PRESTRESSED ANCHORAGES FOR RETAINING STRUCTURES - APPLICATIONS AND CONSTRUCTION

by

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1. INTRODUCTION

Since their first use at Cheurfas Dam, Algeria, in 1934, prestressed ground anchorages have become accepted worldwide as a reliable and cost effective technique for both temporary and permanent applications. In this country, our industry has been particularly well served for three decades by individuals from the bulk of the group represented at this seminar: specialty contractors, certain geotechnical consultants, and materials suppliers. In addition, it has been stimulated - and regulated - by Federal and State authorities, and moulded by bodies such as the Post Tensioning Institute, and, of course, more recently by the Association of Drilled Shaft Contractors. We are also fortunate that the Geotechnical Division of the American Society of Civil Engineers is active in organizing international forums for the exchange of information: the specialty conferences at Cornell in 1990, and at New Orleans in 1992 are excellent examples.

Anchorages have many applications, as collated by Hanna (1982), and Xanthakos (1991) among others. There are several different types of anchorages, and an even wider range of construction methods. This short paper restricts its scope to that of the seminar's theme - ground retention - and provides generic classifications for the various construction activities: in short, it attempts to provide an introductory framework upon which the subsequent papers can be affixed.

Details of design, construction and performance can be gleaned from the works referenced above, from the companion papers presented at this seminar, and from numerous other key papers including Littlejohn and Bruce (1977), Littlejohn (1990a & b, 1992), Bruce (1989, 1991) and Scott and Bruce (1992). With respect to specifications and standards, the following publications will prove to be particularly instructive: FIP (1982), FHWA (1990), PTI (1986) and BSI (1989).

2. APPLICATIONS

Hanna's (1982) book devotes a chapter of 130 pages to 26 different applications of prestressed ground anchorages while Xanthakos (1991) was later able to condense this into 16 categories and 44 pages. Recent conferences such as that sponsored by ASCE at Cornell in 1990 are also replete with excellent case histories.

Applications involving earth and rock retention are self explanatory and may be listed as follows:

- slope stabilization (Figure 1)
- protection for walls for structures threatened by creeping soil (Figure 2)
- cliff stabilization (Figure 3)
- simple excavation support (Figures 4, 5, 6, 7, 8, 9)
- complex excavation support (Figure 10)
- canal structures (Figure 11)
- flood control structures (Figure 12)
- strengthening of existing structures (Figure 13)
- stabilization of underground structures (Figure 14)
- tunnel construction (Figure 15)
- underpinning (Figure 16)

3. DRILLING TECHNIQUES

It will be noted that most of the applications of anchors listed in Section 2 involve drilling through materials other than rock, even where the bond zone is founded in rock. This ability to penetrate through "poor" as well as competent ground is absolutely fundamental for the efficient, and cost effective installation of anchorages, and yet little attention is typically focused on this aspect. In contrast, the more cerebral students of the business tend to devote more attention to the performance of the anchorage, which perversely, of course, can owe a great deal to the drilling method. To further complicate the issue, few drilling equipment suppliers, or contractors, have the knowledge or inclination to consider drilling methods outside those they choose to promote or favor. The consequence is that, especially in the field of overburden drilling, there exists little industry perspective of the range of methods and equipment available. As a result, prospective owners or clients are faced with proprietary or specific drilling options, usually chosen less for their appropriateness to the ground conditions than for their immediate availability in the contractor's yard.

This short review provides a generic guide to the fundamental methods available for use in ground anchorage drilling.

3.1. Rock

Rock anchorage holes can be drilled by one of three principles, depending on the strata, hole diameter and length, the drilling rig available, and the cost/benefit ratio (Figure 17).

1. Rotary Percussive: by top hammer
2. Rotary Percussive: by down the hole hammer
3. Rotary.

The first group involves the use of a mast mounted rotary-percussive head, now usually hydraulically activated. This method, typically used in the shallow blast hole industry, is restricted in terms of both diameter, depth, penetration rate and hole linearity. Air or water flush can be used.

The second group requires a mast mounted rotary head, while the percussion component is provided by a pneumatically powered hammer impacting directly on the drilling bit. This has been proved to be the fastest, cheapest and straightest method of drilling holes in excess of say 4" to depths of over 50' (Bruce, et al., 1991). In addition, recent field researches have demonstrated (Bianchi and Bruce, 1992) that even when this vigorous drilling method is conducted in delicate structures within a few feet of a free edge, minimal damage is caused either by the vibrations (Figure 18) or the expelled compressed air (Table 1).

Elevation of Crackmeter (feet)	Maximum Recorded Movement (in)	Typical Daily Movement due to Temperature Effect only (in)	Approx. Distance from Meter to Hole (ft)
1520.39	0.00239	0.00284	5.0
1510.45	0.00218	0.00284	5.5
1500.34	0.00409	0.00432	5.8
1490.43	0.00510	0.00348	5.8
1480.56	1	0.00353	5.5
1470.17	1	0.00376	5.5
1460.21	1	0.00459	5.5
1450.23	1	0.00440	5.5

1 No discernable movement was detected during the drilling operation.

Table 1. Summary of Crackmeter Data
(Hole 37, Stewart Mountain Dam)

Rotary drilling in the rock anchorage field is invariably associated with high torque, high thrust, low speed drilling using tricone bits or similar. The use of low torque, low thrust, high speed drilling using diamond bits, or coring is simply not economic and indeed is potentially detrimental to rock-grout bond in that it produces a very smooth borehole wall. Rotary drilling can be conducted with either air or water flush, although the latter is not advisable in claystones or similar.

3.2. Overburden Drilling

One may delineate six basic methods of overburden drilling (Table 2), excluding:

a) the rare (in America) technique of open holing through appropriate overburden with "self hardening drilling mud" and

b) the use of vibratory driven casing which is possible only in a very narrow range of soil conditions.

DRILLING METHOD	PRINCIPLE	COMMON DIAMETERS AND DEPTHS	NOTES
1. Single Tube Advancement			
a) Drive Drilling	Casing, with "lost point" percussed without flush.	2-4" TO 100'	Hates obstructions or very dense soils.
b) External Flush	Casing, with shoe, rotated with strong water flush.	4-8" to 150'	Very common for anchor installation. Needs high torque head and powerful flush pump.
2. Rotary Duplex	Simultaneous rotation and advancement of casing plus internal rod, carrying flush.	4-8" to 200'	Used only in very sensitive soil/site conditions. Needs positive flush return. Needs high torque.
3. Rotary Percussive Concentric Duplex	As 2, above, except casing and rods percussed as well as rotated.	3-1/2 -7" to 120'	Useful in obstructed/bouldery conditions. Needs powerful top rotary percussive hammer.
4. Rotary Percussive Eccentric Duplex	As 2, except eccentric bit on rod cuts oversized hole to ease casing advance.	3-1/2 -8" to 200'	Obsolescent, expensive and difficult system for difficult overburden. Largely restricted to water wells.
5. "Double Head" Duplex	As 2 or 3, except casing and rods rotate in opposite senses.	4-6" to 200'	Powerful, newer system for fast, straight drilling in worst soils. Needs large hydraulic power.
6. Hollow Stem Auger	Auger rotated to depth to permit subsequent introduction of tendon through stem.	6-15" to 100'	Hates obstructions, needs care in cohesionless soils. Prevents application of higher grout pressures.

Table 2. Summary of Overburden Drilling Methods for Anchors (Bruce, 1989a)

Within the drilling industry overall, the trend continues towards the use of higher powered diesel or electro-hydraulic self propelled rigs. These have advantages in maneuverability, torque, thrust and pullup and have operational and environmental efficiency unmatched by older models including airtracks. Modern rigs often have long masts to minimize or eliminate rod or casing changes, and mechanical foot clamp/breakout wrench attachments to aid rod or casing handling by making it quicker, safer and less labor intensive. It is reasonable to assume that such rigs promote straight and accurate drilling in the hands of skilled operators - and in this regard the typical industry standard for maximum hole deviation is about 1 in 30 to 50 depending on a myriad of factors.

For cased, or duplex drilling operations, high pressure, high volume water flush is most common in appropriate geologies. Not only does it permit fast penetration but it tends to remove fines from the adjacent soil, thus rendering the subsequent anchor pressure grouting more effective, and so enhancing bond development potential.

Air flushing alone is potentially damaging in urban areas and is therefore quite rare in soils. However, recent developments with foaming additives could well see a resurgence in low volume air based flushing in certain geological or logistical circumstances. Similarly, there is a growing acceptance of polymer slurries in overburden drilling, often to the exclusion of simultaneous casing advancement. Care should be exercised, of course, to verify that any such flushing materials in no way will compromise subsequent anchor performance. For such reasons, the use of bentonite slurry alone as a flush must be avoided, although the advantages of certain "self hardening drilling muds" - cementitious mixes, with bentonite as an integral bulking component - have been demonstrated in voided or cavitated conditions.

Perhaps more than any other aspect of construction, overburden drilling techniques and methods seem to have greatest regional variation across the country. For example, eastern contractors tend to favor the use of some form of casing system, usually with water flush, while on the west coast, the use of larger diameter hollow stem augers has prevailed. Overburden drilling methods should ideally be related directly to the anticipated soil conditions, but appear to be most often dictated by the historical proclivities - and the equipment available - of the individual contractor.

Taken to extremes, this inertia can in fact rule out the possible use of anchors in a given project. For example, in a recent contract in Los Angeles, a relatively high degree of lateral restraint had to be applied to a concrete diaphragm wall to satisfy local design rules. The use of the traditional large diameter hollow stem drilling method would have involved such a large number of low capacity anchors (due to the limited grout/soil bond capacity potential with this method) that the wall's structural integrity would have been threatened by the number and diameter of perforations through it. The use of a flushed casing method permitted the prestress to be distributed into a lower number of higher capacity anchors of much smaller diameter. Thus, the change of drilling method not only reduced the cost of the retention system, but in fact made anchoring a practical option in the first place.

The choice and application of the most appropriate drilling method must also reflect the overall site conditions in general. For example, the use of air - an excellent "scavenger" and an aid to fast penetration - as a flushing medium in urban areas should normally be discouraged as noted above. There have been numerous examples of structural damage to adjacent buildings as a result of ground fracture or upheaval, or simply the consequence of massive air losses acting directly on base slabs. Alternatively, in other applications, air flushing is a possible remedy against the water softening of cohesive soils which can subsequently reduce bond potential.

In such conditions without obstructions, augering may be the most apposite choice, especially if drill depths are not great and subsequent anchor capacity is moderate. However, in soils with low cohesion or very poor cementation, the uncontrolled use of augers may result in severe decompression or cavitation of the soils around the borehole, again leading to reduced bond potential and the risk of adjacent structural distress.

4. GROUTING METHODS AND THEIR CONTROL OVER GROUND ANCHORAGE CLASSIFICATION

Littlejohn (1990a) proposed a classification of anchorage types based largely on the mode of grouting (Figure 19). In broad terms, rock anchorages tend to be largely "Type A" while in softer argillaceous strata "Type D" anchorages are an option, albeit a progressively rarer one. Anchorages in frictional soils are commonly of "Type B" whereas installations in more cohesive deposits are increasingly installed as "Type C". Anchorages installed in soils amenable to the hollow stem auger method are generically of "Type A", thus explaining the relatively low grout/soil bond values they are also associated with.

The key role that grouting plays in anchor construction and performance merits a closer examination of some basic principles (Bruce 1992).

There are fundamentally four types of pressure grouting for soil (Figure 20), if the simple target of void filling is left aside. Void filling occurs when grout under its own head is simply tremied into the hole without the intention of permeating into the soil, densifying the soil or otherwise improving the soil at or away from the borehole interface. Such grouting is used in rock anchors or Type A soil anchors. Jet grouting, with the exception of the field test run in England (Anon, 1988) has not proved a viable grouting method or concept, applicable for anchoring in the United States, although this may change soon.

When grouting anchors in soil, the aims are typically to permeate for some finite distance around the drill hole, to enhance the "effective bulb" diameter, and to cause some compaction of the ground if disturbed during the drilling process. Permeation will occur in coarse sands and gravels, but the phenomenon of "pressure filtration" will normally limit radial travel to a few inches in most cases using typical anchor grouts. This same phenomenon will squeeze out some of the integral mixing water leaving behind an anchor grout of water content considerably lower than that injected, and therefore considerably stronger than the corresponding cube results. For this reason, water/cement ratios used in cohesionless soils can be a little higher than those used for clays and tills and so on, without the drawbacks normally inherent with such mixes (Figure 21: reduced strength, significant bleed potential). Ratios for the former can be as high as 0.55 (assuming significant injection pressures are used), while it is prudent to limit water/cement ratios to 0.45 in cohesives.

Grouts used in post grouting are typically of slightly higher water content than those used in the initial grouting, but still require mixing - to ensure high quality grout - in a colloidal, high speed mixer. The higher pressures needed largely to overcome line and sleeve back pressures can usually only be provided by piston pumps.

5. LONGEVITY AND CORROSION PROTECTION

An excellent discussion of the terms "temporary" and "permanent" was provided by Xanthakos (1991) who noted that the distinction is "arbitrary at best and often academic". Several codes specify the duration of temporary service as 2 years but Xanthakos reminds us that "this guideline should be accepted with caution and full understanding of its limitations, and where soil conditions are fully known and controllable". In addition, the fact that an anchorage is "temporary" does not make it any less important that it performs - in every way - equally as well as its permanent counterpart.

One consequence therefore, is that it is simplistic to mandate that temporary anchors need no special corrosion protection, other than the hardened cement grout. In their study of 35 anchor failures due to corrosion, FIP (1986) found that 9 failed within 6 months, and 10 in the period from 6 to 24 months. Littlejohn (1992) concluded that this "fact ... confirms that where the environment is aggressive, temporary anchorages should be given appropriate protection." He further notes that there is "no single parameter which can be used to predict the risk or corrosion -- bearing in mind that corrosion can be chemical, electrochemical and/or microbiological in nature." He then cites strong international evidence in support of his conclusion that "grout injected in situ to bond the tendon or its encapsulation to the ground should not be considered as part of the designed protective system in aggressive ground."

He then goes on to reaffirm the general world view - which does not reflect general opinion or current practice in the U.S. The choice of protection (Table 3) should logically depend on such factors as consequence of failure, aggressivity of the environment and cost of protection, By definition simple protection implies that one physical barrier against corrosion is provided for the tendon prior to installation. Double protection implies the application of two barriers where the purpose of the outer second barrier is to protect the inner barrier against the possibility of damage during final tendon handling and placement.

Anchorage category	Class of protection
Temporary	Temporary without protection Temporary with single protection Temporary with double protection
Permanent	Permanent with single protection Permanent with double protection

Table 3. Proposed Classes of Protection for Ground Anchorages (Littlejohn, 1990)

Littlejohn concludes by stating "if the conservative assumptions and associated tendon protection systems were communicated more widely in future, confidence in the use of permanent anchorages would be enhanced."

In the United States we regard tendons of the type shown in Figure 25 as "double corrosion protected". Equally we view epoxy coated strands grouted into holes also as having double corrosion protection. By international standards these are merely simple protections and this is one of the key issues which the anchor industry must address itself towards in the years to come, especially as more anchors will be installed in urban and marine environments where natural and artificial corrosive agents are strongly present.

6. FINAL REMARKS

This short paper provides a brief generic introduction to various aspects of anchorage classification and construction. The crucial question of the philosophy of corrosion protection is also touched upon. The serious student of anchorage technology is recommended to pursue all these aspects, in appropriate depth, in the references cited.

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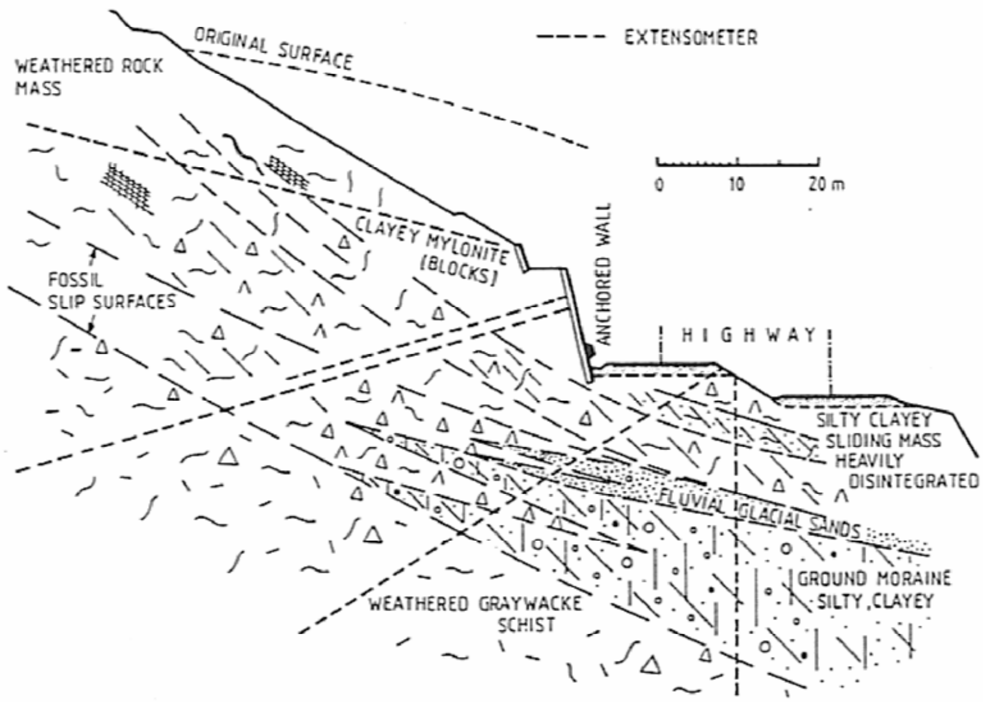


Figure 1. Ground profile showing location of anchored wall and highway (from Hanna, 1982)

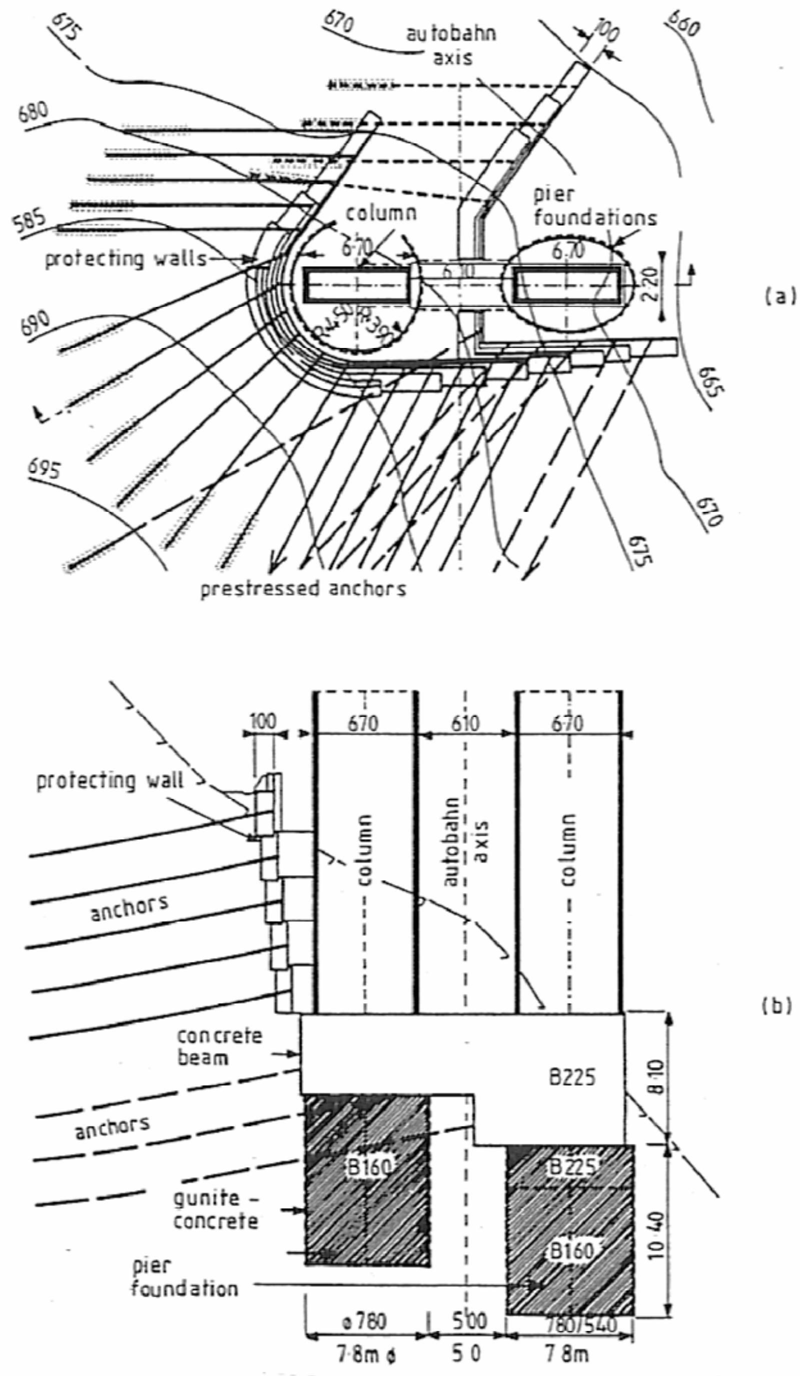


Figure 2. Bridge pier foundation protection against soil creep by an anchored wall (a) plan, (b) elevation (from Hanna, 1982)

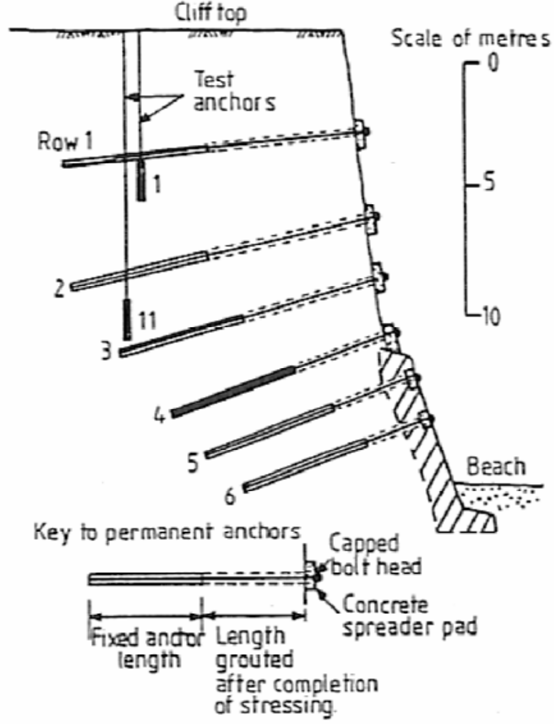


Figure 3. Anchor arrangement for cliff stabilization in chalk, Isle of Thanet (from Hanna, 1982)

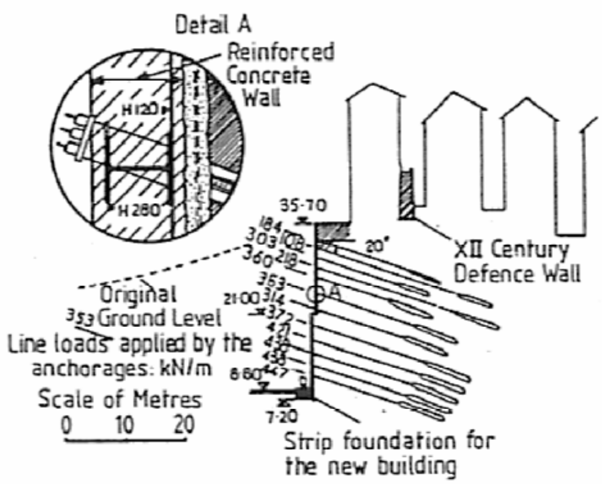


Figure 4. Vertical cross-section through the anchored wall (from Hanna, 1982)

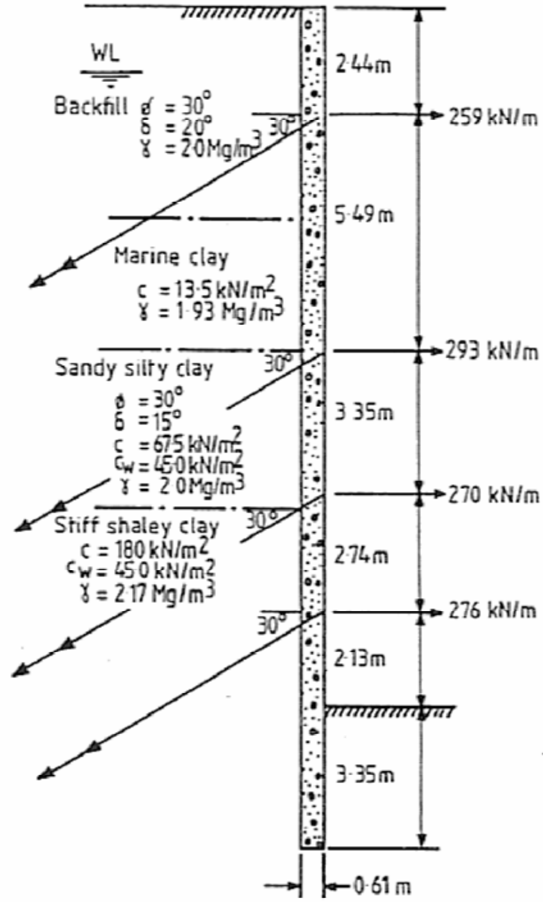


Figure 5. Anchored diaphragm wall for CPF building, Singapore (from Hanna, 1982)

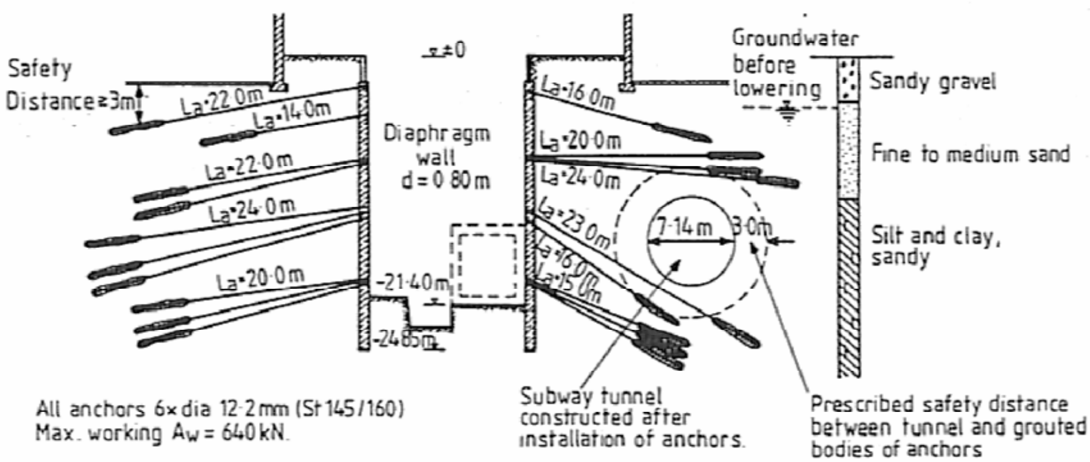
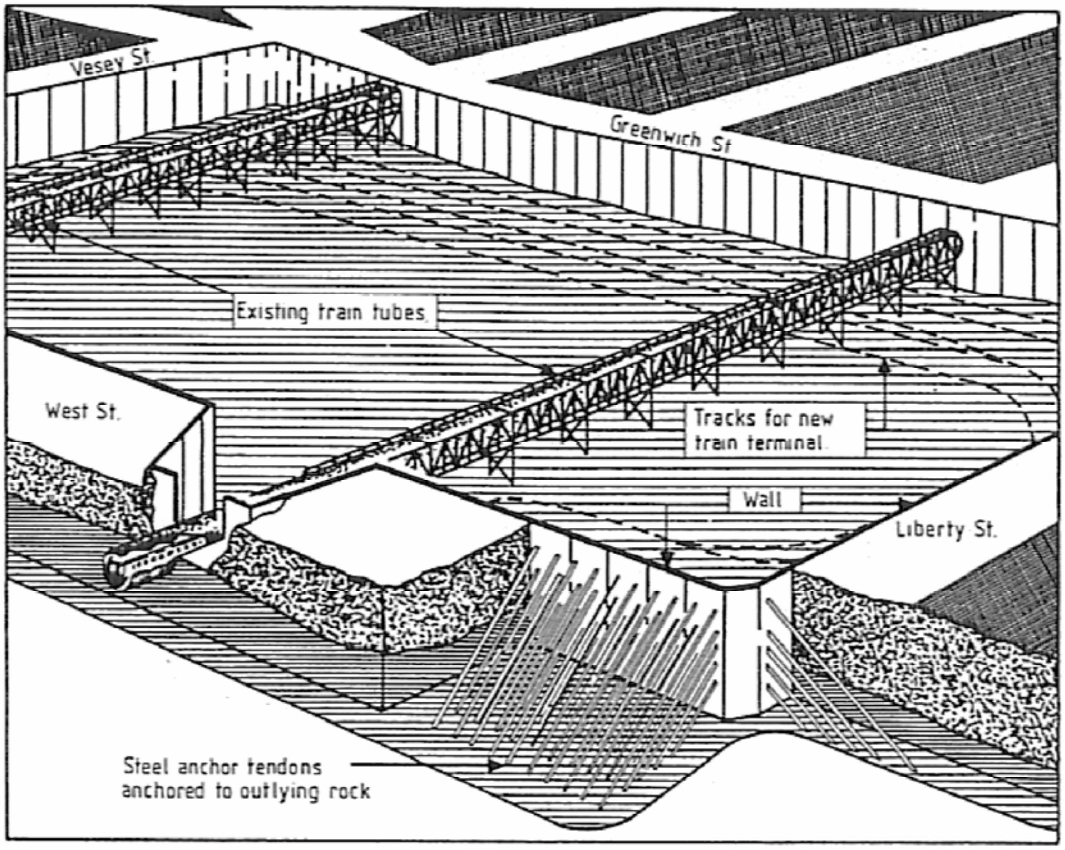
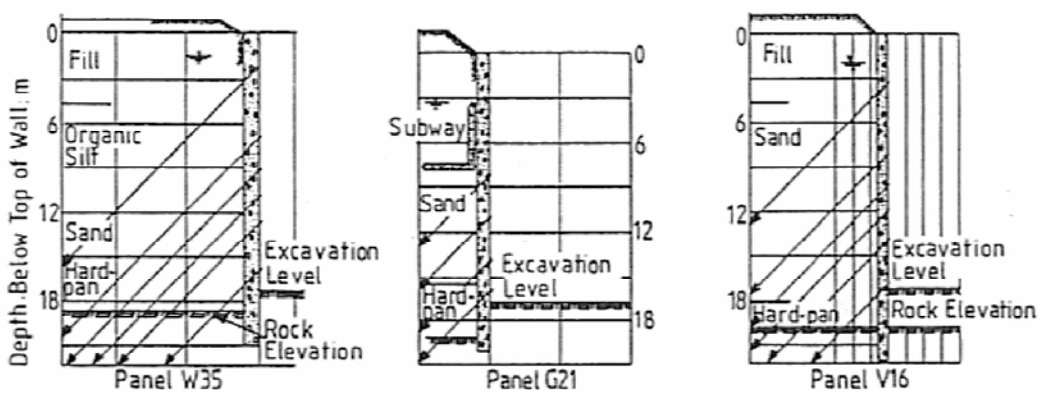


Figure 6. Arrangement of ground anchors to support deep excavation for subway station in Munich (from Hanna, 1982)



General arrangement of deep excavation works for World Trade Center, New York, showing anchored diaphragm walls and existing underground railways



Details of three test panels of anchored walls, World Trade Center, New York

<u>Figure 7.</u> World Trade Center, New York (from Hanna, 1982)

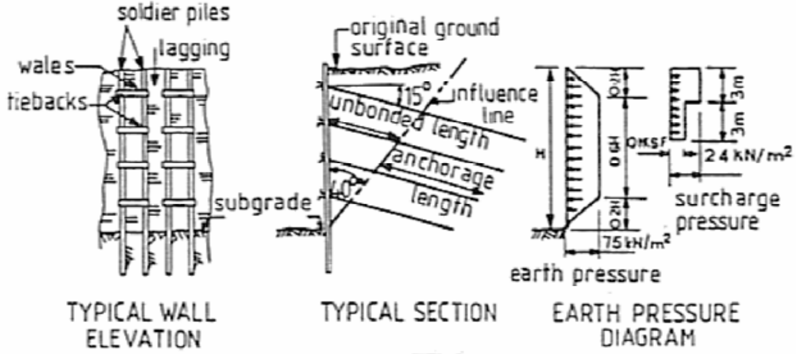


Figure 8. General details of soldier-pile timber-lagged wall (from Hanna, 1982)

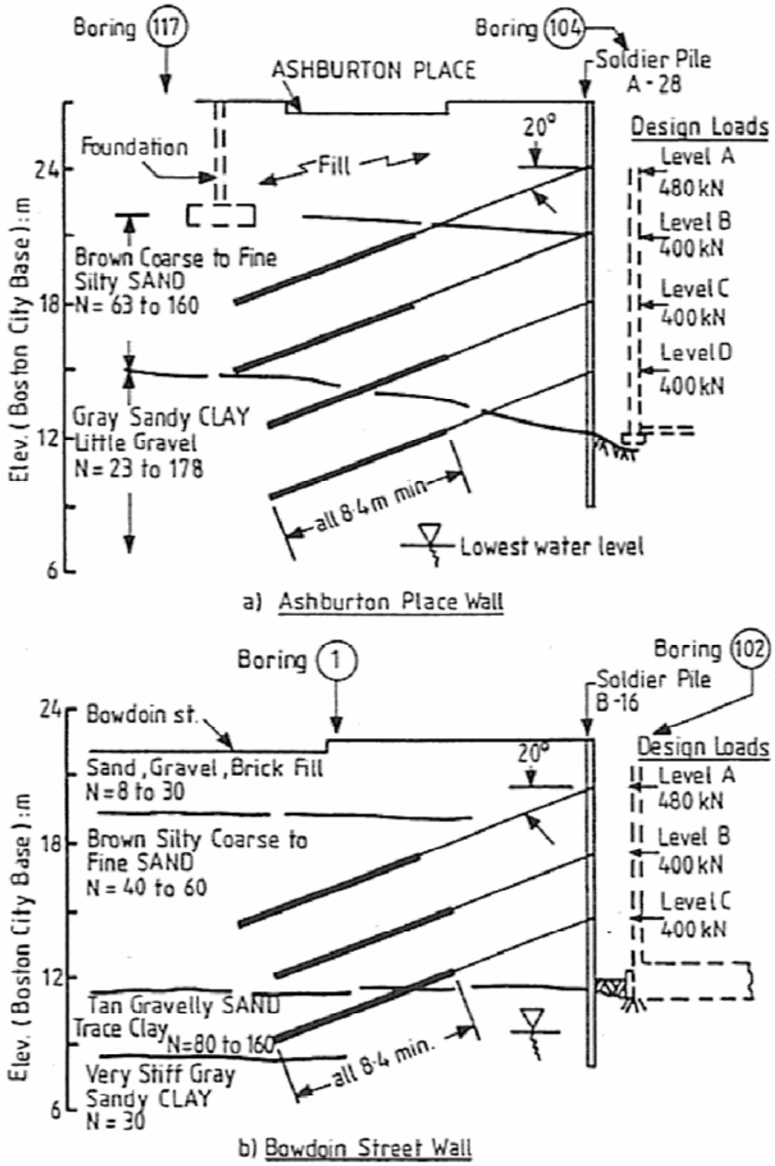


Figure 9. Typical tied-back wall arrangements for Ashburton Place Garage, Boston (from Hanna, 1982)

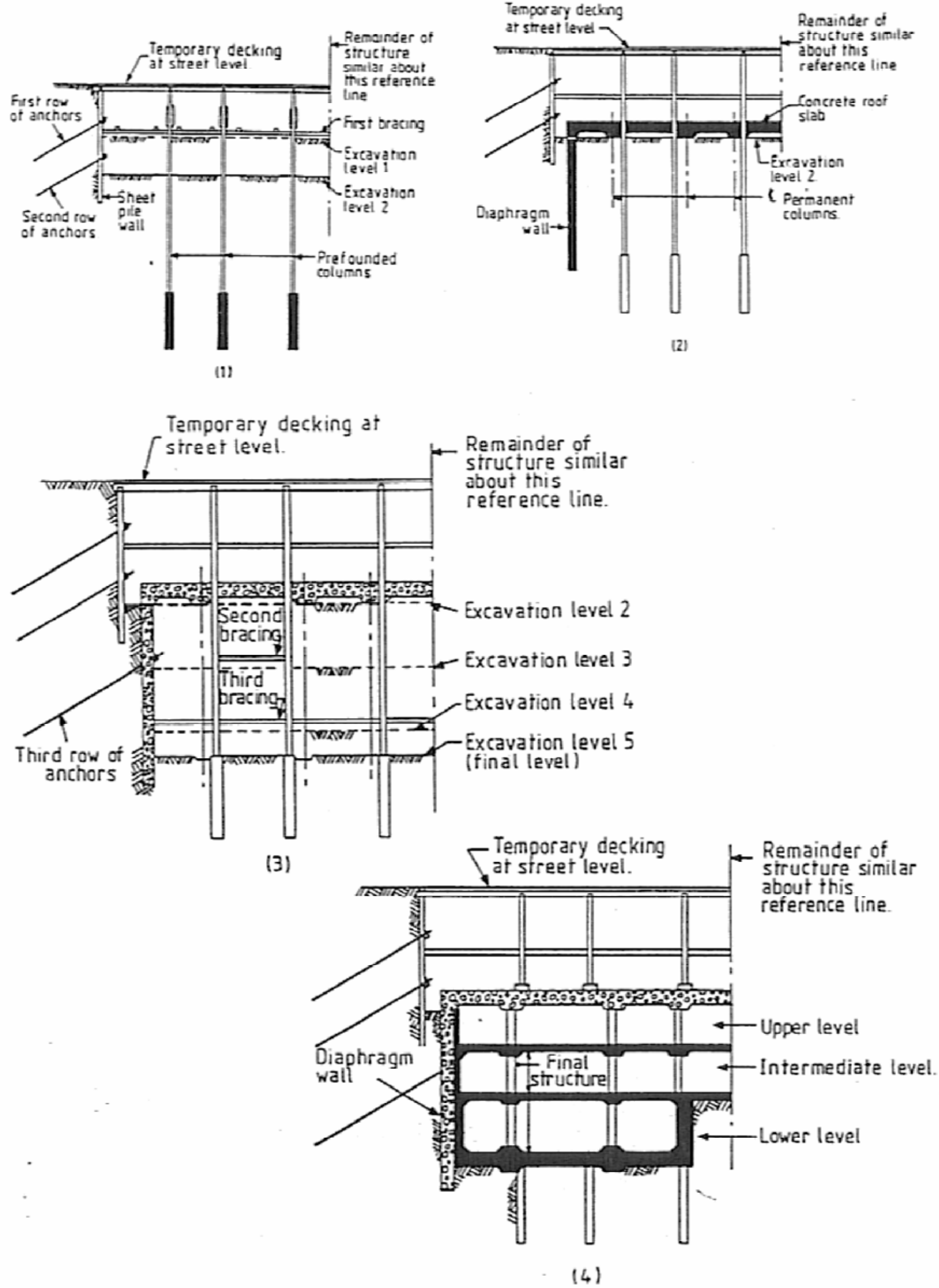


Figure 10. Illustration of construction stages, Alaska Subway Station, Tokyo (from Hanna, 1982)

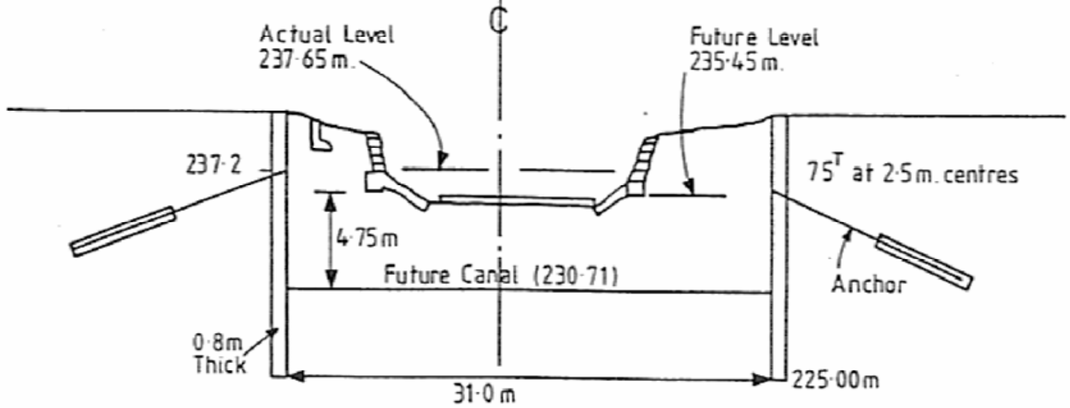


Figure 11. Use of anchored diaphragm wall for deepening of existing canal (from Hanna, 1982)

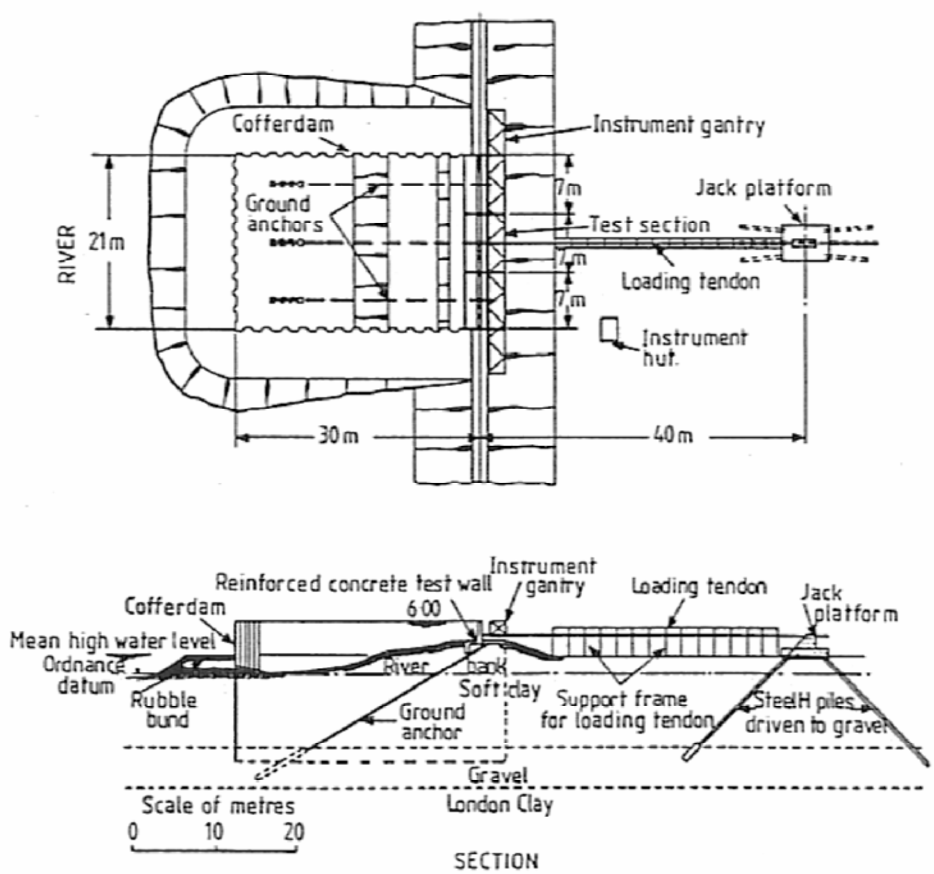


Figure 12. General arrangement of the test facility (from Hanna, 1982)

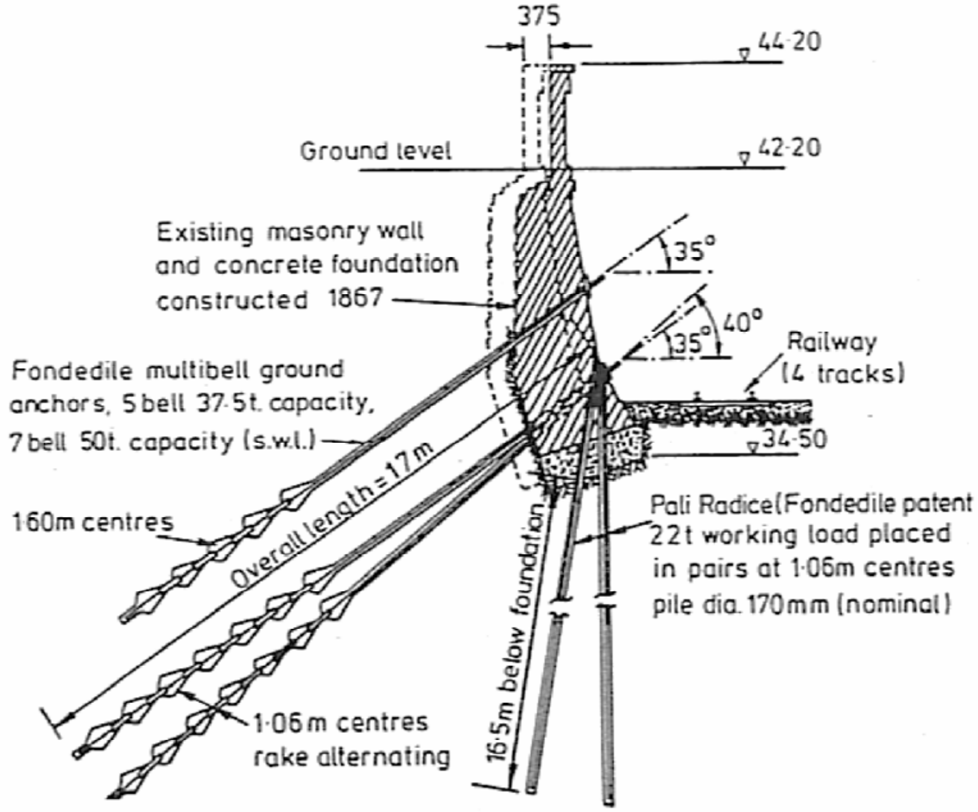


Figure 13. Wall stabilization scheme using prestressed ground anchors and Pali Radice piles (from Hanna, 1982)

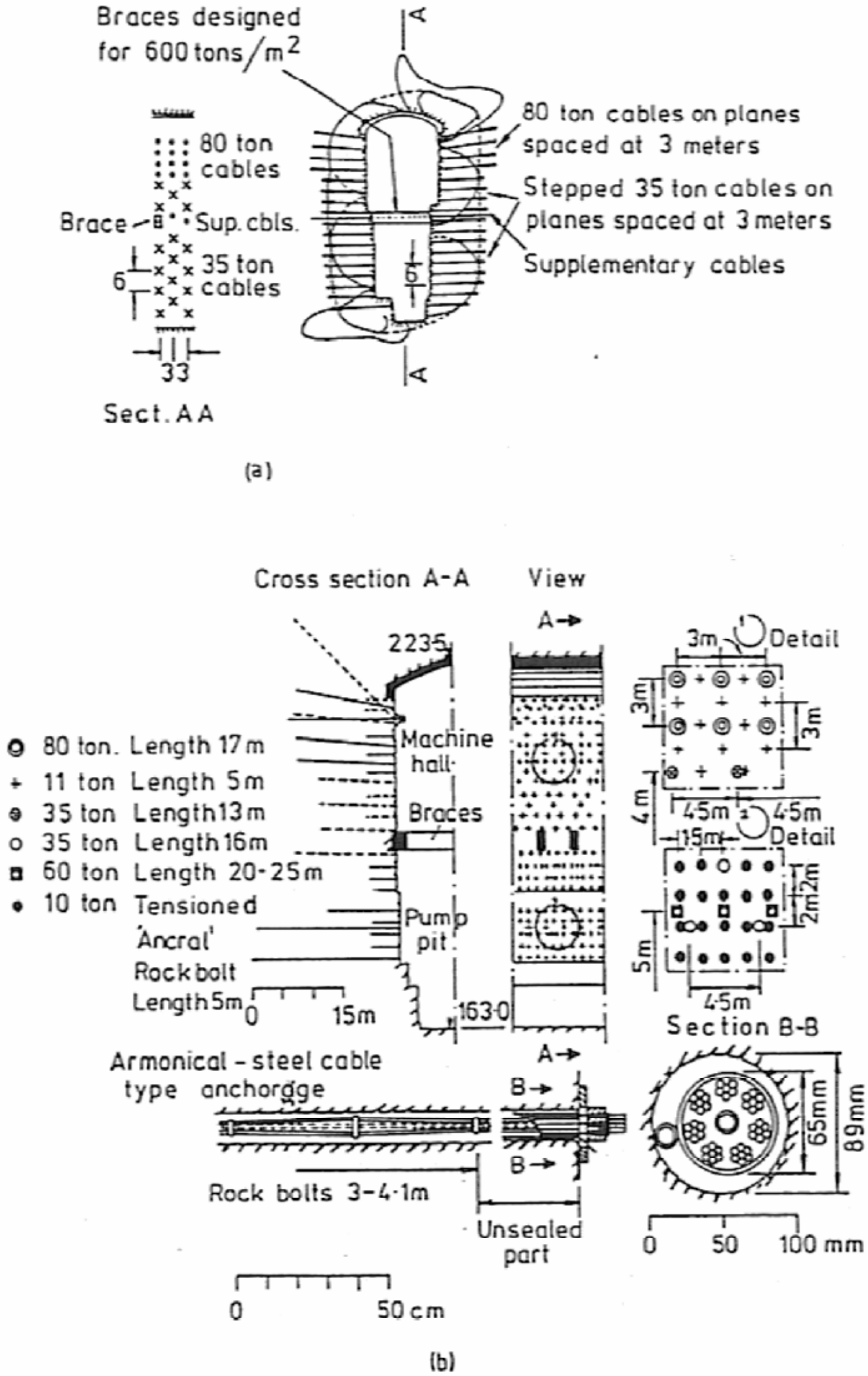


Figure 14. General details of rock wall anchors (a) conceptual design, (b) detail of anchors (from Hanna, 1982)

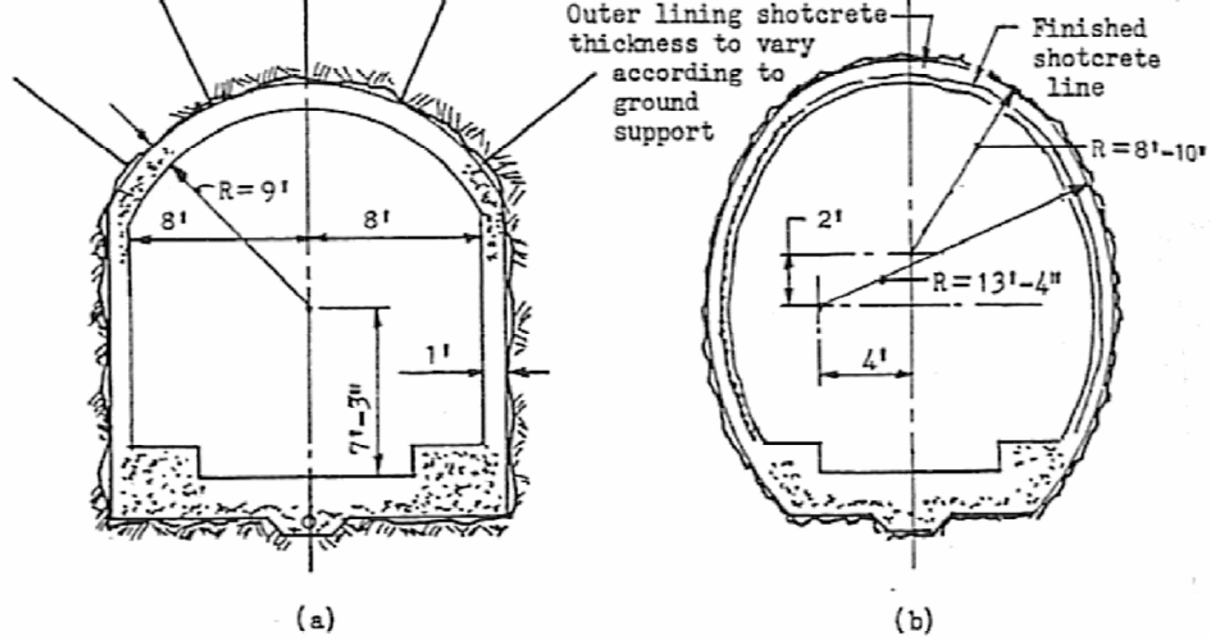
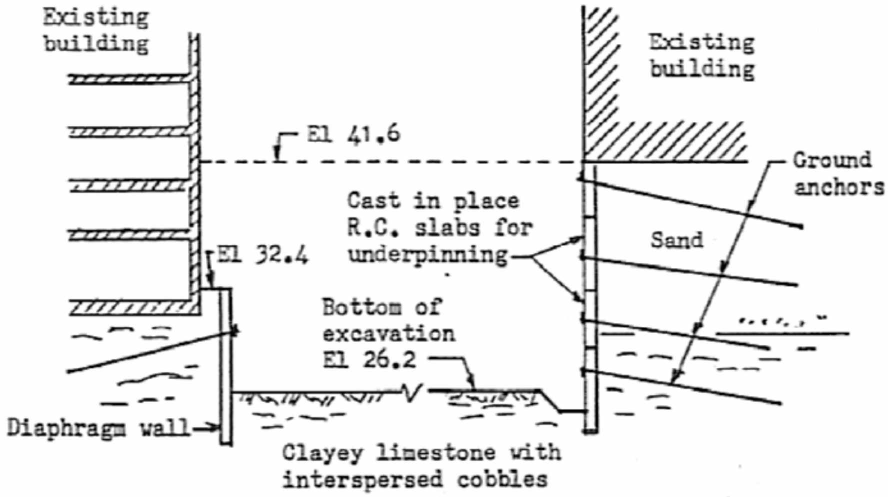
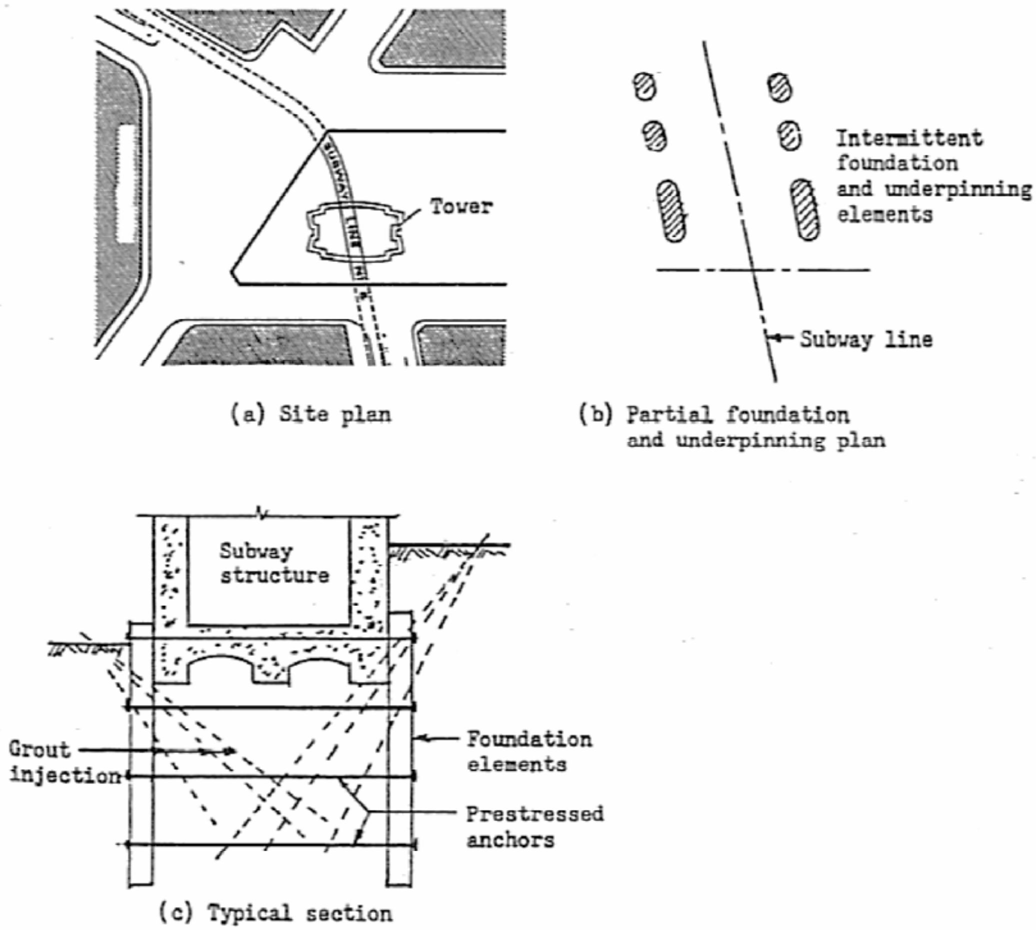


Figure 15. Mt. Lebanon tunnel; (a) cross section of option A; (b) cross section of option B (Xanthakos, 1991)

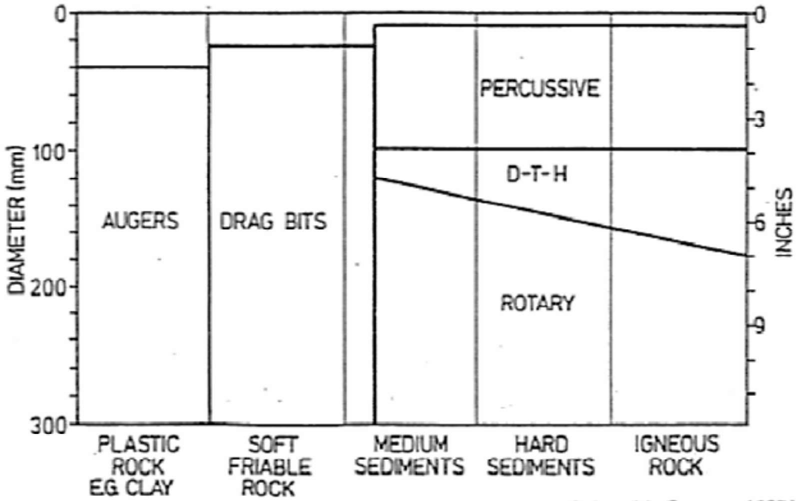


Underpinning of existing buildings, Murat III office deelopment, Paris.



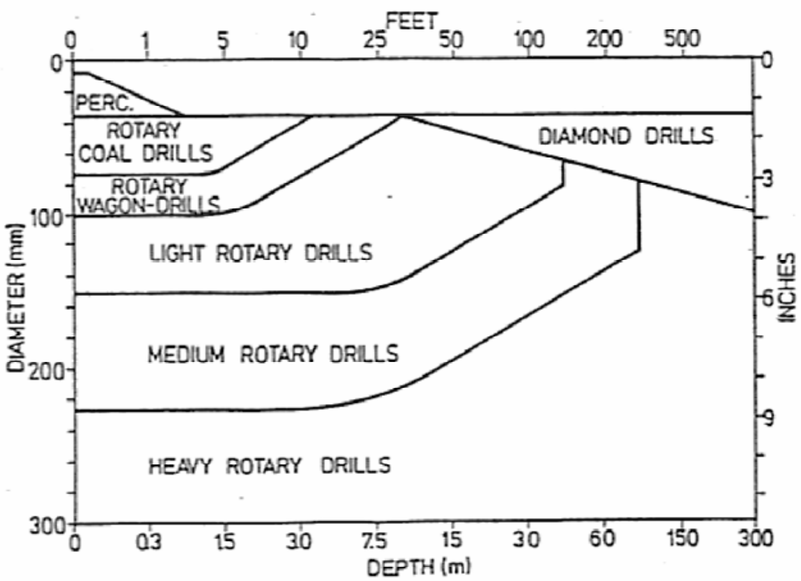
Use of anchorages and strip panel walls to underpin an existing subway

Figure 16. Anchors for underpinning (Xanthakos, 1991)

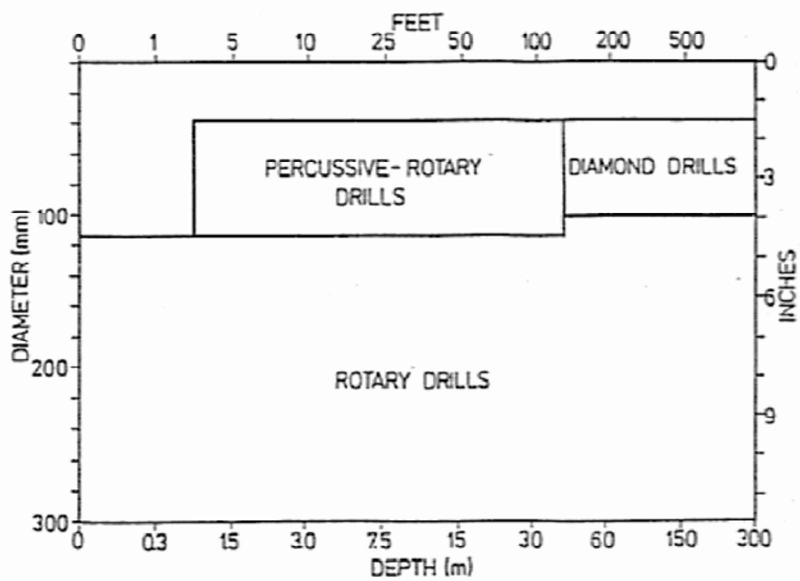


(after McGregor, 1967)

Preferred methods of drilling different classes of rock and at different hole diameters. Depth of hole generalised



Preferred methods in soft friable rocks (after McGregor, 1967)



Preferred methods in variable strata (after McGregor, 1967)

Figure 17. Rock drilling guidelines (Littlejohn and Bruce, 1977)

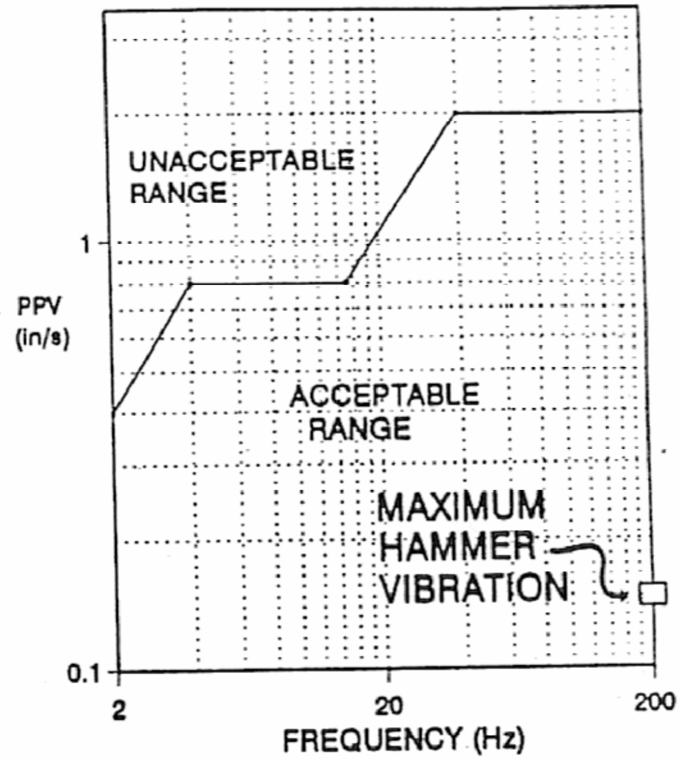


Figure 18. Monitoring of vibrations during drilling. Hole 37 Stewart Mountain Dam, AZ (Bianchi and Bruce, 1992)

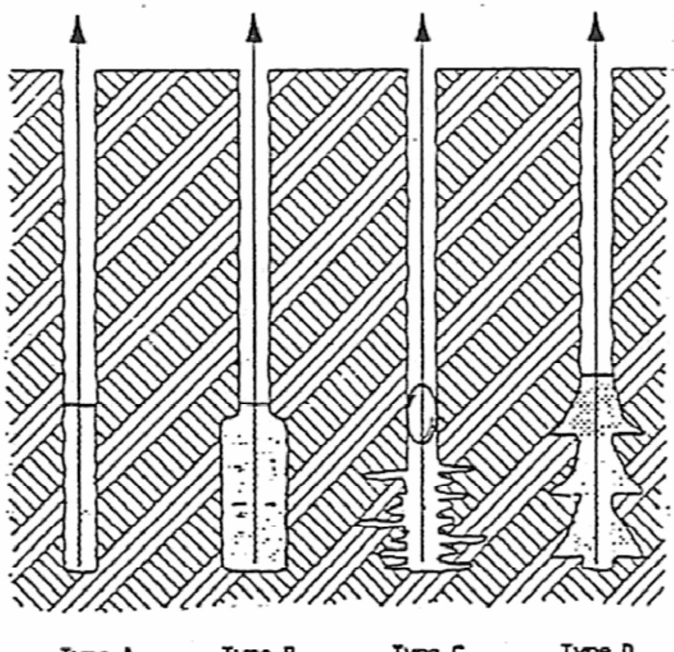


Figure 19. Main types of cement grouted anchors (Littlejohn, 1990)
Type A: straight shaft, gravity grouted
Type B: pressure grouted during installation
Type C: pressure grouted via a sleeved pipe after initial installation grout has set
Type D: underreamed, gravity grouted

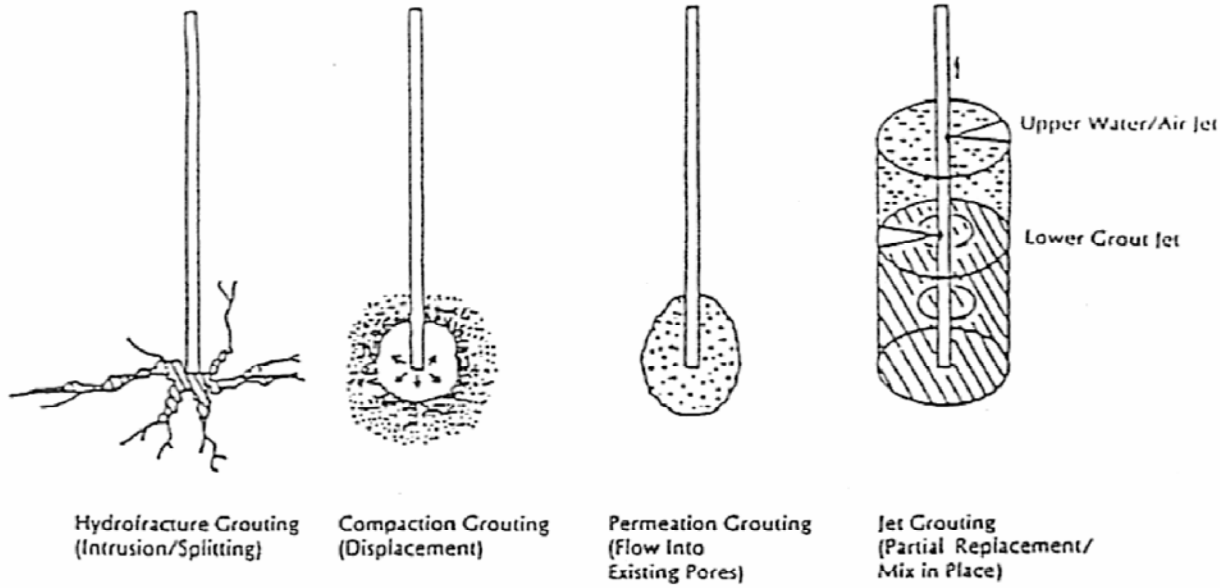


Figure 20. Basic categories of soil grouting methods

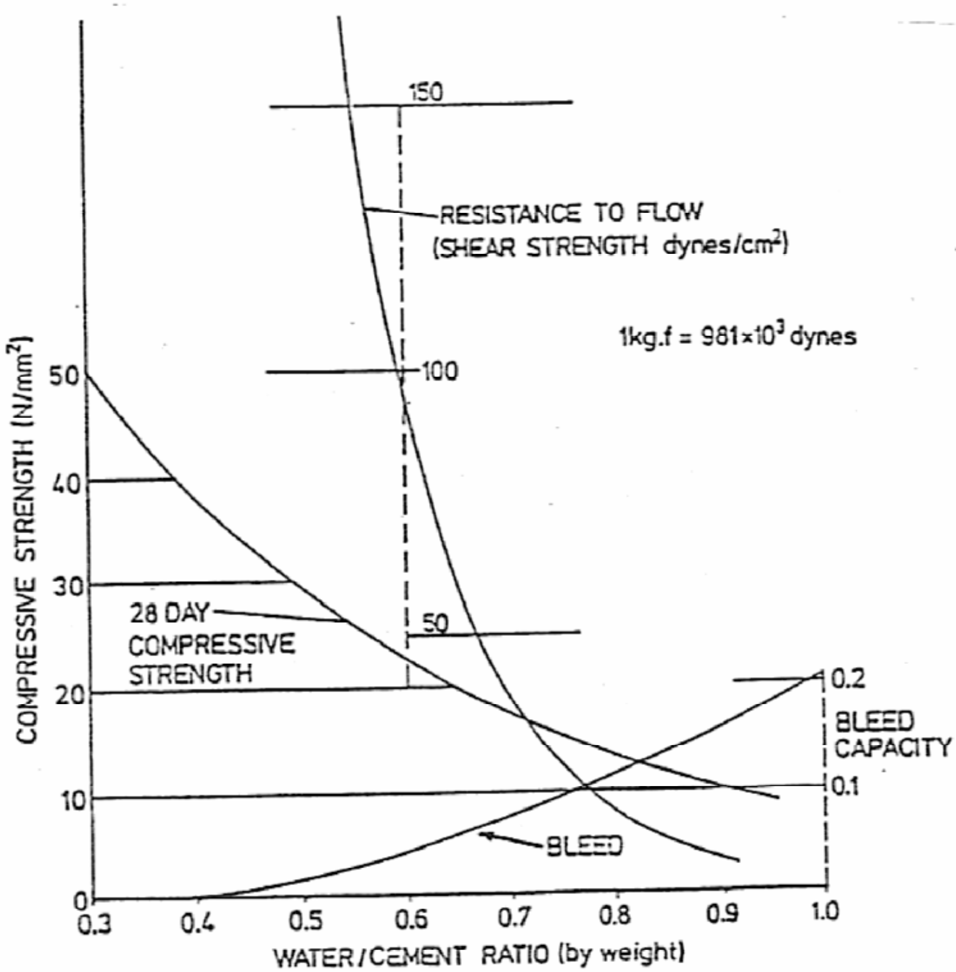


Figure 21. Effect of water content on grout properties
Note: $1\ N/mm^2 \approx 145$ psi

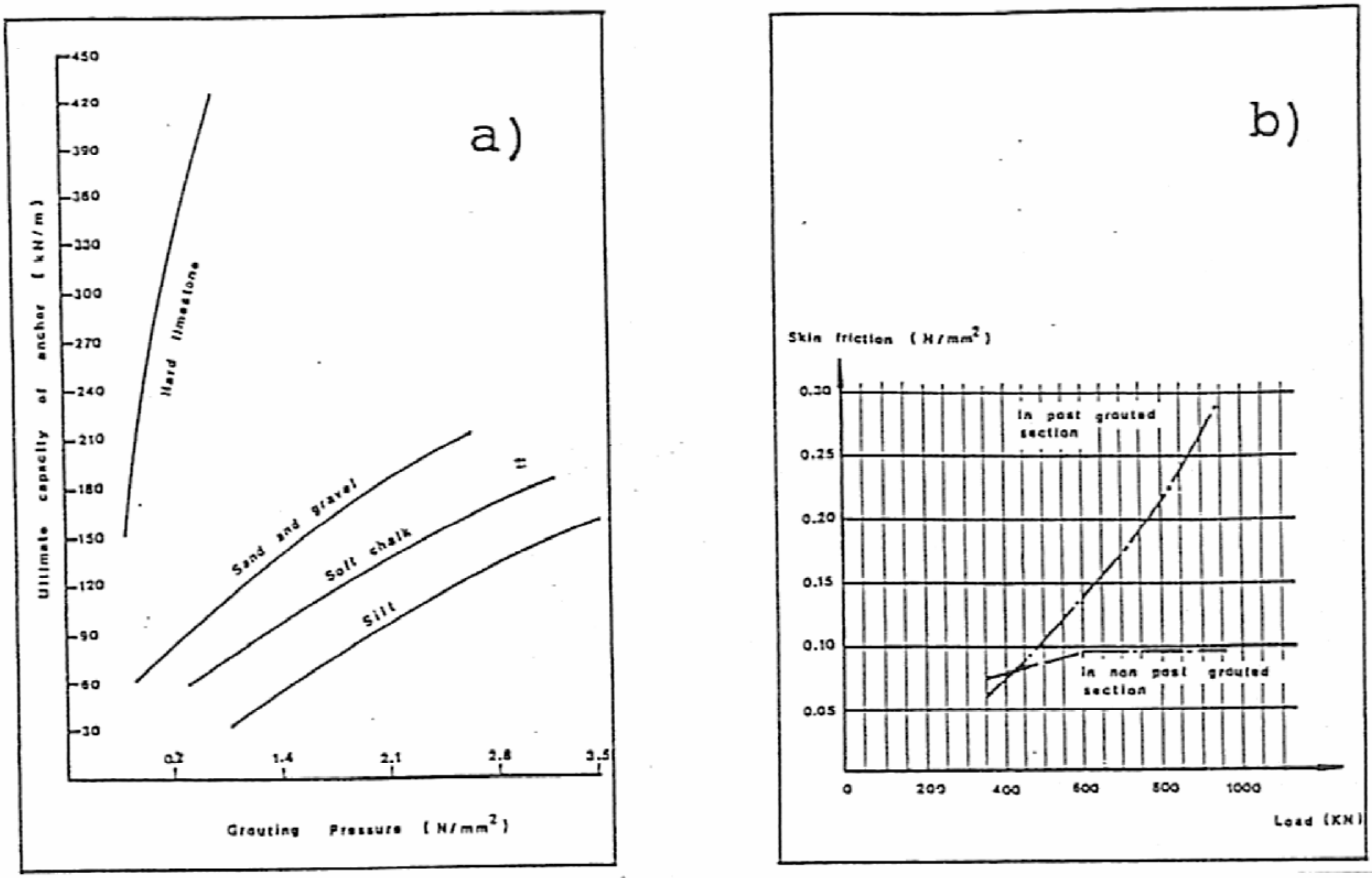
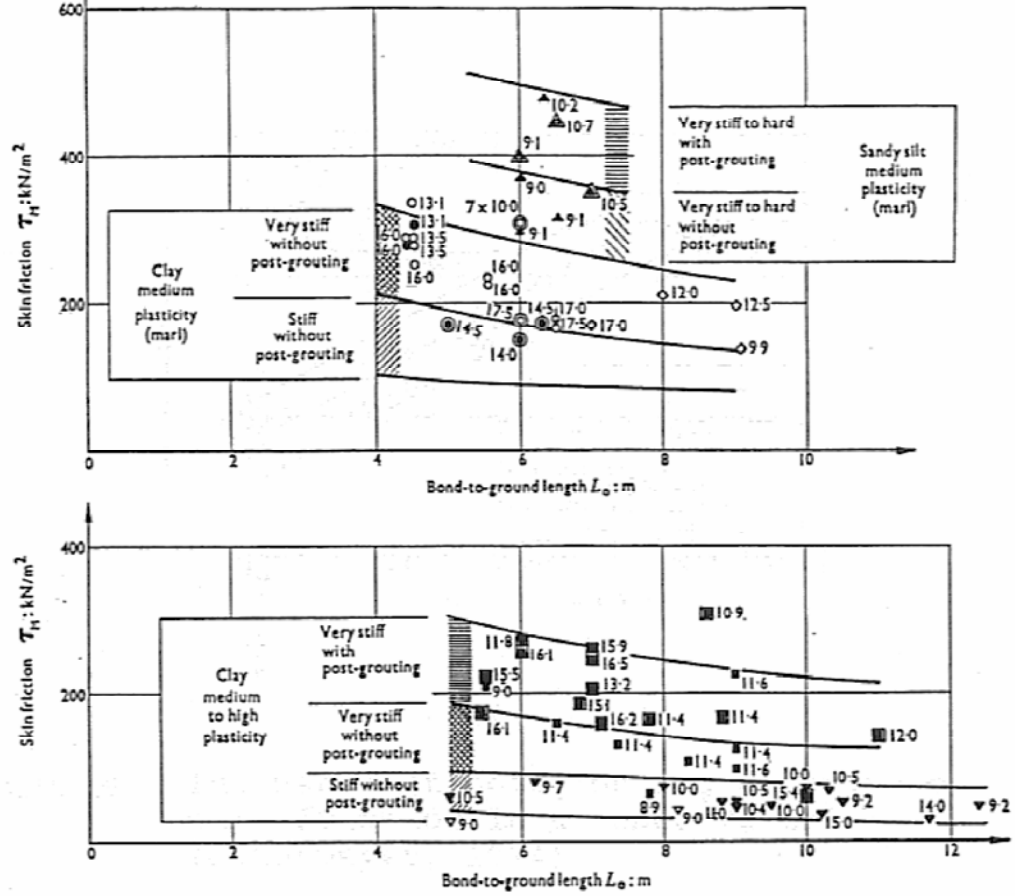
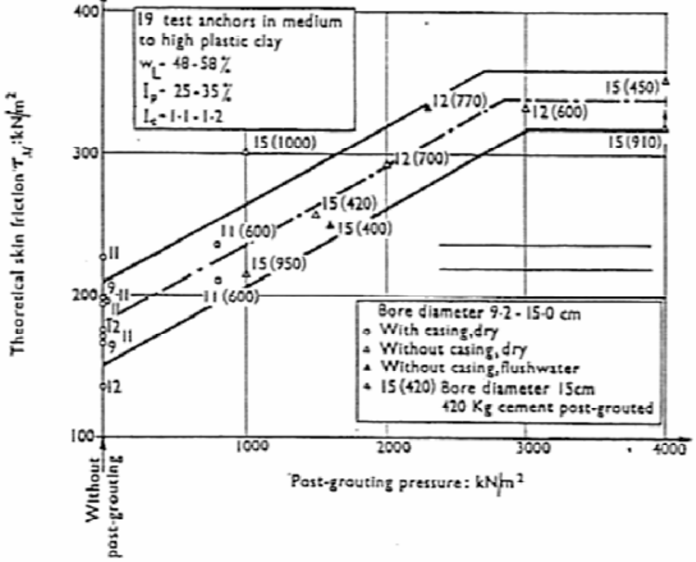


Figure 22. a) Influence of grouting pressure on ultimate load holding capacity (Littlejohn and Bruce, 1977). b) Effect of postgrouting on skin friction (Herbst, 1982)

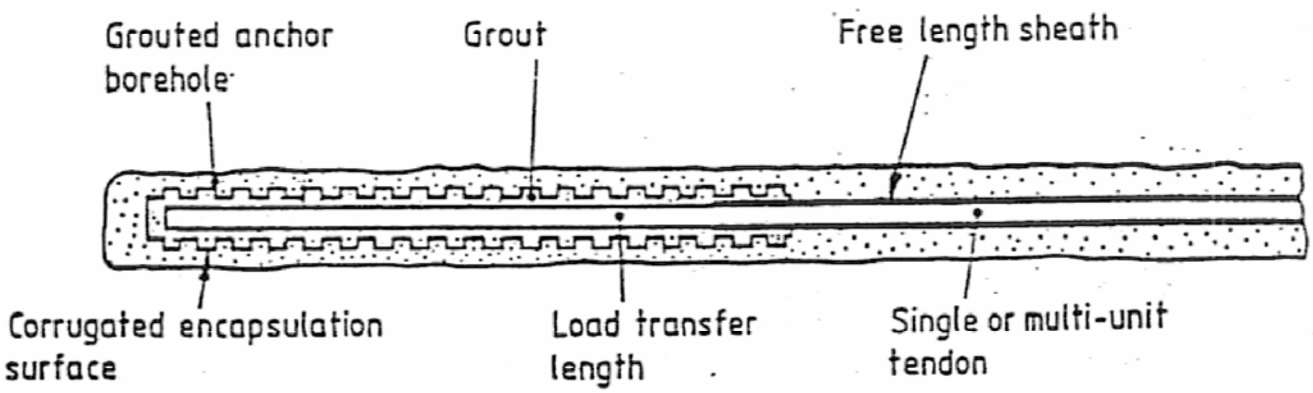


Failure load was reached	Failure load was not reached	Post-grouting	Type of soil	W_L %	I_p %	I_c %
▲	△	Without	Silt, very sandy (marl) medium plasticity	~ 45	~ 22	~ 1·25
▲	△	With				
●	○	Without	Clay (marl) medium plasticity	32-45	14-25	1·03-1·14
●	O	With				
●	○	Without		36-45	14-17	1·3-1·5
●	O	With				
	◇	Without	Silt medium plasticity	23-28	5-11	0·7-0·85
■		Without	Clay medium to high plasticity	48-58	23-35	1·1-1·2
■		With				
▼	▽	Without		45-59	16-32	0·8-1·0

Figure 23. Skin friction in cohesive soils for various bond-to-ground lengths, with and without postgrouting (Ostermeyer, 1974)



<u>Figure 24.</u> Influence of postgrouting pressure on skin friction in a cohesive soil (Ostermeyer, 1974)



<u>Figure 25.</u> Encapsulation of bond length with corrugated protection