Foundation solutions for light and heavy construction on expansive soils: Case studies

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ABSTRACT
In this paper, two building projects of EllisDon Corporation are studied to compare the design and construction challenges involved in expansive soils. The first building is a hospital in Winnipeg, Manitoba which is a six-storey building with a large column spacing of 9-m. The second building is an agricultural building in Vancouver, BC. This building is a lightweight building. Both of the buildings are founded on expansive soil.
Different foundation solutions are studied for both cases. The solutions are designed based on the available geotechnical report. The geotechnical differences of the two expansive soils are also highlighted. The feasibility of the solutions is then investigated based on parameters like constructability, cost, and schedule. Final recommendations are provided for the foundation type for each project.

RÉSUMÉ
Dans cet article, deux projets de construction d'EllisDon Corporation sont étudiés pour comparer les défis de conception et de construction impliqués dans les sols gonflants. Le premier bâtiment est un hôpital de Winnipeg, au Manitoba, qui est un édifice de six étages avec un grand espacement de colonne de 9 m. Le deuxième bâtiment est une construction agricole à Vancouver, en Colombie-Britannique. Ce bâtiment est de type léger. Les deux bâtiments sont fondés sur un sol expansif.
Différentes solutions de fondation sont étudiées pour ces deux cas. Les solutions sont conçues sur la base du rapport géotechnique disponible. Les différences géotechniques des deux sols gonflants sont également soulignées. La faisabilité des solutions est ensuite étudiée en se basant sur des paramètres tels que la constructibilité, les coûts et l'échéancier. Les recommandations finales sont fournies pour le type de fondation de chaque projet.

1 INTRODUCTION

Expansive soils have made significant problem for structures all around the world. The changes in volume of expansive soils could be significant and impose differential heave and therefore extensive stress to the structure. This settlement have a range of 1 to 20 in. The changes in volume related to the moisture content changes of clay. The physiochemical characteristics of clay minerals, percentage of the clay, level of the moisture content and changes in the moisture are the major parameters affecting the volume change in expansive clays.

The design of foundation on expansive soils are much more challenging than ordinary soils. Any ignorance in considering the specific requirements of foundation design in expansive soils could result to expensive repairs. Although the process of soil sampling and in-situ investigation does not have much difference with the one for foundation on ordinary soils, the amount of laboratory testing and analysis of the results are much more extensive.
The design of foundation on expansive soils requires one of the following approaches in concept design:

- The structure is designed to have enough rigidity to tolerate soils expansions
- The foundation is designed in a manner to isolate the main structure from expansive soil
- The soil is improved to have tolerable volume changes for the specific structure

The design concept could also be a combination of above solutions. Shallow foundations should not be used on expansive soils. The stiffened (or ribbed) mat foundations are the most common foundations in moderate expansive soils in which the heave is not excessive.
A deep foundation is primarily used to isolate the structure from the expansive soil. It consists of deep foundation which extend to a layer of soil with tolerable expansive property, a grade beam on top of deep foundation which support the floor and the superstructure. The grade beam would not be in contact with expansive soil to isolate the structure from its adverse effects. Deep foundations also are used for rehabilitation of damaged foundation on expansive soils.
Soil treatment and moisture control options often involves measures to reduce changes of soil moisture content and improving the behavior of clay by affecting its chemical properties. The moisture control involves installation of moisture barriers and/or subsurface drains. The soil treatment usually involves either moisture-conditioning or re-compacting or the use of chemicals to reduce the expansion potential of soil. The successful implementation of soil improvement requires a good understanding of the effect of measures used on soil behavior. This method is often used in combination with other methods.
In this paper, the foundation options for a light weight and moderately heavy weight structure is studied.

2 CASE 1: HOSPITAL BUILDING

The location of the proposed hospital redevelopment is at Winnipeg. The new multi storey health care facility is comprised of a five storey building, a penthouse, and one underground parking. The column loads are in the order of 7300 kN. The property includes a a light commercial building and parking areas. The commercial building had a partial basement. The spread footings supported the individual columns.

2.1 Geotechnical condition

There were considerable geotechnical information in Health Science Centre which helped the geotechnical engineer to plan the field investigations. The scope of work for the field investigation was to include the drilling and sampling off large diameter caissons which would not be carried into the underlying limestone bedrock. They were located at accessible locations along the length of the proposed structure. A preliminary site visit raised concerns about the moisture condition in the subsurface silty clay profile which might seriously impact the performance of a preferred slab-on-grade floor for the underground parkade. As a result, four test holes were planned to be drilled below the basement floor along the length of the existing building.

Four test holes of 390 mm in diameter were conducted using air-driven equipment to a depth of 6.63 to 10.34 meters (Smith Carter Consultant 2010). Disturbed samples were recovered from the auger cuttings and undisturbed samples were obtained using 75 mm diameter Shelby tube samplers.

Another set of four boreholes was undertaken by drilling and sampling of 610 mm in diameter and were carried into the underlying very dense glacial till. Disturbed samples were recovered from the auger cuttings and undisturbed samples were obtained in 75 mm Shelby tube samplers.

The geotechnical investigation reveals that the soil profile consists of (Table 1) layer of fill material over the usual Lake Agassiz lacustrine silty clay which in turn overlies a glacial till deposit which is founded on a limestone bedrock.

The fill has three compositions. In building footprint, it consists of 190 mm of concrete and approximately 90 mm gravel. Outside of building footprint and in paved area, it consists of pavement over 300 to 1160 mm of clay fill. In the landscaped area, the clay fill extended to a depth of 2.44 metres.

The Lake Agassiz lacustrine clay extends to depths ranging from 11.28 to 12.96 metres below surface grades. The silty clay is highly plastic and varies from firm to stiff in its relative consistency with undrained shear strengths (Table 2) ranging from about 40 to 70 kPa, however, within the building footprint above a depth of about four metres, the range was about 60 to 90Pa. The moisture content profiles are generally in the order of 50 percent with a range between 40 and 60 percent.

Table 1. Soil Profiles and Stratigraphy

Layer	Depth (m)
Fill	0.0 - 2.4
Silty Clay	1.0 - 13.0
Glacial till	9.5 - 20.0
Limestone	19.0

The glacial till was encountered beneath the silty clay. The deposit is a heterogeneous mixture of sand, gravel, cobble and boulder size materials within a predominately silt matrix that has a low but variable clay content. The relative density of the glacial till varies from loose to dense to very dense on the basis of the examination of the auger cuttings, moisture contents and drill performance. Visual observations of the glacial till cuttings and the drill performance suggested that it was dense in upper part and very dense in lower part. This was confirmed by the moisture contents of the upper till being in excess of about 8.0 percent and for the lower very dense till ranging from 5.9 to about 7.0 percent.

Table 2. Silty Clay Properties

Type	Value
Qu	25 - 87 (kPa)
PP	45 - 125 (kPa)
VST	37 - 122 (kPa)
γ_d	16.4 - 18.5 (kN/m^3)

The depth to the underlying limestone bedrock was not determined in this investigation. However, from the historic information available from the Health Sciences Centre, the bedrock is at a depth of 19 to 20 m below grade.

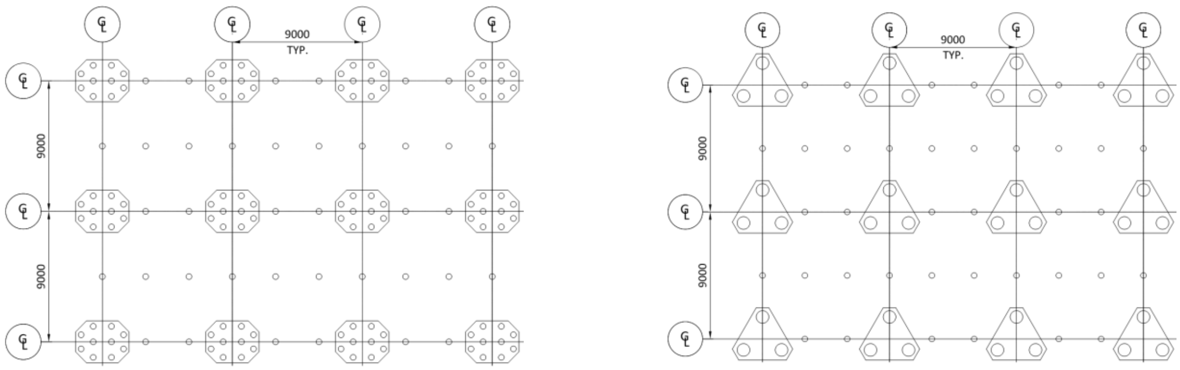
The Atterberg Limits testing was conducted to determine the properties of the fine grained deposit. The plasticity index of the clay samples found to be between 50% to 70%. The swell test revealed a high swell potential with a very high swell pressure of up to 6000 kPa.

2.2 Proposed foundation solutions

The new development is comprised of a five storey healthcare structure with a single storey penthouse and one level of underground parking approximately 4.5 meters deep. The column loads are anticipated to be in the order of 7300 kN. which are high and would suggest that the foundation alternatives could be cast-in-place concrete rock socketed caissons or large diameter cast-in-place concrete caissons (belled or otherwise) which would be end bearing on/in the glacial till deposit.

There were little and/or no experience with rock-socketed pile type of foundation at the Health Care Facility, at least since the early 1970's. It is presumed that the reasons for this are that the foundation loads for the structure were not large, the upper part of the bedrock was highly fractured and the condition of the glacial till cover on the bedrock was not acceptable. Also, the piezometric conditions in the limestone bedrock have been increasing since 1990 and as such have posed significant obstacles to construction.

Another alternative was driven precast concrete endbearing piles which had been the preferred foundation alternative since the 1970's and had been used successfully, with some minor problems.



Option 1 Option 2

Figure 1. Foundation option for Heavy Hospital building

Therefore the rock-socket caissons are not considered a preferred foundation for the support of this proposed structure. Other alternatives, such as the driven end bearing precast concrete piles and/or cast-in-place concrete end bearing caissons are viable alternatives and would be expected to be more economic.

Cast-in-Place End Bearing Concrete Caissons which would be end bearing on/in the underlying glacial till deposit are considered a preferred foundation alternative for the proposed structure. Based on the geotechnical report these piles which may be founded on the dense glacial till and may be sized on the basis of an allowable bearing pressure not exceeding 720 kPa. However, if they were to be founded on the very dense glacial till which is present below the dense till, they may be sized on the basis of 1440 kPa. It may be economic to found the caissons on the very dense glacial till to take advantage of the higher allowable bearing pressure even though it is at greater depth. These piles could be built as a straight shaft or belled caissons.
The bearing surfaces would have to be hand cleaned which would require that the shaft sizes would have to be a minimum of 760 mm in diameter. Problems with boulders and seepage should be expected and could influence the depths at which the caissons would be founded. The caisson bases should be inspected by geotechnical personnel experienced with the construction of these types of Caissons.
Driven end bearing precast concrete piles, as an alternative solution, if driven to practical refusal in the underlying glacial till, may be assigned conventional supporting capacities of 445, 625 and 800 kN for nominal 300, 350 and 400 mm sizes respectively. The piles should be driven with hammers with a rated energy of not less than 40 kJ.

It is expected that the inspection of the driven pile installation to be undertaken by experienced personnel. The presence of cobbles and boulders may result in pile installation problems which should be monitored.
The parkade is about 4.5 metres below the outside grade which may require an excavation in the order of approximately 5.0. At this depth, the excavation will extend below the piezometric surface of the bedrock aquifer (Provincial Groundwater Monitoring Station). A thick layer of silty clay and glacial till prevents the upward movement of this water. However, should a pathway develop to allow groundwater movement upwards, seepage to the excavation and/or the completed structure could become an issue. These pathways could develop by seepage along the pile or caisson foundations or by movement along naturally occurring fractures in the clay. The conditions against base heave are satisfactory.

2.3 Comparison of foundation solutions

The table 3 shows the two options of foundation system as illustrated in Figure 1. The option 1 is comprised of driven 450mm precast concrete piles which are casted into 1800 Cap piles over void form. The option 2 is comprised 900mm cast-in-place caisson piles belled at bottom which are casted into 1800 mm Cap piles over void form. The 900 mm piles should extend to lower layer of glacial till to achieve the higher capacity as geotechnical report suggested. The Table 3, shows that the option 2 is about %23 more expensive than option 1.

Table 3. Foundation Options for Heavy Hospital building

Option	Type	Description/Comments	Total ROM $'s per Column
1	Suspended slab on driven precast concrete slab	Precast HEX Pile 450, 1800 mm Cap pile and, on 150mm void form, and 200mm suspended concrete slab	$22k
2	Suspended slab on cast-in-place concrete end bearing caissons	900mm concrete caissons, 1800 mm Cap pile on 150mm void form, and 250mm suspended concrete slab	$27k

3 CASE 2: LIGHTWEIGHT INDUSTRIAL BUILDING

The subject site is located in a rural portion of the Thompson Nicola Regional District (TNRD) immediately east of Merritt, BC.

The site consists of an undeveloped, rectangular, rural lot with plan dimensions of 220 m north to south by 800 m east to west. The ground surface on the site is generally flat with slight rolling grade changes; the site rises towards a hill feature located at the southwest corner of the site, but the majority of the hillside is located offsite.

At the time of the field work, the site was sparsely vegetated with native grasses, scrub brush and cacti. At the time of geotechnical investigation the proposed development at the site was at a preliminary stage, but would likely consist of a pre-engineered, prefabricated steel frame building with a metal roof and cladding. It is proposed to support the building on shallow reinforced concrete strip and pad footings. A preliminary column spacing of 10 m has been considered. The building loads were not available at the time of geotechnical investigation while based on similar project the following assumption was made:

• Exterior Strip Footing = 60 kN per lineal metre (combined dead and live load)
• Typical Interior Pad Footing = 600 kN
• Slab Live Load = 15 kPa.

A parking area would be located on the north side of the building. Information regarding proposed site grades was also not available at the time of investigation but It is anticipated that final grade on the site would approximately match the current site elevations, with limited cuts and fills required to achieve level building and pavement areas.

3.1 Geotechnical condition

Ten test pits excavated on the site using a tracked excavator (Levelton Consultant 2014). Eight of the test pits were conducted in the vicinity of the building footprint and extended to depths of about 3.5 to 4 m below grade.

Two additional, shallow test pits were conducted north of the building in the proposed parking area. Disturbed soil samples were collected from the test pits for visual classification and moisture content testing. One sample was selected for Atterberg Limits testing and grain size distribution analysis, one sample was selected for Atterberg Limits testing only, and two samples were selected for swell testing.

Geological Survey of Canada Map 8 -1962 indicates the site soils consist of lacustrine and glacio-lacustrine deposits; specifically, thin veneer lake deposits. A surface deposits of weathered clay was observed at the surface. Boulders, as well as sand and gravel deposits, were observed on the hill immediately southwest of the site.

The soils observed at the test pits were generally consistent with the description provided on the surficial geology map. In general, the test pits encountered generally uniform soil conditions across the site. Surficial soils consisted of weathered and fissured clay topsoil with organics and roots extending to depths of about 400 to 800 mm. Soil below the topsoil and weathered / fissured clay consisted of very stiff to hard varved (layered) clay deposits. The varves varied in thickness from 5 to 25 mm in thickness. The test pits were terminated in the very stiff to hard clay deposit at depths ranging from 3.5 to 4 m in the building area and 2 m in the parking lot area. No groundwater seepage was encountered at any of the test pits during the time that they remained open.

Samples collected from the test pits were submitted to the laboratory for physical properties testing. Initial tests were conducted to assist in general soil classification. Based on the results of initial tests the geotechnical engineer was authorized by the Client, to conduct swell testing on two samples.

Tests were conducted on a typical sample of the clay to verify the predicted grain size distribution and determine general physical properties of the deposit. Tests were conducted on two typical samples of clay material; a sample of hard, blocky clay was selected at a depth of 2.4 m, and a sample of hard clay was selected at a depth of 1 m from two test pits. Atterberg Limits testing was conducted to determine the properties of the fine grained deposit. The results of the Atterberg Limits testing are shown in Table 4 as follows:

Table 4. Soil characteristics

Sample Liquid Limit (%)	Plastic Limit (%)	Plasticity Index
73.5	26.2	47.2
54.2	15.4	38.8

The results indicate the samples are a medium to high plasticity inorganic clay. The results of a grain size distribution test by hydrometer on the sample indicate the sample is primarily clay (~85%) with some silt (~15%).

The local knowledge suggests that swelling clays exist in the Merritt area. Swelling clays are primarily high plasticity clays with low in-situ moisture content that “swell” when exposed to moisture, increasing in volume and imparting a “swelling pressure” if constrained. Based on the initial laboratory testing, it was determined that the deposits encountered at the test pits had significant swell potential and that specialized swell testing should be conducted to determine the swelling characteristics of the clay deposit. The geotechnical engineer recommended that swell testing be conducted on two samples. Swell testing involves preparing a test specimen of known volume and saturating the soil. The volume change of the sample is recorded over time to determine the swell potential, following which the sample is loaded to return it to its original size to determine the swell pressure.

The results of the swell test are shown in Tables 5 and 6 as follows:

Table 5. Swell potential

Swell Potential (%)	classification
11.4	Medium
12.6	Medium

Table 6. Swell pressure

Swell Pressure (kPa)	classification
1500	Very High
1500	Very High

Swelling tests were only practical on the blocky hard clay formation found at depth in the test pits.

Based on the Atterberg Limits testing, it is likely that the surficial weathered hard clay formations may have a lower swell potential. More swelling characterization testing is required in the detailed design phase.

3.2 Proposed foundation solutions

From a geotechnical point of view, the site was considered suitable for construction of a light industrial, commercial, or agricultural building; however, the presence of a swelling clay deposit throughout the site poses a unique foundation design challenge. Initially, it was expected that the proposed building could be supported on strip and pad footings, with structural considerations for potential differential movements.

Based on the two swell tests conducted, the in-situ hard, dry clay can be classified as having medium swell potential (predicted volume change during swelling) and very high swelling pressure (stress generated by soils as they swell). Based on these conditions, it is our opinion that swelling must be considered in the design phase to reduce the risk of damaging differential movement in the foundation system.

Given access to a water source, including surface water, storm water, or a water utility leak / break, the swelling soils will saturate, swell and heave upwards. This upheave movement will vary across the affected area, based on the available water and non-uniformity of the swelling deposit, causing differential movements between locations. Predicting upheave of a swelling deposit involves determining the active zone of the deposit (typically a minimum of 2 to3 m) and determining the material specific free swell potential.

Given the swell test results with an average free swell of 12%, and an estimated active zone of 3 m, a surface swell of 120 mm was predicted if the clay becomes saturated. This predicted swell assumes no surficial load is present to ballast the swelling deposit and resist the uplift.

Typically, swelling soils of low induced pressure can be addressed by ballasting the deposit. This design process involves calculating the resulting swell pressure for the maximum allowable surface swell and placing an equivalent pressure on-top of the deposit to keep it compressed through use of fills or building loads. Typically, with a shallow deposit of swelling soils this is accomplished by over excavating a certain amount of the swelling deposit and replacing it with non-swelling fill. In the case of the subject site, the very high swell pressure induced by the material makes this solution unfeasible. For example, to ballast the entire swell potential of 12% would require an equivalent pressure of 1.7 MPa, which would require a prohibitive thickness of ballast fill.

The Canadian Foundation Engineering Manual (2006) outlines three general foundation options for buildings on expansive (swelling) soils. These include:

- Shallow spread footings;
- Pier and beam (pile) system; and
- Stiffened slab-on-grade.

3.3 Comparison of foundation solutions

The Table 7 and the Figure 2 show the available foundation options for the light weight agricultural facility. The option 1 includes a type of soil improvement technique which uses moisture barrier and replacement of a layer of expansive soil. In option 1a, one meter of expansive soil is replaced with structural backfill and in option 1b two meters of expansive soil is replaced with

Table 7. Foundation Options for light industrial building

Option	Type	Description/Comments	Total ROM $'s
1a	Structural Pad under Building with 1 m Deep Structural Fil	Thickened Edge SOG, PVC Membrane under 1m Structural Fill, Allowed for 6m between Header House & GreenHouse to be structural fill as well. If City Site, large amount of fill to be trucked off approx 36,000 cm, Includes Haul off Site Costs of $268k, Includes PVC Membrane Costs of $665k, Includes Struct Fill Cost of $560m	$3.24M
1b	Structural Pad under Building with 2 m Deep Structural Fill.	Thickened Edge SOG, PVC Membrane under 1m Structural Fill, Allowed for 6m between Header House & GreenHouse to be structural fill as well. If City Site, large amount of fill to be trucked off approx 86,000 cm, Includes Haul off Site Costs of $640k, Includes PVC Membrane Costs of $665k, Includes Struct Fill Cost of $1.22m	$4.60M
2a	Concrete Suspended Slab on Pile Foundation	4.1 x 5m Grid, Void Form, 400mm Piles 10m deep, 200mm structural slab	$5.32M
2b	Concrete Suspended Slab on Pile Foundation	4.1 x 5m Grid, Void Form, 600mm Piles 10m deep, Staggered Pattern, 250mm structural slab	$5.95M
3	Struct. Steel Floor Over Crawlspace	Staggered Pattern Grid, 115mm R/C Topping on Deck, 600 mm Piles 10m long, no slab in crawlspace	$5.53M
4	Struct. Steel Floor Over Crawlspace	8.2 x 6.67m Grid, 115mm R/C Topping on Deck, 600 mm Piles 12m long, no slab in crawl space	$7.54M
5	Struct. Steel & Precast Slab Over Crawlspace	8.2 x 6.67m Grid, 90mm R/C Topping, 600 mm Piles 12m long, Reinf Thru, Precast Cores. No slab in crawl space.	$7.29M
6	Precast Slab on Helical Piles	8.2 x 6.67m Grid, 50mm R/C Topping on Precast, 250mm dia Grouted Helical Piles, 10m Deep, no slab in crawl space	$7.80M

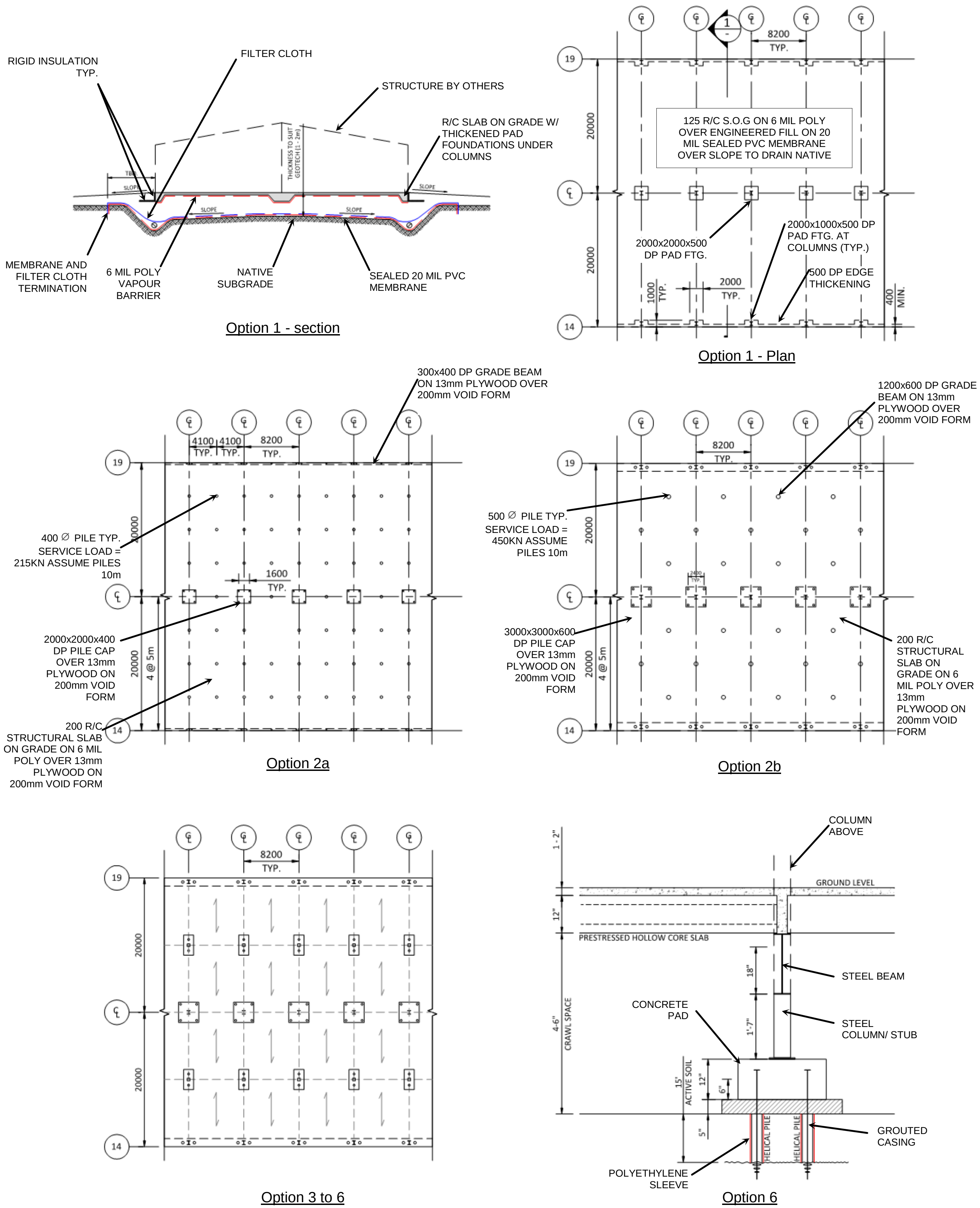


Figure 2. Foundation options for Light building

structural backfill.
The option 2 includes suspended concrete slab supported on piles. The option 2a is using 400 mm driven pile on a gridline of 4.1x5.0 m. The option 2b is using 600 cast-in-place piles in 4.1x5.0 gridline but staggered.
In option 3 to 6, alternative construction methods for suspended slab are considered. These include steel framing with deck and concrete topping and precast concrete slabs.
The cost estimate of the six alternative solutions reveals that the soil improvement technique is the most economical solution for light weight structures.

4 COMPARISON OF FOUNDATION SOLUTIONS FOR LIGHT AND HEAVY BUILDINGS

All the three approaches in resolving the expansive soil problems have been employed in this two case studies. Adding to the rigidity of foundation system is employed in light building case option 1 foundation system. Isolating the structure from expansive soils was employed in all the options of heavy weight structure and option 2 to 6 of light weight structure. Some kind of soil improvement also is employed in option 1 of light weight structure.

It is concluded that for a light weight structure the most economical solutions come from adding rigidity to the foundation system and soil improvement techniques, while for a heavier structure the best approach is to isolate the structure from the effect of expansive soil.

ACKNOWLEDGEMENTS

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