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Geotechnical case studies: emphasis on collapsible soil cases

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Abstract:	Direct exposure of soil to certain atmospheric agents like water can adversely or favourably influence the engineering behaviour of the soil. For instance, saturated and unsaturated / partially saturated soils behave differently, so do soils under seepage and hydrostatic pressures. Many theories in soil mechanics idealise soils as either cohesive or non-cohesive and this has allowed much research to be done on saturated cohesive soils. However, non-cohesive soils have not received as much attention, apart from recent strength and dilatancy theories, yet in some parts of the world certain non-cohesive soils pose significant risk to structures built on them. The most problematic example of such soils is collapsible soils that may not be detected and properly considered in routine ground investigation activities. In this paper some case studies of collapsible soils in the United Arab Emirates (UAE) are examined, to analyse the effect of their collapse on infrastructure and the possible techniques to ameliorate the situation. The case studies include various sites that were found to suffer structural damage traceable to collapsible soils. It is found that in most cases the soil collapse was due to infiltration of rainwater or water from sustained irrigation activities at the surface.
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Geotechnical case studies: emphasis on collapsible soil cases

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Abstract

Direct exposure of soil to certain atmospheric agents like water can adversely or favourably influence the engineering behaviour of the soil. For instance, saturated and unsaturated / partially saturated soils behave differently, so do soils under seepage and hydrostatic pressures. Many theories in soil mechanics idealise soils as either cohesive or non-cohesive and this has allowed much research to be done on saturated cohesive soils. However, non-cohesive soils have not received as much attention, apart from recent strength and dilatancy theories, yet in some parts of the world certain non-cohesive soils pose significant risk to structures built on them. The most problematic example of such soils is collapsible soils that may not be detected and properly considered in routine ground investigation activities. In this paper some case studies of collapsible soils in the United Arab Emirates (UAE) are examined, to analyse the effect of their collapse on infrastructure and the possible techniques to ameliorate the situation. The case studies include various sites that were found to suffer structural damage traceable to collapsible soils. It is found that in most cases the soil collapse was due to infiltration of rainwater or water from sustained irrigation activities at the surface.

Keywords

Site investigation; Failure; Field testing & monitoring

1. Introduction

Civil engineers build different types of infrastructure on various soil types that occur in different parts of the world. The range of infrastructure includes light and heavy overground structures, subsurface installations, slender but tall buildings structures and many more. The structures are supported on variable soils that broadly include both residual and transported soils. Residual soils are those that were formed due to weathering of rocks and have remained at their original locations, whereas transported soils are deposited away from their place of origin (Rezaei et al., 2012). Transportation of soils is caused by gravity movement, wind, water, glacier or human activities. Usually the properties of transported soils are influenced by the mechanisms of transportation and deposition (McCarthy, 2006). Although many soil types are competent as load bearing media, some soils exhibit swelling, dispersing and collapsible characteristics due to change in water content often present a variety of challenges to engineers (Rezaei et al., 2012). Such soils may require special attention and treatment when being considered for use as foundation materials for important structures. Emphasis is being primarily given in this paper on collapsible soil cases and such soils usually sands consists primarily silt sized particles (Kalantari, 2012) possesses characteristics like naturally quite dry, open structure and high porosity (Noutash et al., 2010). The main drawback of these soils seen in the current case studies is that when standard penetration tests (SPT) are carried out in boreholes, they exhibited N-values in the medium dense range (N=4 to 10) as observed from the geotechnical reports, where collapsible soils were attributed finally as the cause for distresses experienced. The penetration resistances observed are majorly due the inter-granular friction between the particles, when they are dry. However, when these soils become wet due to any reason and coupled with loading, they exhibit collapse in their structure leading to reduction in volume (Jotisankasa, 2005), causing settlements to structures being built upon them. Identification of collapsibility of soil was emphasised by many researchers in the past through laboratory tests (Holtz and Hilf, 1961; Jennings and Knight, 1975; Jasmer and Ore,1987; Anderson and Reimer, 1995; Reznik, 2007; Gaaver, 2012; Kalantari, 2012; Rezaei et al., 2012) and field test tests (Reznik, 1993; Houston et al., 1995). Field tests are undoubtedly expensive in ground investigations and most of these laboratory procedures involved performing tests on undisturbed soil samples through direct shear tests and oedometers, which is very difficult to sample in particularly the cohesionless soils in the case studies depicted in this paper. The procedure proposed by Holt and Hilf (1961), which was later verified by Gaaver (2012) and Rezaei (2012) was simplest of all procedures and it involves determining the dry density and liquid limit. As soils in UAE are mostly dry and cohesionless type, cone penetrometer can be used as an alternative to Casagrande apparatus for determining the liquid limit. However, determining accurately the dry density remains questionable, as it is very difficult to retrieve an undisturbed sample in such soils, and simplest way is to use standard correlations between SPT N-values and dry densities. But, SPT tests are generally carried out before the actual construction of project starts, and characteristics of soils will be changed with the ingress of water into ground due to continuous irrigation of landscapes, unnoticed leakage of water lines or sewage lines etc. Also, ingress of water mostly due to irrigation of landscapes was found to be the reason for distresses observed in the case studies described. Thus, it was understood that further research is required to be carried out in this context and long term aim is to develop a methodology for profiling collapsible soils and predicting their effects on structures and how those effects can be ameliorated using ground improvement methods. This paper examines the behaviour of certain collapsible soils in the United Arab Emirates (UAE), how they cause distresses to structures and the possible solutions that engineers can implement to ameliorate the structural distress problem.

2. Collapsible soils

Collapsible soils are found in many regions of the world including parts of the USA, China, Africa, Russia, Central and South America, India and the Middle East (Murthy, 2010). These are loessial type soils (Kalantari, 2012) and are generally unsaturated in state as found naturally (Zhu and Chen, 2009). Examples of such soils are wind-blown sand, loess or alluvial deposits generally found in arid or semi-arid environments where the evaporation of soil moisture is so high that they do not have sufficient time to consolidate under their own weight (Pye and Tsoar, 1990). They are moisture sensitive soils in that moisture increase causes them to undergo sudden volume reduction and settlement (Figure-1), especially under the load of a structure (Bell, 2000). These soils generally possess porous textures with high void ratios and low relative densities (Rezaei et al., 2012).

As recognised by many researchers (Schmertmann, 1955; Graham and Li, 1985; Holtz et al., 1986; Leroueil and Vaughan, 1990; Wesley, 1990) the structure of a soil significantly affects its mechanical properties. Collapsible soils and fills are susceptible to abrupt increase in density due to increase in moisture content or temperature, or as a result of the dissolution of compounds that bond loosely arranged soil particles (Dudley, 1970; Reginatto and Ferrero, 1973; Petrukhin, 1989). In the natural state of collapsible soils, their void ratios are so large as to hold moisture equivalent to the liquid limit value. In the dry state, such soils may offer sufficient resistance to structural loads, but suffer large reductions in void ratio due to wetting and re-arrangement of particles (Jotisankasa, 2005). Additionally, these soil types can show rapid collapse response to saturation (Bolzon, 2010).

Efforts have been made by various workers (Holtz and Hilf, 1961; Jennings and Knight, 1975; Jasmer and Ore, 1987; Anderson and Reimer, 1995; Reznik, 2007; Gaaver, 2012; Kalantari, 2012; Rezaei et al., 2012) to characterise collapsible soils based on laboratory testing. As stated earlier, Holtz and Hilf (1961) suggested that loess-like soils that have a void ratio large enough to exceed its moisture content beyond its liquid limit upon saturation are vulnerable to collapse. A graph (Figure 2) has been developed to help in identifying whether a soil exhibits collapse behaviour or not. The graph requires knowledge of just two basic properties: dry density and liquid limit. Once determined, if the soil falls on/below the line, it shows that that soil is collapsible if there is ingress of water. Later Houston et al. (1993) and Das (2009) also suggested that collapsibility can be evaluated by determining the dry density and liquid limit. Jasmer and Ore (1987) proposed an approach for identifying the collapsibility of soils through the use of direct shear tests on undisturbed and compacted soils. Anderson and Reimer (1995) conducted constant-shear-drained tests using tri-axial methods and concluded that knowledge of stress path is essential to accurately predict the collapse potential of such soils. Reznik (2007) conducted a series of oedometer tests and reported that soil collapse starts when applied stress exceeds the structural pressure level of the soil; ‘structural pressure’ being defined as pressure corresponding to separation ‘point’ between elastic and plastic states of any soil (including collapsible soils) under loading. Reznik (2007) suggested that in-situ void ratio and natural moisture content could be determined using geophysical methods and such data combined with oedometer test results could be used for predicting magnitudes of structural pressures in collapsible soils.

As stated earlier, some researchers (Reznik, 1993; Houston et al.,1995;) have conducted field tests to help characterise collapsible soils. Reznik (1993) conducted field plate loading tests on collapsible soils and reported the tests to be useful for identifying the collapsibility of soils. Houston et al. (1995) developed an in-situ test known as ‘downhole collapse test’, which they utilised on sites of soils known to collapse due to wetting. The results of Houston et al. (1995) work were compared with actual settlements and found to be reasonably consistent.

Though several case studies have been reported earlier by many researchers, few of them have been mentioned below.

i. In semi-arid New Mexico, a commercial building won an award from the city for the year’s most beautiful lawn and landscaping. However, it suffered in foundation damage owing to differential settlement due to wetting of collapsible foundation soils underneath (Houston et al., 2001).

ii. Noutash et al. (2010) had reported that impounding of Khoda Afarin canal located in northern Iran to mitigate existing collapse potential in the area had caused large cracks on both sides of the canal's berms after the pre-treatment technique was completed.

iii. Kalantari (2012) reported a forensic investigation in San Diego, California, where the annual precipitation was about 30 cm before a residential subdivision was built and has been increased to about 170 cm (counting landscape irrigation) after it was built. Such increased level of precipitation had resulted in substantial settlements of the underlying compacted fill. In addition, the lawns were spongy to walk on and the street side curbs had moss growing on them as a result of heavy landscape watering.

In all the above mentioned three cases, the reason for collapse of soil is due to ingress of water either purposely or unintentionally. Similar kind of cases were noticed in UAE, where continual irrigation of landscapes had led to distresses in neighbouring infrastructure like boundary walls, pavements etc. and were elucidated below.

3. Case studies

In this section, two case studies at locations in the UAE are presented, whereby collapsible soils were suspected to have caused structural distress to lightly loaded structures such as boundary walls, pavements, footpaths, landscapes etc. In the case studies, professional Geotechnical companies were commissioned to investigate how the problem occurred, quantify the level of distress and propose methods of reducing the undesirable impacts. In both case studies, it was revealed that collapse of underlying soils was the cause of distresses experienced by the structures. For data confidentiality reasons, the precise project locations and names of the investigation companies or their clients are not disclosed in this paper in order to comply with the conditions under which the data were made available for this research.

3.1 The guest house project

The project was located in Al Ain city of UAE. The site had been developed with Guest house with landscaped gardens and terraces covering 85% of the site. This equates to more than 15000 m^2 of lawn and garden area formed on a 12 m thick fill of top soil. The fill area is bounded by a two-step precast gravity retaining wall structure, which deformed due to uneven settlement of the ground beneath. As deduced later, the settlements were linked to the effect of irrigation water on collapsible soils existing at some depth in the area. Fortunately, the actual Guest house structure did not experience any distresses as it was supported on pile foundations. When settlements were initially observed, it was decided to carry out remedial works in an effort to keep the structures serviceable. However, settlements continued even after the repair works were completed. No settlements were observed during placement of the fill and the associated landscaping works features prior to irrigation. However, as soon as irrigation activities commenced, within 8-10 months, very clear signs of surface settlements and associated distresses were seen. Though distresses were observed on site at several locations, few of them are highlighted below.

i. Kerbstones adjacent to landscaped areas were separated from the walkways by approximately 40mm.
ii. The steps which are in close proximity to landscaped areas of the Guest house structure experienced subsidence, whereas the actual structure (founded on piles) did not (Figure 3).
iii. Large settlements (approximately 80mm) were observed in areas paved with concrete slabs, which are in close proximity with landscaping areas.

The magnitude of settlements observed on site was measured to be in the range of 2-3 cm on the low side and 9-10 cm on the high side. Consequently site investigations were commissioned in order to evaluate and explain the causes for distresses (settlements) observed in the soft and hard landscaped features around the Guest house structure. Ten boreholes of 15 m depth and two others of 20 m depth were drilled along with 4 excavation test pits each 2 m in depth. Additionally, the following field tests were carried out: (i) Standard penetration tests (SPT), (ii) Permeability tests, (iii) Mackintosh probe tests and (iv) Soakaway tests. The general stratigraphy of the site and the observed SPT blow counts are given in Table 1. The mean permeability of the soil obtained from field permeability tests was found to be in the order of $6.83\text{x}10^{-7}$ m/s and is typical for soils with high silt content. Bell (2000) provided an indication of the potential severity of the collapse (Table 2). Collapse potential tests carried out on soil samples from the test pits are shown in Table 3 and the values indicate that the soils are susceptible to collapse and the severity of the problem can is categorized as 'very sever trouble' (Bell, 2000)

Considering the various structural distresses observed at the site and given the vast area of ground to be improved, it was thought that grouting would not be an economic option. Thus, hydro-compaction was recommended as a preferable and inexpensive option. To avoid further distresses due to settlement of soil while hydro-compaction was in progress, it was also recommended to use hydraulic jacks to lift up the existing gazebos and swimming pool structures existing at the site. Upon completion of hydro-compaction and cessation of ground settlements, cement grout would be injected along any resulting gaps, to ensure that the bases of the gazebos and swimming pool structures make complete contact with the ground.

3.2 An infrastructure project

This low-rise housing development in Abu Dhabi (UAE) consists of villas, amenity buildings, community buildings and open green spaces. A network of sector roads traverses the area and connects to the surrounding highway system. Upon completion of construction and during the first year of occupation and service, evidence of distress (due to excessive settlements) began to appear in certain areas of the development. Buildings including villas and other communal or amenity buildings show absolutely no signs of distress since they rest on rigid pile foundation systems. The affected areas were mainly in shallow founded structures/features such as boundary walls, hard landscapes, soft landscapes and internal roads. Although many distresses were noticed on site, quite a few are mentioned below.

i. Footpaths at locations adjacent to landscaped areas experienced settlements (approximately 75 mm) under the effect of continuous water ingress.
ii. Though several boundary walls had distressed on site, those walls which are located with landscaping on either side had suffered the highest level of distress with settlements approximately 260mm (Figure-4).
iii. Flexible pavements, particularly those adjacent to open landscaped areas had experienced distress (settled approximately 100 mm) as well. As, stress transfer under flexible pavements is largely limited to 2.0-2.5 m below ground, it was initially thought that very loose to loose soils that are susceptible to collapse due to movement of water were present at shallow depths. This was later confirmed from the low SPT blow counts observed at very shallow depths (1.0-1.5 m) in the drilled boreholes.
iv. Interestingly it was found that the ground in some green landscaped areas with no structures also subsided (approximately 150 mm). Hence, it was suspected that the ground movements could be due to percolation of the irrigation water down to collapsible soils at depth.

To confirm this, a Geotechnical company was enlisted to carry out thorough investigation of the structural damages and to propose suitable methods of remediation. Two 15 m deep boreholes were drilled close to the areas of observed distress. The boreholes revealed a 1.5-2.0 m thick layer of top soil, which was interpreted to be very loose to loose, based on the recorded SPT blow counts. Also, the groundwater table was encountered at an average depth of 1.5 m below the surface. Under these circumstances, in order to verify how the top loose soils responded to the presence of irrigation water, some open landscaped areas were selected and flooded with water (hydro-compaction) for 15 days to seep through the soil. Such flooding of water on soft landscapes was limited to the height of adjacent hard landscapes (footpaths), to avoid overflowing of water indiscriminately everywhere on the site. It was initially decided to adopt flooding (continuously 12 hours) and desiccation (continuously 12 hours) in equal intervals of time in a day till no further seepage of water into the ground is observed. However, this was continued for only two (2) days and such fixed cycle timings could not be continued due to heavy flooding in a short period of time. Finally the site has reached to such a condition that two (2) hours of flooding time is sufficient for the entire landscaping areas to get flooded and hence the hydro-compaction process terminated with limited number of cycles. This speedy flooding situation could be attributed to less free draining material and high groundwater table on site. Hydro-compaction process was terminated once noticed that no more water was seeping onto the ground. To check whether the seepage of excess water into the soil had improved the density of soil, Mackintosh probe tests were undertaken before and after the hydro-compaction process. As shown in Figure 5, it was found that the soils responded to water movement because the number of blows after hydro-compaction increased for all depths down to 1.4 m. However, the improvement in ground was not noticed locally at depth 0.4-0.6 m and this could be due to saturation of soil instead of responding to collapse of soil structure due to hydro-compaction, which otherwise might have responded to water movement. Similar behaviour was noticed at depth below 1.4m and this could be attributed to the nearness of groundwater table located at 1.5 m below ground. As stated by many researchers (Dudley, 1970; Reginatto and Ferrero, 1973; Petrukhin, 1989; Bell, 2000; Jotisankasa, 2005; Bolzon, 2010; Rezaei et al., 2012) collapsible soil do respond to moisture and their density increases with movement of water due to re-arrangement in soil structure into denser packing, presence of collapse soils in the area of concern was confirmed.

It was considered that hydro-compaction might cause nuisance to the occupants of the villas and so an alternative way of improving the loose soil at shallow depths was explored. Chemical grout (using 35% sodium silicate, 5% amide and 0.5% bicarbonate) was injected under boundary walls and edges of hard landscaped areas, to densify the upper 2m of the soil stratum. For this purpose, holes were drilled down to 2.5 m below ground on either side of boundary walls at 1.5 m centres in a staggered manner and along the lines of private hard landscapes at 1.2 m centres in a linear manner. Under controlled pressure, grouting was done in such a way that upward heaving of the ground was prevented. Upon accomplishing the grouting of all drilled holes, a period of four weeks was allowed for the grout to cure. Mackintosh probe tests were carried out before and after the grouting process to verify the effectiveness of the soil densification process. It can be seen (Figure 6 and Figure 7) that the depth of improvement due to grouting was limited down to 0.6 m compared to hydro-compaction, where the improvement was noticed up to 1.4 m below ground. Such limited depth of improvement in ground due to chemical grouting could be due to non-uniform permeation of grout into soil beyond 0.6-0.8 m below ground. Hence, it was suggested to continue with the hydro-compaction in all areas where settlements were noticed, allowing the settlements to proceed to their maximum values before continuing with repair work to reinstate the distressed structures.

4. Possible solutions

Taking into account the collapsibility of soil, solutions/techniques recommended by various researchers were summarized by Houstan et al. (2001) and are given below.

- Removal of volume moisture-sensitive soil
- Removal and replacement or compaction
- Avoidance of wetting
- Chemical stabilization or grouting
- Pre-wetting
- Controlled wetting
- Dynamic compaction
- Pile or pier foundations
- Differential settlement resistant foundations

However, these possible solutions are recommended to consider based on the site location, type of soil, practicability etc. into consideration. In view of understanding the suitability of above mentioned solutions suggested by various researchers to the specific case studies discussed, complete removal or removal, replacement and compaction of moisture sensitive soil options cannot be considered viable as it is a tedious task and creates chaotic conditions for the existing tenants. Avoidance of unwanted wetting can be considered as solution in terms of controlling any undesirable leakages from underground conduits, provided efficient monitoring system is in place. Chemical stabilization and pre-wetting (hydro-compaction) are feasible solutions on both sites, provided efficacy of such techniques are verifies beforehand. These techniques were tried in the infrastructure project and finally suggested to opt for hydro-compaction compared to chemical grouting, as non- uniform permeation of grout was noticed. Controlled wetting could be considered as solution in both cases provided specific quantum of water supply to the existing landscapes that does not lead to collapse of soil can be calculated and strictly implemented. Dynamic compaction cannot be opted in both case studies, as they are already developed sites and residents are in place. In both case studies, as mentioned earlier that actual structures are already founded on piles and problems are associated with light loaded structures. Pile and pier foundations could be considered as proper solution especially for boundary walls, provided sufficient finances are available. Strap foundations can be considered for founding the boundary walls, which helps in controlling the differential settlements.

Keeping in view those problems associated with collapsible soils in the case studies described in this paper, following solutions could be considered where such soils lie at limited depths not exceeding 2.5 m to 3.0 m below the surface.

- Permanent sheet piling should be installed all along the periphery of villas / buildings founded on shallow footings, provided the development budget permits.
- For low rise buildings / villas, all isolated foundations should be either connected with continuous stiff strap beams or formed of raft foundations.
- Boundary walls should be bearing on long stiff beams all along the perimeter of the building. Optionally the walls could be made with lightweight but sufficiently materials or founded on mini piles.
- Where, greenery (soft landscape areas) is planned around structures with no deep rooted plants, existing soil could be excavated down to the top of collapsible soils and a layer of impermeable membrane inserted followed by backfilling.

However, the deeper layers could be densifed by pre-wetting through boreholes, using overburden pressure to drive the collapse (Houstan et al., 2001)

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5. Conclusions

Case studies of structural damage at locations in the UAE were examined to study the problem of collapsible soils in the area and how human activities such as lawn irrigation exacerbate the problem. Lessons are learnt that the design of foundations in such an environment calls for further considerations beyond the usual bearing capacity and settlement of just the founding soils. The problem lies at greater depths where collapsible soils exist and where infiltration of surface water can cause irreversible collapse of the soils to lead to structural damage over time. Therefore the need to understand and properly consider the site geology in such sites cannot be overemphasised. Prior to development at such sites, a thorough geotechnical exploration is needed to detect and characterise any problematic soils possibly existing at depths far below the levels where borehole would be terminated in straight forward cases. The case-studies discussed in this paper will form part of an on-going Doctoral research project aimed at assessing the mechanisms of structural distress caused by collapsible soils in the United Arab Emirates (UAE).

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Figure captions

Figure 1. Loaded collapsible soil before (a) and after (b) inundation with water

Figure 2. Dry unit weight of soil versus liquid limit

Figure 3. Separation of stairs from adjacent wall due to differential settlement

Figure 4. Cracking and settlement of boundary wall

Figure 5. Mackintosh probe test results

Figure 6. Mackintosh probe test results at boundary walls

Figure 7. Mackintosh probe test results at hard landscaped areas

Table Captions

Table 1. General stratigraphy of the guest house site

Table 2. Collapse percentage as an indication of potential severity.

Table 3. Collapse potential test results

Table 1 General stratigraphy of the guest house site

Depth (m)	Description of soil	Range of SPT N-Value	Relative density (based on SPT)
0.0-1.0	Silty SAND (agricultural soil as fill material)	3-19	Very loose to medium dense
1.0-13.0	Silty GRAVEL / Gravelly SILT (fill material)	3-30	Very loose to medium dense
13.0-15.0	Silty SAND (dune sand)	32-50	Dense to very dense
15.0-19.0	SILT (alluvial soil)	37-50	Dense to very dense
19.0-20.0	Silty GRAVEL (residual soil)	>50	Very dense

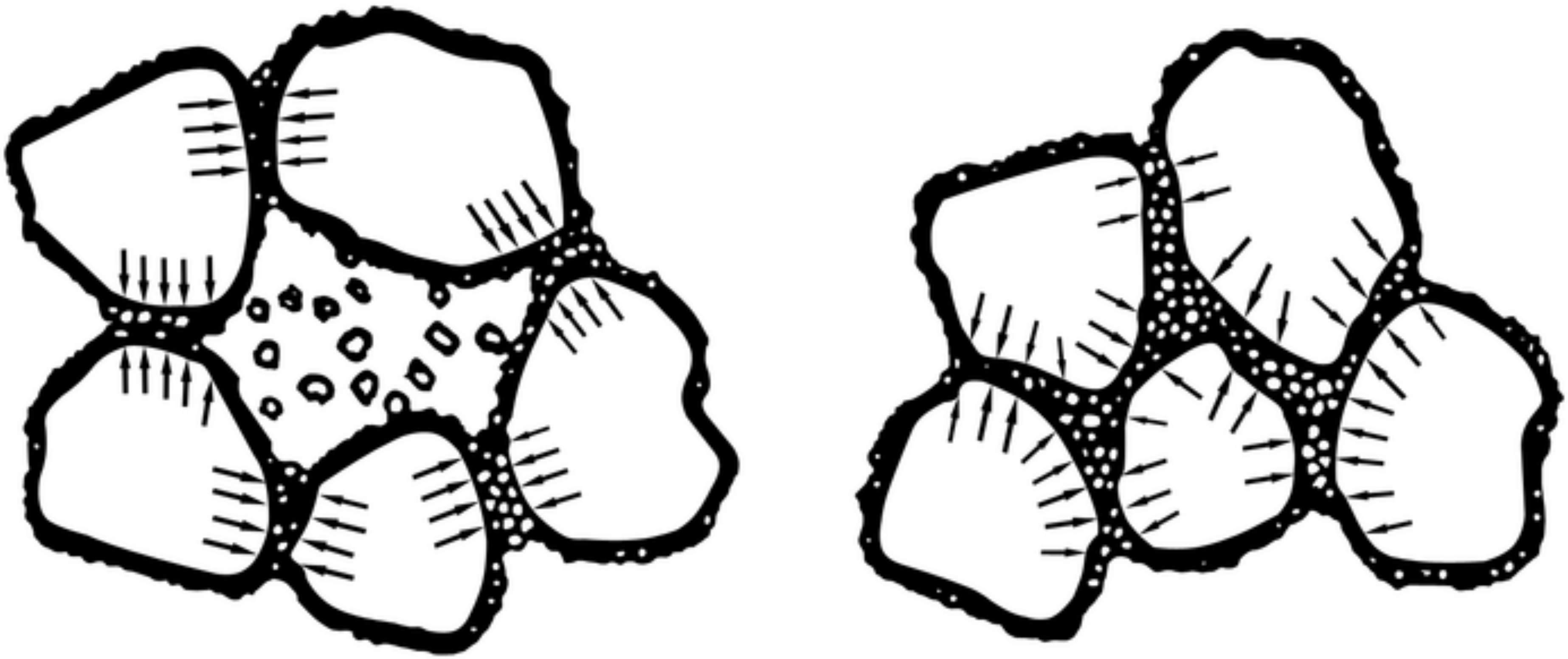
Table 2 Collapse percentage as an indication of potential severity.

Collapse (%)	Severity of problem
0 - 1	No problem
1 - 5	Moderate trouble
5 - 10	Trouble
10 - 20	Severe trouble
Over 20	Very severe trouble

Table 3 Collapse potential test results

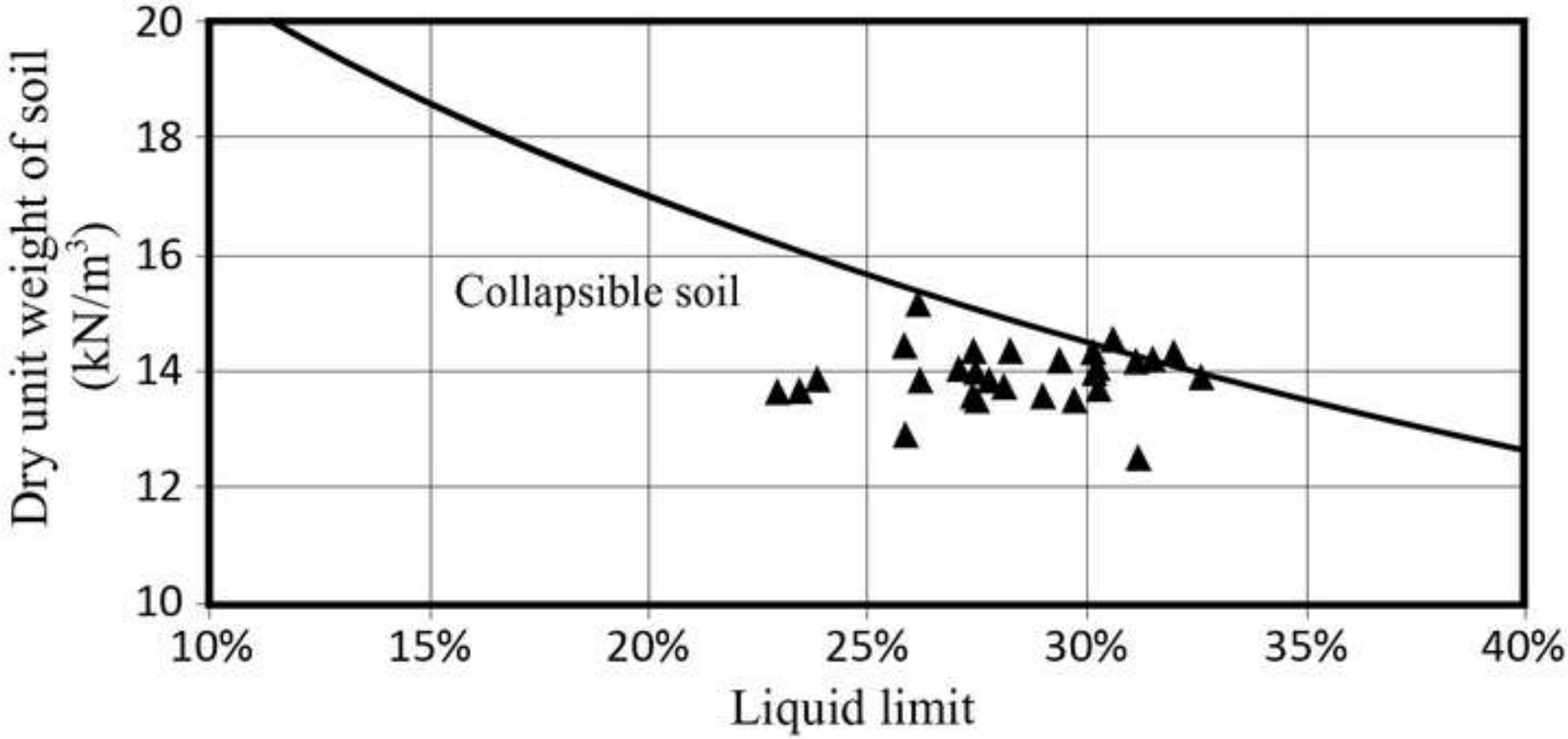
Test pit no.	Depth (m)	Collapse potential (%)
2	0.50	64.5
3	1.20	86.4
4	1.85	86.7

Loaded collapsible soil before (a) and after (b) inundation with water



(a) Dry soil with honeycombed structure before inundation

(b) Soil structure after inundation



Dry unit weight of soil (kN/m^3)
20
18
16
14
12
10
Collapsible soil
10%
15%
20%
25%
30%
35%
40%
Liquid limit

Separation of stairs from adjacent wall due to differential settlement

Cracking and settlement of boundary wall

Mackintosh probe test results

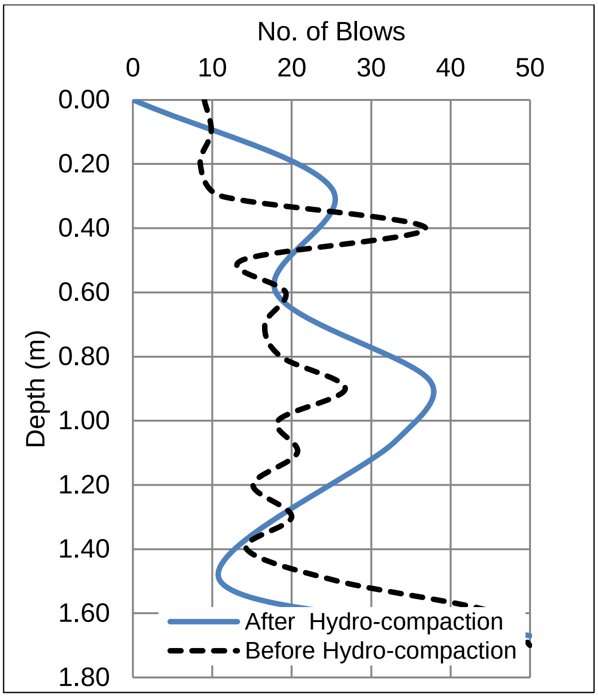


Figure 5 Mackintosh probe test results

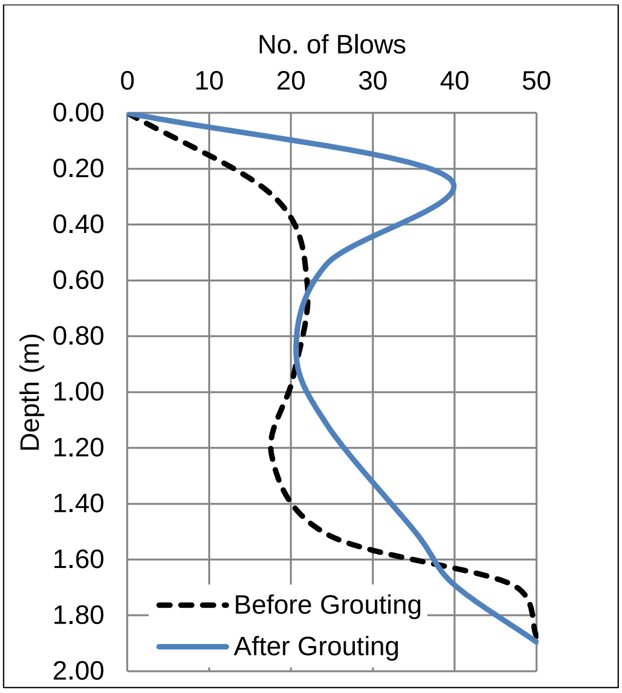


Figure 6 Mackintosh probe test results at boundary walls

Mackintosh probe test results at hard landscaped areas

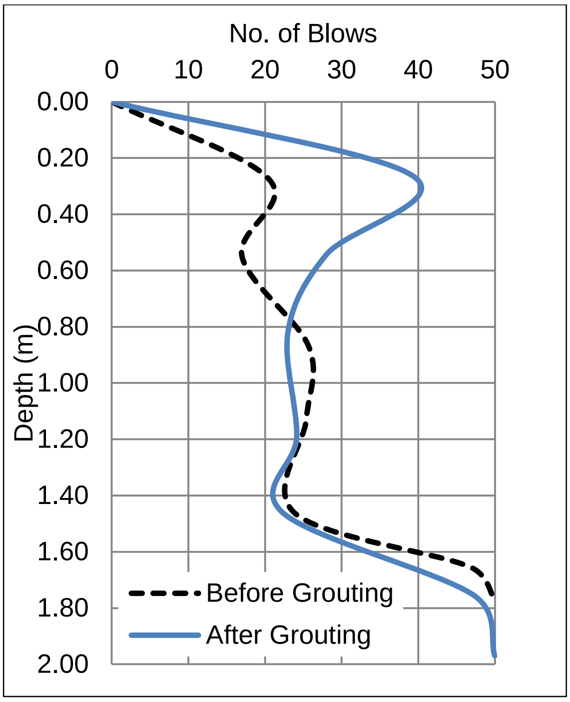


Figure 7 Mackintosh probe test results at hard landscaped areas