Structural Improvement of Alluvial Soils

Amélioration de la Structure de Sols Alluvionnaires

G.A. QURESHI Soils Engineer, Harza Overseas Engineering Company, Chicago, USA
J-J. PARE Department Head, Geology and Soil Mechanics, James Bay Energy Corporation, Montreal, Canada
C. PELCHAT Project Manager, LG-2 Project, James Bay Energy Corporation, Montreal, Canada

SYNOPSIS During the construction of a panel wall cut-off, excessive losses of trench stabilizing fluid were experienced. This paper describes structural improvement of these soils, by injections, leading to successful completion of the cut-off wall.

INTRODUCTION

This paper describes the treatment of the thick alluvial foundation during the construction of the panel cut-off wall for the Dike D-20 which has a maximum height of 50 m and incorporates 8.2 million cu. m. of fill. This sand and gravel dike with till core is an important structure of the LG-2 site, a 5328 MW hydro installation on La Grande River, at the James Bay Hydroelectric development in Northern Quebec, Canada.

In the deepest part, the pervious foundation is provided with a panel cut-off (Arbour et al. - 1979). Individual panels are 0.6 meters thick, and from 6.5 to 7.5 meters long. Continuous wall is made by interlocking cast in place panels. The maximum depth reached as much as 70 meters. Fluid stabilized trenching was carried out to assure excavation of the alluviam to bedrock. The wall was extended 0.6 meters in to a bedrock key (Seemel et al. - 1978).

Geologically, such foundation soils can be termed a fluvio-glacial deposit of clean sand and gravel with cobbles and boulders. Some zones consist mainly of pebbles, cobbles and boulders.

THE PROBLEM

During construction of the panel wall with a Kelly bar or CIS (Circulation Inverse - Soletanche) machine, zones of openwork materials were encountered at depth. The apparent lack of sand and fines made trenching with betonite slurry exceedingly difficult. Sudden slurry losses became so large that the partially dug slots for Panels 22, 25 and 28 (Fig. 1) had to be abandoned and backfilled. The construction plant had a capacity to produce about 100 m^3 of slurry per hour. The fluid losses, which exceeded this capacity, threatened the stability of the trench. The fluid had the following properties:

Mean Density 1.2 g/cm^3 (mud density balance)

Mean viscosity 25 centipoises (direct-indicating viscometer)

40-42 secs. Marsh

Sand content 3.0 to 15.0%

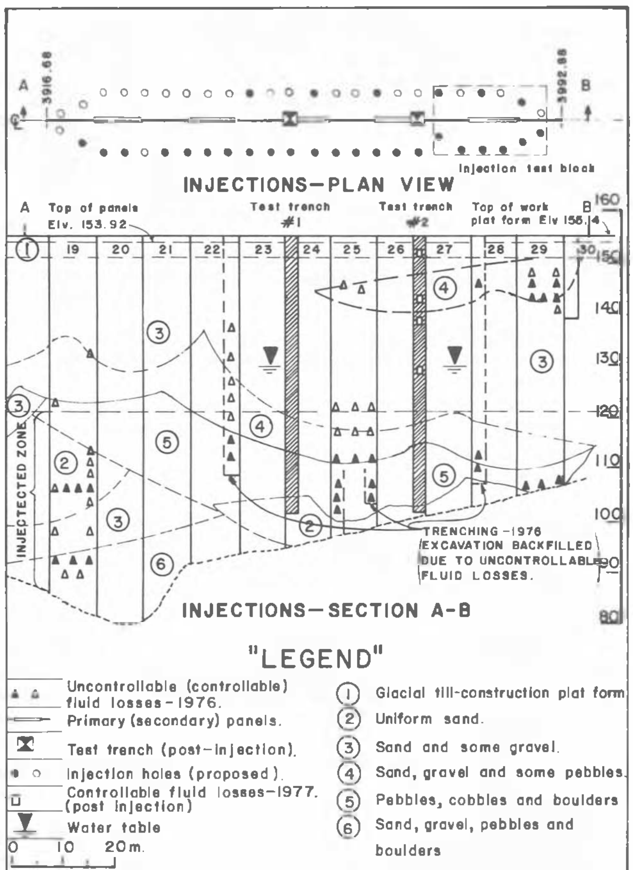

FIGURE — I

Use of local clays, cement and wood shavings added to the slurry to "caulk" the highly pervious strata was found to be ineffective. Even the use of cement, clays and betonite, lowered in sacks with the Kelly and crushed near the open horizons, was of little benefit. However, two panels (Nos. 19 and 29, Fig. 1) were completed by sheer persistence despite very high fluid losses. The rate of trenching was very slow because of the resistance of the thick slurry. Several times trenching had to be suspended due to caving. This procedure of slow trenching through open zones was termed the "Method of Persistence".

It became evident at this point that without somehow improving the foundation soils physically, it would be extremely difficult to complete the cut-off. Moreover, timely completion of this important foundation work lay on the critical path of the scheduled construction of the dike and impending severe winter weather made the problem accute.

BACKGROUND

During geotechnical investigations, a large diameter (100 cm min.) boring was made for a deep dumping well test, using a cable tool machine. Samples recovered from this boring indicated presence of pebbles, cobbles and boulders with some sand. The overall horizontal permeability ranged between 1×10^{-1} to 8×10^{-1} cm/sec. However, locally the permeability could be an order of magnitude higher. Ground water conditions, as observed during construction of already completed portions of the wall, could be considered favourable.

From these observations and a close review of the existing date, it could be concluded that the presence of the open horizons was the main cause of "excessive slurry losses".

ALTERNATE SOLUTIONS

The following alternate solutions were considered:

A) Shorter panels or even interlocking piles in the area of high fluid losses.

Conditions:
It would require additional and/or different machines.

Remarks:
Would increase number of joints.
Scheduled time might be too short.

B) "Method of Persistence".

Conditions:
Would require more machine and materials.

Remarks:
Too expensive and time consuming.

C) Detour in the location of the panel wall.

Conditions:
Extent of horizon in third dimension unknown.

Remarks:
May have required a detour in dike alignment; therefore unacceptable.

D) Improvement of foundation soil properties by grouting the open horizons.

Conditions:
Would require mobilizing drilling and grouting machines.

Remarks:
Would constitute extra work. Could be carried out without affecting project schedule. Improved soil structure could provide desired quality of wall.

E) Grouted cut-off (i) complete grouted cut-off between two farthest complete panels, (ii) partial grouted cut-off in the area of high fluid losses below the completed portion of the panel wall.

Conditions:
Would require mobilizing drilling and grouting equipment.

Remarks:
Any version of grouted cut-off would provide a cut-off of different quality then required. Would affect project schedule. Would constitute extra work.

For any solution to be acceptable, the following conditions had to be considered:

1) Completed cut-off of quality inferior to that specified would be unacceptable.

2) Construction schedule had to be maintained for timely completion of the project.

3) As little extra work should be required as possible.

Solution "D" above was considered to have the best chances of meeting these conditions. Before working the details of solution "D", the following objectives were established:

i) The drilling method should be rapid and able to locate open horizons.

ii) The work should be started before the spring thaw, and should reach substantial and acceptable completion before schedule restart of cut-off wall construction.

iii) Maximum use of on site plant should be made to save some of the mobilization costs.

iv) The hardened grouts should not appreciably rigidify and alter the soils at the panels wall location.

v) The grouts should incorporate inexpensive materials.

vi) The grouts should have a short gel time in order to restrict dispersion.

THE SOLUTION

Solution "D" was retained. It was decided to form an enclosure of grouted soil around the panel alignment, rather than to grout along the actual alignment with a single line of holes. It was considered that a scheme of two grouted curtains, 5 meters apart, upstream and downstream of the centerline of panel wall should be constructed.
Although the actual number of holes to be drilled would be twice as many as at a single centerline location, the physical separation of grouting and trenching locations offered the possibility of simultaneous panel-wall trenching and grouting, if needed (Fig. 1).

As most of the open ground was encountered some 30 meters below the surface, it was decided to grout only from that depth to bedrock. Primary holes were located 6 meters apart, with secondary holes split-spacing the primaries

Of the 48 total holes planned, the southern 12 adjacent to panels Nos. 27 to 29 were designed as a "test block". It was intended that the test block holes should be drilled and grouted first, to assess and optimize the program.

Becker hammer drills were considered best suited to the requirements of speed and ability to sample the soils continuously. Information regarding the density index of the soils penetrated was also available, and grouting could be performed by lowering the grout pipes through the drill stem casting.

Initially the following seven trial mixes as proposed by the Contractor were used in the test block.

CEMENT (Kg)	BENTONITE (Kg)	SILICATE SOD. (Kg)	SAND (Cm^3)	WATER (m^3)	VISCOSITY MARSH SECS.
145.0	22.50	7.25	—	0.75	53
145.0	27.25	7.25	—	0.75	50
145.0	32.00	7.25	—	0.75	55
181.5	32.00	7 25	—	0.75	60
181.5	32.00	7.25	33.0	0.75	46
181.5	32.00	7.25	49.0	0.75	46
218.0	27.25	14.50	—	0.75	47

THEORETICAL CONSIDERATIONS

The above mixes were chosen on the basis of the following: The reaction with free lime derived from cement causes conversion of sodium bentonite into calcium bentonite (Xanthakos 1974). The calcium bentonite is then flocculated by the excess of cations (mainly calcium) in the continuous solution phase. The flocs so formed are still gelatinous and prevent the sedimentation of relatively coarse particles of cement. On the other hand, the addition of sodium silicate to the bentonite suspension leads to the development of a yield point and thixotropy at low solid concentrations. But it is also known that sodium silicate reacts with salts of some elements such as calcium, magnesium, and aluminium (Hernson et al. - 1976).

The lime from cement exchanges base from sodium silicate to form a solution of calcium silicate, with variable setting time (Tallard et al. - 1977).

The final reaction of a cement-bentonite-sodium silicate mix is to produce in relatively short time of one to two hours a pudding like gel which is not affected by water. Further gelefication forms a stiff mortar.

Ideally the most efficient grout would be the one which could be pumped in the shortest possible time. It should have a radius of penetration (r) equal to the horizontal distance between hole being injected and centerline of the panel wall. The gelation time should be equal to the travel time of this distance.

Theoretically:

$$r = 0.620 \sqrt[3]{\frac{QT}{n}} \quad \text{--- (1)} \quad \text{(Herndon et al. - 1976)}$$

r = Radial distance of grout penetration (m)

Q = Average rate of grout take - (m^3/hr)

t = Pumping or gelation time (hours)

n = Porosity of soil, espressed as fraction

Having fixed the grout volume Q and the grout penetration radius r, it is possible, by varying the mix design to determine the most economical grouting time t. The fixed value of safe grouting pressure is taken either from litterature or again by trial or error. It is reasoned, that a sudden change in grouting pressure may be due to hydraulic fracturing (Herndon et al. - 1977).

PROCEDURE

The following procedure for the test block was established:

1) With bedrock elevation being previously known, a 14 cm (OD) pipe was driven to bedrock with the hammer drill.

2) A complete log was made of soils removed during driving and water level was established.

3) A special grouting packer was lowered inside the drive pipe with a 5 cm ID grout pipe. The packer was seated and checked for water tightness.

4) The drive stem was lifted 1,5 meters (half the length of the drive tubing sections).

5) Grouting with the appropriate mix was completed, injecting to "Refusal".

   "Refusal" was calculated as fixed quantity of grout mix which would theoretically fill the voids of a soil sphere of 1,5 meters radius under gravity. If a back pressure of 11.0 kPa/m corresponding to less than half of overburden pressure was registered before fixed quantity injection, grouting was terminated.

   For secondary holes the back pressures build-up was the only refusal criteria.

6) The drive tubing was then lifted to the next higher section and step (5) was repeated.

TEST BLOCK INJECTION

During drilling some sand flows were experienced. This affected the drilling rate and caused temporary stoppages. This problem was overcome by closing the bottom of the drive pipe with a tight fitting conical plug, which was then pushed out by grout pipe and sacrificed before injection. However, no sampling was possible with this method. In order to get information on the nature of the soil, a few holes were sampled. The grout takes in these holes were then used as an indication of open zones. Drilling and grouting of the test block took about 2 weeks, but enough information became available, to permit elimination of 3 holes out of the 12 in the test block.

The following group mixes were established on the basis on the test block injection results:

MIX DESIGNATION	CEMENT (Kg)	BENTONITE (Kg)	SILICATE SOD. (Kg)	SAND (Cm^3)	WATER (m^3)
I	109.0	29.5	—	—	0.75
I-A	145.0	29.5	—	—	0.75
2	145.0	29.5	7.25	—	0.75
3	145.0	29.5	14.50	—	0.75
3-A	145.0	29.5	14.50	49.0	0.75

Mix 1 Starting mix for "P" holes
Mix 1-A Starting mix for "S" holes
Mix 2 Bulk grouting mix for "P" & "S" holes
Mix 3 Cut-off or "refusal" mix for "P" holes
Mix 3-A Cut-off or "refusal" mix for "S" Holes

INSPECTION AND TRENCHING TESTS

The following is a summary of the work completed during actual injections, using the above procedure:

Total number of holes required = 28 (instead of 48)

Total linear meters drilled = 1,499

Total linear meters injected = 688

Days worked on drilling
(two drills - 24 hours per day) = 30

Hours of injection (2 pumps) = 750

Sacks of cement used = 10,958

Sacks of bentonite used = 2,000

Injection equipment consisted of standard cement grouting equipment with the addition of a high speed mixer/digester for bentonite mixing.

Two test panel trenches (Fig. 1) were dug after grouting operation. No fluid loss was experienced in the injected area. Traces of grout were encountered during trenching. Bentonite consumption averaged 58 Kg/m^2 during the trenching and reached 185 to 200 Kg/m^2 in the open zones. The average trenching rate was 2.7 m^2 per hour per machine.

PANEL WALL CONSTRUCTION

The whole injection program was completed in less than 8 weeks. Work on the actual cut-off wall commenced as scheduled and was completed in time (Seemel et al. - 1978). The comparative Kelly bar excavation rates of different soils and bentonite consumption are given below:

	EXCAVATION WITH KELLY BAR					BENTONITE CONSUMPTION			
	SAND AND GRAVEL SQ.M.	DENSE SAND AND GRAVEL SQ. M.	WITH SLURRY LOSSES SQ. M.	SQ M/HR PER MACHINE	TOTAL SURFACE SQ. M.	TOTAL Kgm	TOTAL Kgm/SQ. M.	AREAS WITHOUT SLURRY LOSSES Kgm/SQ. M.	AREAS WITH SLURRY LOSSES Kgm/SQ. M.
1976	3240	420	512	2.0	4432	312767	70.6	46.7	326
AFTER SOIL STRUCTURE IMPROVEMENT (POST INJECTIONS)									
1977	4168	1273	619	2.5	6979	363474	52.1	39.5	185 1
TOTAL	7408	1693	1131	2.3	11411	676241	59.3	42.4	252.3

The completed panel wall has since been subjected to full reservoir head and has displayed no apparent deficiency. The project schedule was maintained and the the project was commissioned in time.

CONCLUSIONS

The injection of inexpensive grouts can be used to improve soil properties.

The effectiveness of such injections permitted normal and timely completion of a deep cut-off panel wall at Dike D-20 at James Bay.

REFERENCES

Arbour R., Paré, J.-J. (1979) Seepage control for the D-20 dyke at La Grande (31-9) Water Power And Dam Construction - 32-36

Herndon, J., Lenahan, T. (1976) Grouting of Soils (1) A State of-the-Art Report 66-68 - 99-119, U.S. Department of Commerce - National Technical Information Service.

Seemel, R.N., Qureshi, G.A. (1978). Deep Cut-offs in Dam Foundations at James Bay, A.S.C.E, Annual Convention, Chicago.

Tallard, G.R., Caron, C. (1977). Chemical Grouts for Soils. Available Materials (1) U.S. Department of Commerce - National Technics Information Service 23-24.

Xanthakos P.P. (1974). Underground Construction in Fluid Trenches 50-54 National Educational Seminar, University of Chicago.

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