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Design and Construction of a Railway Yard, Embankment and Foundations under Difficult Groundwater Conditions

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SYNOPSIS This paper describes four design and construction cases that were identified, investigated and solved at the site of a major industrial project located in Central Alberta, Canada. These consisted of cases where: (i) high artesian water pressures were encountered in a stratum that was planned to be excavated during cut and fill operations for a railway yard area, (ii) an embankment was to be built over an area that became waterlogged due to over excavation, (iii) a year round drainage system was to be provided below a railway embankment where an icing (pingo shaped) problem was encountered in winter, and (iv) bored, belled, cast-in-place concrete pile design had to be revised due to the existence of artesian pressures in layered and fractured bedrock stratum where pile bases were to be founded.

SITE CONDITIONS

The subsurface information at this site was obtained by carrying out investigations in four phases. The first phase consisted of preliminary geotechnical work and the second phase consisted of hydrogeologic investigations for the railway yard area. The third phase consisted of detailed geotechnical investigations for the railway yard and the plant site. The fourth and the final phase of investigation was necessary because the most economical arrangement for the railway yard was not feasible due to water problems, and a new area had to be investigated.

Findings of these investigations will be summarized in this section. Detailed information on these are available in various reports (EBA, 1981 and 1982 and Hardy 1982).

First Phase of Investigation

The purpose of this work was to define basic soil conditions, their suitability for subgrade support and the use as fill material. A total of 34 boreholes, 10 within the confines of the plant site and 24 in the railway yard area, were drilled and have been shown in Figure 1 as indicated by BH 101 thru 134. Field sampling included standard penetration tests, split-spoon samples, undisturbed shelby tube samples and bulk samples. Laboratory test included the determination of moisture contents, Atterberg limits, grain size analyses, moisture-density relationships, unconfined compressive strengths and soluble sulphate concentrations. A total of sixteen sealed standpipes were also installed in the area.

The generalized subsoil conditions for the plant area consisted of organic topsoil below which were layers of sand and silt over glacial clay till to an average depth of about 7.5m below ground level. This glacial till was of low to medium plasticity with an occasional zone of high plasticity material. The consistency of the till generally ranged from stiff to very stiff. Below this till lay bedrock which primarily consisted of siltstone with interspersed layers of clay shale. In general the bedrock appeared relatively uniform and intact with the exception of borehole BH 102 where the clay shale was highly fractured. Based on unconfined compression test results on a few samples of this rock, it was recommended to use belled piles founded on top of bedrock with an allowable bearing capacity of 452 kN for 500 mm shaft diameter and 1000 mm bell base diameter.

Water levels in the bedrock ranged from 10.9m below ground in hole BH 102 to 14.5m below ground in BH 104 after six days of drilling. These holes were dry on completion of drilling. In the clay till, water levels were about 11m below ground surface. One sealed piezometer installed in a surficial deposit of sand in BH 106, which was a low depression area, recorded a "perched" water level at about 1.0m below grade. Other than this the overall plant site indicated low water levels.

Typical soil conditions for the railway yard area were similar to the one described above for the plant site. This area, however, had a larger extent of surficial sands and silts; clay till was wetter here than the plant area. Thin layers of sand were encountered within the clay till in BH 115 and 119. Water levels measured in standpipes in the North end of this area ranged from about 2m below to about 1.5m below grade; other areas indicated no water in standpipes. Borehole BH 118 displayed artesian conditions which at that time was attributed to the existence of an adjacent wind pump and pond. The apparent high water table at the North end of the site was believed to be a "perched" water table in the sand layers. It was interpreted that the sand became saturated due to

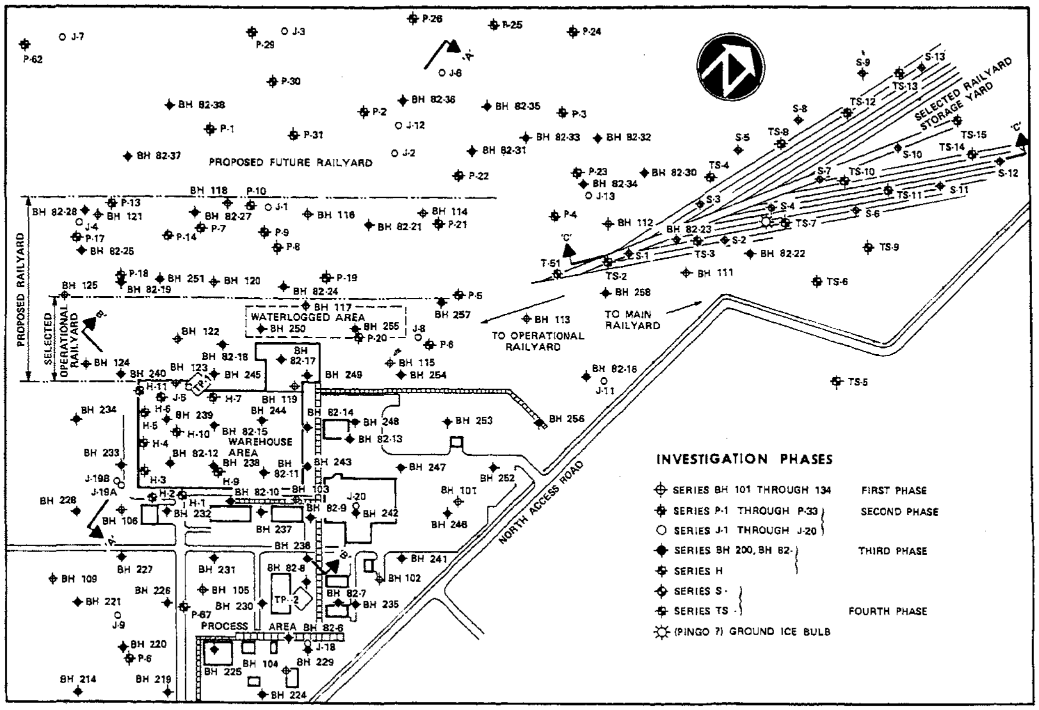

Fig. 1. Borehole Location Plan

surface runoff draining towards the low depression. It was recommended that subsurface drains should be installed in the centre part of the site in order to drain saturated sand lenses existing below the clay subgrade. This would drain "perched" water tables. In general it was concluded that if localized soft and wet areas were drained then normal cut and fill operations could be successfully carried out at this site.

As described in a later section, the cut and fill operation in the railway yard area could not be carried out successfully because of continuous groundwater flow from the apparent "perched" water table area. At this stage a detailed hydrogeological investigation was initiated at this site.

Second Phase of Investigation - Hydrogeological

The purpose of this investigation was to determine hydrogeological stratigraphy in and around railway yard area and then to recommend effective methods of draining the so-called "perched" water table. The recommended work based on the results of this investigation was to be executed without adversely affecting local sources of water for the existing wells in the area or the quality of the surrounding agricultural land. A total of 73 test pits, identified by prefix P, were dug and 20 boreholes, identified by Prefix J, were drilled in the proposed railway yard area. Sealed standpipes were installed by lowering a 40 mm PVC pipe with a slotted tip down an open borehole to the zone of interest. These zones were various surficial deposits and the bedrock. The main purpose of these water level monitoring installations was to observe both horizontal and vertical hydraulic gradients in water-bearing formations, to evaluate changes in water levels in these formations due to a controlled hydraulic disturbance, such as, withdrawal of water from bedrock and observation of any indication of hydraulic continuity between water-bearing formations.

In order to determine hydraulic characteristics (transmissivity and storage coefficients) of the apparent main water-bearing formation and to assess the effects of withdrawing water from this stratum on other water-bearing layers, a constant yield (353m³/day) pumping test was carried out. Results indicated that hydraulic continuity existed between bedrock and the surficial deposits. Pre-test investigations suggested that the water-bearing formation was a multi-layered aquifer possibly consisting of primary permeability sandstone, secondary permeability shale, weathered bedrock and surficial deposits of variable permeability. Piezometric observations in these formations supported this.

This investigation revealed that artesian conditions existed in bedrock over the entire site. The water pressures in the surficial materials in the railyard area were lower than piezometric pressures in the bedrock to the surficial materials. The elevations of the proposed grade surface was approximately 101.6m (108 ft.) on the North side and 100.7m (105 ft.) on the Southeast side. This meant that over a significant portion of the railway yard the cut would extend beneath the piezometric pressure heads both in the bedrock and the surficial materials. It is expected that where the cut penetrated the bedrock surface, considerable flow could be expected out of the cut surface. If this cut penetrated the surficial sands again, considerable flow was expected. Due to high pressures and high permeability of till in the area where this cut extends, trafficability may be a problem. Thus in order to have a safe cut in the area, the three potential problems that were considered were groundwater discharging from bedrock and sand, poor trafficability in high moisture content surficial deposits and unacceptable high uplift pressures.

Third Phase of Investigation - Detailed Geotechnical

In January - March 1982 further site specific geotechnical investigation work was carried out for both the railway yard and the plant site. This work consisted of drilling and digging a total of about 100 boreholes and test pits as shown in Figure 1 by borehole series (BH 200) and(BH 82) and test pit series (H). Results of this work confirmed earlier interpreted soil stratigraphy including the results obtained from hydrogeologic work. Figure 2 exhibits typical sections across the site.

Table 1 provides a summary of the water levels monitored in standpipes and piezometers installed in the formations of interest. Standpipes (#203 and 208) installed in clay till showed that water pressures existed in this material. As expected standpipes in clay till showed only a small change in response to rainfall while water pressures in gravel (#J2b) appear to be responding to rainfall data as supplied by Environment Canada (1982). Fluctuations in water levels in gravel during the period of these records were within an approximate range of ±0.5m. Water levels in bedrock varied from about 1.8m to 11.4m below ground levels. Most of the holes were dry on completion of drilling and water started seeping in with time. Free water was observed in some holes on completion of drilling. In almost all cases, no water was observed in holes when the top metre or so of bedrock was pierced. Test pits, indicated by series H, were dug during the design phase and it was observed that in some cases pits were dry when bedrock was excavated but in other cases water started gushing in as soon as bedrock was cut. This indicated that the bedrock was under pressure and contained water in fractured layers. When these layers were ruptured water filled the hole. Piezometers at different elevations in bedrock confirmed that the bedrock was under artesian pressures.

No definitive response pattern with rainfall was exhibited in bedrock piezometric pressures. Artesian pressures in bedrock at shallow depths remained at the same levels while pressures at deeper elevations were different. This probably was due to fractures and the layered nature of bedrock. Fractures provided continuity for a short distance and therefore pressures did not change unless the points of measurements were far apart.

Fourth Phase of Investigation - Final Geotechnical

The fourth and the final phase of investigation was carried out to determine the sub-soil conditions below the proposed railway yard storage area and the area connecting this storage yard to the operational yard area. A total of 13 boreholes were drilled by a Mobile B-61 drilling rig and 13 test pits were dug by backhoe. Standard field and laboratory tests, as described earlier for previous investigations, were carried out at the site. A total of 13 standpipes and 3 pneumatic piezometers were installed to identify the piezometric pressures in the bedrock and the groundwater table within the overburden soils. Figure 2C presents a section along the site and confirms that generalized soil conditions were similar to other areas that have been described earlier.

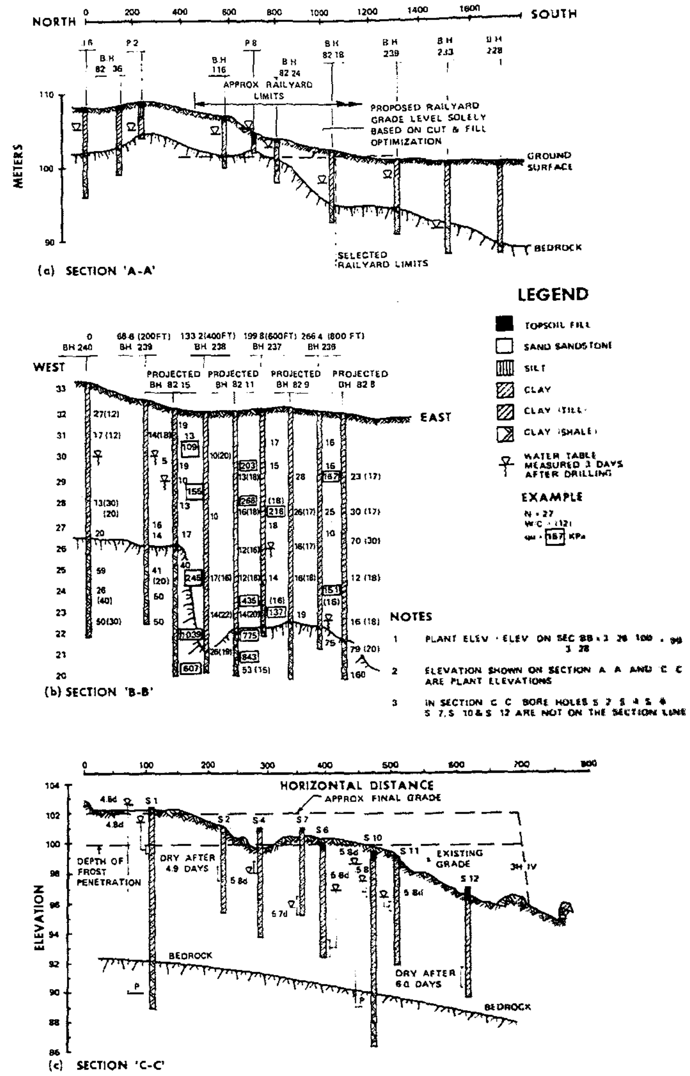

Fig. 2. Soil Stratigraphic Sections

TABLE 1. Water Levels - Below Ground Surface

HOLE NO.	TYPE OF INSTRUMENT	TYPE OF MATERIAL	WATER LEVEL (METERS) (DAYS AFTER DRILLING)		
203	Standpipe	Clay Till	2.4	(180)	(a)
208	Standpipe	Clay Till	3.5	(150)	(a)
222	Standpipe	Bedrock	7.25	(135)	(a)
232	Piezometer	Bedrock	2.4	(135)	(a)
237	Standpipe	Bedrock	6.1	(3)	(a)
239	Standpipe	Bedrock	2.5	(3)	(b)
82-8	Piezometer	Bedrock	9.4	(16)	(a)
82-9	Standpipe	Bedrock	---		(a)
82-15	Standpipe	Bedrock	3.1	(0)	
82-15	Piezometer	Bedrock	2.5	(4)	
J-2a	Piezometer	Bedrock	1.8	(0)	(c)
J-2b	Piezometer	Surface Gravel	2.9	(0)	(d)
J-3a	Piezometer	Bedrock	4.0	(0)	(e)
J-3b	Piezometer	Bedrock	4.5	(0)	(e)
J-14a	Piezometer	Bedrock	11.4	(0)	(c)
J-14b	Piezometer	Bedrock	5.6	(0)	(c)
J-14c	Piezometer	Bedrock	5.2	(0)	(c)

(a) Dry on completion. (b) W.L. @ 3m below on drilling. (c) Almost steady for 9 months. (d) Fluctuates - related to rainfall. (e) Steadily increasing with time for 9 month monitoring.

However, this area in general was a groundwater discharge area which was confirmed by ponded water approximately 200 meters west of borehole S-3. Pneumatic piezometers installed in the bedrock in boreholes S-1 and S-10 indicated higher piezometric pressures in bedrock than overlaying soil indicating an upward seepage condition. An area around and south of borehole S-4 had a high groundwater table and the rest of the site had no such water conditions in surficial deposits.

In the following sections four problem cases and their solutions will be discussed. The geotechnical and hydrogeologic conditions described above will be referred where applicable.

CASE 1: WATER PROBLEM ENCOUNTERED DURING SITE PREPARATION - CUT AND FILL OPERATION

Results of the first phase of geotechnical investigation indicated that cut and fill operations for the proposed railway yard area could be carried out if local depressions were drained and localized soft spots were removed. This work was to be done by establishing a final grade elevation of 101.6m (108 ft.) across the railway yard area. With this background the cut and fill operation in the railway yard area was commenced in Fall 1981 which was scheduled to be completed before the freezing weather. During excavation operations extensive groundwater was encountered in the area that could not be stopped. The reasons for this continuous groundwater flow were not obvious from the sub-surface information available then. Hydrogeological investigations for the area were therefore undertaken, results of which have been summarized earlier. These investigations revealed that the bedrock was under high artesian pressures and, in certain locations of railway yard area, the bedrock was at shallow depths below ground surface. Figure 2a shows the soil, bedrock and water profiles. This indicated that if excavation, as planned to elevation 101.6m, was carried out in railway yard area then it would cut the bedrock which had artesian pressures. This would result in uncontrollable water flow. In order to proceed with the construction of the railway yard site preparation work some form of mitigative action was therefore required to deal with this hydrogeological condition. The following six design alternatives were studied before a final solution was selected.

Alternative 1: Move the entire railway yard area to some other location where hydrogeological conditions were more acceptable. This alternative would not be attractive at this late stage of design because fresh land negotiations and other permit requirements would delay the project.

Alternative 2: Rearrange the railway yard configuration by avoiding the area where bedrock is at shallow depths and has high piezometric pressures. This meant the railway yard would be relocated along the eastern boundary of the property and extend northwards beyond the existing railway yard limits.

Alternative 3: Raise the elevation of the entire railway yard subgrade to 104.6m (118 ft.) from the previously planned subgrade elevation of 101.6m (108 ft.). This would avoid cutting into high pressured bedrock surface and would also provide enough surcharge to counteract piezometric pressures in the bedrock. Local trafficability problems in soft areas due to water could be solved by providing proper drainage and gravel blanket over geotextile fabric.

Alternative 4: In order to reduce the cost of raising the whole site to elevation 104.6m, as described above, it was proposed to have a finished grade elevation of 102.5m (111 ft.) instead of 101.6m (108 ft.). This was a compromise between 104.6m and 101.6m elevations. This alternative would, however, require an elaborate drainage and sealing arrangement such as: temporary dewatering with shallow wells, provision of about 3.5m (10 ft.) thick compacted clay seal above bedrock to minimize seepage, and then providing geotextile fabric and a gravel blanket over clay seal. A network of 200 mm diameter perforated pipes would also be required at the base of gravel to provide drainage. Sub-ballast and ballast

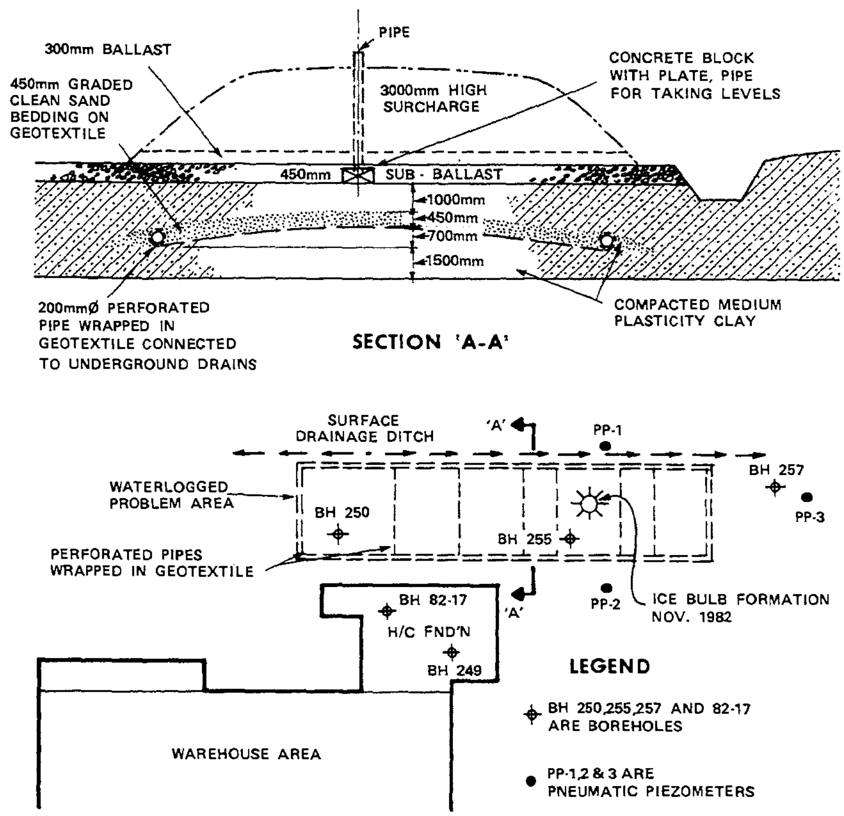

Fig. 3. Water Problem Area and Solution

would then be placed over this gravel layer. Temporary dewatering could then be ceased.

Alternative 5: Install a permanent well point dewatering system so as to lower water pressures in the bedrock. A pumping test carried out earlier had indicated that this was a feasible solution.

Alternative 6: Arrange the railway yard elevation such that it has two terraces: one with a subgrade elevation of 104.6m in the northwest area where piezometric problems in high bedrock elevations was a problem and the second terrace with subgrade elevation of 102.5m near plant boundary where bedrock was deeper. The idea is to have sufficient surcharge to counteract piezometric pressures at the same time minimize cut and fill operations.

Alternative 1 was not acceptable because of construction schedule; Alternative 5 needed further detailed environmental impact and regional groundwater studies and Alternatives 3 and 4 were expensive solutions, and their long-term trouble-free operational feasibility was doubtful. Alternative 6 appeared to be the most cost effective solution if only cut and fill costs were considered. This, however, required that more powerful locomotives would be required to work in a terraced railway yard.

Finally Alternative 2 was adopted. This utilized the lower elevation (102.5m) terrace of Alternative 6 as an operational yard area and extended the railway yard northwards for this purpose. Selected operational yard and storage yard areas are shown in Figure 1.

CASE 2: EMBANKMENT CONSTRUCTION OVER WATER LOGGED AREA

In Summer 1982, during site preparation work for the railway yard area attempts were made to remove soft surficial materials so that the railway embankment could be placed on a firm base. This planning was successful in most of the area except in an area covering about 150m x 30m in the railway operational yard area (Figure 1). Here the bedrock was about 3.0m below grade and the soft-wet surficial materials were sands, silts and clays. In the process of removing soft surficial material, bedrock had been exposed which resulted in water flow and made the area very soft and waterlogged. Attempts were made to drain and seal off this area by backfilling it with compacted clay till. Since it was not possible to place clay in 0.15m compacted layers backfilling consisted of placing about 0.6m (2 ft.) thick clay layers and then compacting it to 95 percent Standard Proctor maximum dry density.

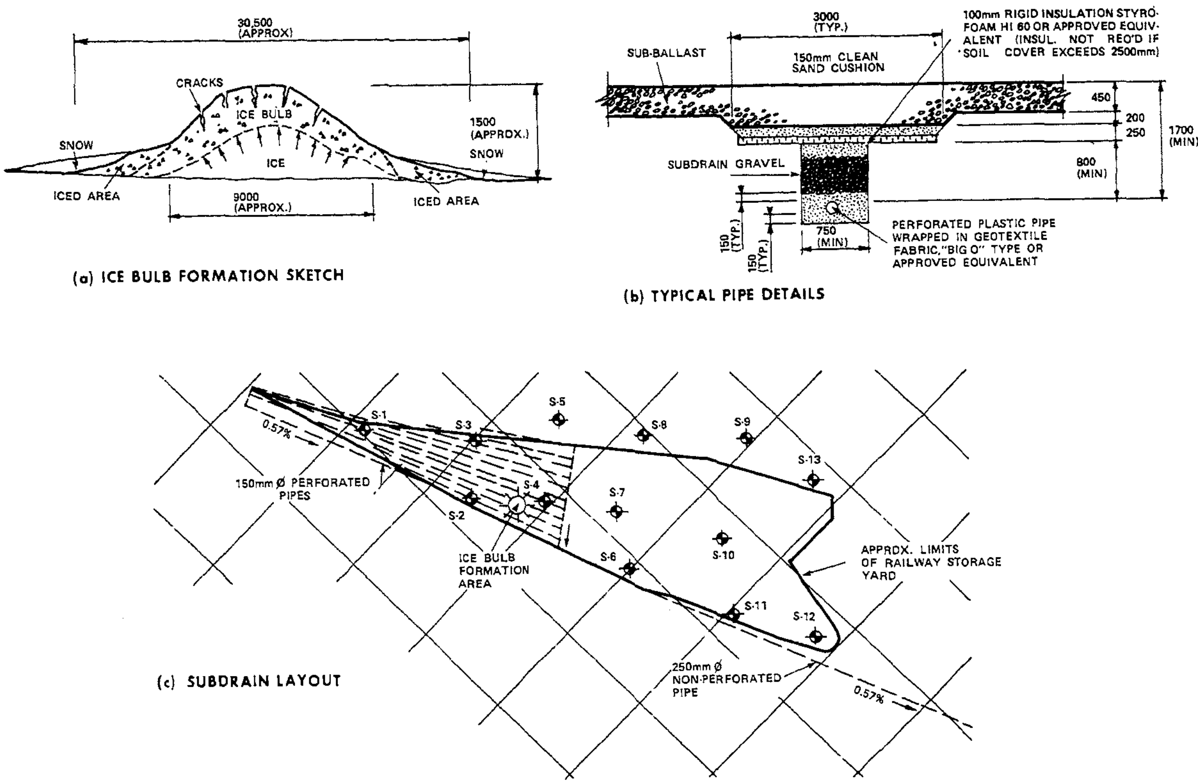

Fig. 4. Ice Bulb Problem-Solution :Railway Storage Yard

This meant that only in the top 0.15m layer the desired compaction could be achieved. The result was that the remaining 0.45m of backfill was placed in loose form. It was, therefore, observed that after a few days (4 to 5 days) water made its way up through the backfill which again made the area extremely soft and wet. This operation was repeated three times and on each occasion similar situations arose within a few days.

The solution to the above problem consisted of backfilling above bedrock to about 1.5m height and compacting it to 95 percent Standard Proctor dry density in 0.6m thick layers as was done earlier. Woven, monofilament, NILOS 70/05 geotextile reinforcement was placed above this backfilled material. This geotextile reinforcement was then connected to perforated pipes that had geotextile wrapping. A 450 mm thick compacted graded sand blanket was placed above this area to enhance drainage. This area was preloaded and monitored for six months before final grade wasprepared. Three piezometers PP-1, 2 and 3, each with two tips, one in bedrock and another in clay till, were installed in the area. Their readings over six months indicated that water levels both in bedrock and clay till were about 1.0m below ground surface. This system is shown in Figure 3. Ballast and rails were then finally placed over the top. The system has since been operating satisfactorily.

CASE 3: ICE BULD FORMATION IN AN AREA WHERE AN EMBANKMENT WAS TO BE BUILT

Growth of an ice bulb, during the Winter 1981 - 82, was noticed near the N-W corner of the railway yard area as shown in Figures 1 and 4. This ice bulb (Pingo-shaped) was kept under observation for a few weeks and its growth in size was regularly reported. The total area encompassed by this bulb was about 30m in diameter as shown in Figure 4. The convex part of this bulb exhibited extensive cracks indicating upward fluid pressure. When this ice bulb was cut by a bulldozer it discharged a significant amount of water flow even though the air temperature was near -30°C. Formation of this ice bulb became a matter of concern since the area lay below the proposed rail line approach track. No information on soil and ground water conditions was available for this area at the time of this observation. The fourth phase of geotechnical investigations, as detailed earlier, was therefore carried out for this area. In general the soil in the area is clay till above bedrock. Local sand layers were observed within the till. Results of these investigations also indicated that groundwater was high in standpipes placed in holes S1 and S3. Also the bedrock piezometric pressures in S1 were about 0.3m above grade level. The overall area was a groundwater discharge area. The existence of ponded water in a low area surrounding boreholes S2, S3 and S4 was recorded. It is possible that a sand layer or a pocket or more pervious zones in till was being fed by upward flow due to its continuity with bedrock which probably was feeding the surface water. Whatever the source of water, the high groundwater obstruction to this flow due to surface freezing of the area appeared to be the reason for the growth of this ice bulb. The obvious solution would be to provide a year around drainage for this water. A few alternative solutions to achieve this goal were proposed.

The adopted solution consisted of a series of trenches located about 10m apart. In these trenches were placed perforated plastic pipes wrapped with geotextile, which were surrounded by clean sand for effective drainage. Where these pipes were at depths shallower than 2.5m, rigid Styrofoam HI-60 insulation was placed above them to inhibit freezing. This method has been effectively used on many projects as a frost protection technique (Dow Chem. 1980, and Sharma 1981). These drains were then connected to a collector drain which finally discharged into an existing drain. In order to have free flow these pipes were placed at a minimum slope of 0.5 percent. At the outlet end of the discharge pipe, provisions were made to keep it open throughout the year. Arrangements such as a heated and insulated sump pump system near the outlet, heat tracing of the last 20m metal section of pipe at discharge point, and blowing hot air at outlet, if required, were also provided. Construction of this system was finished about one year prior to completing the final levels of the railway storage yard. This ensured that the groundwater table had effectively been lowered and would be kept low year round.

CASE 4: DESIGN REVISION FOR CAST-IN-PLACE BORED AND BELLED PILES

The fourth case related to revising the pile design of an earlier established design criteria. As mentioned in the "First Phase of Investigation" it was recommended to use end bearing belled piles founded on top of bedrock with an allowable bearing capacity of 452 kN for a 500 mm shaft diameter and 1000 mm bell base diameter. Detailed geotechnical investigations, however, indicated that a part of this site may have weaker foundation conditions evidenced by shallower bedrock depths and higher piezometric pressures. Typical soil and water conditions across the plant site are shown in Figure 2b. Water level details across the entire site are summarized in Table 1. This information indicated that shallower bedrock was at about 6.5m below the ground and piezometric levels were about 4m above bedrock. On the other hand, where bedrock was at about 10m below ground, the water in the bedrock had a head of about 0.5m. In view of such variations in soil and water conditions across the site, it was decided to carry out two sets of pile load tests each with one axial compression, pull out and lateral load tests in two representative areas. One area represented the worst soil conditions (called area TP-1) while the second area represented the best soil conditions (called area TP-2) and are shown in Figure 1. Details of all these tests are provided elsewhere (Sharma 1982). Only axial compression test results will be summarized here and discussions will be limited only to axial downward pile capacity. Load test results indicated that allowable axial compression values for area TP-1 near warehouse structure was 250 kN and for area TP-2 near Process Area was 570 kN. When compared with the earlier estimated end bearing capacity of 452 kN it was clear that the allowable bearing capacity value from pile load tests was significantly lower in the area where bedrock was at shallower depths and had higher piezometric pressures. Load test values for the second area were closer to the estimated capacity.

Because the reasons for such substantially lower pile capacities based on pile load test were not obvious, it was decided to excavate the tested pile and visually examine its shaft and bell. The excavation was carried out by a backhoe and it was observed that the pile base and shaft were structurally intact. Seepage was, however, noted around the bell especially around the area surrounding the neck of the bell and the shaft. A gap was also observed between pile and surrounding soil above the bell. The clay appeared to be soft for a length of about 1.5m along the shaft above bell. Possible reasons for a lower pile capacity in Warehouse Area than originally estimated based on average strength for the site may be attributed to: (i) About 50 mm of standing water was observed at pile base when concrete was poured. Also about one hour delay was experienced in pouring concrete into the drilled hole. This may have resulted in softening the bell base. (ii) Water pressure information on the nearest hole #82-15 indicated that at completion of the hole, the water level in the hole was about 2.9m above bedrock and 3.5m above bedrock after four days. This hydraulic uplift would encourage rebound and swelling of rock surface. Representative swelled rock samples should therefore be tested to determine undrained strength values for bearing capacity evaluation of this rock. (iii) The possibility of the roof of the bell collapsing would have a potential for causing the movement of soil around the neck. This may have resulted in creating a gap between pile and the soil. Actually when the soil around the pile in this area was removed by hand excavation the accumulated water in this region flowed out of the gap.

After this test and above observations, additional boreholes, series numbers (82 -), were drilled and weathered bedrock samples under water were taken for unconfined compressive strength tests. Revised calculated and actual load test values for both areas are summarized in Table II.

TABLE II Summary of Allowable Axial Compression Loads Calculated and Load Test (Revised Bearing Values)

	AREA TP-1 WAREHOUSE AREA TEST PILE	AREA TP-2 PROCESS AREA TEST PILE	EARLIER ESTIMATED VALUE FOR THE SITE
Calculated Value Qa, kN, (FS = 3)	280	500	452
Load Test Value Qa, kN, (FS = 2.5)	250	570	

Revised pile bearing capacities were assigned to each area in Table II based on the evaluation made as discussed above. Pile installation problems encountered and solutions provided are described elsewhere (Sharma et al, 1983). A total of about 1500 bored, belled, cast-in-place piles were successfully installed and loaded at the site during 1982 and 1983.

CONCLUSIONS

1. In the development of a site investigation program, all surface features and anomalies should be considered and analyzed before a field drilling work is executed. In this particular instance the existence of a wind mill well and permanently waterlogged areas on the high ground enabled recognition of a potential bedrock artesian condition prevailing at the site. In selecting a final solution where bedrock was under artesian conditions, factors such as, geotechnical, hydrogeological, environmental, maintenance and capital and operational costs were evaluated.

2. A system of clay backfill, geotextile, pervious soil, perforated pipes and clay cap can effectively be used to seal the source of water and dissipate excess pressures.

3. Formation of ice bulbs in an area of high ground water can be controlled by providing year around drainage system. Also properly designed and installed extruded polystyrene insulation can effectively be used to inhibit ground freezing.

4. In an area of high piezometric pressures, shear strengths of representative swelled soft weathered bedrock samples should be used to estimate bearing capacity of bored cast-in-place piles founded on such stratum. Calculated pile capacities based on using such strength parameters agreed reasonably well with pile load test results.

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