Geotechnical considerations in trench design for lifelines

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Abstract: This paper considers Vancouver area geotechnical design and construction practices for trench excavation and support systems accommodating lifelines. Attention is given to the approach commonly used in Vancouver, in which a pipe is placed in an unsupported trench with shoring only provided at locations where workers enter the trench to join the pipe. Backfill is then undertaken to above the spring line using water compaction of free-running sand. The objective of the paper is to review current practice and to discuss some soil conditions influencing unsupported trenches. Conventional trenching practice is discussed, as well as a method for assessing trench stand-up time and impact of previously backfilled excavations.

Introduction

This paper considers Vancouver area geotechnical design and construction practices for trench excavation and support systems constructed to accommodate lifelines at up to 6 m below street level. Trenching is a simple process; the skill lies in economically excavating the trench in such a way that:

- Trench sides are adequately supported to ensure worker safety and prevent damaging movements to adjacent properties and utilities.
- Groundwater is controlled.
- The trench bottom is adequately prepared to provide uniform support.
- The ground surface is properly reinstated, and does not undergo settlement with time.

Trench safety

In the U.S., trench cave-ins account for about 1% of work-related deaths. This amounts to about 1000 work-related injuries per year, and between 60 and 100 deaths per year (Thompson and Tanenbaum 1977; Surada et al 1988; and National Institute of Occupational Safety and Health (NIOSH) 1995). Moreover, Occupational Health and Safety Administration (OSHA) (1990a) issues an average of 746 citations per year for trench safety violations, mostly failure to adequately slope or shore trenches (Surada et al 1988). Statistics are difficult to obtain and are often misleading because trench cave-ins are frequently included under the statistical classification of "struck by objects other than vehicles or equipment" or "caught in or between objects other than vehicles or equipment". For this reason, reliable statistics could not be obtained from the BC Workers Compensation Board (WCB). However, interpretation of available records suggests that local trench cave-in fatalities are very low.

Thompson and Tanenbaum (1977) and Surada et al (1988) studied US trench cave-in statistics to identify common factors and recommend improvements to OSHA codes. They concluded the following:

- Most deaths occurred in sewer line construction.
- 79% of trenching fatalities occur in trenches less than 4.5 m deep, and 38% occur in trenches less than 3 m deep.
- Almost all fatalities occurred in unshored trenches, indicating that even some shoring provides protection against death. Several incidents occurred in shoring cages. Except for one case, shoring cage accidents happened when workers exited the box, or were outside the box while it was being moved.
- The major external factors influencing trench cave-ins were the presence of construction equipment operating near the edge of the trench, and adverse climatic conditions.
- Trench stand-up times (interval between completing excavation and cave-in) were mostly between 0.5 and 10 hours but ranged from 5 minutes to 4 months. Longer stand-up times often reflected collapse initiated by external factors. (Thompson and Tanenbaum 1977)
- Young construction workers and smaller companies account for the greatest proportion of the fatalities.

Evolution of local Vancouver practice

Historical development

Trench support in Vancouver has traditionally consisted of hand-built shoring evolved over the past 100 years from forms of timbering introduced by ex-miners employed by the city (SRK-Robinson 1997). Timber trench support is well-suited to hand excavation. The advent of large excavators able to excavate quickly to design depth encouraged the development of support systems capable of rapid installation with greater spacing between cross braces to facilitate access. Shoring procedures evolved to meet the accelerated excavation rate, but often still rely on timber supports and bracing. Pre-built timber sets, which can be lowered in sections into excavated trenches, have largely replaced hand-built timbering. Steel components are replacing timber components in traditional systems.

While timber is still most common for routine work, metal shoring cages and steel sets are gaining popularity for ease of construction and ability to be re-used.

Current City of Vancouver practice

Procedures for excavation and shoring of sewer trenches in the City of Vancouver are presented in *Safe Practice Guide for Shoring* (SRK-Robinson 1997). The Guide is based on a structural assessment of timber elements required to safely withstand lateral earth pressure. The Guide has a set of standard shoring designs based on four basic soil behaviour conditions and three species of timber. Trench widths and depths are typically less than 3.65 m and 6 m, respectively. The soil classification used is similar to that developed by OSHA (1990b).

Structural design is based on three timber species: Douglas fir – Larch, Hemlock – Fir, Spruce – Pine Fir. Of these, the most commonly used is Hemlock – Fir as supplies of Douglas fir are limited and expensive because of demand from other markets. Commonly available lumber sizes are: 4"x 4", 4"x 6", 8"x 8", and 4"x 12", and so design tables have been developed for these sizes, but they are not necessarily the minimum dimensions required for a particular trench depth. Consequently, support for a trench of a given depth maybe oversized for the conditions and provides an additional measure of safety.

The components of a typical generic timber shoring design include stringers (walers), bracing (struts, spreaders, cross braces), and uprights (sheeting) and is depicted on Fig. 1.

Comments

Experience during and following to the development of the *Safe Practice Guide for Shoring* has:

- Support systems not meeting all classical geotechnical design requirements sometimes perform satisfactorily. This is explained by conservatism in conventional design approaches, and indicates the need to account for local experience.
- A generic shoring design was prepared for trench depths to 9 m; however, for trenches deeper than 6 m, the WCB requires site-specific evaluation.
- Normally timber sheeting is left in place or cut off above the pipe because pulling sheeting can lead to voids in the pipe zone and disturbance of the bedding. Ideally sheeting should be cut off at a level no higher than about 1.8 m below grade to avoid localized hard zones following settlement of the surrounding ground.
- It is often assumed that pulling thin steel sheeting does not result in similar disturbance, but steel trench cages pulled along the trench can cause substantial disturbance and possible pulling apart of joints. Ideally, a trench cage should ride above the top of the pipe on a thin shelf cut to accommodate the cage.

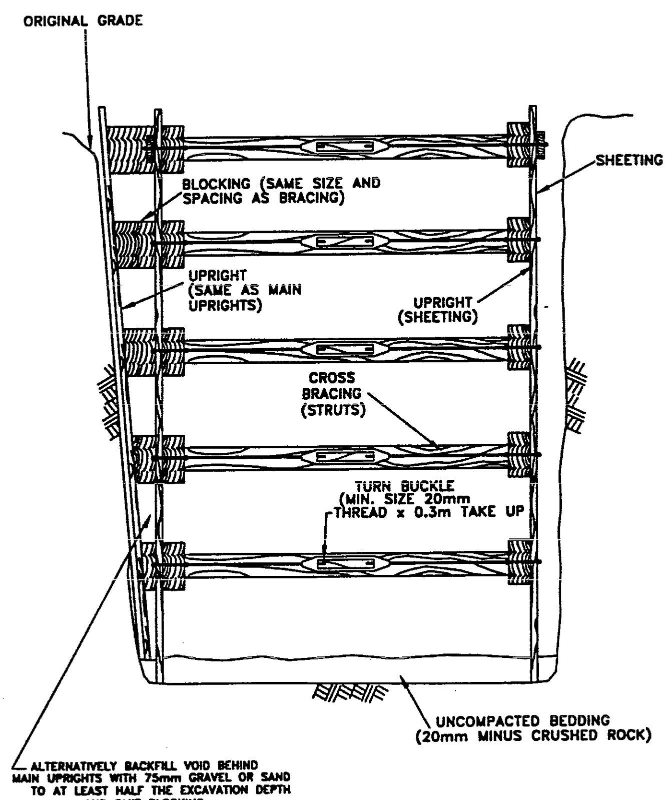

Fig. 1 City of Vancouver generic shoring design for sewer trenches

Table 1. Soil classification system

Class	Description	Undrained shear strength	Allowable slopes
A	Hard and solid soils (undisturbed glacial till, sandstone and siltstone)	greater than 70 kPa	0.75 H : 1 V
B	Soils likely to crack and crumble (weathered glacial till/stiff glacio-marine sediments)	between 25 to 70 kPa	1 H : 1 V
C	Soft, sandy fill or loose soils (loose sand/granular fill, some organic and clayey silt)	less than 25 kPa	1.5 H : 1 V
D	Soils subject to hydrostatic pressure (saturated silt, sand, gravel, and peat)	all soils	

Alternative trench construction

Alternative trench construction approaches have been developed to reduce project cost, improve worker safety, and reduce social costs (impact on the community through disruptions to business, traffic, etc.). An obvious example is "trenchless" construction methods. We observed early evidence of "trenchless" techniques during construction of the 45th Avenue watermain, which reflected the contribution of the ex-miners. The excavation for the new watermain intersected old sewers, which were located in small timber-supported tunnels. The tunnels had been hand-excavated under the streets and were typically about 1.2 m high and about 1 m wide. They remained in place when the sewer was laid, but had been partly filled with sand to protect against collapse.

Another alternative is the use of narrow trenches, often excavated with continuous trenching machines, and pipe-laying methods that obviate the need for workers to enter the excavation. In Vancouver, this approach entails installing long pipes in unsupported trenches with shoring only provided at joint locations where worker entry is required. Backfill to above the spring level is undertaken by water jetting using lances to produce flow of free-running sand backfill. Water compaction can result in high densities provided adequate procedures are used (USBR Earth Manual 1974). Water compaction allows the trench width to be minimized since only about 0.3 to 0.5 m clearance is required on either side of the pipe for backfilling. Compaction above the pipe is undertaken by either plate compactor supported by excavators (hoepacks), or by workers operating self-propelled compactors within shoring cages. Successful use of this approach requires that the trench wall remain unsupported for around 2 to 3 hours, and that reliable assessments of ground movements are made.

Case History

Construction in 1998 of the 45th Avenue GVWD watermain by B. Cusano Contracting Ltd. illustrated the advantage of the unsupported trench approach. The project involved installing a 1.5 m diameter steel pipe from Griffiths Avenue, Burnaby to Central Park, and then west of Boundary Road to Victoria Drive mainly along 45th Avenue, a total length of around 6 km. The alignment traversed several shallow depressions located between much wider rounded ridges, and extended along suburban streets and crossed some busy intersections including Rupert Street at 45th. Trench depths were typically about 4 m below road grade, but, in soft soil, extended to 5 m to accommodate increased bedding requirements.

The soils in the higher areas comprised dense to very dense silty sand with some gravel and cobbles, till-like material overlain by fill and road pavement layers. The till-like materials were mainly non-plastic and had fines contents of up to 50%. SPT "N" values were around 70 to 100, and moisture contents were between 10 and 12%. Perched water was sometimes encountered within the fill directly above the till.

The soils in the shallow depressions comprised road pavement layers and fill over peat and soft clayey silts, with dense till-like material present as deep as 8.5 m below road grade. The road fill varied from about 0.9 to 2 m thick. The native soils consisted of peat to a maximum depth of 6 m below grade, over very soft to soft grey clayey silt with occasional fine sandy layers, over dense till-like materials. Moisture contents in the peat ranged from about 500 to 630%, and undrained strengths varied between 20 and 40 kPa, depending on moisture content. Undrained shear strengths in the clayey silt were typically around 20 to 30 kPa, but were as low as 12 kPa in places. The watertable was about 2 m below ground. The thickest occurrence of peat and soft clayey silt extended over about 500 m from Earles Street to east of Lancaster Street, where up to 6.5 m of soft soil overlay till-like soil.

Construction proceeded by excavating trenches to design depth, placing uniformly-sized granular bedding, landing 18 m long steel pipe sections by slinging from two excavators, placing shoring cages at joint locations, pouring free-running sand around the pipe followed by water compaction, welding and corrosion protection at the pipe joints, and finally filling to grade and surfacing. A construction cycle from excavation to placement of backfill to above pipe crown was between 1.5 to 2 hours, but varied depending on soil and groundwater conditions and impact of existing utilities. Workers only entered the excavation at joint locations and during final stages of backfilling, and were always protected by shoring cages.

In the areas of till-like subgrade soils, trenches with vertical sidewalls remained stable without shoring for long periods, even when subject to machinery vibration. However, it was specified that excavators handling the pipes should be at least 1.5 m back from the edge of the trench to prevent collapse of the overlying road fill. Minor sloughing of the road fill occurred.

The original geotechnical report indicated that in peat-filled depressions, even with dewatering, temporary excavation slopes would not stand steeper than 1 H:1 V. The report also warned that dewatering would result in unacceptable settlement and distress to the overlying road pavement. Unfortunately, no space was available to accommodate 1 H:1 V slopes due to congested utilities. Previous experience and monitoring of excavation performance were used to develop excavation protocols in the soft soil consistent with the short stand-up times required. The final excavation design consisted of 70^0 to 80^0 sidewalls protected by polyethylene sheeting, with stringent monitoring requirements. One cave-in occurred after 20 minutes in a section of vertical trench excavated through 1 m of fill, over 1.8 m of peat, over 1.2 m of very soft clayey silt. Back calculation showed that the Factor of Safety (FOS) based on undrained shear strength (Total Stress Analysis, TSA) was about 0.97.

The project proceeded with few problems. The following aspects were observed:

- Longitudinal cracking of up to 40 mm wide parallel to the excavation indicating incipient cave-in was observed in several locations. The cracking mainly coincided with the location of adjacent backfilled trenches. Collapse was averted by backfilling the trench.
- Soil conditions were observed to deteriorate rapidly over short distances, often in response to almost imperceptible topographic variations. Careful monitoring protocols are required to cope with such changed conditions.
- Extensive use was made in earlier times of timber corduroy placed perpendicular to the roadway to stabilize peat sections. These tree trunks contributed to increased and irregular excavation overbreak, and more onerous spoil disposal requirements.
- The trench was more stable when it was entirely within peat, and less stable when the base penetrated the silt.
- In several cases, flooding of the trench occurred when saturated granular backfill from other trenches was intersected.

Trench design aspects

Prediction of stand-up time

Without considering other factors such as cohesion, soil structure and strain rate, short-term stability of a temporary slope in a cohesive soil is enhanced by virtue of pore pressure reductions induced by a reduction in total confining stress as the material is excavated. Following excavation, the soil mass swells as the pore pressures increase to equilibrate with those governed by seepage conditions. The ultimate pore pressures are higher than those obtaining following excavation, and the FOS declines to its lowest value when persistent seepage conditions prevail. Therefore, cohesive soils are most stable directly after excavation. Pore pressure equalization occurs at a rate dependent on the permeability and degree of over-consolidation as shown generally by Fig. 2.

Various publications differentiate between short- and long-term stability. National Bureau of Standards (NBR) (1988) considers short term as less than 24 hours, whereas Irvine and Smith (1983) differentiate between short- and long-term at one week. Leroueil et al 1990 have investigated the relationship between short- and long-term stability, and conclude that the time needed to obtain steady conditions is highly variable. In the case of cohesionless soils, drainage and consequently strength loss occurs rapidly. In the extreme case, saturated sand will flow and become totally unstable when excavated giving rise to "running sand". Therefore, discussions related to stand-up time are only applicable to cohesive soils, and to partially saturated, fine cohesionless soils where apparent cohesion is a result of surface tension.

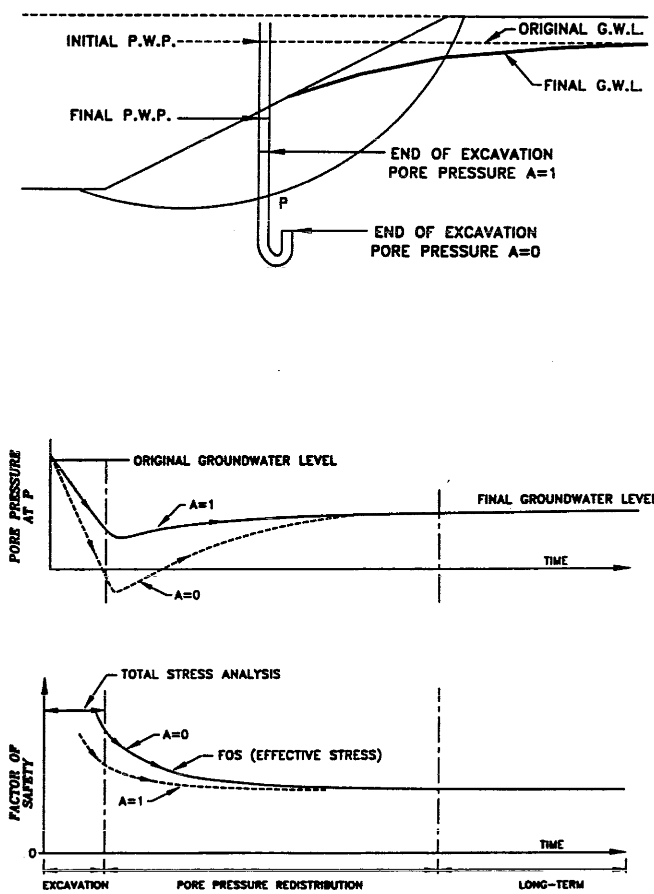

Fig. 2. Change in time of pore pressure and excavation Factor of Safety (Bishop and Bjerrum 1960)

Mitchell (1983) approaches temporary slope design by assuming that long-term conditions will be achieved at 3 months and 6 months for silty clays and intact clays, respectively. According to Mitchell, many soils with an FOS of 2.0 calculated using TSA would yield a long-term FOS of close to unity. Therefore, if M is the required time in months for which an excavation must remain open, then the initial FOS is given by:

[1] $FOS = 1.3 + 0.2\,M$ for silty clay

[2] $FOS = 1.3 + 0.1\,M$ for intact clay

Trench stand-up time is analogous to tunnel design where models to predict stand-up time are based on an empirical correlation between rock quality (e.g. Rock Mass Rating, Bieniawski 1989) and observed tunnel stand-up time. FOS calculated using TSA provides a similar measure of soil quality and geometric constraints. This approach was used by Leroueil et al 1990 who related the reduction in FOS with time for excavations in Mexico City clay. However, these and other published results (e.g. Bazett et al 1961, Skempton and Golder 1957, Kwan 1971, Lafleur et al 1988, etc.) are generally for stand-up times far in excess of the 3 hours required for unsupported trenches.

A review was made of published case histories, for which FOS and stand-up time were available and the results compared to local experience (see Tables 2 and 3). Uncertainties in assembling the results related to allowance for tension cracking, whether vane shear results should be corrected for rate effects, and strength contribution of granular fill layers and asphalt surfacing. The results have been plotted in Fig. 3, which illustrates that for stand-up times of around 3 hours, an FOS calculated using TSA should be in excess of about 1.1 for silty clay to clayey silt materials (Type C soils from Table 1).

Table 4 has been developed from observations and experience, and provides suggested stand-up times for trenches of up to 4 m deep, for which workers will not enter.

Prediction of ground movement

Restricting ground movement adjacent to trenches is frequently more onerous than ensuring worker safety against cave-in because restricting movement demands non-yielding support, which does not optimally utilize the self-supporting characteristics of the surrounding ground. Prediction of movement has received considerable attention in the literature and is therefore not discussed in this paper.

Impact of adjacent backfill trenches

Because of extensive previous development, most trench alignments in Vancouver now encounter previously backfilled trenches, and consequently the predominant soil type is C and D (Table 1). A particularly hazardous situation occurs where thin columns of undisturbed native material remain between trenches, and the presence of a backfilled trench is not suspected. Further, water often collects in granular trench backfill, and, when intersected by excavation for a new trench, results in additional hazards of flooding contributing to piping and running sands.

If the width of the intervening column of undisturbed native soil is too narrow, failure can occur by slumping or toppling, or occasionally as a localized breakthrough. Warning of an impending failure is often indicated by longitudinal cracking along the previous trench line.

Fig. 4 indicates the effect of a backfilled trench on stability of a new trench excavated in till-like soils with an undrained shear strength of 70 kPa. Stability was evaluated by treating the column of undisturbed soil as a gravity wall subject to lateral load from the previous trench backfill. Estimation of the lateral load is complicated because of the narrow confines of the trench, and the likely presence of residual compaction stresses. For the example of the till-like soil, Fig. 4 shows that the spacing (S) between trenches needs to be greater than 30% of the depth (D) to ensure stability.

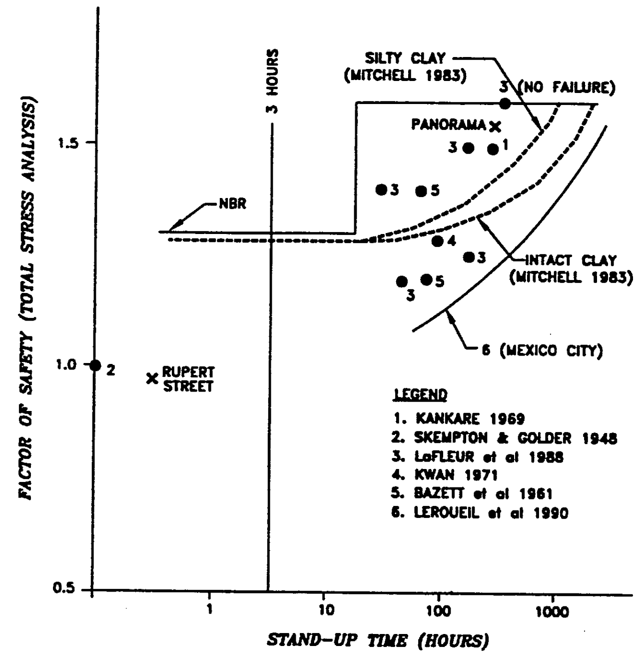

Fig. 3 Relationship between Factor of Safety and standup time

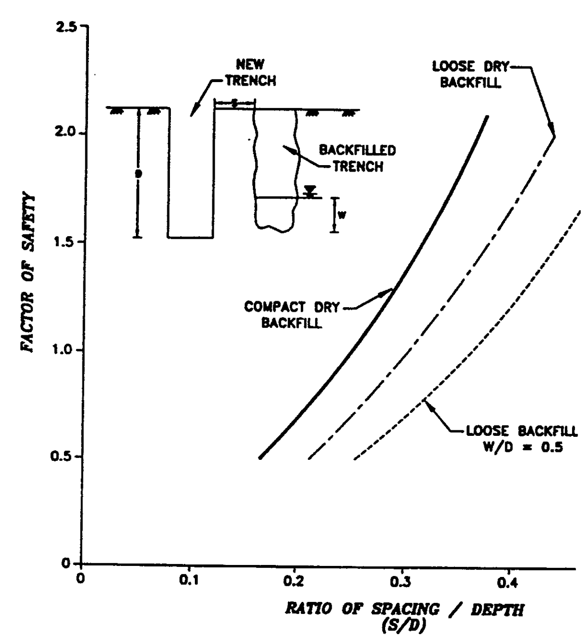

Fig. 4 Impact of adjacent previously backfilled trench

Conclusion

The main conclusions are as follows:

- Research on trench cave-ins shows that most fatalities occur during sewer construction in trenches of less than 4.5 m deep. Few fatalities occur in shored trenches.
- In Vancouver, sewer trench construction is codified in the *Safe Practice Guide for Shoring*. This guide is largely based on timber support.
- In Vancouver, the use of long pipes placed in unsupported excavations with backfill compacted using water jetting has been found to be cost effective.
- The main geotechnical design issues are prediction of stand-up time, prediction of ground movement, and influence of adjacent backfilled trenches on new trenches. For short-term stand-up times of around 3 hours, Factors of Safety of 1.1 using a total stress analysis can be used. More case histories are required to improve short-term stand-up time prediction.

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Table 2. Local Case Histories used

Location	Depth(m)	Soil Profile	FOS	Observations
45 Ave/Rupert	4	1.8 m fill/1.4 m peat/0.8 m soft silt	0.97	Failed in 20 min, slow seepage
45 Ave/Rupert	4	As above, but slopes cut to 70^0	1.2	No cave-in after 2 hours
104 Ave/Surrey	8.5	Very stiff till-like material	4.6	Good performance
Highway 10	5	1 m fill/1.5 m peat/1.5 m soft silt	1.29	Stood for 2 hours but failed when surcharged by excavator
Panorama	2.6	1.1 m fill/1.5 m soft silt fill	1.55	Failed after 2 weeks
UEL	2	Soft silt, previous trench backfill	1.1	Immediate collapse of backfill

FOS is based on 5 Su/ɣH
All trenches have vertical walls
No seepage unless indicated.

Table 3. Published case histories

Depth (m)	Description and soil profile	FOS	Observations	Reference
7	4 m stiff to very stiff clay/3 m firm clay	3.9	Failed after 3 hours Influence of fissures	Jennings 1977
8	London clay remoulded to 14kPa	1.10	Critical height at failure	Skempton & Golder 1953
5	34^0 slope in sensitive soft clay	1.5	Failed after 7 days?	Lafleur et al 1988
6	45^0 slope in sensitive soft clay	1.2	Failed after 2 days	Lafleur et al 1988
8	34^0 slope in sensitive soft clay	1.25	Failed after 7days?	Lafleur et al 1988
8	27^0 slope in sensitive soft clay	1.4	Minor distress, 30days	Lafleur et al 1988
8	18^0 slope in sensitive soft clay	1.6	No failure after 1 year	Lafleur et al 1988
10	Vertical excavation in firm clay	1.29	Failed after 4 days	Kwan 1971
10	Lightly over-consolidated soft clay	1.5	Failed after 9 months But immediate small slips	Kankare 1969
3.5	45^0 slope in organic clayey silt and peat	1.35	Failed after short time	Calabresi et al 1977
3.5	45^0 slope in organic clayey silt and peat	1.37	Failed after short time	Calabresi et al 1977
4.0	45^0 slope in organic clayey silt and peat	1.15	Failed after short time	Calabresi et al 1977
5.0	35^0 slope in organic clayey silt and peat	1.27	Failed after short time	Calabresi et al 1977
5.0	35^0 slope in organic clayey silt and peat	1.28	Failed after short time	Calabresi et al 1977
4.4	83^0 cut in soft very sensitive clay	1.6	Failed immediately	Bazett et al 1961
6.6	83^0 cut in soft very sensitive clay	1.2	Failed after 3 days	Bazett et al 1961
6.7	83^0 cut in soft very sensitive clay	1.4	Failed after 3 days?	Bazett et al 1961

Table 4. Suggested trench stand-up times for trench depth up to 4 m

Soil Class	Side Slope	Recommended Stand-up Time
A	Vertical	Up to 4 weeks
B	Vertical	Up to 1 day, but require protection against rainfall
C	70^0	Up to 3 hours, but careful monitoring required
D	Variable	Each situation needs to be reviewed.