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The Santa Helena Dam on Compressible Foundation

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SYNOPSIS

The Paper describes the performance of a 28 m high compacted earthfill dam for water supply in Northeast Brazil. For mainly economic reasons, the dam was founded, in part, over soft to very soft silty clays to a depth exceeding 8 metres. Large diameter sand drains were provided to accelerate consolidation in the foundation during construction. Maximum construction settlements of approximately 240 cm were recorded. Results of settlement of spillway structures, pore pressure measurements in the fill and long term settlements, are described in this paper. Transverse cracking of fill occurred as a result of large deformations in the compressible sediments. The cracks were repaired, and the dam is currently operating satisfactorily.

INTRODUCTION

The Santa Helena water supply dam is located on the River Jacuipe, approximately 60 km north of the city of Salvador, in Bahia, Northeast Brazil. The works for this water supply project consist of a homogeneous earthfill dam, approximately 20 metres above natural ground level and a reinforced concrete gated spillway with a maximum discharge capacity of 1750 m^3/s. The abutments of the dam are founded in dense residual soils. However, part of the dam fill is founded directly over an 8 metre thick deep trough of soft compressible clay. Large diameter sand drains were installed in the soft clay to accelerate the consolidation and consequently develop adequate shear strength during construction of the dam. Large settlements were therefore expected and subsequently measured. The principal reasons for placing the dam directly on the compressible foundation were both economic and technical. Excavation of soft river sediments would have entailed extensive dewatering, deep excavation and subsequent backfilling with compacted fill. Indeed, the significantly increased direct costs and time required for the above operations could have rendered the project unattractive.

TABLE 1 Dam Characteristics

Max. Height (at spillway):	28.5 m
Dam Crest Elevation: (excluding camber)	El. 23 m
Length Along Dam Axis:	260 m
Crest Width:	7 m
Upstream Slopes (H:V):	2.5:1, 5:1 and variable
Downstream Slopes (H:V):	2:1, 3:1 and 14:1
Volume of Earthfill:	308500 m^3
Reservoir Storage Volume:	$240x10^6$ m^3

Fig. 1. The Santa Helena Dam

This paper presents the deformations recorded both during and after construction of the dam. Localized transverse cracking of the fill occurred while the construction was in progress. These cracks were repaired and the dam is currently in operation satisfactorily.

Table 1 lists some principal characteristics of the project. A downstream view of the dam at the end of construction is shown in Fig. 1.

THE SUBSOIL

Good information on subsoil at the dam site is available from 51 drillholes and a number of test pits. Fig. 2 shows the subsoil profile

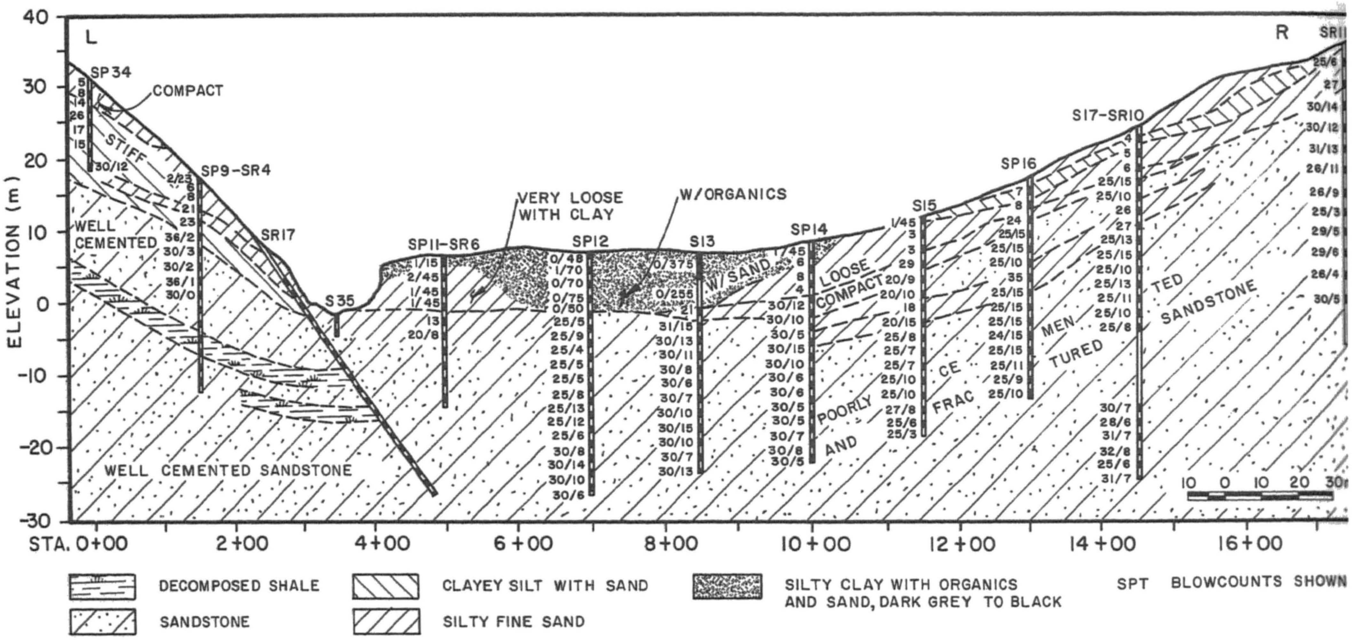


Fig. 2. Subsurface Profile Through Dam Axis

with typical SPT blowcounts, through the axis of the dam. The right abutment of the dam is founded in mature dense silty fine sand residual soil derived from intense weathering of the underlying sandstone. The left abutment is comprised of clayey silt interbedded with silty fine sand. The sandstone in the right abutment is poorly cemented and friable in comparison with that in the left abutment. Due to intense weathering at the site, the residual soils exposed in the abutments do not depict structure of the parent rock. In general, the residual soils are non-plastic with clay content varying between 10-20%. However, zones of more clayey material with L.L.=30-45% and P.I.= 20-26% are also found. The laboratory oedometer tests indicate a compression index, C_c of the order of 0.2. Fig. 3 shows typical grain size ranges of these soils.

As indicated on Fig. 2, compressible silty clay deposits are encountered in the valley floor, directly underneath the foundation of the dam, to a depth of approximately 8 metres. These silty clays are soft to very soft (often zero SPT blowcount), dark grey to black, and contain some organics. The clay fraction generally varies between 60 and 75%. The soft deposits between Stations 4+50 and 6+00 are significantly more sandy. Fig. 3 shows the typical grain size band for these clays. The liquid limits range from 33 to 123%; these values are generally above 70 for the very soft deposits. Laboratory oedometer tests on 4.7 cm diameter Shelby tube samples indicate the compression index to vary between 0.33 to 2.70. Some samples show an apparent preconsolidation pressure of 0.75- 1.0 kg/cm^2. The vertical coefficient of consolidation, C_v, ranges from 10^{-3} to $2x10^{-4}$ cm^2/s. Very soft clay samples in the middle of the valley show initial water contents above 200%. These samples are also found to indicate values of compression index in the range of 2.0 to 2.70. The unconsolidated undrained shear strengths range between 0.01 to 0.28 kg/cm^2 The strain at failure, both in the simple compression test and in consolidated undrained triaxial tests with pore pressure measurement lies in the range of 6 to 15%. The shear strength parameters in terms of effective stress can be represented by C'=0',Ø=26.5°. An effective cohesion intercept of 0.1-0.2 kg/cm^2 is also observed in some tests. The pore pressure parameter A at failure (Skempton, 1954) is found to vary between 0.78 and 1.55.

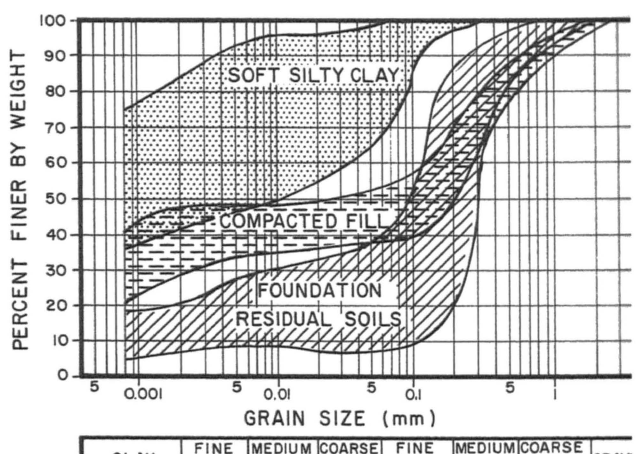


Fig. 3. Grainsize Distribution Curves

BORROW MATERIALS

The borrow fill material consisted of a sandy clay residual soil of medium plasticity with mean values of 38 and 23% for liquid limit a

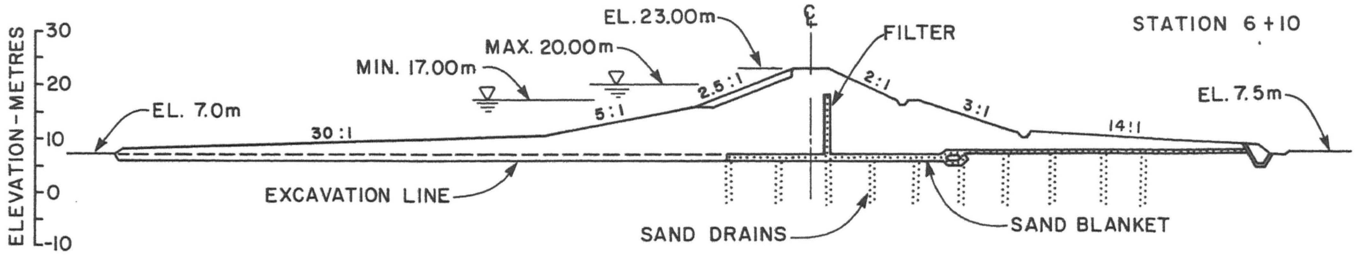


Fig. 4. Typical Dam Section

plastic limit respectively. A maximum dry density of 1.69 t/m^3, at optimum water content of 19%, can be achieved using the Standard Proctor effort in the laboratory. The effective shear strength parameters for samples moulded at optimum water content can be expressed by $C'=0.25$ kg/cm^2; $\phi'=27°$.

THE DAM

A typical cross-section of the dam founded over soft clay is shown in Fig. 4. A 1 m thick vertical sand filter and a 2.0 m thick horizontal drainage blanket have been provided for control of seepage in the fill and the foundation. Below Elevation 10.60 m, the upstream slope incorporates a compacted fill berm at a slope of 30(H):1(V). This berm serves as an impervious blanket as well as for stabilization during construction. The pervious fill in the downstream slope below El. 11.0 m, at a slope of 14:1 also serves as a stabilizing berm. In the abutments, where the dam is founded on residual soil, a minimum 5 m deep cutoff trench, 6 m wide at the base, has been provided to reduce flow through the upper, more pervious, sandy soils.

Removal of the soft compressible deposits was not attempted in view of the complex and time-consuming dewatering and excavation methods which would necessarily be required. The location of these deposits in the valley floor also implied that all excavation and backfilling would have to be completed in one dry season.

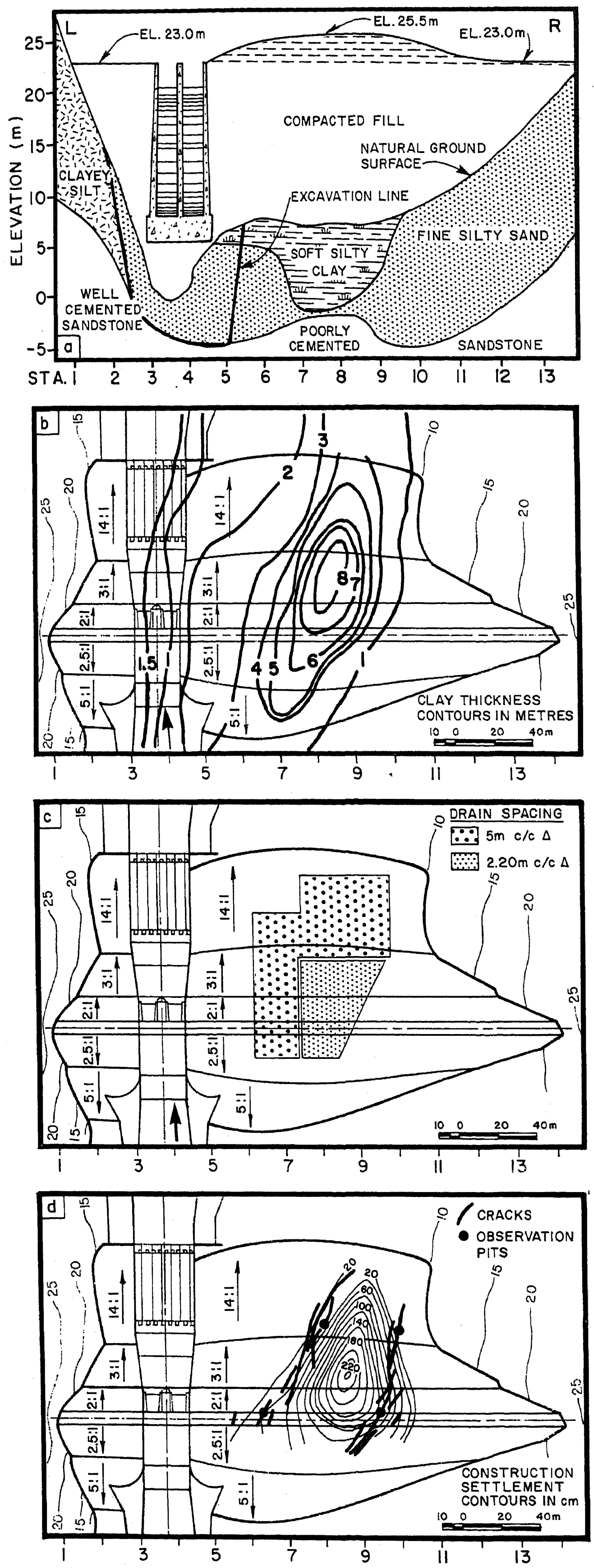
Stability analyses indicated the need for accelerated pore pressure dissipation during construction to ensure stability. A pore pressure ratio, $r_u = u/\sigma v$ (Bishop 1954) equal to 0.5 in soft silty clay was required to attain a factor of safety of 1.3 at any stage of the construction. A factor of safety of 1.5 was deemed to be desirable, for which an average $r_u = 0.3$ was required. With a vertical coefficient of consolidation value of 10^{-4} cm^2/s, a pore pressure ratio equal to 0.7 could be expected at the end of construction. This slow rate of dissipation was unacceptable. Consequently sand drains were designed (Johnson, 1970) to achieve a minimum degree of consolidation in the foundation of 60% at the end of the stipulated ten month construction period. The horizontal coefficient of consolidation, C_h, was assumed to be 3 times C_v.

Fig. 5(a) shows a simplified section through the axis of the dam, while Fig. 5(b) shows the thickness of soft clay in the foundation. Fig. 5(c) shows the distribution of sand drains in plan. Two equilateral triangle grid spacings of 5 m and 2.2 m were adopted depending on the height of fill and the consolidation characteristics of the compressible foundation soils. The 42 cm diameter, non-displacement type sand drains were installed using vibratory equipment. An open steel casing was suspended from an external vibrator providing an eccentric moment of 5000 kg.cm. The steel casing penetrated the soft foundation easily until it rested on top of residual soil. Another steel casing with its bottom flap valve open was next driven inside the first casing. The second casing was subsequently removed with its bottom valve now closed to remove the clay. The sand drains were formed by backfilling with clean sand while the outer casing was simultaneously extracted and vibrated. The sand drains so formed are therefore considered to be well compacted. By using two hoists, two external casings and one internal casing, 579 sand drains with a total length of 5610 m were installed over a period of 40 days. On average, 22 drains were installed in an eight hour shift. On two occasions, 33 drains (329 m) and 32 drains (343 m) were installed in each shift.

A heavy reliance was placed on instrumentation to control the rate of construction and to assess the stability of the dam. The instrumentation used was selected on the basis of simplicity and ready availability in the area. The instrumentation consisted of 20 pneumatic piezometers, 16 open standpipe piezometers, 9 sets of telescopic settlement plates, 27 surface setlement plates, 8 surface horizontal deformation measuring stations and one seepage flow measurement gauge.

THE SPILLWAY

Fig. 6 shows a longitudinal section of the spillway which was located in the river channel. The excavation was taken down to approximately El. -4 m until either well cemented sandstone or dense residual soil was encountered. The inlet canal and the gated spillway section are founded on 6 to 9 metre thick compacted residual soil fill. The alternative of backfilling the excavations with concrete was deemed to be very costly in comparison with the solution adopted.



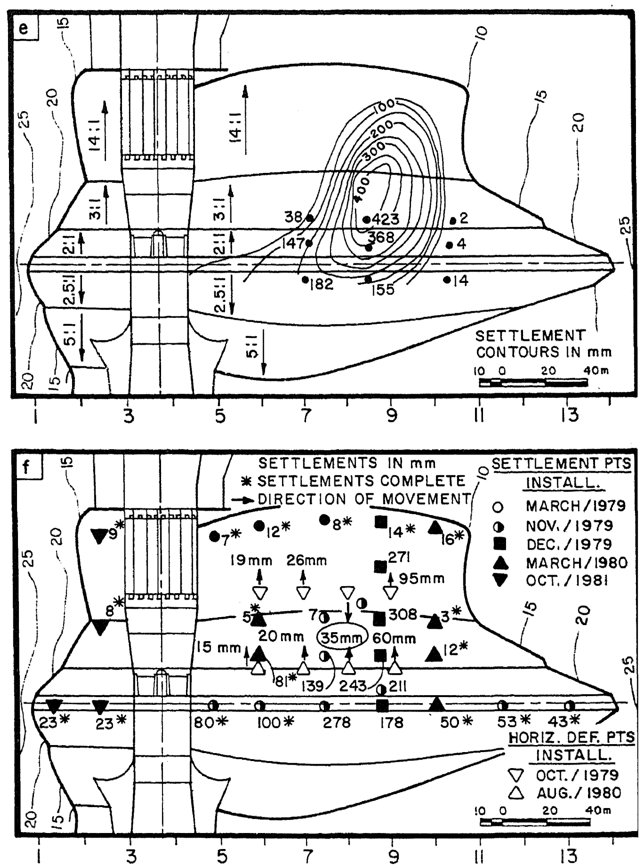


Fig. 5 (a): Simplified Section Through Axis
(b): Thickness of Soft Clay in Plan
(C): Layout of Sand Drains
(d): Construction Settlements in Found tion, and Observed Cracks
(e): Post-construction Foundation Se tlement
(f): Post-construction Deformation Fill and Foundation

The spillway apron is cast on a compact gravel blanket overlying sand fill (Fig. 6 The dissipation basin concrete slab, 3.50 thick, is cast directly over the dense fou dation. Hence, varying foundation conditio are encountered for the spillway walls. Si teen settlement measurement points were i stalled on these walls.

The excavation for the spillway structure prov to be a difficult task in view of the large i flow of water. Limitation of working area al required some excavation slopes to be steep than 1:1. Consequently extensive dewateri using well points and deep pumping wells w necessary. Over 700 well points and 38 de wells, as shown in Fig. 7, were required enable the foundation preparation to procee The water table was lowered by approximate 10.7 metres to El. -3.7. The average flow fr the seven pumping stations reached 200 m^3/hr the final excavation depth for the spillway.

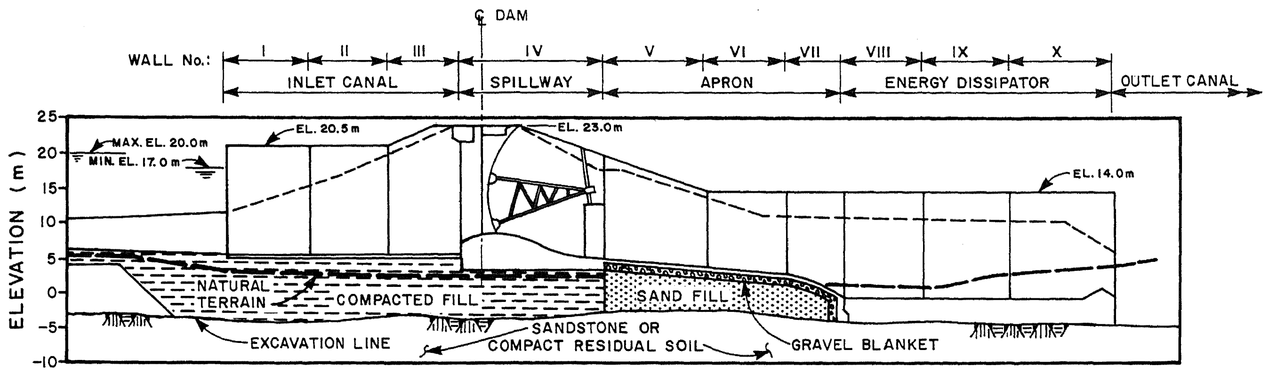


Fig. 6. Foundation Conditions Beneath Spillway Structures

The excavation along the cofferdam was slow in view of the presence of soft clays and loose to medium dense sands. Considerable slumping of excavation slopes along the cofferdam was experienced. These slopes were supported by using timber crib walls to maintain their steep angle. Significant delays were also caused as a result of flooding from heavy rains (188 mm in 24 hours), in October 1977 when the compacted fill had been placed in the foundation of the spillway. The considerable difficulty enountered in the excavation for the spillway justified the decision to place the dam directly on the deposit of soft clay.

BEHAVIOUR OF THE DAM

a) Settlements

Fig. 5(d) shows the distribution of settlements in the soft clay at the end of the twelve month long construction period. The settlements caused by placement of a 2 m thick sand blanket, which also served as a working platform for installation of sand drains were not recorded. The settlements shown in Fig. 5(d) were obtained from settlement plates installed on top of the compacted sand blanket. The maximum settlement during construction reached 240 cm at the maximum section at Station 8+10. The settlements generally follow the contours of thickness of soft clay shown in Fig. 5(b).

Fig. 8 shows a comparison between the predicted and measured settlements for the maximum section at Station 8+10 m. The measured settlements after twelve months of construction under the centreline of the dam, at drillhole S-13, are almost half of the estimated settlements. After almost five years of settlement data, it can be stated with confidence that the final consolidation settlements under the centreline would be approximately 250 cm in comparison with the estimated consolidation settlement of 420 cm. It would appear that in the case of very soft silty clays, with SPT blowcounts of zero, the value of C_c has been grossly overestimated in laboratory oedometer tests. Some of the difference between observed and measured settlements can also be attributed to the installation of large diameter, compacted, sand drains, which would reduce the mass compressibility of the

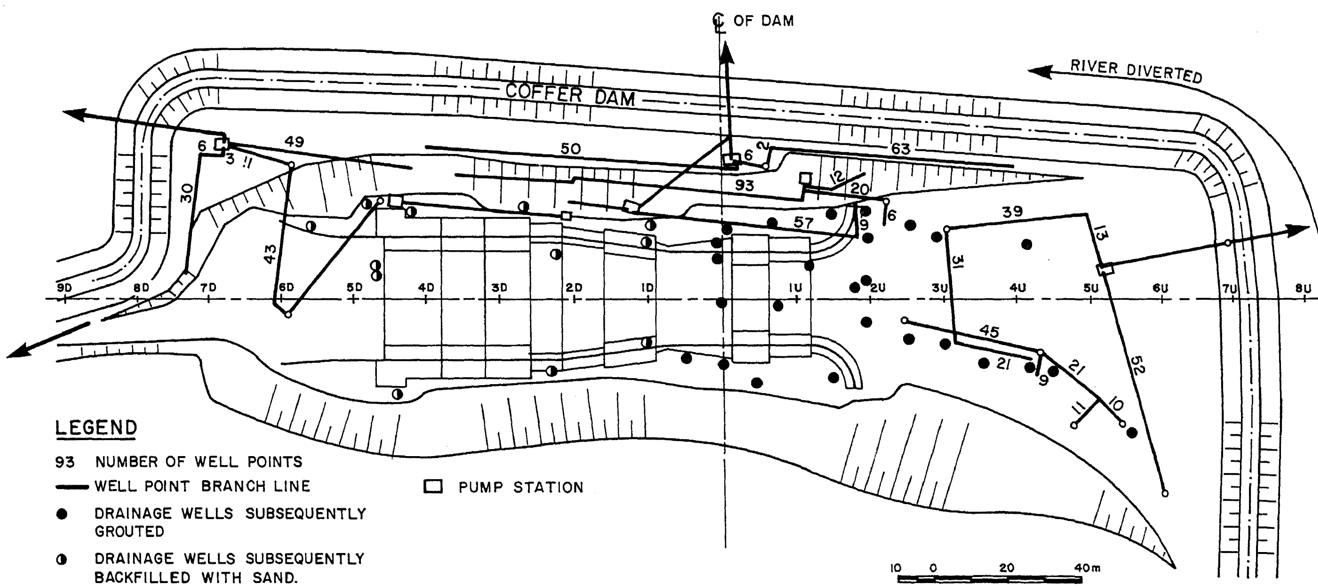


Fig. 7. Dewatering in Spillway Excavation

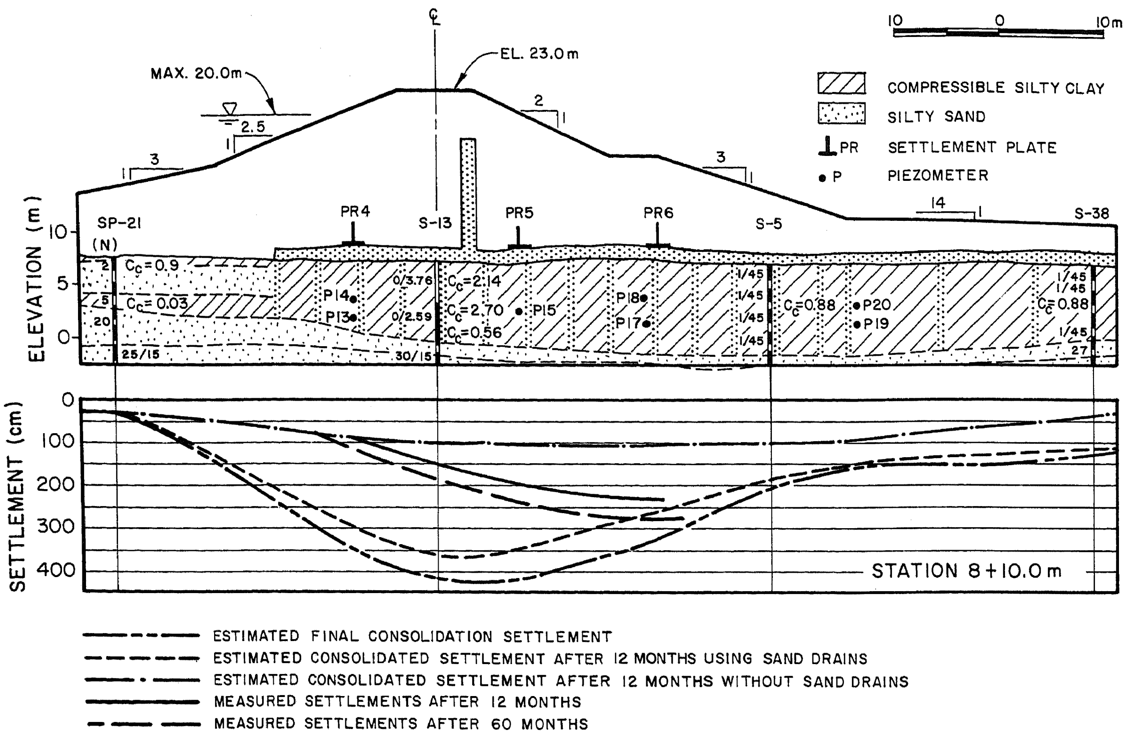


Fig. 8. Estimated and Observed Settlements at Maximum Section

soft deposit. Also, in view of the relative stiffness of the sand drains, the load would tend to concentrate on them due to an arching effect, in a manner similar to that for stone columns. The difference in observed and estimated settlements is also a reflection on the variability of fluvial sediments since there is a practical limitation on the quantity of testing that can be undertaken on any project of this nature. Table 2 shows a summary of estimated and measured settlements in the foundation of the dam.

TABLE 2 Estimated and Observed Settlements

Sta	Offset (m)		Gauge No.	Settlement (mm) Estimated 12mos	Estimated Final	Measured 12mos	Measured 60 mos
7+00	8.50	U/S	PR1	1900	2350	600	782
	7.50	D/S	PR2	2100	2300	530	677
	21.00	D/S	PR3	1600	1800	200	238
8+10	8.50	U/S	PR4	2500	2900	960	1115
	7.50	D/S	PR5	3320	4000	2090	2458
	21.00	D/S	PR6	2100	2350	2350	2773
10+10	8.50	U/S	PR7	80	80	95	109
	7.50	D/S	PR8	400	410	124	128
	21.00	D/S	PR9	800	900	90	92

Fig. 9 shows five years of record of settlemen and pore pressure measurements at the maximu section with increase in height of fill and res ervoir elevation. Over 90% of total settlement occurred during the first 18 months. Durin early stages of loading, the rate of settlemen increased significantly reaching 15 to 1 mm/day in early October. It is interesting t note that this accelerated rate of settlemen coincided with alarmingly high pore pressur measurements. Indeed, on the bases of instru mentation readings, the fill placement operatio was suspended for 36 days during October an first week of November 1978. The rate of settle ments decreased to less than 0.5 mm/day si months after the end of construction. The rat of settlement is currently (Oct. 1983) less tha 0.05 mm/day and decreasing.

The diminishing rate of settlements is exempli fied in Fig. 10, where settlements of both fil and foundation are plotted on a log time scale The settlements have been plotted subsequent t January 1980, when over 90% of consolidatio settlements had occurred. The thickness of sof clay under settlement points PS18 and 19 was and 4.5 metres, respectively. Hence, a strai rate of approximately 0.05% per log cycle is ob tained for the 560 to 1500 day interval. Th strain rate shows a trend for a significant de crease after 1500 days, possibly as a result c drained creep behaviour. The new strain rat is estimated to be less than 0.02% per lo cycle.

Fig. 5(e) shows post-construction settlement contours of the clay foundation. Settlement

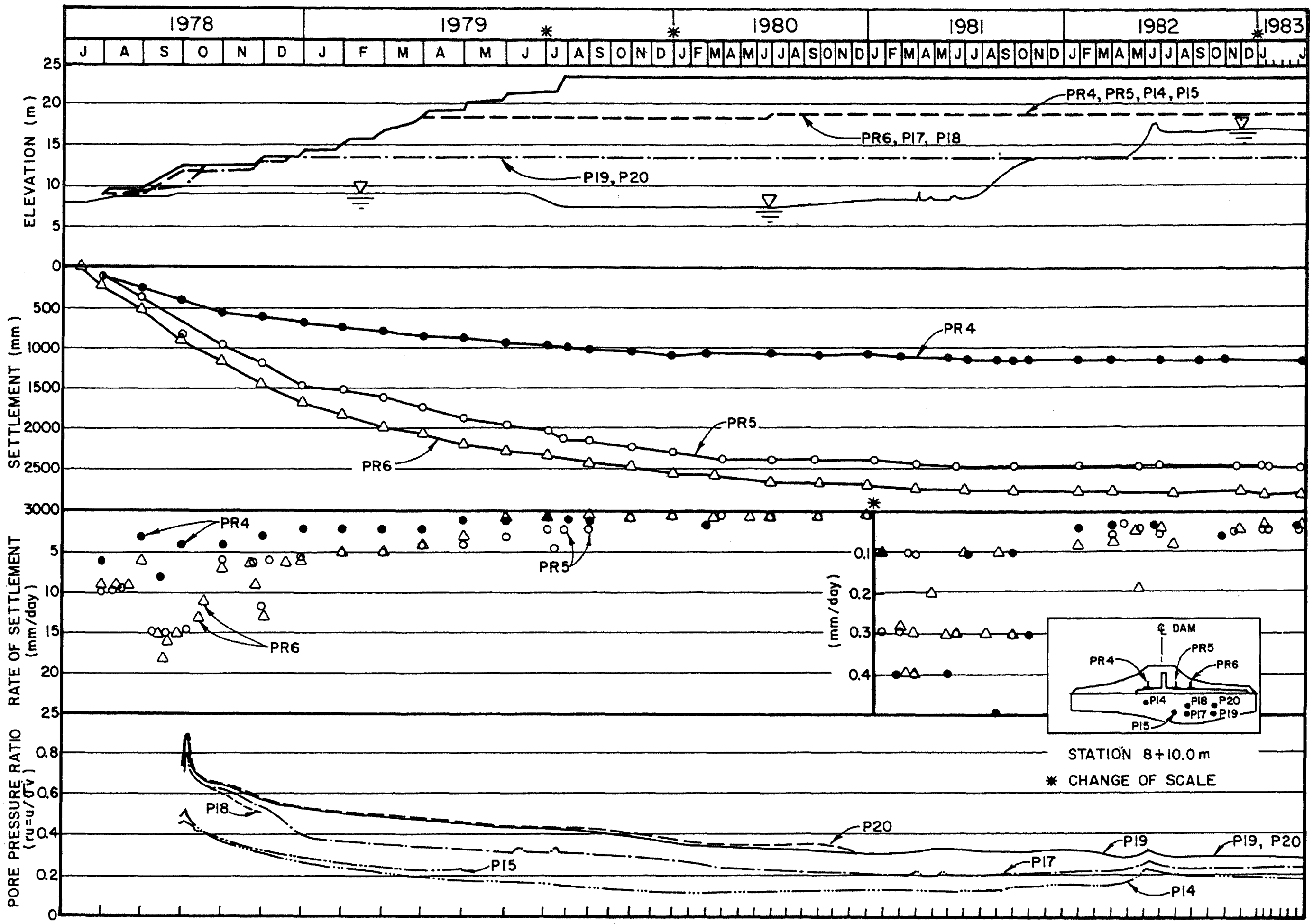


Fig. 9. Variation of Settlement and Pore Pressure with Time

exceeding 420 mm occurred at Stations 8+00-9+00 from August 1979 to October 1983. Fig. 5(f) shows post construction settlements and horizontal movements of both fill and foundation, as observed after installation of the various surface plates. The asterisk denotes measurement points where all settlements have ceased at present.

The four horizontal movement gauges installed 50 m downstream of the slope in October 1978 show a maximum horizontal downstream movement of 95 mm to date. However the gauge at Station 8+10 indicates movement in the upstream direction as a result of large settlements in the centre of the bowl of soft clay. The horizontal movements have stabilized since Dec. 1980. Another set of four surface horizontal movement gauges was installed 20 m downstream of the dam axis in August 1980 to monitor movements of the thicker deposit of soft clay. The largest downstream movement, 60 mm, was again observed at Station 9+00. The readings from the second set of installations have stabilized since Sept. 1981 and are consistent with the small rates of settlements noted earlier. Horizontal deformation measurements have been relatively insensitive to increases in water elevation in the reservoir.

The spillway walls (Fig. 6) suffered differential settlements due to varying foundation conditions. A maximum settlement of over 110 mm was observed on Right Wall II founded on compacted fill. A large proportion of settlements occurred while fill placement in the dam, adjacent to the walls was in progress. During this period a maximum settlement of 0.6 mm/day was observed at Wall II R. Table III summarizes the recorded settlements.

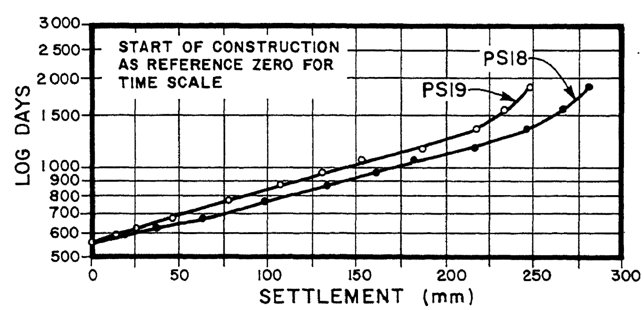


Fig. 10. Long Term Settlements vs Log Time Plot

TABLE III: Spillway Wall Settlements

Wall	No.	Settlements (mm) from April 1978 Sept. 1978	Jan. 1981	July 1981
I	R	82	83	83
	L	12	16	18
II	R	82	106	111
	L	29	52	54
IV	R	61	102	107
	L	60	81	83
V	R	56	80	85
	L	55	70	74
VI	R	17	41	44
	L	13	26	27
VII	R	7	4	0
	L	20	21	0
IX	R	7	6	0
	L	2	1	0
X	R	2	0	0
	L	3	0	0

The settlements of the spillway walls on compacted fill were surprisingly almost three times higher than anticipated. The reason may lie in the fill being placed at a water content above the optimum. Also, the flood of October 1977 inundated the compacted fill then in progress. While a substantial volume of upper softened fill was subsequently excavated and replaced, it is possible that the remainder of the fill also experienced some swelling and softening.

b) Pore Pressure

The pore pressure observations have been relatively uneventful except in the early stages of fill construction when high pore pressures were recorded. It is likely that in the very early stages of loading, before the clay could undergo significant consolidation, the foundation was very close to local failure. Under such a condition, local yielding would result in increased rate of pore pressure generation (Burland, 1971, Leroueil et.al, 1978).

c) Cracking of the Embankment Fill

Localized cracking in the fill occurred as the dam height was raised. The first two cracks occurred when the fill was at approximately elevation 10 m. The cracks at Stations 6+10 and 9+10 were parallel to each other at an angle of 70° with the dam axis. It is of interest to note that these first cracks occurred when the fill placement was temporarily suspended as a result of high piezometric readings ($r_u > 0.5$). Narrow trenches transverse to the cracks were excavated to a depth of approximately 3 m. The cracks were cleaned by injecting water under low head and were subsequently filled with a 5% bentonite mix. Fig. 11 shows the treatment of a crack ending at the sand chimney drain.

The next two cracks, both near Station 9+10, were observed when the fill reached El. 18.0 m. The cracks maintained an inclination of 70° to the dam axis. These cracks were located only on the downstream slope, and were treated as described previously. At the completion of the dam, a new set of nine major cracks was obs ved (Fig. 5(d)). Three cracks were ag located at Station 9+10 m, two of wh occurred on the downstream slope while third was located upstream. Two cracks Station 6+10 also extended to the upstr side. The crack at Station 8+00 was restric to lower elevations of the downstream slc The cracks were a maximum of 3.5 m deep were generally of hairline width. However places, a maximum opening of 20 mm observed. These cracks were also grouted us a 5-7% bentonite mix. The transve observation trenches have been backfilled w compacted residual soil alternating between 30 cm thick layers of gravel. PVC pipes grouting and bleeding have been installed the gravel layers for any treatment which be necessary in the future. No additio cracking has been observed since January 198

The first appearance of cracking was prob associated with large shear strains in the foundation, and the simultaneous developmen high pore pressures. However subsequent tr verse cracking appears to be related to the ferential settlements in the fill. Fig. and b) show a sharp change in thickness of clay near Stations 9+00 and 6+10 where crac occurred. It is also of interest to note high pore pressures in the foundation were observed with cracking at the higher fill el tion. This mechanism of cracking is diffe from the more common case of squeezing ou clay from beneath the embankment to upwards the toe of the embankment. In the latter c tension cracks begin at the bottom of the (Chirapuntu and Duncan, 1977). Similar tr verse cracking has been observed at Duncan (Gordon and Duguid, 1970) where over 4.3 differential settlements occurred at a dist of less than 120 m from the rock abutment.

Fig. 11. View of a Crack in the Fil

As of October 1983, the Santa Helena Dam is operating satisfactorily with reservoir at El. 19.00 metres. Seepage through both fill and foundation in the right flank of the dam is currently measured to be 4.4 l/s, in comparison with the design estimate of 2 l/s.

CONCLUSIONS

This case history is an illustration of the application of the art and science of engineering and the use of the observational approach to construction (Peck, 1969). A major water retaining structure has been constructed over soft foundation at a relatively low cost to the client.

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