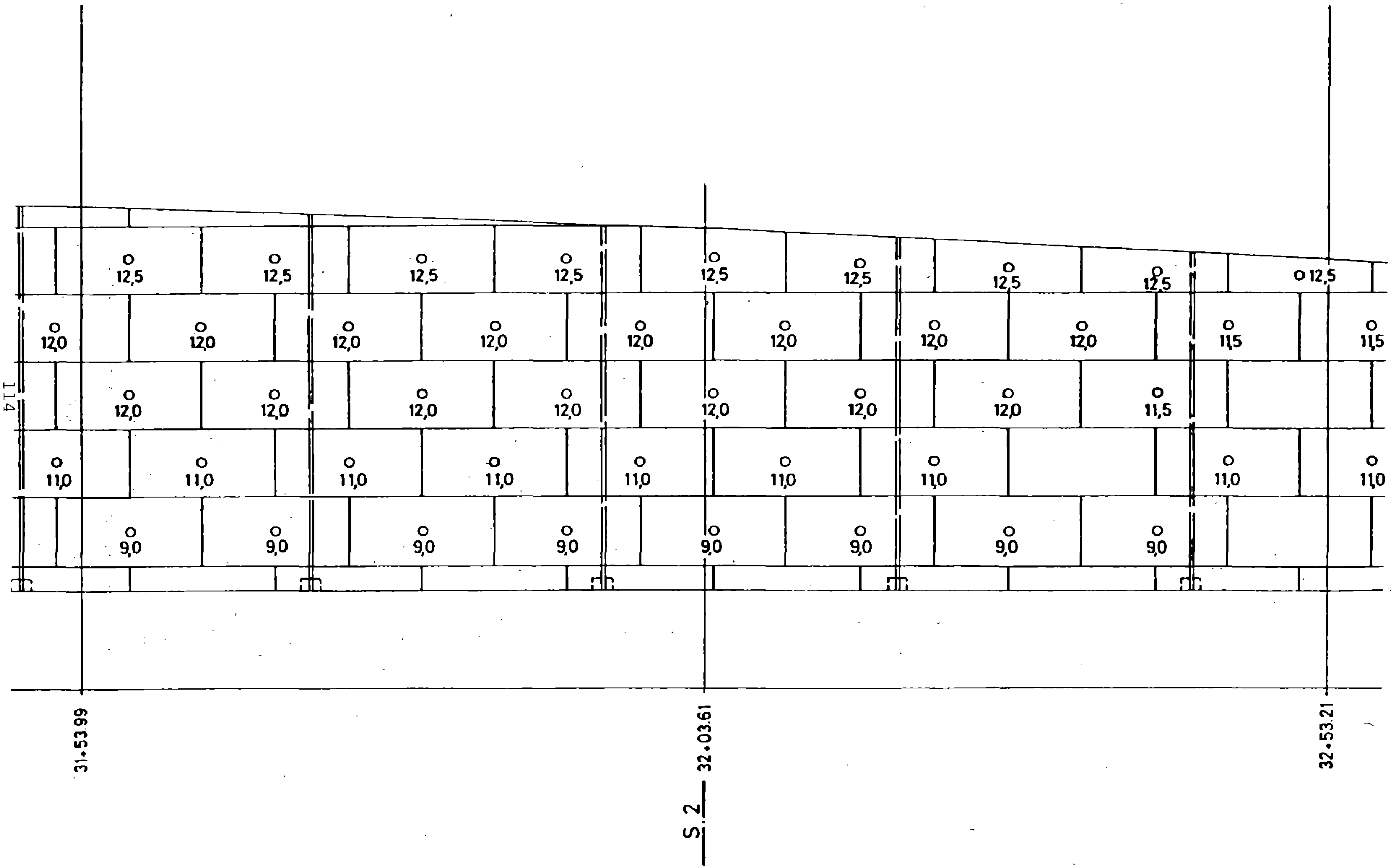

Figure 53. Wall layout and cross section (continued).

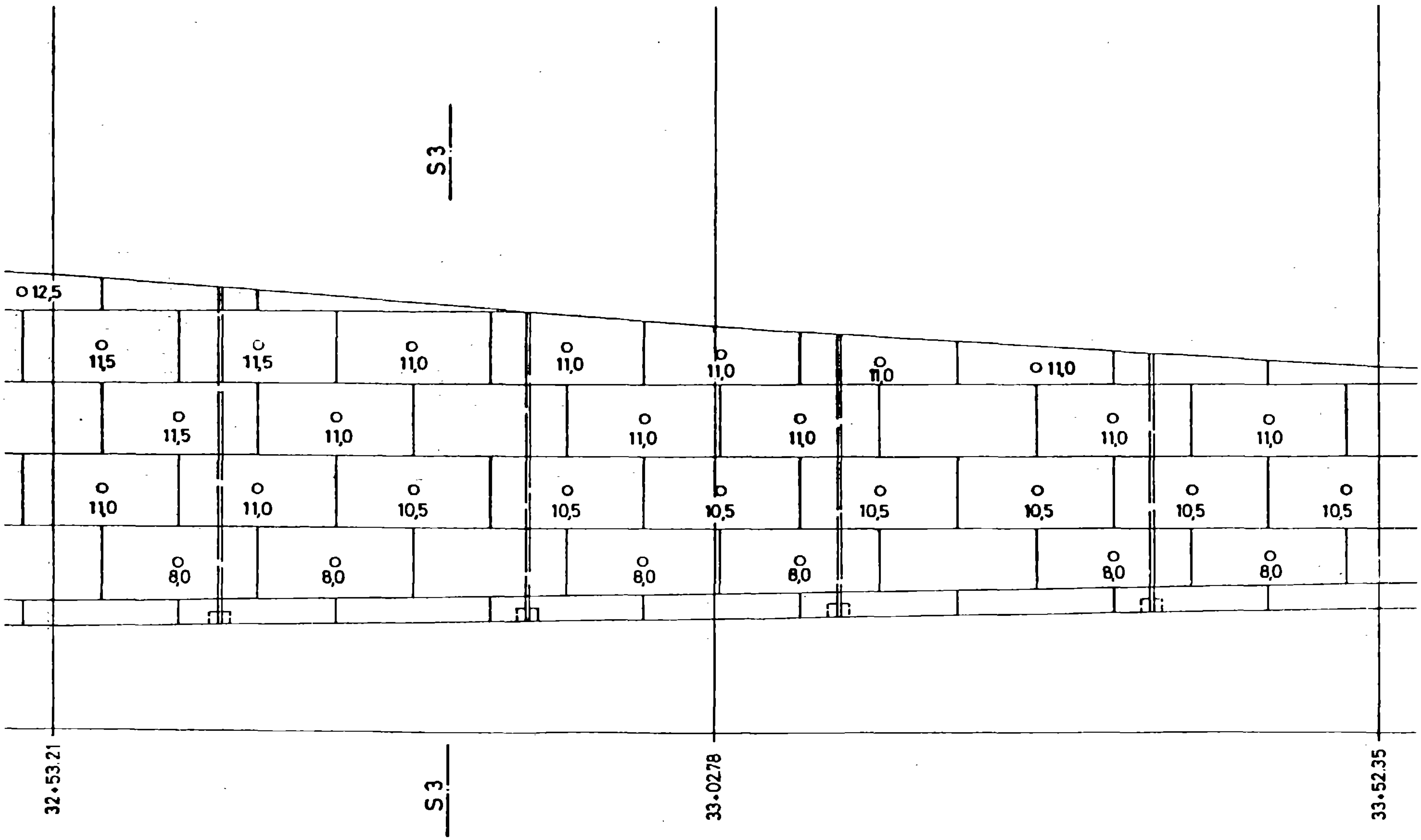

Figure 53. Wall layout and cross section (continued).

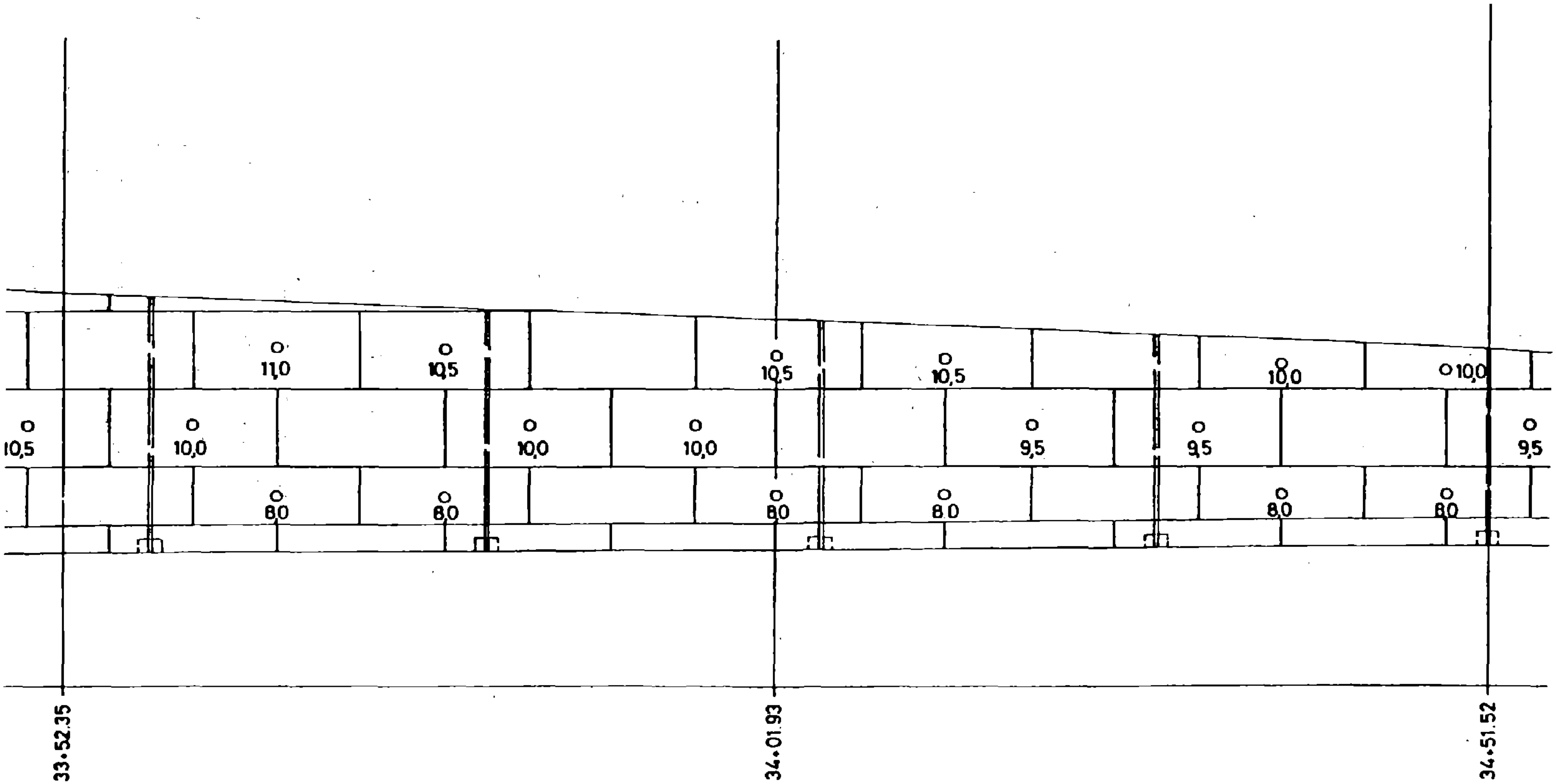

Figure 53. Wall layout and cross section (continued).

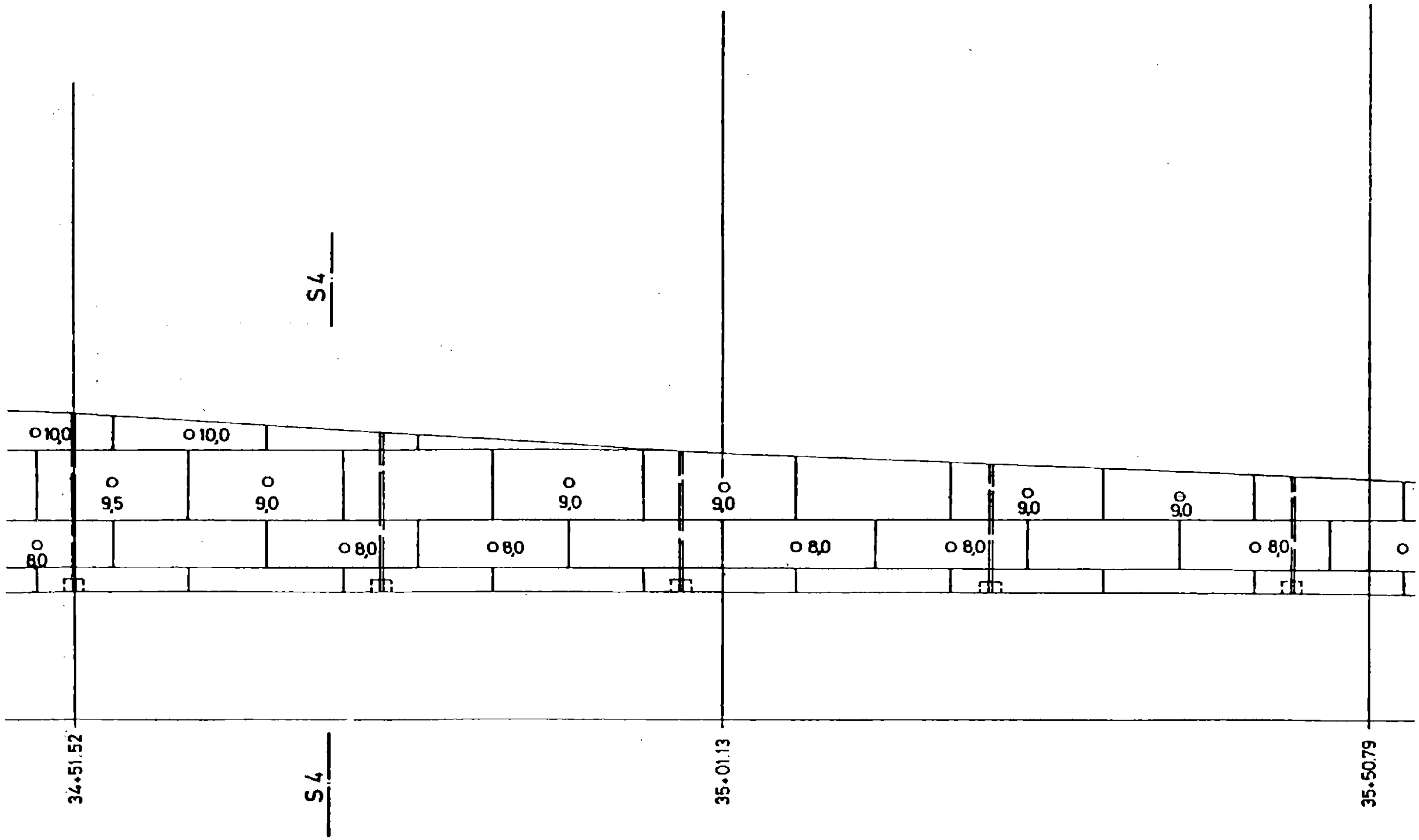

Figure 53. Wall layout and cross section (continued).

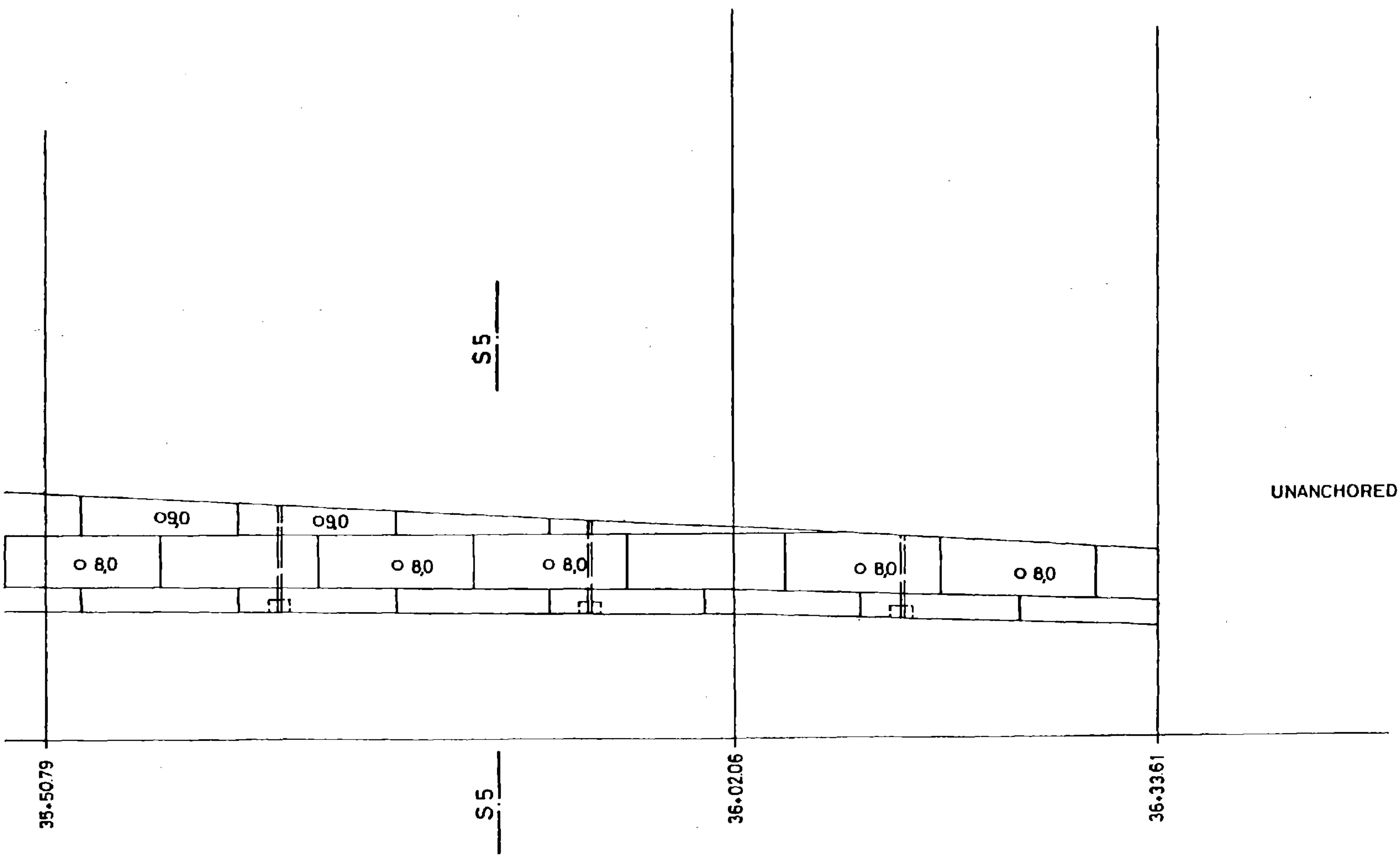

Figure 53. Wall layout and cross section (continued).

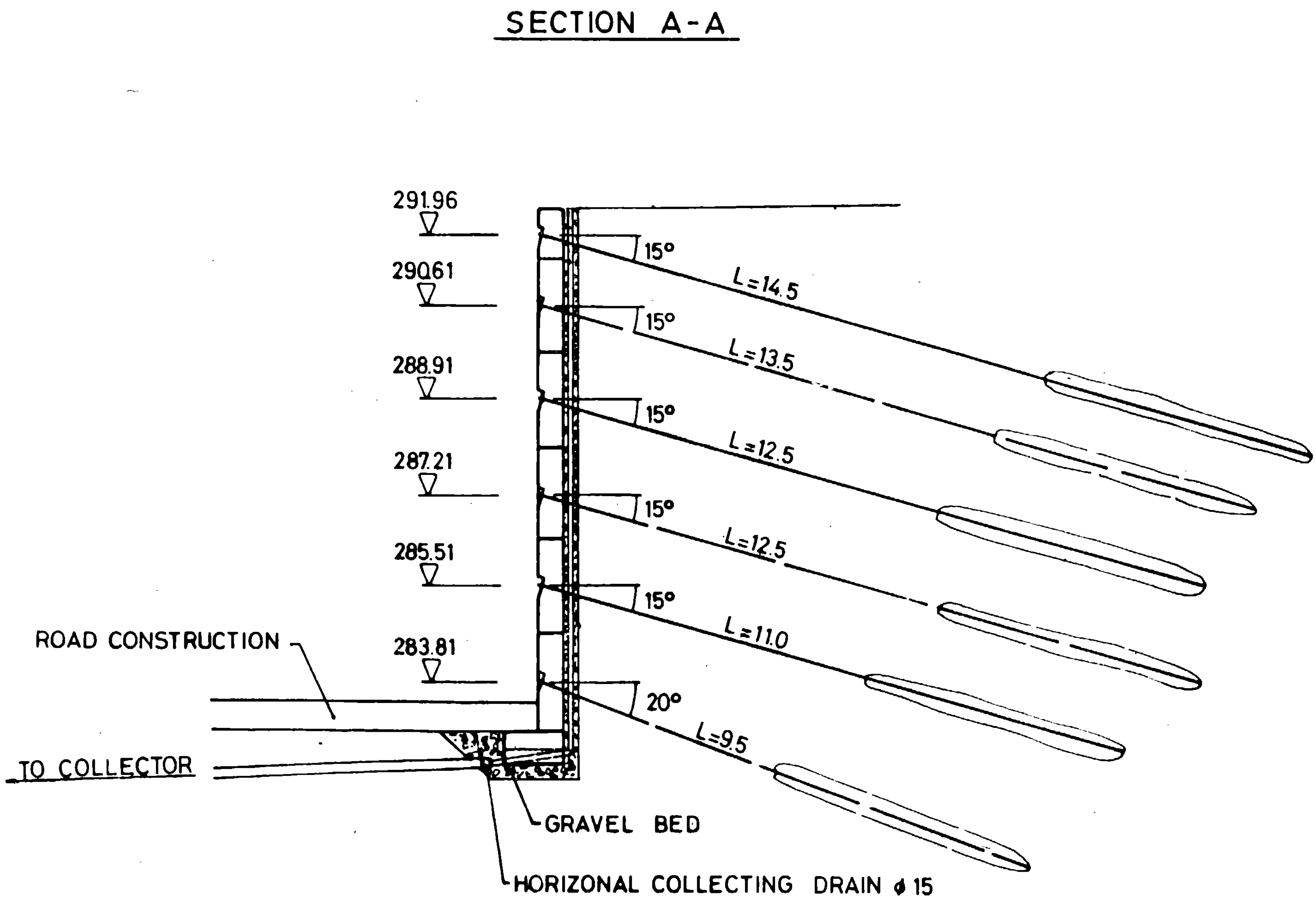

Figure 53. Wall layout and cross section (continued).

c) The foot of the wall must be placed without forming, so that its support effect, which has been assumed in the calculation, is fully effective.

d) The elements must be concreted the same day that the ground is excavated.

e) Before carrying out the project the contractor must check to see if the anchors cut across any buried pipes (gas and water mains, etc.)

Monitoring

In order to observe the wall-anchor-soil system and to interpret correctly the qualitative and quantitative changes in its behavior, we recommend that the following measuring stations be set-up in three sections:

a) deformation stations

A borehole is drilled in each section at a distance of about 1.0 m behind the wall and extending to a depth 5-6 m below the foot of the wall. A further row of 3 measuring stations is set-up in the same sections, each station being located about 15 m behind the wall. These boreholes must be constructed before the excavation work is carried out. The back row of boreholes may be made 3 m shorter than the front row.

b) force measurements

In each of the three measuring sections the anchors should be fitted with electrical force transducers (or other precision load measuring cells) in each anchor position in an alternate manner left and right of the measuring tube. Both deformation and force measurements should be carried out after the construction of each row of elements and possibly before and after stressing the anchors.

The monitoring could be extended to include geodetical measurements. In each case a measuring program should be worked out and executed (especially with regard to the number of measurements).

The structure should be observed periodically after being turned over to the client.

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FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion, and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements that affect the quality of the human environment. The goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge and technology of materials properties, using available natural materials, improving structural foundation materials, recycling highway materials, converting industrial wastes into useful highway products, developing extender or substitute materials for those in short supply, and developing more rapid and reliable testing procedures. The goals are lower highway construction costs and extended maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural and hydraulic designs, fabrication processes, and construction techniques to provide safe, efficient highways at reasonable costs.

6. Improved Technology for Highway Construction

This category is concerned with the research, development, and implementation of highway construction technology to increase productivity, reduce energy consumption, conserve dwindling resources, and reduce costs while improving the quality and methods of construction.

7. Improved Technology for Highway Maintenance

This category addresses problems in preserving the Nation's highways and includes activities in physical maintenance, traffic services, management, and equipment. The goal is to maximize operational efficiency and safety to the traveling public while conserving resources.

0. Other New Studies

This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.