

CURRENT TRENDS IN GEOTECHNICAL INVESTIGATION TECHNIQUES

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ABSTRACT – The new emerging trend in modern geotechnical investigation is to place a greater emphasis on in-situ tests. These tests can be effectively used to predict foundation behaviour with a higher factor of reliability. The paper presents four different types of advanced in-situ testing techniques with three case histories to demonstrate effective use of such advanced in-situ tests – static cone penetration tests, pressuremeter tests, geophysical tests and aquifer test.

1 INTRODUCTION

1.1 Geotechnical Investigations – The Need

Geotechnical engineers have a vital role to play in solving some of the world's most pressing problems of space utilization, transportation sector, construction in difficult soil conditions, etc. (Gopal Ranjan, 1996). Geotechnical investigations provide the necessary input for design of foundations, which is of paramount importance for reliable performance of civil engineering structures.

A carefully planned and properly executed investigation can result in a greater degree of confidence in the design and in addition to cost effectiveness. Advance geotechnical testing techniques play an important role in generating high quality data for design and increasing the reliability of the foundation system.

A thorough geotechnical investigation with proper interpretation of data is the basis for safe and stable structures (Sanjay Gupta, 1993). With the current trend for fast track projects, the thrust is on superior, maintenance free performance, stringent design criteria and tight time schedules for completion.

1.2 The Challenge

It is becoming increasingly important to relate engineering solutions not only based upon overall stability, but also on an acceptance / serviceability criterion based upon its anticipated performance.

For adequate performance of such structures, geotechnical investigations play a vital role. There is a greater responsibility on the geotechnical engineer to develop reliable and economic designs involving heavier loads and difficult soil conditions. The need is to predict the behaviour / performance of foundations / structures to higher degree of reliability. This necessitated new trends

in advanced in-situ testing methods to predict the behaviour more rationally and accurately for developing most suitable stable / economical foundation designs.

It is in this context that a carefully planned site investigation is an important pre-requisite for all civil engineering projects. In-situ tests should form an integral part of a modern geotechnical investigation program to enhance the level of reliability of geotechnical prediction. Sufficient field tests backed up by a detailed laboratory testing program on disturbed and undisturbed soil samples are essential to develop soil parameters that reflect the in-situ condition of the substrata.

2 INVESTIGATIONS TECHNIQUES

Developing a geotechnical investigation program that simulates the project requirements is more of an art than an exact science. To select a soil profile and parameters for foundation design requires a lot of ingenuity, tempered by field experience and understanding of soil behaviour.

The most commonly used method of site investigation is to drill boreholes, conduct dynamic cone penetration tests, plate load tests. Standard Penetration Tests (SPT) are conducted at every 1.5 to 3.0 m depth intervals in the boreholes and undisturbed soil samples are collected at every 2 to 3 m depth intervals. The analysis is performed based upon SPT value and laboratory test results.

In situ tests can provide a better insight to soil behavior and should be relied on to a greater extent. Some in-situ tests that can improve the quality of prediction of foundation behavior are discussed below.

2.1 Pressuremeter Tests

This is an advanced state-of-the-art test. A probe with a rubber membrane is lowered into the borehole and expanded under pressure. The pressure-volume relationship is correlated to various engineering properties of the soils. The prediction of soil bearing capacity and settlement from pressuremeter data is more realistic than other available methods. Figure 1 presents a photograph of the control panel and probe of a Menard's pressuremeter.

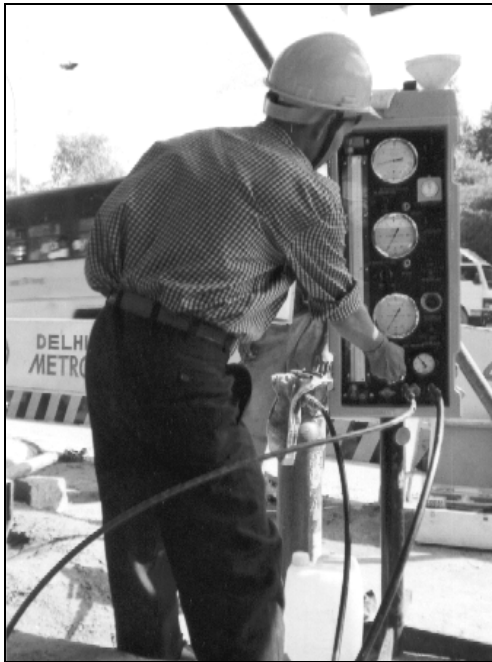


Figure 1 : Pressuremeter Test Setup

Figure 2 presents a flow chart explaining the pressuremeter test set up.

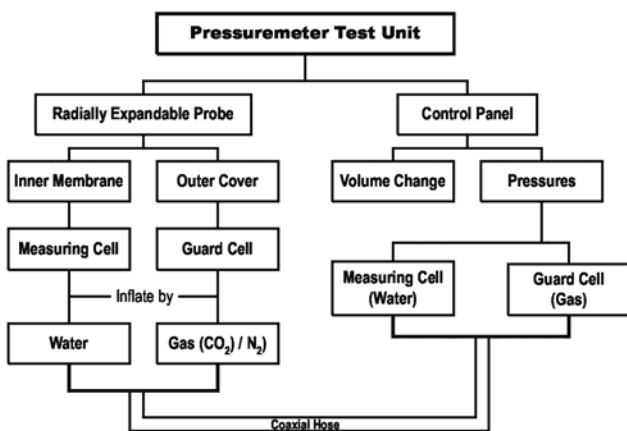


Figure 2 : Pressuremeter Test Schematic

Figure 3 presents typical results of the pressuremeter tests. The field curve and calibration data (air calibration and pipe calibration) are presented together with the corrected pressure versus volume curve.

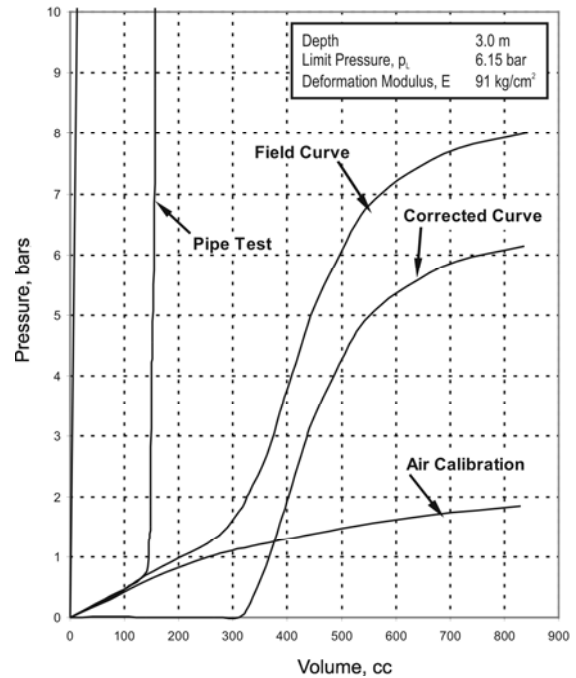


Figure 3 : Typical Results of Pressuremeter Test

The pressuremeter data may be correlated to the various soil properties. Table 1 presents the typical values of limit pressure for different types of strata.

Table 1 : Range of Limit Pressure for Different Soils

Soil Classification	p_L m (kN/m ²)
Clay	0 – 1200
Silt	0 – 700
Firm Clay or Marl	1800 – 4000
Compressible Sand	400 – 800
Compacted Silt	1200 – 3000
Soft on Weathered Rock	1000 – 3000
Sand and Gravel	1000 – 2000
Rock	4000 – 10000
Very Compacted Sand and Gravel	3000 – 6000

The advantages of the pressuremeter tests are:

- In-situ stress-strain behavior of soil and rock can be evaluated

- There is minimum disturbance to in-situ conditions, hence quality of results is superior
- In weathered rocks, where core recovery is poor, pressuremeter test is the only test, which can give realistic data
- Bearing capacity analysis and settlement analysis for shallow foundations and pile capacity analysis using pressuremeter data gives more realistic estimate of actual soil behavior.

The disadvantages of this technique are:

- In sandy strata, where boreholes collapse, it may be difficult to conduct the test
- Test cannot be conducted in bouldary strata
- In fractured rocks, the membranes may get damaged if the membranes get stuck between the fissures.

2.2 Static Cone Penetration Tests

This test gives a continuous record of penetration resistance with depth and is useful to identify presence of soft layers, local variations etc. The cone tip resistance can be correlated to undrained shear strength of clays and density condition of sands. It can provide a better assessment of bearing capacity and settlement, pile capacities etc. Figure 4 presents a photograph of the static cone penetration test in progress.



Figure 4 : Static Cone Penetration Tests

Typical results of static cone tests results are presented on Figure 5. The data includes cone tip resistance, friction resistance on the jacket and friction ratio.

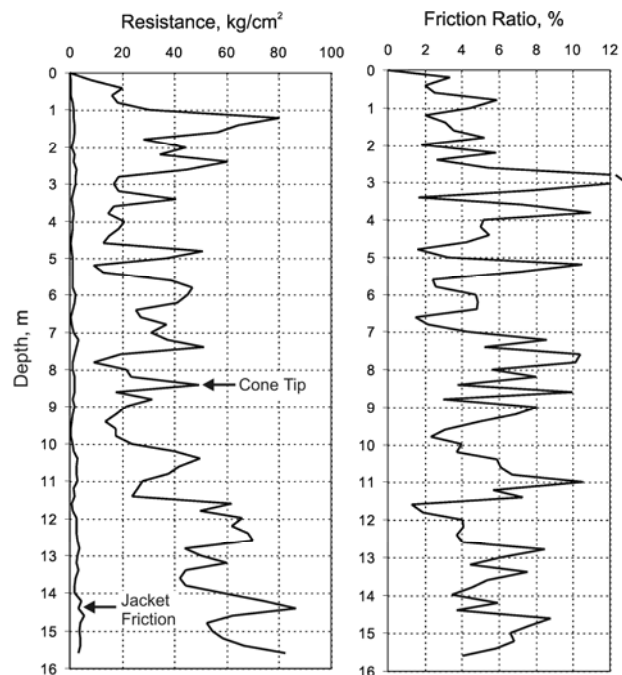


Figure 5 : Typical Results of Static Cone Penetration Tests

Table 2 presents correlation of cone tip resistance with SPT values.

Table 2 : q_c versus N

Density Description	SPT (N) Value	Static Cone Tip Resistance (kg/sq.cm)
Very Loose	0 to 4	< 20
Loose	4 to 10	20 to 40
Medium Dense	10 to 30	40 to 120
Dense	30 to 50	120 to 200
Very Dense	> 50	> 200

The advantages of SCPT are:

- It gives a continuous profile of penetration resistance with depth
- The cone is hydraulically pushed, hence human errors such as problems in borehole cleaning, improper stroke of SPT hammer, etc. are eliminated
- The equipment can be used together with electrical cone / piezo cone with data logger for digitized data collection
- The test can be effectively used for compaction control and quality control of embankment

construction, fill placement and ground improvement checks

- The data can be used directly for geotechnical analysis of bearing capacity and settlement analysis of open foundations, pile foundations, etc.

The disadvantage of the technique is that it is not suitable for boundary strata and shallow rocks since refusal will be met.

2.3 Geophysical Tests

Geophysical tests such as electrical resistivity tests and seismic refraction tests are being increasingly used to supplement the borehole data. These tests can confirm continuity of the various strata, depth of layers, groundwater conditions etc. In strata containing boulders or rock, substantial savings in cost and time can be achieved by judicious inclusion of electrical resistivity tests in the geotechnical investigation program (Ravi Sundaram & Sanjay Gupta, 2001).

For analysis of resistivity tests to assess the layers, the inverse slope method proposed by Sanker Narayan & Ramanujachary (1967) is used. In this method, the data is plotted as a graph of “a” versus “ a/ρ_a ” (a = electrode spacing, ρ_a = apparent resistivity). The plot is analyzed as per the inverse slope method to identify the layers.

The resistivity data is analyzed in conjunction with the borehole data to assess the probable stratigraphy at the required points. Using the resistivity data from the various locations at which the tests are conducted, a three dimensional picture of the stratigraphy is visualized so as to interpolate the soil profile at the required locations. Based on this analysis, a geo-electric litholog that matches with the anticipated stratigraphy is generated.

2.4 Aquifer Pump-out Tests

Aquifer tests (full scale pump-out tests) are in-situ test methods used to determine hydraulic parameters such as drawdown-time relationships, Transmissivity, hydraulic conductivity, well storage coefficient, specific capacity, well efficiency, etc. Hydro-geologic parameters derived from the test, averaged over the spatial zone of influence of the test, are used to design dewatering system and to develop a hydro-geological model.

2.4.1 Step Drawdown Test (SDD Test)

Step drawdown test is used to establish short-term yield-drawdown relationship. Stage pumping is done to approach the estimated maximum yield of the well.

Ground water level measurements are taken in pump well and observation wells close to the pump well at

frequent time intervals. The average discharge for each step is recorded. The results are plotted as step drawdown vs. time for each step and discharge vs. step drawdown (BS 6316).

The data is also plotted as specific drawdown (drawdown/ discharge) vs. discharge at the end of each step. The relationship is used for estimating the well parameters like – (a) formation loss coefficient and (b) well loss coefficient.

The test results are analyzed to estimate the discharge value for steady state/constant discharge test so as to stress aquifer for proper response.

2.4.2 Constant Discharge Test (CD Test)

Constant discharge test provides data on hydraulic characteristics of the aquifer within the radius of influence of the pump well. The pump well is continuously pumped at constant discharge rate so as to ensure the desired depression of water level at steady state. Water level readings are recorded at the pump well and observation wells at regular time intervals till the near steady state/equilibrium is reached. The test is continued over a period of about 72~96 hours depending upon the response.

The test results are plotted as corrected drawdown vs. time on log-log scale for pumpwell and observation wells. Analysis is performed by Theis method and Cooper Jacob’s method, as applicable, to compute various parameters.

2.4.3 Recovery/Recuperation Test

After completion of pumping out test, the pump is shut down for the recovery test. During recuperation, the water level measurements are recorded in the same sequence as that of during pumping stage. The recovery test data is used to compute aquifer parameters based upon Theis’ recovery method.

2.4.4 Concepts of Analysis

The various analyses approaches of unsteady flow and equilibrium methods are applicable for confined aquifer and fully penetrating wells.

The analyses assume uniform, homogeneous soil mass with uniform properties. It is further assumed that the permeability of strata below the well tip is very low, as such data is analyzed considering fully penetrating well. Further, if the bottom of well casing is plugged with sufficient bottom blank casing portion, only radial flow will occur. The vertical flow shall be negligible.

The observed drawdown is corrected for confined aquifer condition as per Jacob using the equation:

$$s_c = s - \frac{s^2}{2b}$$

From the analysis, aquifer parameters like Coefficient of Transmissivity (T), Coefficient of Permeability / Hydraulic Conductivity (K) and Storage Coefficient (S) for CD test and recovery test are computed by Theis method and Cooper-Jacob method (Todd, 1995).

As per Cooper-Jacob analysis, the various expressions are: -

$$T \text{ (m}^2\text{/day)} = \frac{2.3 Q}{4\pi \Delta s}$$

$$K \text{ (m/sec)} = \frac{T}{b}$$

$$S = \frac{2.25 T t_0}{r^2}$$

where	s_c	=	corrected drawdown, m
	s	=	observed drawdown, m
	b	=	aquifer thickness, m
	Q	=	constant discharge, m ³ /day
	Δs	=	change in drawdown, m
	t_0	=	extrapolated time at zero drawdown, days
	r	=	radial distance of observation well from pump well, m

3 CASE STUDY 1: ANANDPUR SAHIB

3.1 Project Details

The Khalsa Heritage Memorial Complex (also called "Ajooba"), commemorating 300 years of the Khalsa Panth is planned to be a grand monumental structure, replete with classical architectural features and traditional Sikh elegance. Planned on a plot of 70 acres area, the project will have about 25000 m² built up area. The various facilities planned include the following:

Complex A	:	includes library and associated facilities
Complex B	:	a pedestrian bridge connecting Complexes A and C
Complex C	:	exhibition halls, theatre and related units
The Nishan-e-Khalsa	:	a tall tower structure

A lake is planned between Complexes A and C. For architectural reasons and to generate the required floor space, the hills will be cut by nearly 10 to 18 m to achieve the required final finished levels.

Complex D, the Nishan-e-Khalsa is planned on another hill on the northern side. Figure 6 is an artist's impression of the project.



Figure 6: The Khalsa Heritage Memorial Complex at Anandpur Sahib Punjab

A layout plan illustrating the field investigation locations is shown in Figure 7.

3.2 Site Conditions

Anandpur Sahib is located on the bank of River Sutlej. The soils deposited in this area are alluvial deposits of this river. The river has changed course over the millennia resulting in erosion and subsequent deposition. This probably resulted in the formation of valley flanked by hills on either side. The alluvium to about 4 to 5 m depth is of recent origin and is underlain by soils of Pleistocene Age.

The site is in the foothills zone of the Himalayas. The soils consist of sand mixed with pebbles / cobbles (varying from 20 to 200 mm size) with intermediate clay / sand layers.

3.3 Scope of the Geotechnical Investigation

Since the founding level at Complexes A and C were planned to be in the range of 10 to 20 m below the existing ground level, it was decided to start the detailed investigation from below the founding level. This meant that a blank borehole was drilled to the founding level and the various tests were started below this level. This included SPT and sample collection in boreholes, the pressuremeter tests as well as the static cone penetration tests.

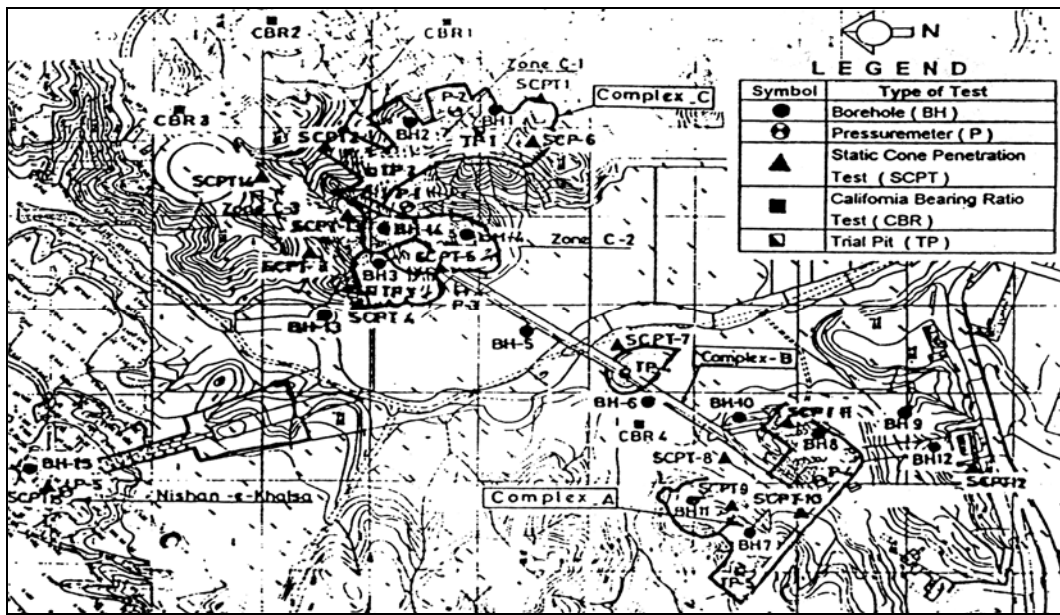


Figure 7 : Layout Plan

On review of the site conditions and the structural loads, the scope of field investigation comprised of drilling of boreholes upto 40 m depth, conducting static cone penetration tests and collection of samples for laboratory testing, performing static cone penetration tests and pressuremeter tests. In addition, field and laboratory CBR tests, Proctor compaction tests etc. were also done for earthwork and compaction control.

3.4 Presentation of Results

Typical borehole data from Complex C is presented on Fig. 8. It can be seen that the soils are primarily granular and contain pebbles / cobbles / gravel with a 1-3 m thick clay layer at about RL 304 m.

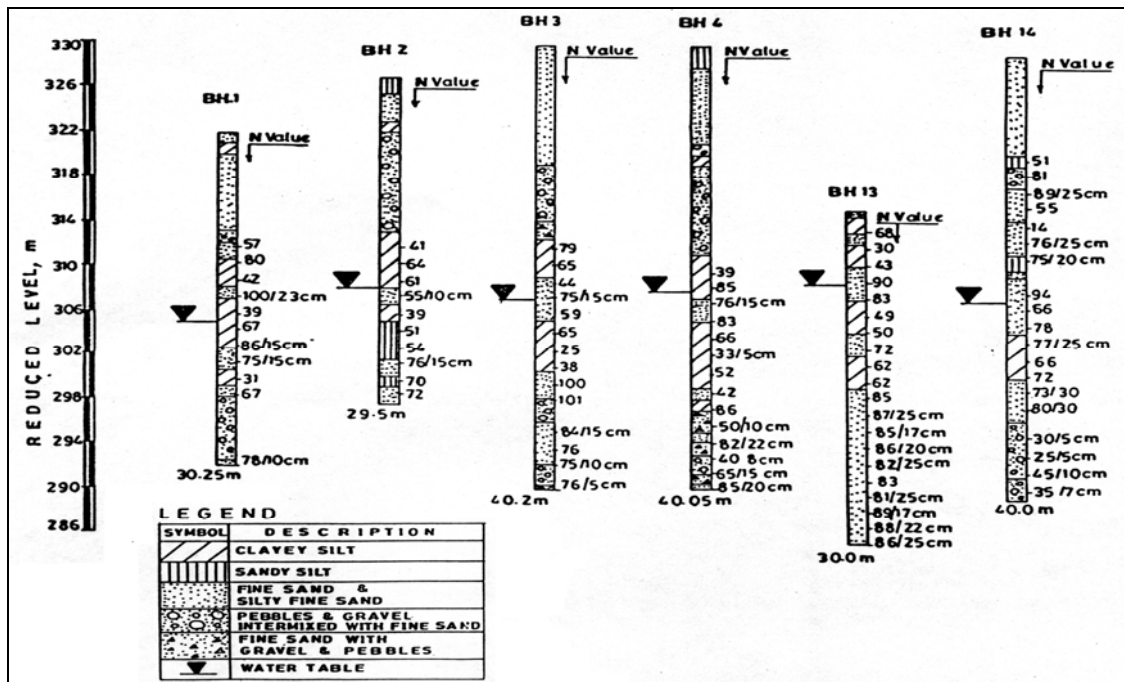


Figure 8 : Borehole Profiles – Complex C

Figure 9 shows the interpreted profile of limit pressure and modulus of elasticity with depth based on the pressuremeter tests conducted at different depths in an NX size borehole.

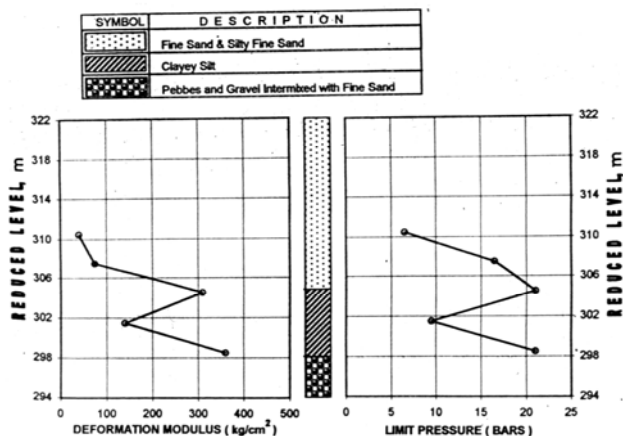


Figure 9 : Profile of Pressuremeter Parameters – Anandpur Sahib

3.5 Analysis of Results and Foundation Scheme Proposed

The settlement analysis for open foundations was done by four alternative methods. These include:

- N - values
- Static cone penetration test results
- Classical theory (elastic + consolidation settlement)
- Pressuremeter test results

Based on a detailed analysis of the results, net bearing pressures for the various facilities of the project were worked out. Due to the high quality of the data, it was possible to justify a safe net bearing pressure as high as 35 to 40 T/m² for the various facilities. The estimated settlement under the design bearing pressure was worked out to be in the range of 15 to 35 mm, which is well within the tolerable limits for adequate performance of the proposed structure.

Results of a typical analysis for a foundation in Complex C by the various approaches is presented below:

Foundation Size	3 m × 8 m
Net Bearing Pressure	30 T/m ²
Computed Settlement	
N Value	33.6 mm
SCPT Data	27.0 mm
Classical Theory	31.3 mm
Pressuremeter	13.6 mm

It was demonstrated that while substantial saving was achieved in costs and time by eliminating the need for piles (as was originally planned in view of the very high column loads), a higher soil bearing capacity could be used in design.

The settlements under the design bearing pressures were then confirmed by plate load tests at founding level. The test was conducted on a 75 × 75 cm size square plate. The evaluation of the test results matches well with the settlement predictions as worked out by static cone penetration tests and pressuremeter test results.

4 CASE STUDY 2: BRIDGE IN NEPAL

4.1 Project Details

Geotechnical investigations for several bridges on the Kohalpur-Mahakali Highway in District Kailali of south-eastern Nepal revealed the presence of layers of boundary strata in the foothills zone of the Himalayas. Boreholes through such strata containing large sized pebbles and boulders were too expensive and time consuming. One case study pertaining to the bridge across river Charaila is presented here. The bridge is 96.64 m long with 3 spans.

4.2 Investigation Program

Six boreholes were planned to be drilled, two at abutment locations, two at pier locations and two for the approaches on either side. However, at the project site, the borehole progress was very slow due to the presence of large sized boulders and cobbles. Chiselling as well as rotary drilling were attempted however, the progress that could be achieved was very slow.

The boreholes were drilled upto the depths of 15 m to 40 m at various locations. Since the drilling process of the borehole resorted to chiseling to break larger size boulders / cobbles to smaller pieces for progressing the boreholes, the stratigraphy could not be ascertained properly.

To evaluate the stratigraphy to the required depth of 30 m at pier locations and 40 m at abutment locations (as per the project specifications), it was decided to conduct electrical resistivity tests. The tests were conducted in accordance with IS 3043-1987 at several locations along the bridge alignment as well as at upstream and downstream locations. The tests were conducted using the Wenner electrode configuration. Eleven resistivity tests were conducted across the river.

4.3 Analysis and Interpretations

The stratigraphy has been interpreted based on the evaluation of the electrical resistivity tests results as well

as visual observations. Typical results from one test at River Charaila are presented on Fig.10.

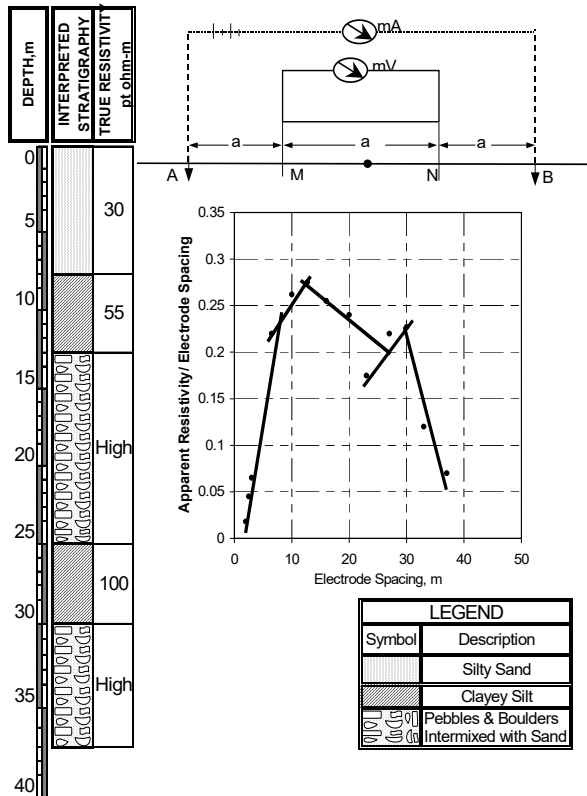


Figure 10 : Interpretation of Layers by Inverse Slope Method

It may please be noted that the interpreted stratigraphy is a geo-electric litholog and is an average over the width investigated. The maximum spacing between the current electrodes was 120 m in this case; thus the interpreted profile is probably an average stratigraphy over the 120 m stretch.

Since resistivity ranges for the different soil types overlap, careful judgment is required to assess the layers met. This was done by comparing the results with the borehole data and resistivity results at nearby locations.

A generalized subsurface profile / geo-electric litholog along the centre line of the bridge over River Charaila is presented on Fig.11. The stratigraphy is shown beyond the depth investigated by the boreholes as obtained by interpolating and projecting the geo-electric profiles along the centre line of the bridge alignment.

Although analysis using resistivity is useful, economical and time saving, it should be realized that the test has its inherent limitations. However, at this site, the

approach was useful since the geology and details of layers present were known from limited borehole data. From the geo-electric litholog, geotechnical characteristics of various layers can be estimated based upon the resistivity values of different types of strata. The litholog also helps in establishing continuity of various formations along the bridge axis. The more realistic geotechnical parameters could thus be selected for the foundation analysis.

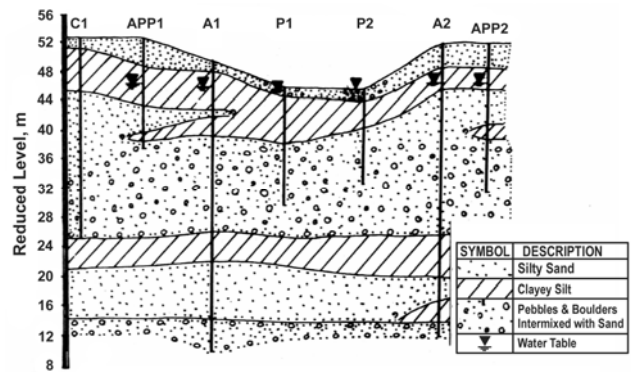


Figure 11 : Interpreted Geo-electric Litholog

5 CASE STUDY-3 : AQUIFER TEST IN DELHI

Deep excavations were planned below groundwater level at some sites in Delhi. This necessitated a thorough hydro-geological study of the area, evaluation of the aquifer parameters for a detailed and realistic design of the dewatering system. In view of the presence of existing buildings in the vicinity, it was necessary to control the drawdown in the zone beyond the excavation limits.

For a geotechnical and hydrological study of the formation, aquifer tests were conducted. The stratification for aquifer tests over Delhi area can be classified under 3 groups viz. – (a) Soil / Alluvium, (b) Rock and (c) soil / alluvium and rock.



Figure 12 : Pumpwell Installation

5.1 Details of Wells

In general, for conducting test in the alluvium, pump well of 400 mm diameter with MS casing of 200 mm diameter and in rock formation, the pump well of diameter 300 mm with MS casing of 150 mm diameter were installed. Figure 12 shows a pumpwell installation in progress. The observation wells were 150 mm in diameter with 100 mm diameter MS casing. The typical sketch of pumpwell in alluvium is presented in Fig.13.

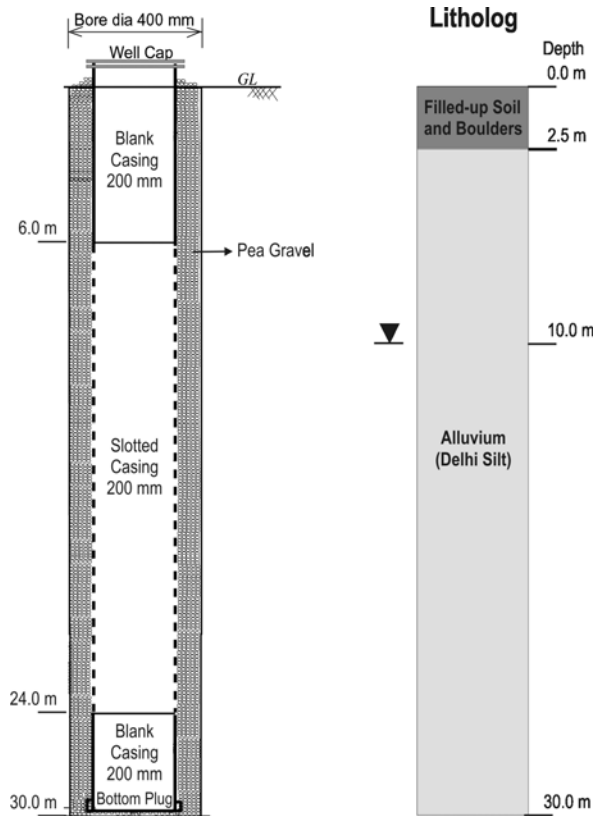


Figure 13 : Pumpwell in Alluvium

In the wells, MS casing as Blank and slotted with pea gravels in annular space are provided. The slotted pipe section is suitably provided as per project requirements and aquifer conditions.

The pump wells and observation wells are developed for free flow of water.

The control valve and flow meter are attached to the delivery line. The water is discharged by hosepipe into a silt trap tank, prior to discharging the same into the storm water drain/discharge point away from the test section so that test section of aquifer does not get recharged by the pumped out water.

In the case of soil-rock formation, where pump-out test is to be carried out in rock formation, i.e. water from rock formation is to be pumped out during pumping stage, the test set up has to be modified to suit the requirements.

The Delhi Quartzite is highly fractured at interface of alluvium and rock. Hence, water inflow through such fractured zones may occur. Due to this, hydrostatic pressure is generated in rock formation. Therefore, the testing procedure was to be modified to simulate the actual conditions during construction.

For this purpose, the alluvium portion in the pumpwell and the rock portion in the observation wells were grouted. A typical section of pumpwell in the soil-rock deposit is presented in Fig.14. The water was pumped out from the pumpwell in the rock mass and the drawdown was observed in the alluvium at the observation wells.

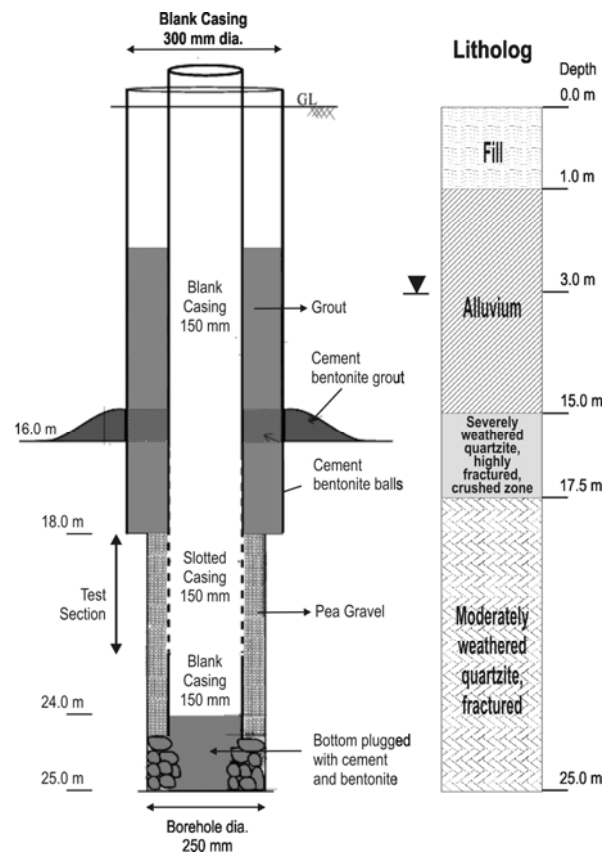


Figure 14 : Pumpwell in Soil-Rock Formation

5.2 Presentation of Results

Typical results of step drawdown tests are presented in Fig.15.

Table 3 : Aquifer Parameters

Strata	Method	Aquifer Parameters			Coefficient of Permeability from Packer permeability test in boreholes, cm/sec.
		T_{av} (m ² /day)	K_{av} (cm/sec)	S_{av}	
Alluvium Strata (Delhi Silt)	Theis	65	3.5×10^{-3}	5×10^{-3}	7.0×10^{-5}
	Cooper-Jacob	71	4.0×10^{-3}	4×10^{-3}	
Rock Strata in Soil- Rock Formation (Delhi Quartzite)	Theis	385	2.0×10^{-2}	8×10^{-3}	1.7×10^{-3}
	Cooper-Jacob	420	2.2×10^{-2}	9×10^{-3}	

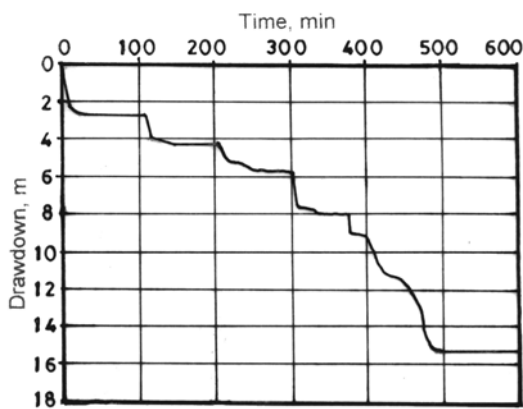


Figure 15 : Time vs. Step Drawdown

The discharge vs. step drawdown as obtained from step drawdown tests are presented in Fig.16. From the results of step drawdown test (Fig.16), constant discharge value is selected for steady state test so as to stress aquifer for proper response.

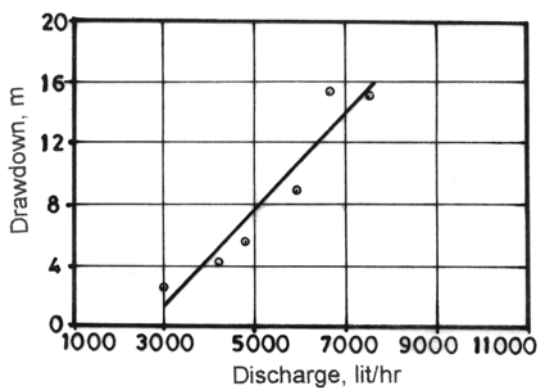


Figure 16 : Discharge vs. Step Drawdown

The typical results of CD test are presented in Fig.17.

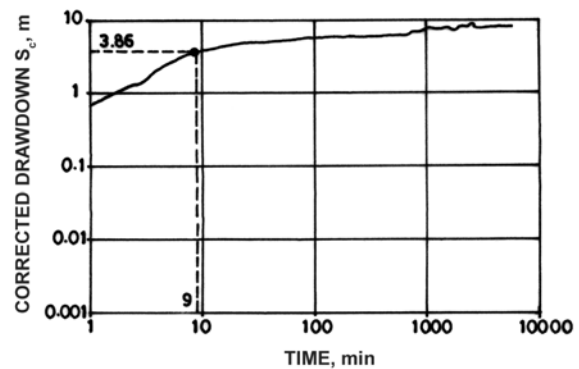


Figure 17 : Typical Result of CD Test

Table 3 presents the typical results for Alluvium and fractured rock formation (Delhi Quartzite) in Alluvium – rock formation with a comparison to coefficient of permeability as obtained from borehole packer test (pump-in-test).

It can be observed that in-situ coefficient of permeability as obtained from pump-out test is about two orders of magnitude more than that computed from borehole packer test (pump in test).

CONCLUDING REMARKS

The projects, today demand stable, safe structures with high degree of performance / serviceability at the minimum cost. Advanced in-situ testing techniques provide more realistic parameters with higher degree of reliability for foundation analysis.

In the new millennium, the thrust is on the use of better equipments and mechanization for higher production and performance. On fast track projects for better infrastructure facilities, of course, one requires superior technologies for effective implementation.

In such a scenario, it is equally important to have engineering parameters also based upon advanced technologies rather than using decades-old methods and analysis. If the input parameters are more realistic/accurate, one can design structures with lower factor of safety, yet with higher factor of reliability in the prediction of its performance.

We geotechnical engineers are now in transition phase, moving away from conventional investigation methods to advanced in-situ testing techniques. We should now change our mind set to understand sub-strata characteristics in terms of cone resistance / limit pressure, etc.

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