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DESIGN OF SHALLOW FOUNDATIONS FOR A LARGE POLYSILICON PLANT IN CHINA

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ABSTRACT

This paper presents the development of a shallow foundation system consisting of mat foundations for support of all structures including heavily loaded, settlement sensitive structures at a recently completed Polysilicon Plant in China. The site grading for the 700 by 700 m (2297 by 2297 ft) site included up to 15 m (49.2 ft) of fills and 18 m (59.1 ft) of cuts. The soil conditions consisted of native residual stiff clays overlying siltstone/mudstone and conglomerate bedrock. Soft soils were present in the canyon bottoms and at the contact between stiff soils and bedrock.

Settlements of native soils and bedrock were measured during site grading when up to 15 m (49.2 ft) of fill was placed to achieve the finished site grade. These measurements provided accurate assessment of compressibility of the native soils and fills. Full-scale test fills measuring 20 by 20 m (65.6 by 65.6 ft) (top dimensions) by 6 m (19.7 ft) high were placed at the finished grade to measure compressibility of both native soils and fill. Where settlements were unacceptable (in deep fill areas and where soft soils were present), surcharge of up to 10 m (32.8 ft) was placed to reduce the compressibility of the soils and post-construction foundation settlements. The entire plant was supported on shallow mat foundations designed for settlement using compressibility data obtained by full-scale settlement monitoring of test fills. The mat foundations traversed variable soil conditions consisting of deep fill, residual soils, and bedrock. Measurements of settlement were made during construction of major structures and tanks and these values compared favorably with predicted settlements.

INTRODUCTION

A large Polysilicon plant was constructed in the City of Xinyu, Jiang Xi Province, Peoples Republic of China. Based on a preliminary geotechnical investigation at the site, local Geotechnical Engineers recommended pile foundations for the support of for all heavy, important, and settlement sensitive structures. This is quite common in China where the engineers generally do not support heavy and settlement-sensitive structures on compacted fill. Since the project had variable subsurface conditions including shallow bedrock, stiff residual soils, and deep 12 m (39.4 ft) canyon fills overlying bedrock under a single long 218 m (715.2 ft) Reactor foundation, it was expected that the foundations for critical structures would be deep pile foundations installed into siltstone/conglomerate bedrock.

The project designers had proposed a very aggressive construction schedule. Time required for the construction of large number of piles made it very difficult to meet the schedule if deep foundations were selected for the support of the plant structures. Furthermore, due to variable depth to bedrock, pile lengths would vary significantly under each structure. Measurements of settlement during fill placement and under test fills indicated that it might be possible to support the structures on shallow foundations without exceeding total and differential settlement requirements. Calculations of settlement for various structures were based on the compressibility parameters obtained from full-scale test fills. Where settlements were unacceptable (in deep fill areas and where soft soils were present), surcharge of up to 10 m (32.8 ft) was placed to reduce the compressibility of the soils and reduce post-construction foundation settlements. Soft soils at the boundary of stiff residual soils and bedrock, where present near the foundation elevation, were removed and replaced with compacted fill or gravel. Although surcharge has been used extensively for improving soft soils and reducing post-construction settlements (Bhushan, 2000, 2004), little data are reported in published literature where surcharge was used to reduce settlements of stiff soils and compacted fills. This case history presents such data.

PROJECT DESCRIPTION

The project consists of development of a new 15,000 tons per year Polysilicon manufacturing facility. The site of the manufacturing facility is located north of the Zhangjing expressway and west of Shuima Road in the City of Xinyu. The area for the plant was about 700 by 700 m (2297 by 2297 ft).

Major structures at the site consisted of three parallel Reactor Buildings, a Product Handling Building at the end of the Reactor Lines, and a Converter building. Each Reactor Line is about 20 m (65.6 ft) wide and 218 m (715.2 ft) long. The Reactor and Product Handling Building at the eastern end is about 30 m (98.4 ft) wide and 130 m (426.5 ft) long. The Reactor Lines had column loads ranging between 5.52 and 1.07 MN (1,240 and 240 kips) and the column spacing ranges between 8.5 and 6.0 m (27.9 and 19.7 ft). The Product Handling Building and the Converter Buildings had column loads of about 3.34 MN (750 kips). Other structures included: two Cooling Towers; 6-level Pipe Racks; a Utility area with three Water Tanks, two Fuel Oil Tanks, a Boiler House, and other buildings; three Compressor Lines, and three TCS Lines supporting each Reactor Line; a Switchyard; a Tank-farm; a Wastewater Treatment area; and other miscellaneous plant structures.

SITE AND SUBSURFACE CONDITIONS

The overall site, footprint of the buildings, and preliminary boring locations are shown in Fig.1. The site topography in the plant area consisted of rolling hills. The original site grades in the area ranged between about El. 70 m (229.7 ft) at the southwest corner and El. 101 m (331.4 ft) in the northern area of the site. Four main canyons traversed the plant area in generally north-south direction as shown in Fig.1. The maximum grade difference at the site was about 31 m (101.7 ft). Existing ponds were present at the end of the canyons into which they drained.

Preliminary pre-grading geotechnical investigation at the site consisted of 15 control borings penetrating to about 10 m (32.8 ft) and 30 standard borings penetrating to about 5 m (16.4 ft) into moderately decomposed bedrock. The preliminary investigation and a few test pits excavated in the valleys indicated a soil profile consisting of:

Layer 1: Residual Soil - Gravel-Sand-Clay Mix

The surface layer of sand-gravel-clay mixture is generally described as medium dense based on blow count data from driven cone. However, due to about 50% fines content, the blow counts may not provide a realistic assessment of the density and compressibility of this layer. The soils were very resistant to probing with a probe penetrating less than 25 mm (1 in.) in the sides of the test pit.

Layer 2: Residual Soil - Hard Red Clay (CL)

This layer is present under the gravelly soils in most areas. Based on pocket penetrometer readings this layer is generally hard in consistency with pocket penetrometer readings between 0.29 MPa (3 tsf) and 0.43+ MPa (4.5+ tsf) at very shallow depth below the surface of the layer. The clays have liquid limits between 30 and 48 and plasticity index between 10 and 20. The liquidity index of the soils ranges between –0.1 and 0.28 with an average of about 0.17 indicating highly overconsolidated very stiff to hard clay. This soil is expected to have low compressibility.

Layer 2A: Slope Wash - Brown to Red Sandy Clay (CL)

This layer is present at the ground surface in the valleys and is soft to medium stiff near the surface and very stiff to hard at depths of about 2 to 3 m (6.6 to 9.8 ft). The near-surface clays appear to be wet. Ruts up to 0.46m (18 in.) in depth were observed under the wheel loads of front-end loaders operating at the site.

Layer 3: Highly Decomposed Rock

This layer consists of zones of hard clay and intact rock. The clays are hard with pocket penetrometer readings of 0.43+ MPa (4.5+ tsf) and rock has typical unconfined compression strength of 1.2 MPa (25 ksf). The rock pieces crumble to gravel size material with moderate effort.

Layer 4: Moderately Weathered Siltstone/Claystone / Conglomerate

This layer consists of alternating layers of siltstone/claystone and conglomerate. This is generally intact rock with unconfined compression strength ranging from 0.96 to 1.7 MPa (20 to 36 ksf) for Claystone/siltstone and 7.2 to 17.2 MPa (150 to 360 ksf) for the conglomerate.

The thickness of the layers is quite variable across the site and many layers are absent in the profile. Groundwater was generally present below elevation El. 76 m (249.3 ft) and was also found at or close to the boundary of bedrock and overlying residual soils. With finished plant grade at El. 83 m (272.3 ft), groundwater was generally not a critical factor in foundation design.

Table1. Summary of Settlement Plate Data

1 ft = 0.305m
1 kcf = 157 kN/m^3

Plate	Date Installed	Plate Elevation (m)	Finish Grade (m)	Estimated Fill Below Plate (m)	Fill Above Plate (m)	Measured Settlement (mm)	Modulus of Subgrade Reaction (kN/m3)	Notes
SP-1	10/05/07	76.5	82.0	1.0	10.9	39	5049	Mouth of Canyon 2
SP-2	10/08/07	72.5	82.0	3.1	10.0	57	3158	Interior of Canyon 1
SP-3	10/12/07	70.0	82.0	3.0	12.1	92	2367	Mouth of Canyon 1
SP-4	10/14/07	74.1	83.0	3.0	9.7	52	3358	Interior of Canyon 4
SP-5	10/18/07	74.4	83.0	3.0	8.4	43	3504	Mouth of Canyon 3
SP-6	10/21/07	69.3	83.5	2.0	13.4	118	2044	Mouth of Canyon 4
SP-1A	11/03/07	82.3	87.4	6.5	5.2	52	1786	Surface Plate Canyon 2
SP-2A	10/22/07	76.2	82.7	6.7	6.5	49	2388	Mid height Plate Canyon 1
SP-3A	10/26/07	75.7	82.0	8.7	6.3	86	1329	Mid height Plate Canyon 1
SP-3B	12/20/08	82.1	82.0	15.1	5.9	74	1442	Surface Plate Next to Plate 3
SP-6A	11/07/07	76.3	83.5	9.0	6.7	87	1378	Mid height Plate Canyon 4
SP-7	11/09/07	83.2	83.0	9.0	4.9	41	2164	Surface Plate Reactor Finger 1
SP-8	11/11/07	83.0	83.0	2.0	5.2	15	6216	Native Soil Reactor
SP-9	11/12/07	83.1	83.0	1.0	5.1	18	5100	Native Soil Reactor
SP-10	11/17/07	83.0	83.0	10.5	6.0	62	1742	Surface Plate Convertor
SP-11	11/17/07	83.0	83.0	11.5	6.0	82	1317	Surface Plate Convertor
SP-12	11/21/07	83.1	83.0	8.0	6.0	32	3375	Surface Plate Reactor Finger 1
SP-13	11/25/07	78.9	83.0	0.0	8.9	18	8900	Bedrock Plate
SP-13A	12/15/07	83.0	83.0	4.0	4.3	44	1771	Surface Plate in Shallow Bedrock
SP-14	11/29/07	82.50	83.0	6	6.0	39	2769	Surcharge Canyon 2
SP-15	11/29/07	82,7	83.0	6	6.0	42	2571	Surcharge Canyon 2
SP-16	11/29/07	82.80	83.0	6	6.0	38	2842	Surcharge Canyon 2
SP-17	12/05/07	82.70	83.0	6	6.0	57	1895	
SP-18	12/05/07	82.90	83.0	10	6.0	43	2512	
SP-19	12/05/07	82.60	83.0	4	6.0	29	3724	
SP-20	12/05/07	82.70	83.0	4	6.0	25	4320	
SP-21	12/11/07	82.90	83.0	4	6.0	56	1929	
SP-22	12/11/07	82.40	83.0	4	6.0	43	2512	
SP-23	01/02/08	83.00	83.0	4	6.0	56	1929	
SP-24	01/03/08	83.00	83.0	3	6.0	33	3273	

Notes:
1. Plates 1-6 were placed at the bottom of the fill in the canyons.
2. Plates 1-A through 6A were mid height plates
3. Plates 7-16 were surface plates in the Reactor / Convertor area

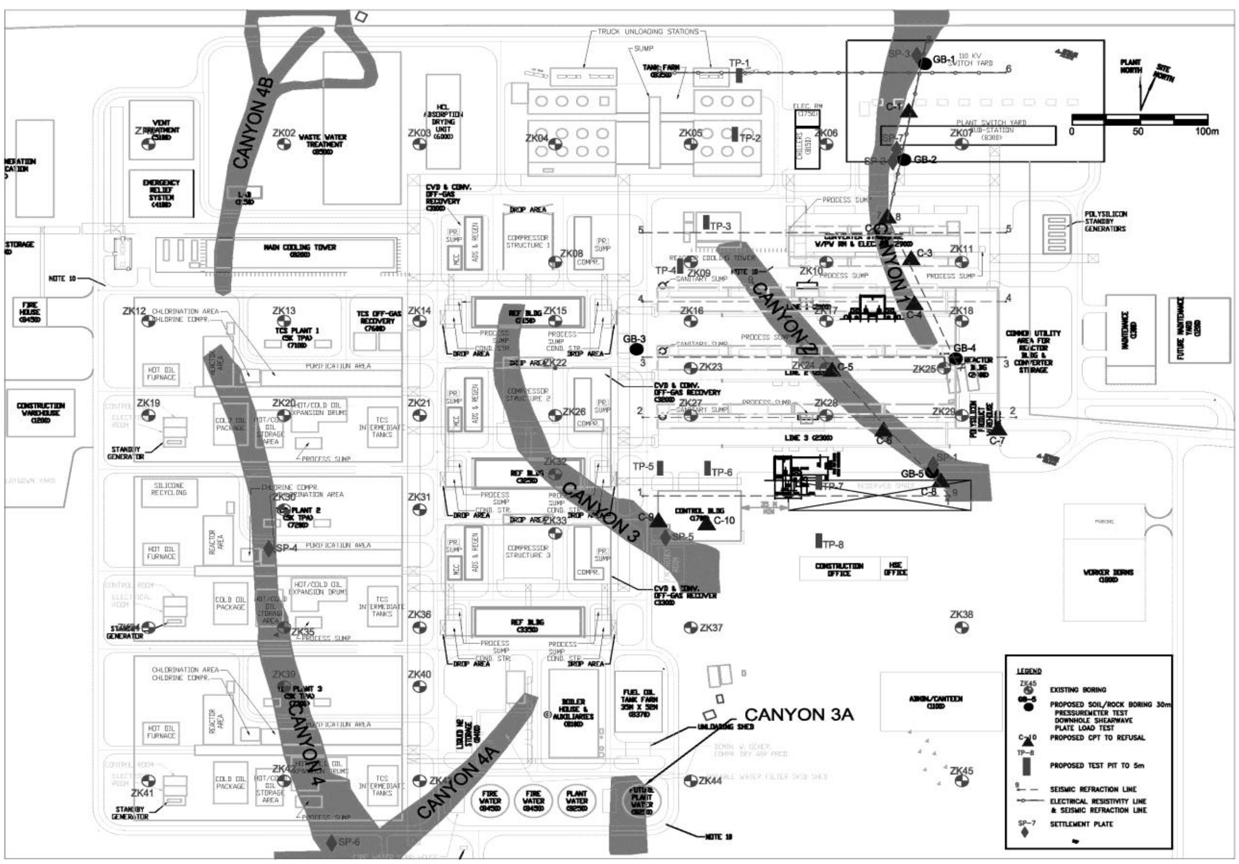

Fig.1. Location of Plant and Existing Borings

SITE GRADING

The grade difference between the highest point El. 101 m (331.4 ft) and the lowest point El 70 m (229.7 ft) at the site was about 31 m (101.7 ft). To obtain a level site at El. 83 m, (272.3 ft), required cuts of up to 18 m (59.1 ft) and fills of up to 13 m (42.7 ft). Three contractors, hired by the local government development agency, performed the site grading. The government prepared a level site for delivery to the owner (LDK) and Engineer (Fluor) for construction of the plant. Site grading consisted of clearing and grubbing, removal of the top soil, vegetation, and roots, excavation and disposal of soft soils in the canyon bottoms, installation of sub-drains in the canyons, cutting of the hills, and filling the canyons with fill compacted to 95% relative compaction in accordance with ASTM D-698 to obtain a level site. The maximum dry density ranged between 1.8 and 2 g/cm^3 (112 and 125 pcf) and the optimum moisture content between 14 and 16%.

Soft soil removal in the canyons ranged from 2 to 5 m (6.6 to 16.4 ft) in depth. The fill was placed in 250 to 300 mm (9 to 12 in.) lifts and was compacted with 18-ton vibratory rollers. Dry density and moisture content of the fill were verified with a nuclear gage. The site grading began on October 1, 2007 and most of the filling of the canyons was completed by middle of December 2007. Rock (siltstone/mudstone and conglomerate) was encountered generally at depths of 8 to 12 m (26.2 to 39.4 ft) below the top of the hills. Although a single shank large dozer could rip siltstone/mudstone, the contractors chose to excavate all rock by blasting. Localized removal of rock was also done by using hoe rams (hydraulic hammers mounted on an excavator). To minimize rock excavation during plant construction and to reduce differential settlement between structures supported on rock and compacted fill, all rock areas were overexcavated by 4 m (13.1 ft) and replaced with compacted fill.

SETTLEMENT MONITORING

Since fills of up 15 m (49.2 ft) were placed in the canyons, it was decided to measure insitu compressibility of native soils and the compacted fills by monitoring settlements under the fill loads with settlement plates. During placement of the fills, a total of 10 settlement plates were placed; six at or near the bottom of the fills and four at mid height of the fills. The settlements were monitored on a daily basis as the fill placement progressed.

In addition to monitoring settlements during fill placement, test fills measuring 20 by 20 m (65.6 by 65.6 ft) (top dimensions) by 6 m (19.7 ft) high were placed on the graded site at various locations to determine the insitu compressibility of native stiff-to-hard residual clays, deep compacted fill, and shallow compacted fill overlying bedrock. The width and height of the test fills were selected to provide loading equivalent to that anticipated under the reactor foundations. In addition to the test fills, surcharge fills were placed in various areas and settlement under surcharge monitored by more than 100 settlement plates. The measured settlement under test fills represented a full-scale "load test" and provided accurate estimates of soil compressibility that allowed various structures to be designed on shallow foundations while minimizing differential settlements. A summary of the data obtained from 24 settlement plates in the Reactor-Converter area is provided in Table 1. Due to the overconsolidated nature of the native residual soils and the compacted fill, the settlement occurs rapidly with the application of the load and about 90% of the settlement is completed within one to two weeks. Locations of the settlement plates in the Reactor-Convertor Area are shown in Fig. 2. Typical settlement vs time data from selected plates are shown in Fig.3.

FINAL GEOTECHNICAL INVESTIGATION

After the site was graded to the final elevation of El. 83 m ± (272.3 ft), a detailed geotechnical investigation was performed which included drilling 285 borings with a total length of 5216 m (17113 ft), performing 616 Cone Penetration Tests (CPTs) with a total penetration of 4323 m (14183 ft), and five downhole shear wave velocity tests to depths of 30 m (98.4 ft) each. The maximum depths of borings and CPTs were 34.1 m (112 ft) and 21.5 m (71 ft), respectively. The borings and CPTs were spaced at 20 to 30 m (65.6 to 98.4 ft) in accordance with the GB 50021 Chinese Code requirements. Laboratory tests were performed on the soil and rock samples that included moisture content and dry density, specific gravity, liquid and plastic limit, pocket penetrometer, direct shear tests, consolidation tests, soil corrosivity tests, electrical resistivity, and grain size distribution tests.

Due to the fast schedule of the project, CPT data and pocket penetrometer readings were used to provide undrained shear strength of the clays, and full-scale test surcharge and settlement plate monitoring data were primarily used to provide soil compressibility for the foundation design. A total of 47 separate geotechnical reports were prepared for various structures and units at the site. CPT data was extremely useful for the design of the fast-track project, even though all four geotechnical firms bidding on the investigation claimed that CPT testing was not feasible at the site. It was through CPT data that the weak/soft zones present at the boundary of the residual soils and bedrock were discovered that were totally missed by the preliminary investigation and the soil borings. Although an extensive laboratory-testing program was completed, these data were generally not available at the time of preparation of geotechnical reports as the construction on the fast-track project was underway.

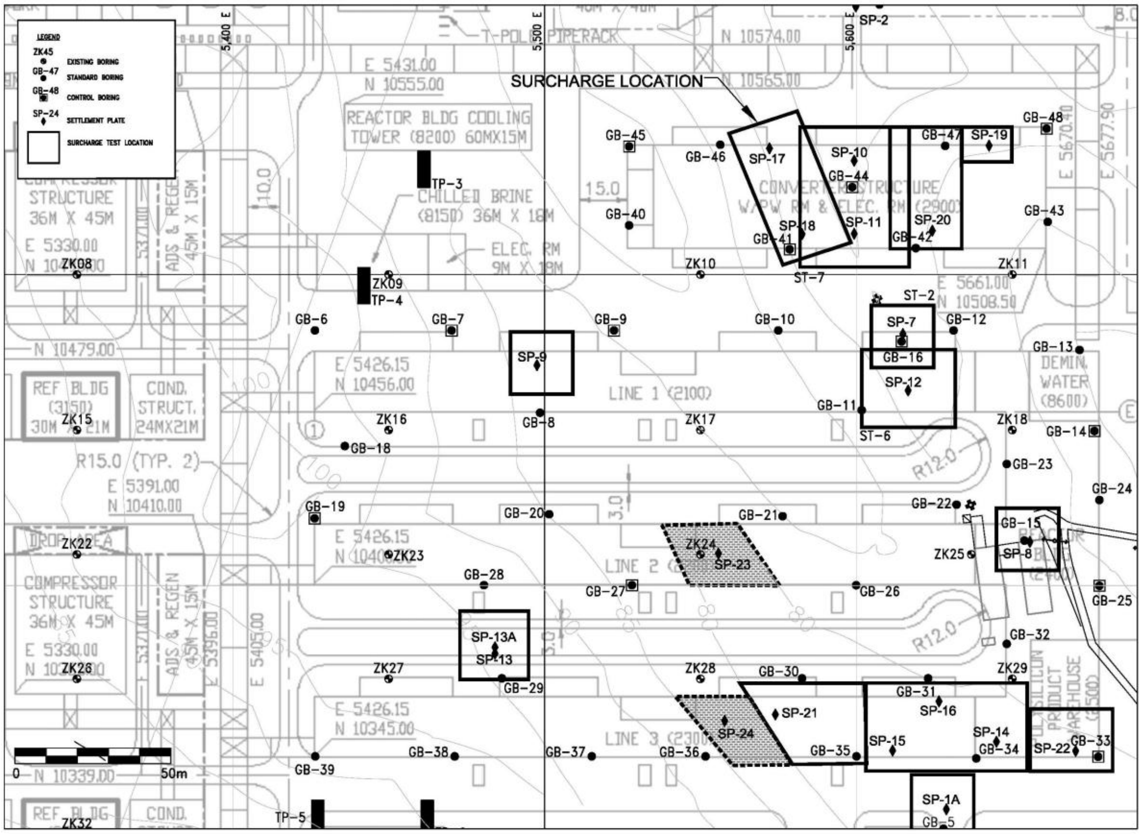

Fig.2. Boring and Settlement Plate Location Plan, Reactor Area

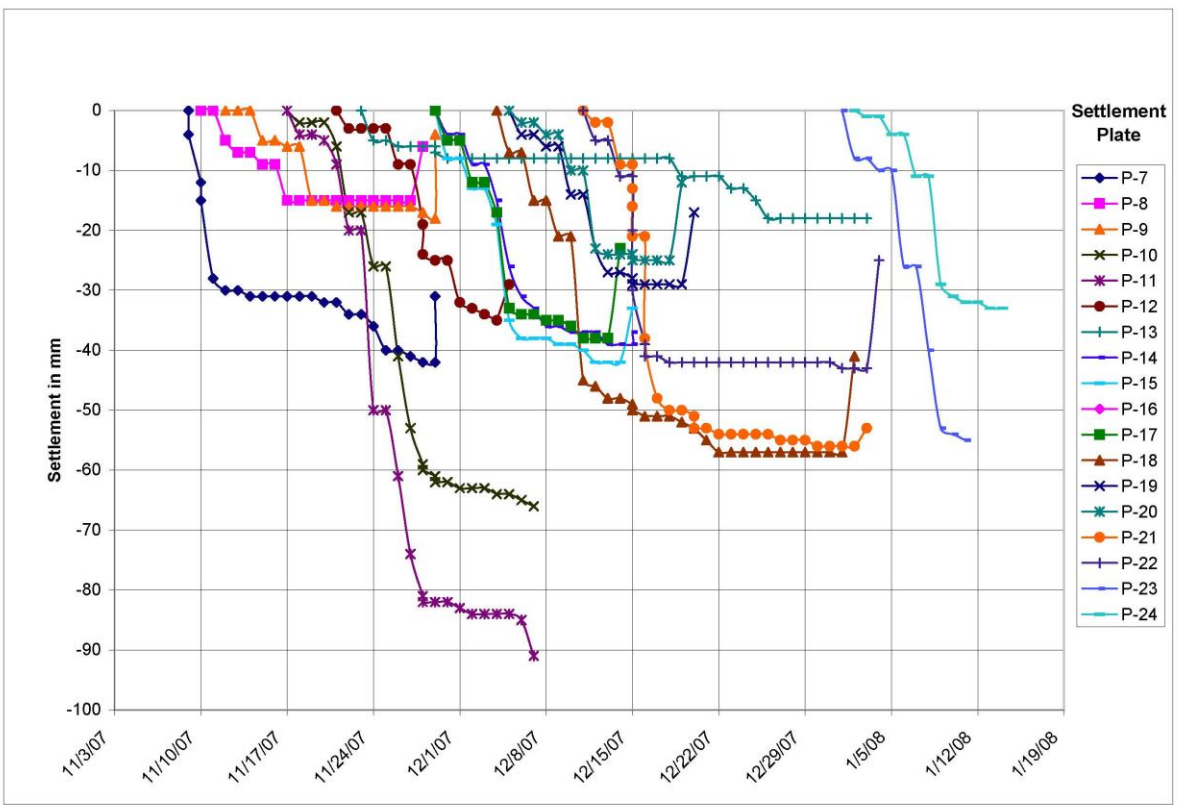

Fig.3. Time Settlement Plot, Plates 7-24

FOUNDATION DESIGN CONSIDERATIONS

Key geotechnical issues affecting the foundation design at the site are:

1. Large and Heavily-Loaded Structures

A typical Reactor Building is 20 m (65.6 ft) wide and 218 m (715.2 ft) long with maximum column loads of 5,518 kN (1,240 kips). Water tanks are 28 m (91.9 ft) in diameter and 8 m (26.2 ft) high. Fuel Oil Tanks are 16 m (52.5 ft) in diameter and 11 m (36.1 ft) high. Pipe racks includes up to six levels of pipes/cable trays/air coolers and have heavy column loads of up to 2,180 kN (490 kips). TCS units include a heavy pipe rack with 1780 kN (400 kip) column loads, a reactor building, and 9 towers up to 61 m (200 ft) in height with dead plus live loads of over 4,500 kN (1,000 kips) and an overturning moment of 13,300 kN-m (8,945 ft-kips). The main Cooling Tower is 177 by 25 m (580.7 by 82.0 ft) in plan and has column loads of up to 3,560 kN (800 kips). Most structures are long and heavy concrete structures with stringent differential settlement requirements.

2. Extremely Variable Soil Conditions

Due to significant cutting and filling at the site, highly variable native soils, and structures with lengths exceeding 200 m (656.1 ft), soil/bedrock conditions under a single foundation are extremely variable. For example, under Reactor Line 3 Building with a total length of 218 m (715.2 ft), the soil conditions below the mat foundation include:

Distance from Western End	Zone	Soil/bedrock profile
0-85 m (0 - 278.9 ft)	1	2 to 3 m (6.6 to 9.8 ft) of compacted fill over bedrock
85-140 m (278.9 – 459.3 ft)	2	2 to 9 m (6.6 to 29.5 ft) of residual soils over bedrock
140-218 m (459.3 – 715.2 ft)	3	9 to 13 m (29.5 to 42.7 ft) of compacted fill/residual soil over bedrock

With such variability of soil conditions, the anticipated differential settlements had to be carefully considered for any shallow foundation design. Similar conditions are present under other long structures and pipe racks.

3. Presence of Soft Soils at Rock/Residual Soil Boundary

Although not discovered during preliminary investigation, the CPTs disclosed presence of 1 to 4 m (3.3 to 13.1 ft) thick random zones of weak/soft soils at the boundary of the stiff to hard residual soils and the bedrock. The surface of the bedrock appeared to be very uneven and variable and contained isolated zones or small cavities filled with weak soil or a combination of soil and water. The extent and location of these zones was very erratic and random. Based on the site grading, these zones could occur a short distance below the foundations or at large depths below the foundations.

TYPICAL FOUNDATION DESIGN

For sake of illustration, foundations for three typical plant areas are discussed in the following sections.

REACTOR FOUNDATIONS

To meet the aggressive construction schedule, Reactor Lines 1-3 and Converter foundation design was completed and the construction started before the detailed geotechnical investigation was performed. The design was based on limited data from preliminary geotechnical investigation completed before grading and extensive full-scale load tests by test and surcharge/preloading fills. Typical test and surcharge fills placed in Reactor Lines 1-3 and Converter areas are shown in Fig. 2. Settlement was measured by surface plates 7-24 shown in Figure 2 and is summarized in Table 1, Summary of Settlement Plate Data. Typical settlement data are shown in Fig. 3. Back-calculated modulus of subgrade reaction values are also shown in Table 1.

Allowable Bearing Capacity

Undrained shear strength of natural soils and compacted fill below the 1.5-m (4.9 ft) thick mat foundation supporting the Reactor Lines and Converter building ranged between 96 and 192 kPa (2 and 4 ksf). For the mat foundation supported on 2 to over 10 m (6.6 to over 32.8 ft) of compacted fill and/or residual soils over bedrock, we recommended an allowable bearing capacity of 200 kPa (4.2 ksf) which has a minimum factor of safety of 3.75 against a bearing capacity failure.

Modulus of Subgrade Reaction

Modulus of subgrade reaction values of 3140 to 12560 kN/m^3 (20 to 80 kcf) were initially estimated for areas with deep natural soils or compacted fill overlying bedrock and areas of shallow bedrock, respectively. These values were applicable to mats with widths of 20 m (65.6 ft) or more and were estimated using E/C (elastic modulus / undrained shear strength) values of 100 to 150 for the stiff to hard onsite clays. After it was decided to have a minimum of 4 m (13.1 ft) overexcavation of all rock areas, a minimum of 2 m (6.6 ft) of compacted fill was present below a 1.5 m (4.9 ft) thick mat foundation. The modulus in the shallow rock areas was reduced from 12560 kN/m^3 (80 kcf) to 9420 kN/m^3 (60 kcf).

These initial modulus values were to be confirmed by test fills in the deep native residual clay areas and compacted canyon fill areas.

Two test fills were performed to provide estimate of modulus for areas of shallow residual soils over bedrock. These test fills represent area with about 5 to 7 m (16.4 to 23.0 ft) of native soils over bedrock with less than 1 m (3.3 ft) of cut and fill. The load-settlement data for Plates SP-8 and SP-9 indicate that a total settlement of 15 and 18 mm (0.6 and 0.7 in.) was measured under a fill of about 5.1 m (16.7 ft) or a loading of 18 x 5.1 = 92 kPa (1.9 ksf). The calculated modulus of subgrade reaction for this loading ranges between 5110 kN/m^3 (32 kcf) and 6130 kN/m^3 (39 kcf). Upon removal of the test fills, a rebound of 9 to 14 mm (0.4 to 0.6 in.) was measured with a rebound modulus of subgrade reaction of 6643 kN/m^3 (42 kcf) to 10,220 kN/m^3 (65 kcf). These data indicated that the recommended modulus of subgrade reaction of 3140 kN/m^3 (20 kcf) was conservative for the native soils of less than about 8 m (26.2 ft) overlying bedrock for loads up to 100 kN/m^2 (2 ksf) and foundation widths of about 20 m (65.6 ft).

Plate 13 was placed in area of shallow hard rock and showed a settlement of 18 mm (0.7 in.) under 8.9 m (29.2 ft) of surcharge fill yielding a modulus of 8,900 kN/m^3 (56 kcf). Plate 13A, a surface plate adjacent to Plate 13 with 4 m of fill above the bedrock showed 44 mm (1.7 in.) of settlement under a load of 4.33 m (14.2 ft) of soil yielding a very low modulus of subgrade reaction of 1771 kN/m^3 (11.3 kcf). This low value probably represents a localized undiscovered zone of soft soil close to the bedrock surface.

The project structural engineer required a minimum modulus of subgrade reaction of 3140 kN/m^3 (20 kcf) for the Reactor and Converter foundations. The measured modulus of subgrade reaction in deep canyon areas, Plates 7, 10, and 11 was lower (1317 to 2164 kN/m^3 or 8.4 to 14 kcf) than the desired modulus of 3,140 kN/m^3 (20 kcf). Therefore, it was decided to improve the modulus of subgrade reaction by surcharging areas where the modulus was less than 3140 kN/m^3 (20 kcf) with 6 m (19.7 ft) of surcharge fill. The top dimension of the fill was selected as the width of the Reactor / Converter foundation. The surcharged areas are shown in Fig. 2 and the Settlement Plate data are shown in Table 1.

In order to estimate the degree of improvement due to preloading (surcharge), the fill from a previously surcharged area around SP-1A was removed, heave measured, and the area was re-surcharged and settlement measured. These data are shown in Fig. 4 and indicate that the settlement after preloading was about 18 mm (0.7 in.) resulting in a calculated reload modulus of subgrade reaction of 5024 kN/m^3 or 32 kcf. Another way to estimate reload modulus is to determine the unload modulus and assume that the reloading will recompress the heave during unloading. Based on this test and the unload modulus of subgrade reaction for plates SP-8 and SP-9, the post-surcharge modulus may be assumed to range between 4710 and 6280 kN/m^3 (30 and 40 kcf).

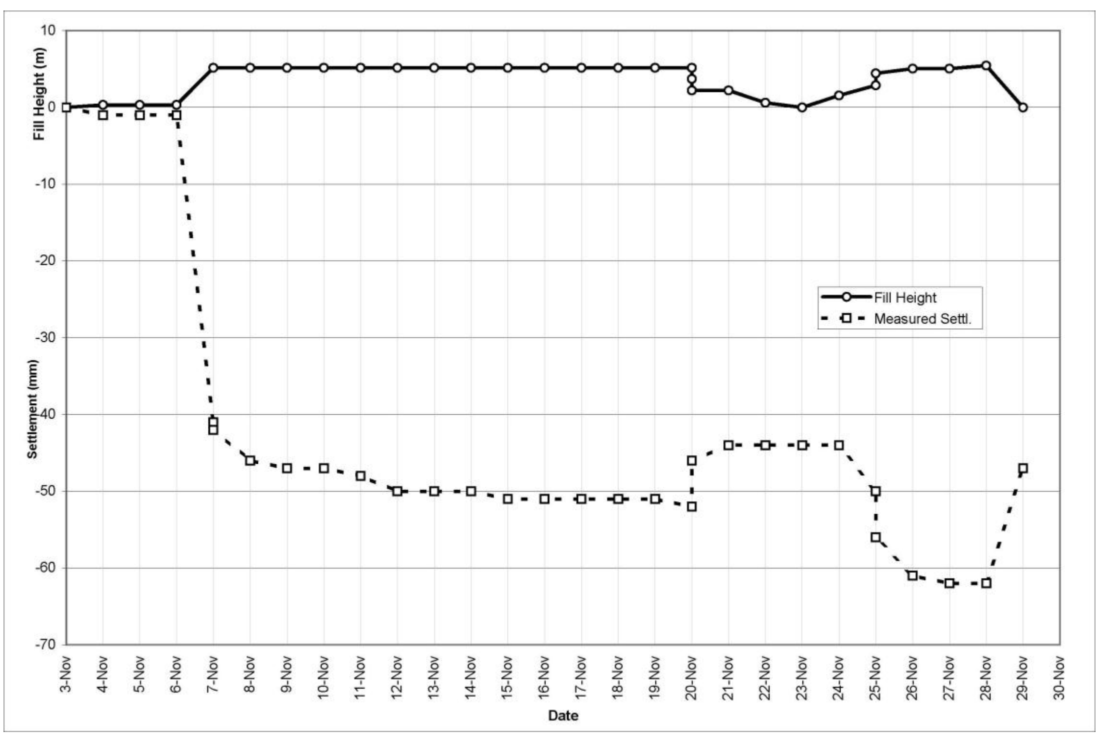

Fig.4. Settlement Plate SP-1A

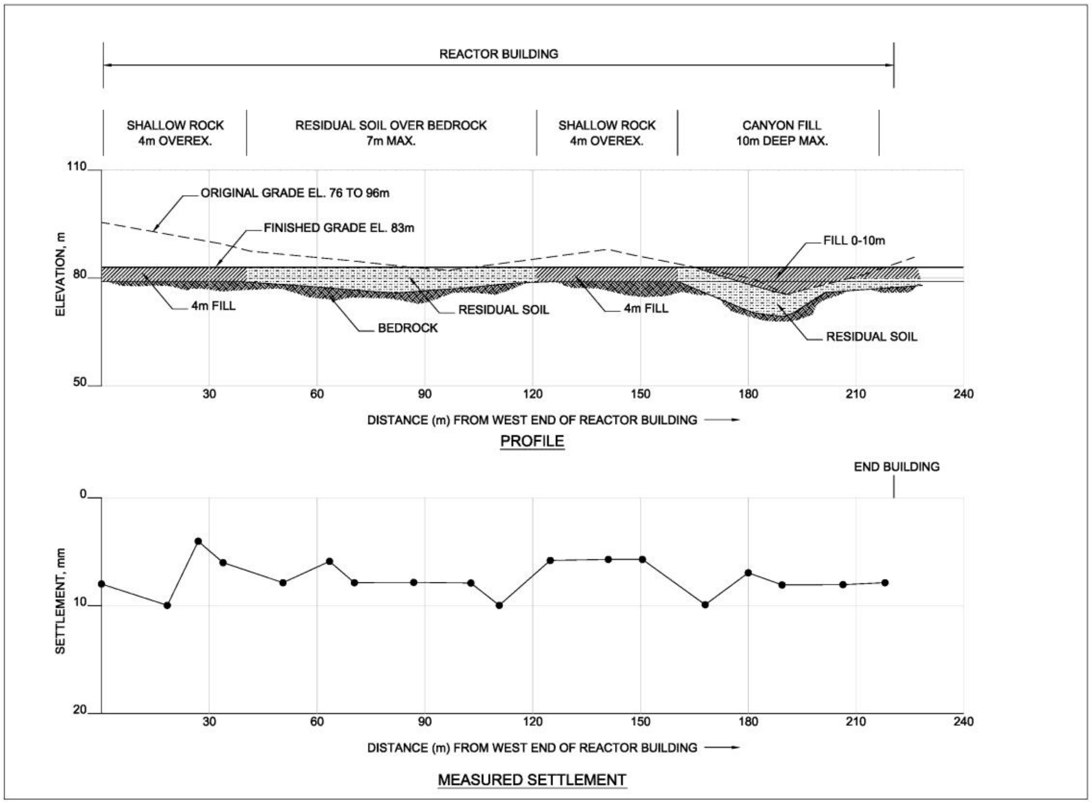

Fig.5. Measured Settlement for Reactor Building

The final modulus of subgrade reaction, in areas with shallow bedrock and a minimum of 2 m of compacted fill, was recommended as 6280 to 9420 kN/m^3 (40 to 60 kcf). For the native residual soils and deep canyon surcharged areas, we recommended a modulus of subgrade reaction of 3140 to 6280 kN/m^3 (20 to 40 kcf). The measured modulus of subgrade reaction without surcharge ranged between 1256 and 2200 kN/m^3 (8 and 14 kcf) and would have resulted in unacceptable settlement, if these areas were not surcharged.

Predicted Settlements

Average loading under the Reactor mat was 120 kPa (2.5 ksf) including the weight of the mat and about 86 kPa (1.8 ksf) without the weight of the mat. Since the settlement due to weight of the mat was essentially completed as the concrete was placed, it was not included in the analysis. Under an average mat loading of 86 kPa (1.8 ksf), the anticipated settlement ranged between 9 and 27 mm (0.4 and 1.1 in.) based on modulus of subgrade reaction of 3140 to 9420 kN/m^3 (20 to 60 kcf). Based on settlement plate readings, maximum differential settlement was expected to be 19 mm (0.8 in.) over 23 m (75 ft) or 0.0008 L. Both the total and differential settlements are within the GB code requirements (slope due to differential settlement of 0.004 and maximum settlement of 200 mm). The structural engineer’s requirement of maximum 38 mm (1.5 in.) settlement under the total load of 120 kPa (2.5 ksf) controlled the design.

Measured Settlements

Measured settlements at 54 points established on rebars projecting out of the top of the mat (before the mat was poured) for Reactor Line 1 are shown in Fig. 5. These measurements were made when about 60% of the load was in place and indicate a maximum measured settlement of 10 mm (0.46 in.) and differential settlement of 6 mm (0.2 in.) between adjacent columns. Settlement monitoring was discontinued after this time due to budgetary constraints. It is expected that at full load, the maximum settlement will be less than 25 mm (1 in.) and differential settlement between adjacent columns will be less than 12 mm (0.5 in.).

TANK FOUNDATONS

Utility area includes Boiler Room, Boiler Stack, Fuel oil, Plant and Fire Water tanks, and a number of other buildings and Pipe racks. The fuel oil and water tanks have diameters of 16 and 28 m (52.5 and 91.9 ft) and heights of 11 and 8 m (36.1 and 26.2 ft).

Subsurface Conditions

The subsurface conditions consist of 10 to 13 m (32.8 to 42.7 ft) of fill and residual soils over bedrock. Undrained shear strength of the compacted fill and native residual soils, was generally above 100 kPa (2 ksf). Weaker soils with undrained shear strength less than 47 kPa (1 ksf) are present near the bedrock at some location(s).

Modulus of Subgrade Reaction

For design of footings or mat foundations supported on 11 to 13 m of fill (36.1 to 42.7 ft) and residual soils over bedrock, we recommended a modulus of subgrade reaction of 1256 kN/ m^3 to 2355 kN/ m (8 to 15 kcf) in the tank area.

Settlement

Estimated settlements using the modulus of subgrade reaction from the Reactor area shown in the previous section ranged between 50 mm (2 in.) and 70 mm (2.75 in.) for the Fire Water and Plant Water Tanks.

A 5-m (16.4-ft) high test surcharge placed at one of the tank locations indicated measured settlements of 23 to 36 mm (0.9 to 1.4 in.) or a back-calculated modulus of subgrade reaction of 5370 to 2355 kN/ m (25 to 15 kcf), respectively. Using these data, the maximum estimated settlement of the tanks ranged between 33 and 46 mm (1.3 to 1.8 in.).

Measured settlements at the tanks with diameters 16 and 28 m (52.5 and 91.9 ft) and heights of 11 and 8 m (36.1 and 26.2 ft) during hydrotest on 8 points around the tank perimeter ranged between 9 and 30 mm (0.4 and 1.2 in.). The maximum differential settlement between two adjacent points on the tank perimeter was 12 mm (0.5 in.).

TCS UNIT-3 FOUNDATIONS

Three TCS Units measuring 180 m by 125 m (590.6 by 410.1 ft) in plan each are present in the western section of the project. Major structures include an E-W Pipe rack - 10 by 157 m (32.8 by 515.1 ft) with dead plus live column loads of 1780 kN (400 kips), a Reactor structure - 21 m by 39 m (68.9 by 128.0 ft) with loads of 1869 kN (420 kips), a Purification area with 9 towers ranging in height from 34 to 61 m (111.6 to 200.1 ft) weighing up to 4673 kN (1,050 kips), miscellaneous Tanks, Sumps, a Furnace, and a Control building.

Subsurface Conditions

The subsurface conditions at the site of TCS-3 were determined by 29 borings and 43 CPTs. The area is underlain by up to 12 m (39.4 ft) of canyon fill and remaining area is underlain by residual soils. Typical undrained shear strengths from 43 CPTs are plotted in Fig. 6.

The CPT data indicate that random zones of weaker clays with undrained shear strength of less than 50 kPa (1 ksf) are present near the bedrock-residual soil contact. It appears that these weak zones are 1 to 5 m (3.3 to 16.4 ft) thick and are likely the result of higher sand content and water softening the stiff clayey soils where it accumulates near the bedrock-residual soil contact. These weak soils are present in 19 out of 43 CPTs and 10 out of 29 Borings within and adjacent to the TCS-3 area.

The weak soils are generally present more than 10 m (32.8 ft) below the finished grade and therefore they are not likely to affect the bearing capacity of foundations in the TCS-3 area. However, the deep weak soils can result in excessive settlements of the foundations supported in areas where they are present.

Surcharge of TCS-3 Area

Due to the presence of a large number of CPTs and borings showing zones of weak materials at depths between 10 and 20 m (32.8 and 65.6 ft), there was concern regarding long-term settlement in TCS-2 and TCS-3 areas.

Based on this concern, we set up six surface monitoring points to evaluate if settlement was still occurring under the load of the up to 12 m (39.4 ft) of fill placed to raise the site to the El. 83 m (272.3 ft). The data from these points, monitored between May 30, 2008 and August 10, 2008, indicate that settlement due to the placement of the original fill to raise the site grade to El. 83 m (272.3 ft) had been completed.

To reduce the post-construction settlements, we also recommended surcharge of the TCS-3 area, where deep soft soils were present, with 10 m (32.8 ft) of surcharge. A total of 23 plates in the TCS Area 3 were installed and monitored during the surcharge program and the results of typical measurements are provided in Fig.7. In areas where 9 to 10 m (29.5 to 32.8 ft) of surcharge was placed, measured settlement ranged between 94 mm and 192 mm (3.7 and 7.6 in.) with a differential settlement of about 98 mm (3.9 in.). The settlement leveled off after about 30 days. Similar but less extensive surcharge was performed on TCS-1 and TCS-2 areas.

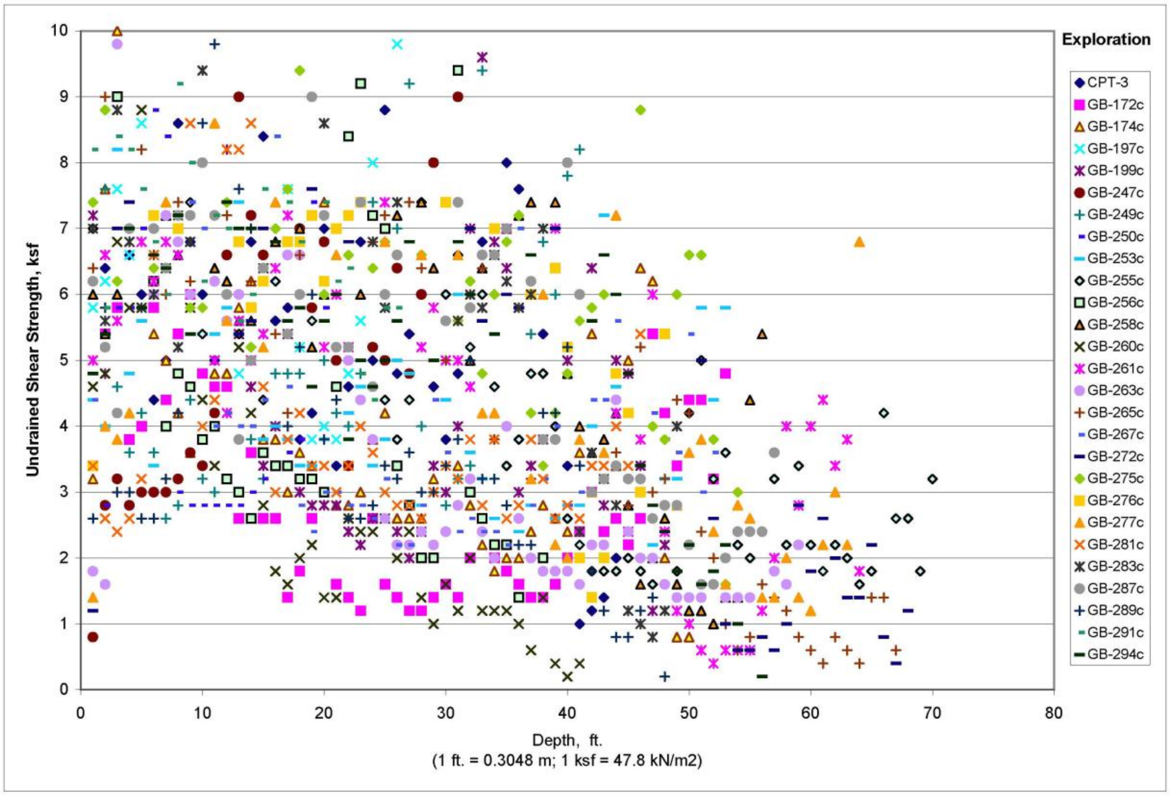

Fig. 6. Undrained Shear Strength, TCS-3 Area

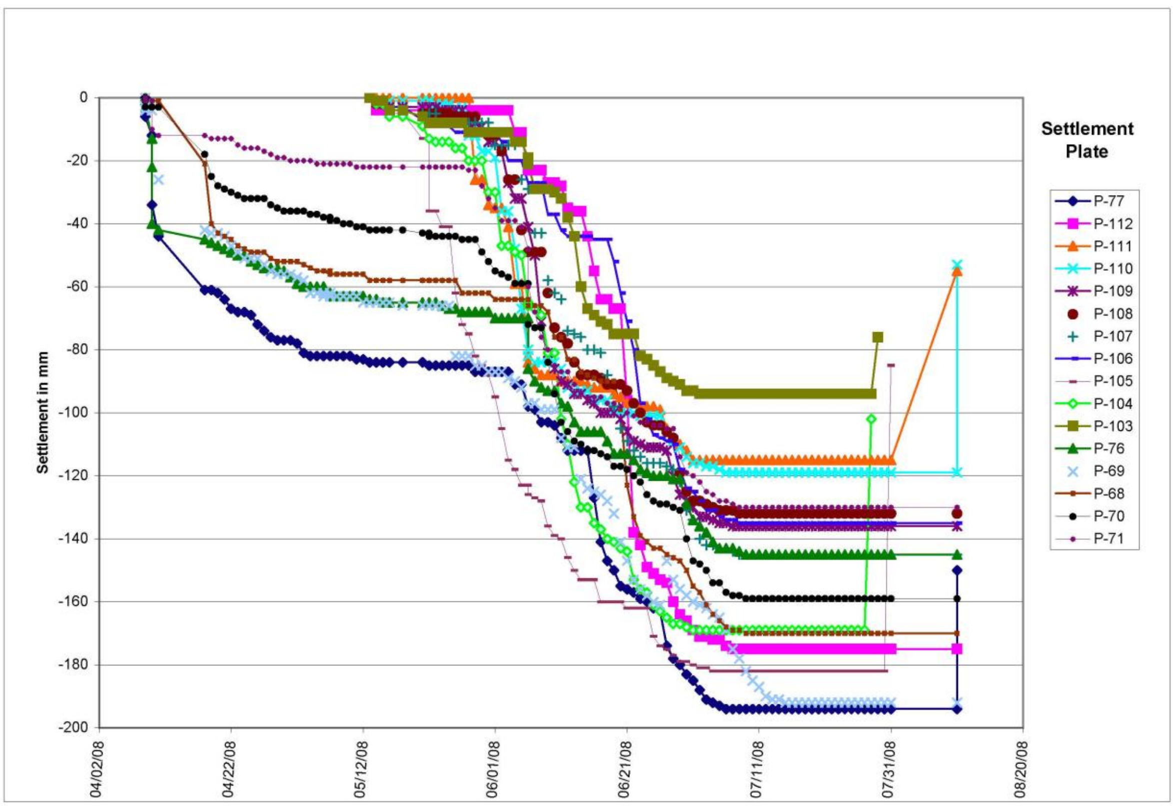

Fig. 7. Time Settlement Plots, TCS-3 Area

Allowable Bearing Capacity

Major structures in TCS area, Purification columns, main Pipe rack, and Reactor area are supported on mat foundations with thickness of 0.8 to 2 m (2.6 to 6.6 ft). We recommended an allowable bearing capacity of 150 kPa (3.0 ksf).

Modulus of Subgrade Reaction

For design of mat foundations supported on 11 to 12 m (36.1 to 39.4 ft) of fill in the canyon areas that had been surcharged, we recommended a modulus of subgrade reaction of 4710 kN/m^3 to 2355 kN/m^3 (30 to 15 kcf). The modulus of subgrade reaction in un-surcharged areas was recommended to be between 1256 to 2355 kN/m^3 (8 and 15 kcf).

Settlement

Estimated settlements of TCS-3 area after the surcharge ranged between 25 and 50 mm (1 and 2 in.) with a differential settlement of 25 mm (1 in.).

DYNAMIC PARAMETERS FOR COMPRESSOR FOUNDATIONS

Field measurements of the shear and compression wave velocity were performed at five locations within the three Compressor Unit areas. The subsurface conditions ranged from shallow bedrock to up to 10 m of fill / residual soils overlying bedrock. The shear wave velocity in the fill/residual soil ranges from 200 to 300 m/sec (656 to 984 ft/sec). The shear wave velocity in the siltstone / mudstone ranges between 1,000 and 1,700 m/sec (3280 to 5577 ft/sec) while the velocity in conglomerate ranges between 3,000 and 3,300 m/sec (9843 to 10827 ft/sec).

CONCLUSIONS

The following conclusions can be made from this case history.

1. Structures underlain by highly variable soil conditions can be supported on shallow mat foundations provided settlement analyses are based on full-scale testing and compressibility is established by back-calculation of full-scale test fills.

2. Surcharge has been routinely used and numerous case histories are reported in the literature (Bhushan et al., 2000, 2004) to improve soft soils and reduce post-construction settlements. However, data presented in this paper indicate that surcharge can also be used successfully to reduce the compressibility of stiff soils and compacted fills.

3. Weak zones at the boundary of bedrock and stiff residual soils can also be improved by surcharge. However, shallow weak zones, where present close to foundation, must be removed and replaced with compacted fill.

4. Settlement of stiff to hard residual clays can be estimated by elastic modulus of 100 to 150 times the undrained shear strength.

5. Due to overconsolidated nature of the soils, most settlements occur as the loading is applied and little long-term settlement is present.

6. The use of shallow foundations can result in significant savings in cost as compared to pile foundations. These savings result from two areas. There is direct saving of foundation cost since shallow foundations are cheaper to build than deep foundations. The other savings come from the schedule impact of shallow foundations. The use of shallow foundation can result in early completion of the project with resulting savings in interest cost on the investment. As an example, on a large project with total project cost of 1.5 billion dollars, an early completion by three months is equivalent to savings of about $40 million in interest cost.

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