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RESIDUAL STRENGTH OF SAND FROM DAM FAILURES IN THE CHILEAN EARTHQUAKE OF MARCH 3, 1985

by

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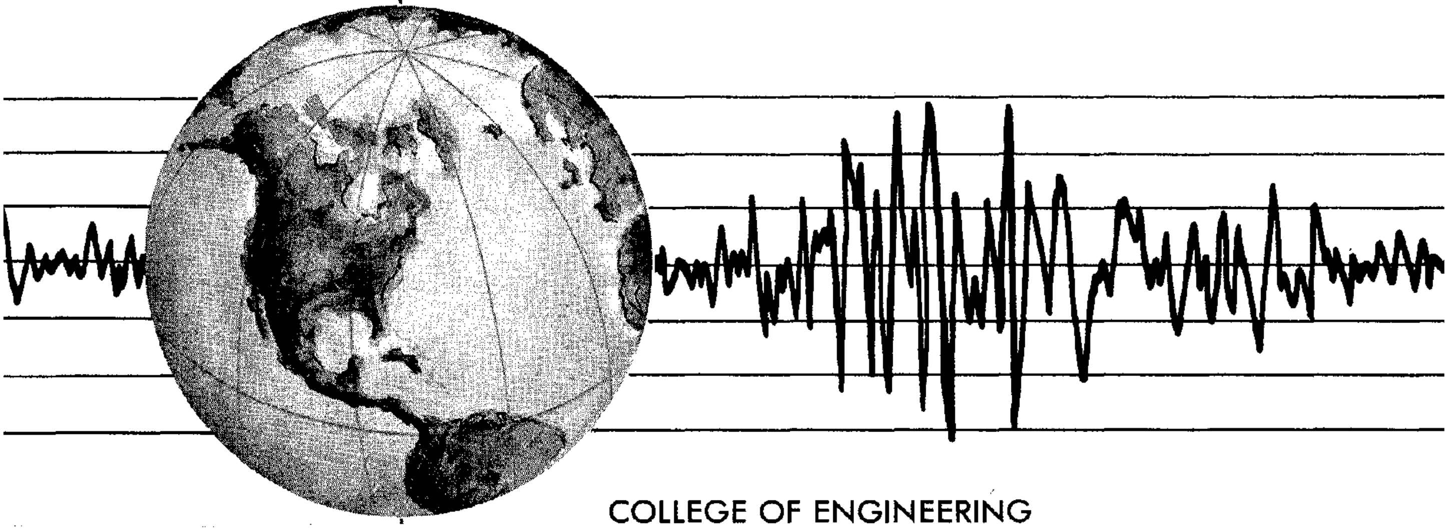

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The slope failures at La Marquesa and La Palma Dams in the Chilean earthquake of March 3, 1985 were apparently due to liquefaction of loose sand layers near the base of the embankments. The dams were low structures (4 to 10 m high) and the horizontal movements were substantial. However the configuration of the embankments after the failures suggests that the liquefied soil retained a small but significant strength after liquefaction occurred.

Field explorations provided a basis for assessing the characteristics of the liquefied sands before the earthquake occurred and analyses of the configuration of the slide material provided a basis for evaluating the probable ranges of residual strengths for the liquefied soils.

The two cases of dam failure described are of special interest because they provide examples of the performance of very low dams, in which static stresses are quite low, under strong earthquake conditions and they also provide two additional studies to add to the relatively sparse record from which residual strengths of liquefied soil can be determined on the basis of field performance.

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1. Introduction

Within 50 miles of the epicenter of the Central Chile earthquake of March 3, 1985, are 30 small storage reservoirs, retained by earth embankments ranging in height from 4 to 26 m (13 to 85 ft). The reservoir embankments were typically built of silty clayey sands weathered from the granodioritic Coast Batholith. The majority were constructed using light equipment and have a relatively low degree of compaction. While 14 of these embankments suffered cracking, it is remarkable that only two of them (La Marquesa and La Palma Dams) suffered extensive damage during this major earthquake (Retamal et al., 1986a); in both cases, the magnitude of the deformations suggested that liquefaction of the embankment or its foundation was the probable cause and that, for some time during the earthquake, the soil strength had been reduced to its residual value. In view of the scarcity of field data for this type of failure, a study of these two dams, La Marquesa and La Palma de Quilpué, was undertaken to determine their internal geometry and define the characteristics of the materials that failed.

2. The Earthquake of March 3, 1985

The earthquake with Magnitude $M_s = 7.8$ occurred at 22:47 GMT on March 3, 1985 in the subduction zone formed where the Nazca Plate moves under the South American plate at a shallow angle. Fig. 1 shows the location of the epicenter

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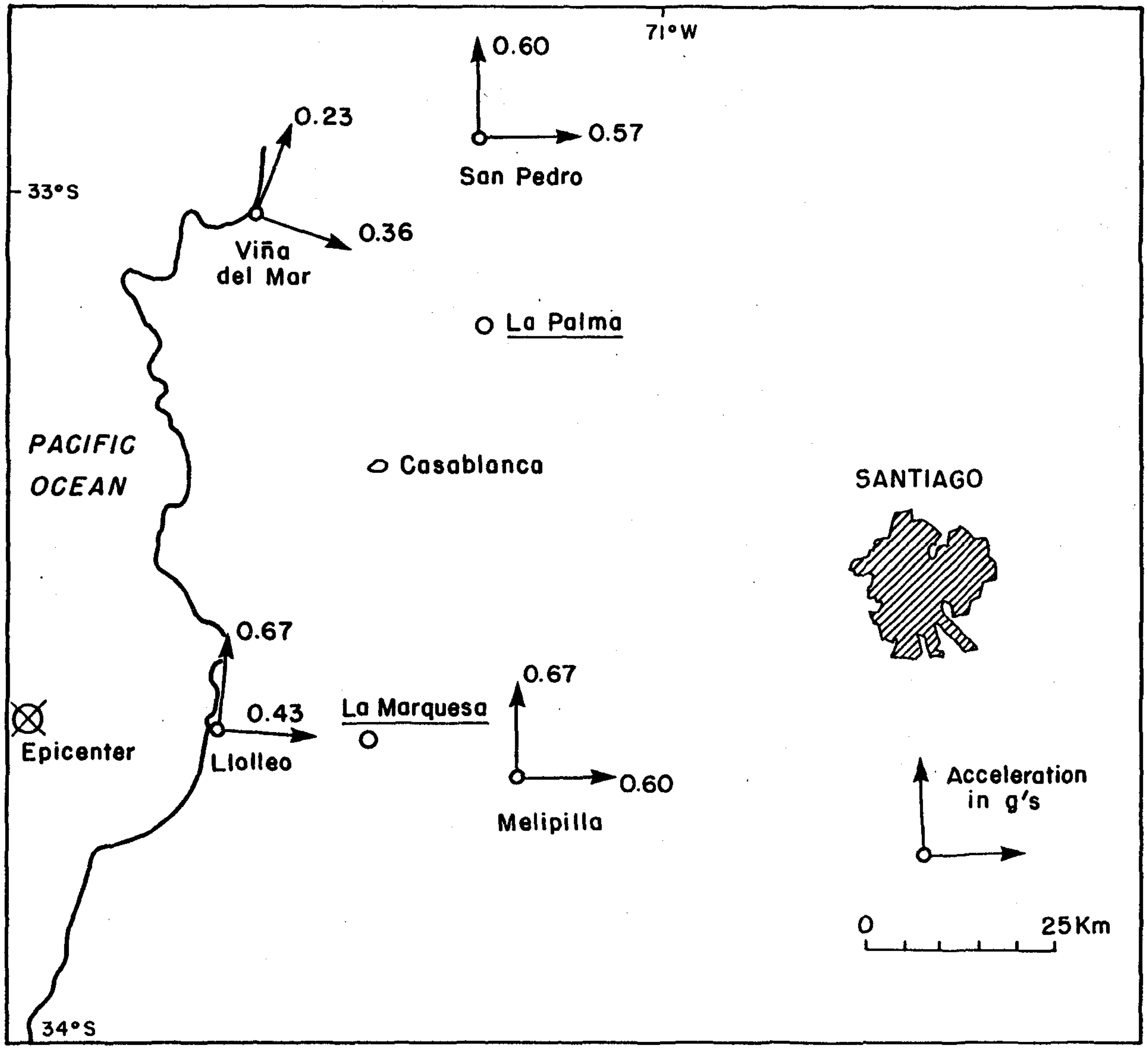


Fig. 1 MAP OF CENTRAL CHILE SHOWING EPICENTRAL AREA AND RECORDED PEAK ACCELERATIONS IN EARTHQUAKE OF MARCH 3, 1985
(from Saragoni et al., 1986)

and values of peak ground accelerations recorded at those stations closest to La Marquesa and La Palma dams.

As shown in Fig. 1, the recording stations closest to La Marquesa Dam are Llolleo and Melipilla; the dam is roughly midway between these stations on an East-West line, and at a distance of 45 km (28 mi.) from the epicenter. Peak accelerations recorded at these stations were as follows (Saragoni et al., 1986):

Llolleo (19 kms from the dam-site):	N10E:	0.67g
	S80 :	0.43g
	Vertical:	0.85g
Melipilla (20 kms from the dam-site):	NS:	0.67g
	EW:	0.60g
	Vertical:	0.59g

On the basis of these recordings, it seems reasonable to consider that the peak horizontal ground acceleration in the vicinity of La Marquesa was about 0.6g.

The level of peak acceleration at La Palma Dam is not as well defined as it was at La Marquesa. Fig. 1 shows the location of the dam, and the peak accelerations recorded at the two closest recording stations:

Viña del Mar (31 kms from the dam-site):	N70W:	0.23g
	S20W:	0.36g
	Vertical:	0.17g
San Pedro (25 kms from the dam-site):	NS:	0.60g
	EW:	0.57g
	Vertical:	0.38g

The damage observed at the nearest town, Casablanca, indicated a MM intensity of VIII. Retamal et al. (1986b), based on the recorded accelerations and

Chilean attenuation laws, proposed that the acceleration at the site was about 0.46g.

3. Performance and Evaluation of La Marquesa Dam

Performance of Dam During Earthquake

La Marquesa Dam had a pre-earthquake maximum height of 10 m (33 ft), a crest length of 220 m (722 ft) and a storage capacity of 204,000 cu. m. (267,000 cu.yd.). The existing dam was built in 1943 over the remains of an older dam that was overtopped during a flood in 1928. Construction materials were the local silty and clayey sands borrowed from the reservoir area and the abutments; the more plastic material was placed in the center of the section to form a core. The last recorded modifications were made in 1965, when the embankment was raised by about 1.5 m (5 ft.).

During the 1985 earthquake, the dam experienced major sliding of both the upstream and downstream slopes, with the largest displacements occurring in the upstream slope. The resulting loss of freeboard was about 2 m (6.6 ft.) in the middle one-third of the embankment, thus dropping the crest of the dam to the same elevation as the spillway crest. An emergency breach was opened near the right abutment and the reservoir was emptied within a few days of the earthquake. Subsequent surface inspection showed an extensive pattern of longitudinal cracks, especially in the upstream slope, with crack widths of up to 0.8 m (2.6 ft.) and depths of up to 2 m (6.6 ft.). Fig. 2 shows cracking in the upstream slope. Horizontal displacements in the zone of greatest damage reached about 11 m (36 ft.) at the toe of the upstream slope and 6.5 m (21 ft.) at the toe of the downstream slope.

Fig. 2 CRACKING AND SLUMPING IN UPSTREAM SLOPE OF LA MARQUESA DAM

Field Explorations

Explorations at La Marquesa concentrated on a cross-section through the most damaged part of the embankment, section A-A in Fig. 3. Results from two studies were used to define the internal geometry at this section. Retamal et al. (1985) had reported results from a test pit and two SPT-borings carried out approximately at this location during studies for a possible reconstruction of the dam. To confirm and extend these results, an additional four SPT-borings and a test pit were made in the Summer of 1986 under the direction of the writers. Three of these borings, B-1 to B-3, were made along section A-A; an additional boring, B-4, was located 28 m (92 ft) away from this section, outside the major damage zone.

All the 1986 borings were made using a Joy 12B skid-mounted drill rig with a separate tripod. Drilling was carried out with a 3.875-in. tricone bit using water as the drilling fluid. The hole was provided with 4-in ID casing. A conventional donut hammer was used for the SPT-tests; rods were AW size, and the spoon was provided with a liner. Borings B-II and B-III of the 1985 study was carried out with similar equipment, except that the spoon was unlined.

Recent studies by Seed et al. (1985) have emphasized the need for measuring the energy delivered by the SPT hammer to the drill rods. A Binary Instruments model 102 SPT calibrator was used during the 1986 explorations to measure the imparted energy. A total of 8 energy tests were carried out in two different borings with an average of 15 energy measurements in each test. The results, expressed as an energy ratio, ER (fraction of maximum theoretical free-fall energy), ranged from ER = 52% to ER = 59%, with a mean value of 56%.

Figure 4 summarizes the results of the two exploration programs. SPT N-values have been normalized, as suggested by Seed et al., for the effects

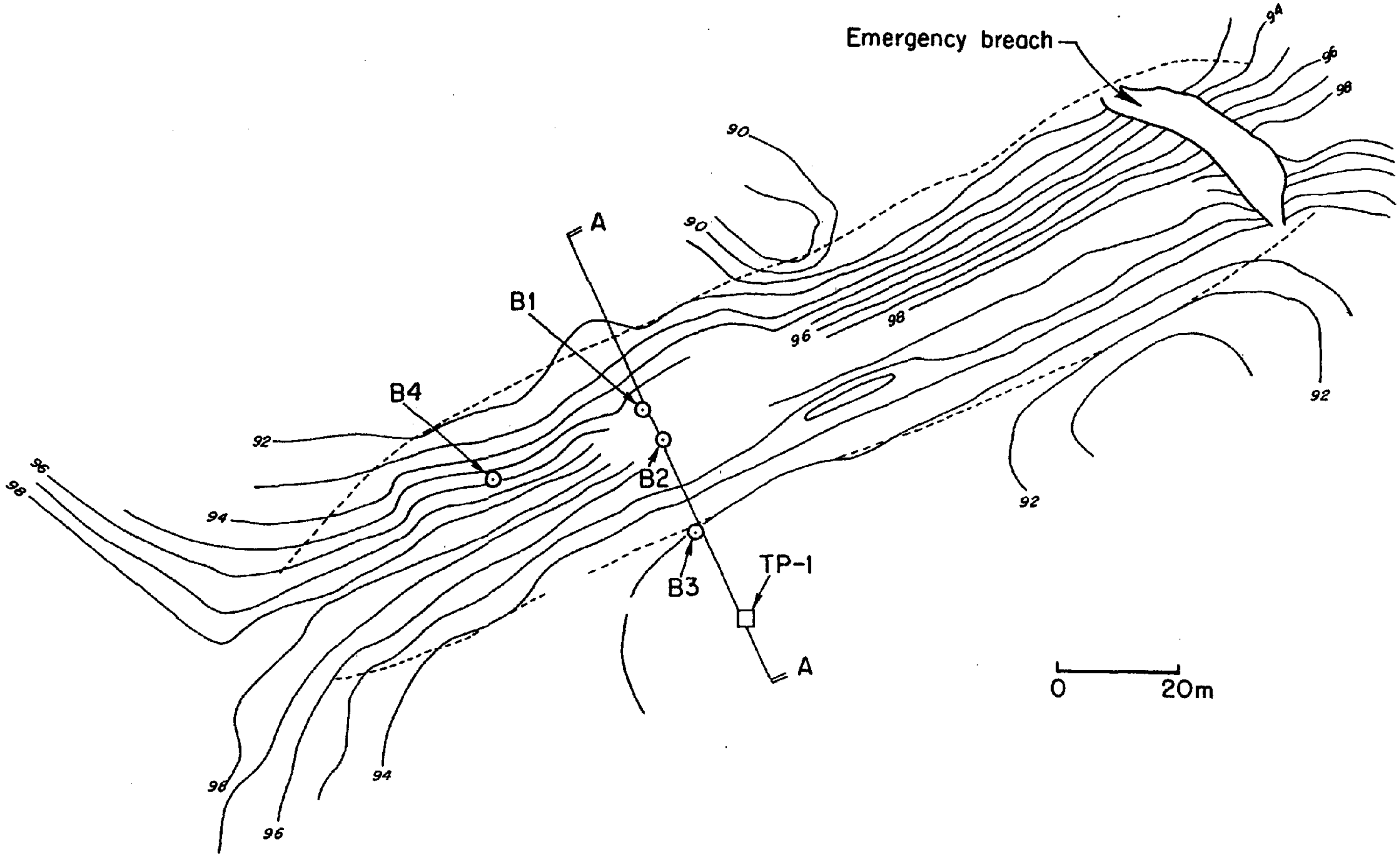


Fig. 3 PLAN OF LA MARQUESA DAM SHOWING BORING LOCATIONS

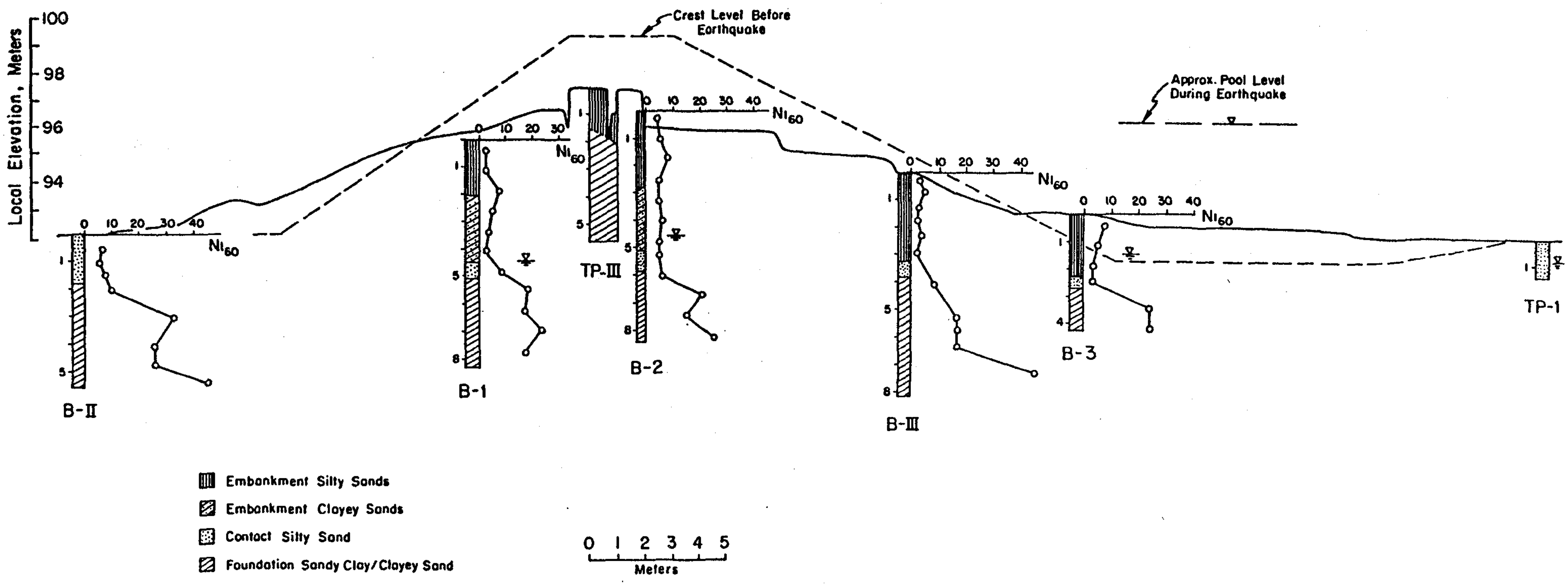


Fig. 4 CROSS-SECTION THROUGH FAILURE ZONE OF LA MARQUESA EMBANKMENT

of vertical effective stress and hammer energy. Blowcounts shown are those that would be observed for a vertical effective stress of 1 tsf, calculated as for level ground, and a hammer energy of 60%, i.e. $(N_1)_{60}$. In addition to the $(N_1)_{60}$-values, the fines content and plasticity index of each split-spoon sample were determined in the investigation program.

The most easily defined layer in the foundation is the clayey sand/sandy clay which appears at about local elev. 90.5 meters (referred to subsequently as the base sandy clay). Blowcounts, fines content and plasticity all increase simultaneously in this layer. This clay is a residual material, and it is exposed in the abutments and in the main channel upstream of the dam. Immediately above the base sandy clay, a layer of silty sand was identified in all the borings of section A-A; this will be referred to subsequently as the contact silty sand.

Figure 5 compares downstream boring B-1 of section A-A with boring B-4, also located on the downstream slope, but away from the major damage zone, at about 28 meters (92 ft.) from B-1. A boring on the upstream slope would have been more desirable, but it was not possible to locate an undamaged portion of the upstream slope that was not practically at the abutment. The downstream slope at B-4 showed only minor cracking, with no significant movement. It is interesting to observe that what is evidently the same contact silty sand appears above the base sandy clay in Boring B-4, although the thickness of this layer at this location is about 1.4 m (4.6 ft), compared to 0.75 m (2.46 ft) in boring B-1.

Erratically-distributed silty and clayey sands were encountered above the contact silty sand at section A-A, constituting the remains of the embankment itself. Despite the fact that these materials are moved significantly

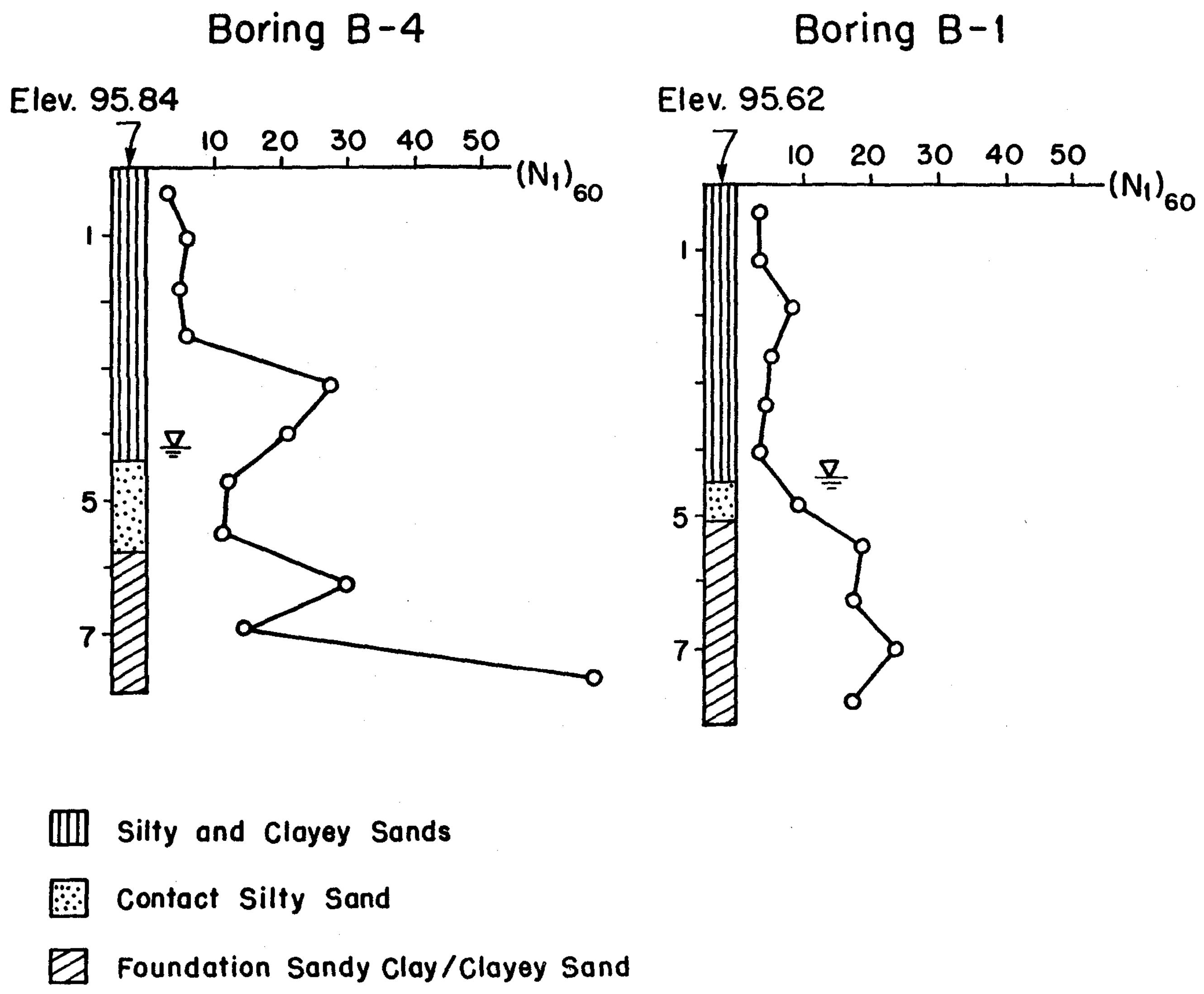


Fig. 5 COMPARISON OF BORINGS B-1 AND B-4 AT LA MARQUESA DAM

from their original locations, their distribution permits some conclusions to be drawn about the internal geometry of the embankment. In test pit TP-III at the crest of the dam, a sandy clay analogous to the foundation sandy clay was found under about 1.7 m (5.6 ft) of silty sand. This upper sandy material probably represents the most recent (1965) raising of the embankment, while the lower clayey material is part of the older (1943) dam. This lower plastic material is probably the same sandy clay encountered at about 2.8 m (9.2 ft) in boring B-2, above the contact silty sand.

Figure 6 shows a reconstruction of section A-A based on the exploration results. The existence of a relatively narrow core zone of more plastic material at the center of the dam was confirmed by borings B-2 and TP-III, as well as by the damage pattern observed at the crest of the dam. This core location is also consistent with the zoning specified in the 1943 reconstruction plans.

The core is surrounded by silty and clayey sand shells; these shell zones were built directly on the contact silty sand, whereas this sand was removed under the core. This removal was called for in the specifications for the 1943 reconstruction, and is confirmed by the fact that the dam operated for many years without significant seepage at the downstream toe.

The reconstructed section in Fig. 6 shows a somewhat larger thickness of the contact silty sand than that reported in the borings, where a maximum value of 0.8 m (2.6 ft) was encountered under the embankment. Study of the damage pattern suggests that the large horizontal and vertical upstream deformations were probably due to liquefaction of the contact silty sand as will be discussed subsequently. The assumption of a larger pre-earthquake thickness for the contact silty sand under the major damage section is supported by the 1.4 m (4.6 ft) thickness of this material in boring B-4 outside the major

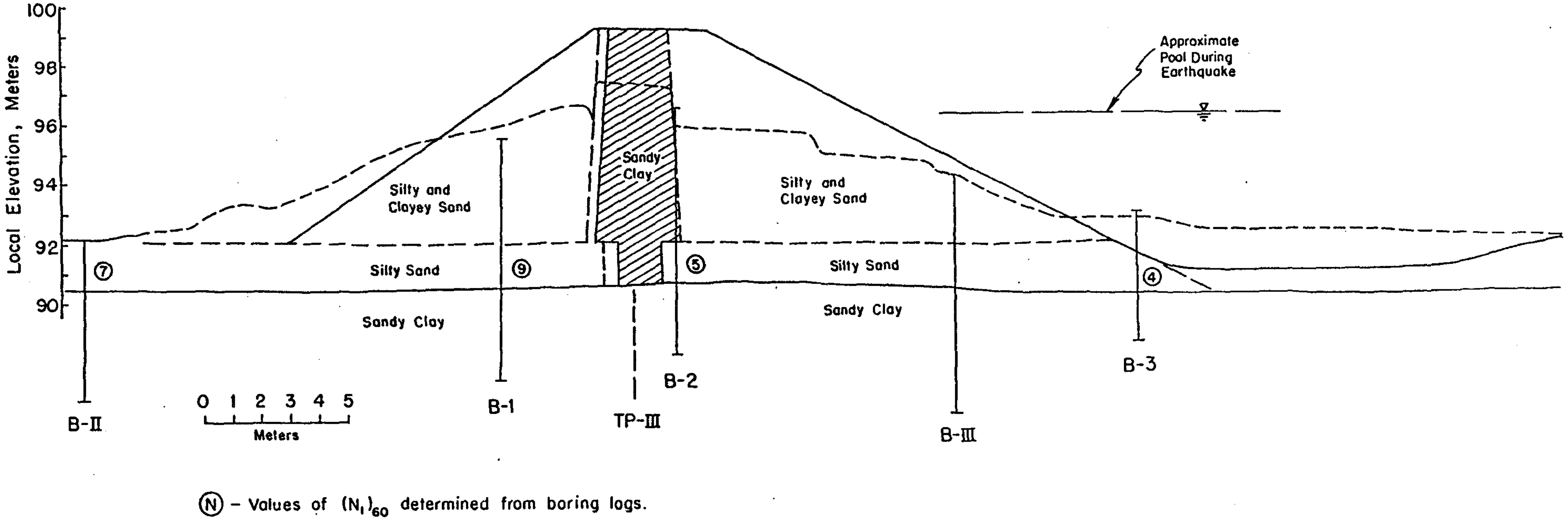


Fig. 6 RECONSTRUCTED CROSS-SECTION THROUGH FAILED PORTION OF LA MARQUESA EMBANKMENT

damage zone, by the 1.7 m (5.6 ft) thickness measured in B-II beyond the downstream toe, and by the 1.4 m (4.6 ft) of sand found in test pit TP-1 upstream of the embankment. The reconstructed upstream slope is shown extending down below the top of the silty sand layer, since it appears that the sand layer immediately upstream of the dam was removed and placed in the upstream slope, a practice observed at other small dams in the area.

This belief is consistent with the large deformations of the upstream slope, and it accounts for the large volume which is now missing from the upper part of the pre-earthquake section, (see Fig. 6), while remaining consistent with the observed surface disruption beyond the toe of the slope.

Under the downstream slope, the contact silty sand was somewhat denser and the volume missing from the upper part of the pre-earthquake section approximately matches the volume of present surface disruption (above the pre-earthquake ground surface elevation) beyond the downstream toe.

It is of interest to compare the values of $(N_1)_{60}$ measured in the silty sand layer in Borings B-1, in the major damage zone, and B-4 where damage was minor, (see Fig. 5). The $(N_1)_{60}$ value measured in Boring B-4 was 11 bpf and that in Boring B-1 was 9 bpf, a difference of only 2 bpf. The fines content was similar at both locations. From this comparison, it would appear that pre-earthquake blowcounts under the embankment may not have been substantially different from the values measured after failure, and that as a first approximation these latter values can be considered indicative of the pre-earthquake values. It is also important to note that the data from boring B-II of the 1985 study is not considered representative of conditions under the embankment. The surface disruption pattern beyond the downstream toe suggests that liquefaction did not extend as far as this boring. While blowcounts at this location are comparable to those under the embankment it should be noted that the fines content (37%) and, more importantly, the plasticity of the fines (PI = 11%) are significantly greater in this area than in the contact silty sand at other locations under the embankment, and the clayey nature of this soil may well have prevented liquefaction from occurring at this location.

Based on the results of the field investigations it was estimated that the average $(N_1)_{60}$ value of the contact silty sand under the upstream shell was about 4 and the fines content was about 30%. Allowing for the effects of the fines, as proposed by Seed (1987), the equivalent clean sand values of $(N_1)_{60}$ for this sand is thus about 6.

For the silty sand under the downstream shell, the measured value of $(N_1)_{60}$ was about 9 and the fines content about 20%, leading to an equivalent clean sand value of about 11.

Analyses of Failure

Based on the field investigation described above, it was a simple matter to calculate that the inertia forces induced by earthquake motions with a peak acceleration of 0.6g in the contact layer of silty sand with a $(N_1)_{60}$-value of 6 to 11 would induce liquefaction early in the period of earthquake shaking. Thus the postulated failure mechanism for both the upstream and downstream slopes was essentially as follows: The contact silty sand layer on which the main body of the embankment was constructed liquefied soon after the outset of the strong motions. Under the upstream slope, where the silty sand was looser, a major flow of this liquefied material took place, driven mainly by the weight of the embankment. The embankment material, which generally contained more plastic fines, probably had not liquefied at this point and instead broke up into blocks, which slid downwards and outwards on the liquefied layer. A similar mechanism is assumed to have operated in the downstream slope; in this zone, however, the contact silty sand was denser and thus movements were somewhat smaller (about 6.5 m (21 ft) of horizontal movement at the downstream toe as opposed to 11 m (36 ft) upstream).

The relative simplicity of the sliding mechanism involved in such a failure made it possible to attempt to evaluate the residual strength of the contact silty sand after liquefaction occurred, or at least to establish reasonable upper and lower bounds for the residual strength of the liquefied soil. In an analysis of this type, allowance had to be made for the fact that both the upstream and downstream slopes suffered major changes in geometry during the failure and that liquefaction of the base sand layer caused the embankment shell zones to become at least partially uncoupled from the base motion. Two extreme cases were thus postulated for analysis:

(a) Examination of potential failure surfaces involving the original pre-earthquake section of the embankment. This case represents an upper bound to the gravity driving forces acting on the sliding mass, which subsequently spread out horizontally; the embankment was also considered to be coupled loosely to the foundation, so that some inertia forces due to earthquake accelerations below the base of the embankment were also included in the analyses.

(b) Examination of the final configuration of the slopes after the earthquake. In this case, different potential failure surfaces were studied to determine a lower-bound residual strength, i.e. that required to obtain a safety factor of unity immediately after the earthquake.

Figure 7 summarizes the different potential failure surfaces considered in the case (a) analyses for the upstream and downstream slopes. As

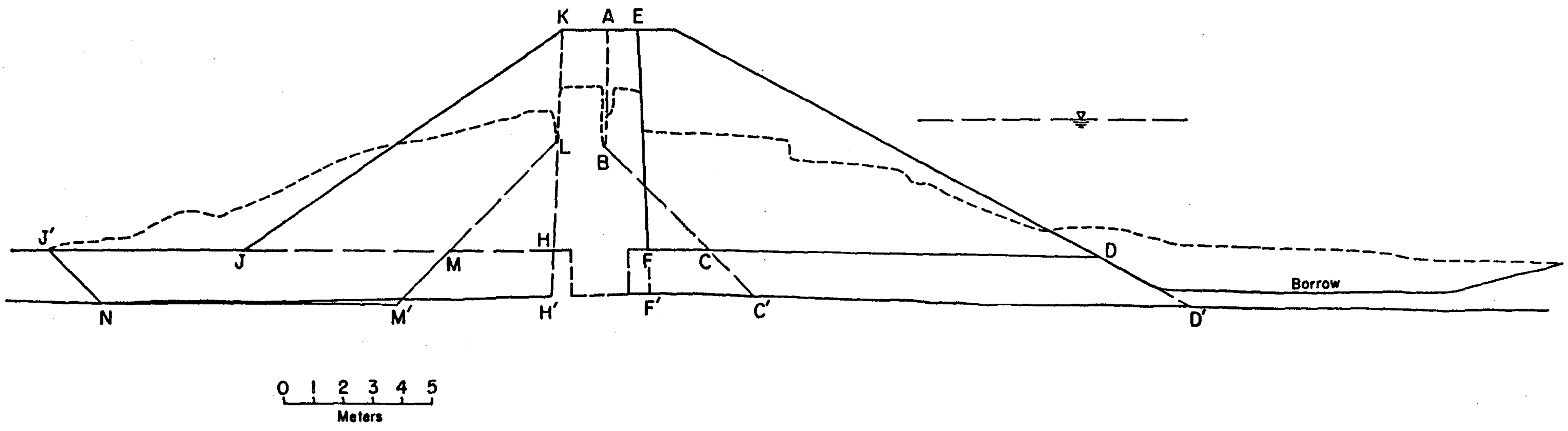


Fig. 7 POTENTIAL SLIDING SURFACES THROUGH ORIGINAL SECTION OF LA MARQUESA EMBANKMENT

previously noted, both the effects of the gravity forces and the earthquake-base motions were considered using a Newmark-type dynamic deformation analysis. Deformations were considered to be indicated by the simplified charts of Makdisi and Seed (1977). Thus, given the measured horizontal deformations and the free-field acceleration, it was possible to back-calculate an approximate value for the yield acceleration, k_y, defined as the average acceleration (as a fraction of g) producing a horizontal inertia force on the potential sliding mass which induced a condition of incipient failure defined by a factor of safety of unity. Accelerations above the yield acceleration will cause the mass to experience permanent displacements. It was felt that an upper bound for the residual strength of the liquefied soil might be obtained by this approach.

In the case of the upstream slope at La Marquesa, the horizontal displacement was estimated to be about 11 m (36 ft). Based on the recorded accelerations at the nearest stations, a peak horizontal ground surface acceleration of 0.6g was adopted for the site. For these values, a yield acceleration $k_y = 0.012g$ was obtained. A first series of calculations was then carried out simply assuming that the full effective at-rest earth pressure of the core acted against the shell along the core/shell boundary, EF. This force together with the force corresponding to the yield acceleration and resisted by the residual strength of the contact silty sand acting along the surface FD, were considered to bring the slide mass to a condition of failure. The calculated yield acceleration and the core earth pressure force (assuming an at-rest earth pressure coefficient of 0.5 and a soil unit weight of 1.93 ton/m^2) were applied to the sliding wedge EFD, leading to a computed residual strength along FD of 320 psf. A similar value, of about

330 psf, was obtained for wedge E F'D', assuming sliding along the top of the base sandy clay.

A second series of type (a) analyses was carried out assuming failure along surfaces ABCD and ABC'D' in Fig. 7, suggested by the damage pattern of the upstream slope, as shown. These were simple sliding wedge analyses, carried out considering a uniform residual strength along BCD or BC'D' and the same yield acceleration previously calculated. Residual strength values on the order of 290 to 340 psf were obtained by these analyses. These values represent an upper bound to the residual strengths that might be calculated for these surfaces, since if a higher undrained strength is assumed for the part of the failure surface that passes through the more plastic embankment materials along BC, then the residual strength required for equilibrium along CD or C'D' will be lower.

The same type of analysis was carried out for the downstream slope, considering the surfaces indicated in Fig. 7. In this case, for a horizontal deformation at the toe of about 6.5 m (21 ft), a yield acceleration of 0.03g was determined and used to quantify the residual strength of the liquefied silty sand. The results of these analyses for the downstream slope led to a computed upper limit for residual strength of 580 psf.

Type (b) analyses, to determine the residual strength required for a factor of safety of unity once earthquake shaking had stopped, were carried out for the surfaces shown in Fig. 8 and the results are summarized in Table I. Again, a uniform strength was assumed along the sliding surfaces considered. In this case, it is clear that very low strengths could be obtained, depending on the sliding surface considered; however, the values of interest are those consistent with the failure mechanisms proposed. For

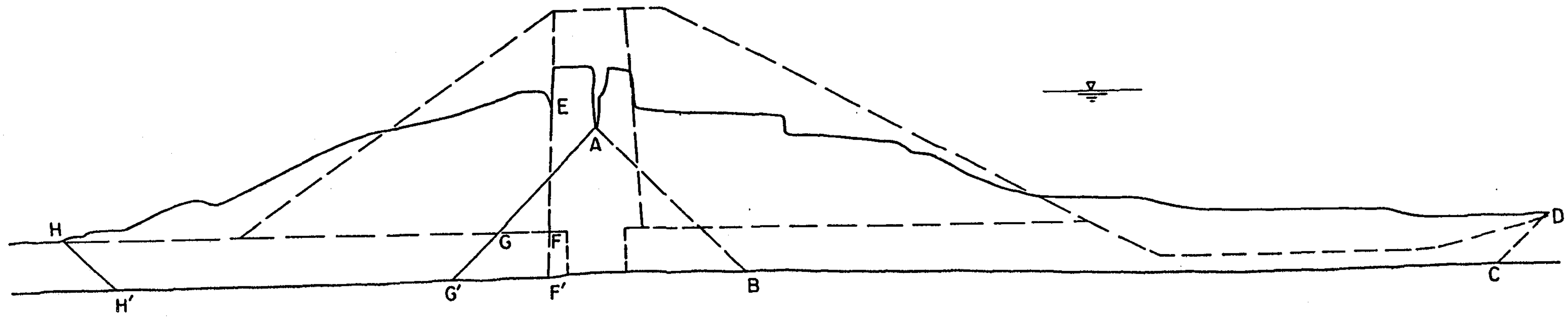


Fig. 8 POTENTIAL SLIDING SURFACES FOR POST-EARTHQUAKE STABILITY ANALYSIS OF LA MARQUESA EMBANKMENT

Table I

Residual Strength Calculations for La Marquesa Dam -- Post-Earthquake Section

(a) Upstream Slope

Surface	Residual Strength (psf)
ABCD	76

(b) Downstream Slope

Surface	Residual Strength (psf)
A G'H'H	266
E F'H'H	164
AGH	190
EFH	152

this section, these values were 266 psf for the downstream slope and 76 psf for the upstream slope.

Based on these analyses it was concluded that the residual strength of the liquefied silty sand could be expected to be within the following ranges:

Base of upstream shell:	76 to 340 psf
Base of downstream shell:	266 to 580 psf

4. Performance and Evaluation of La Palma Dam

Performance of Dam During Earthquake

La Palma Dam had a pre-earthquake maximum height of 10 m (33 ft), a crest length of 140 m (459 ft) and a storage capacity of 56,000 cu m (73,300 ft). This dam was over 50 years old at the time of the 1985 earthquake. During the earthquake, major sliding took place in the upstream slope, while the downstream slope remained relatively undamaged. Fig. 9 shows the dam shortly after the earthquake.

The damage pattern observed upstream is somewhat similar to that at La Marquesa Dam; the middle third of the embankment suffered major displacements, with the upstream toe moving out about 5 m (16 ft) and the failed embankment zone breaking up into blocks between longitudinal cracks. A major crack 80 m (260 ft) long with a maximum width of 1.2 m (4 ft) developed along the crest; in the area of greatest damage the upstream side of the crack settled more than 2.6 ft (80 cm) relative to the downstream side as shown in Fig. 10. A second major 60-m (197 ft)-long crack developed about 2 m (6.6 ft) below the crest in the upstream slope, with a surface width of about 80 cm (2.6 ft) and a drop of 1.5 m (4.9 ft) between the downstream and the upstream sides. Although the water level in the dam was relatively low at the time of the

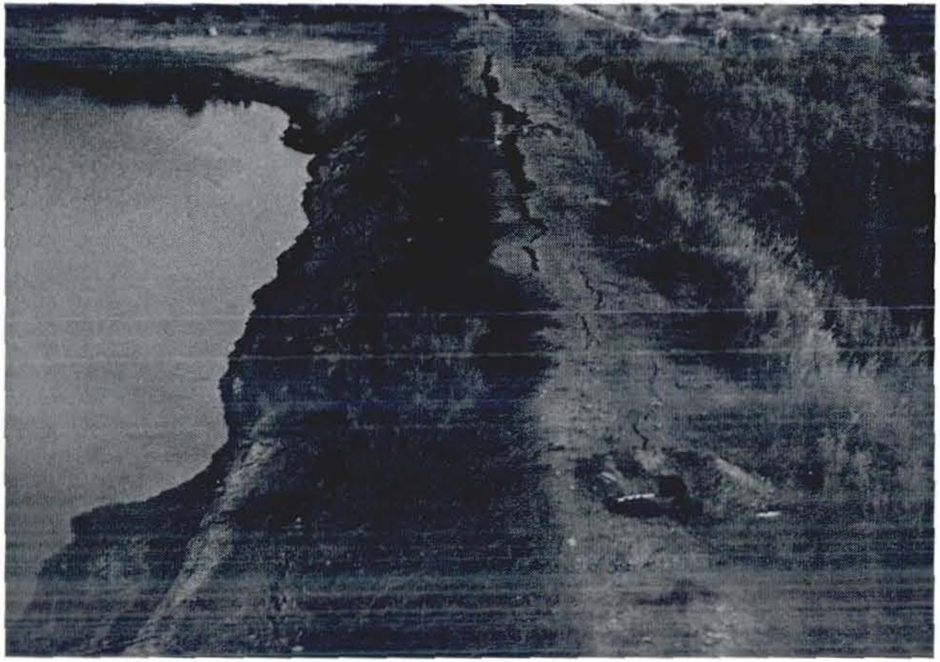
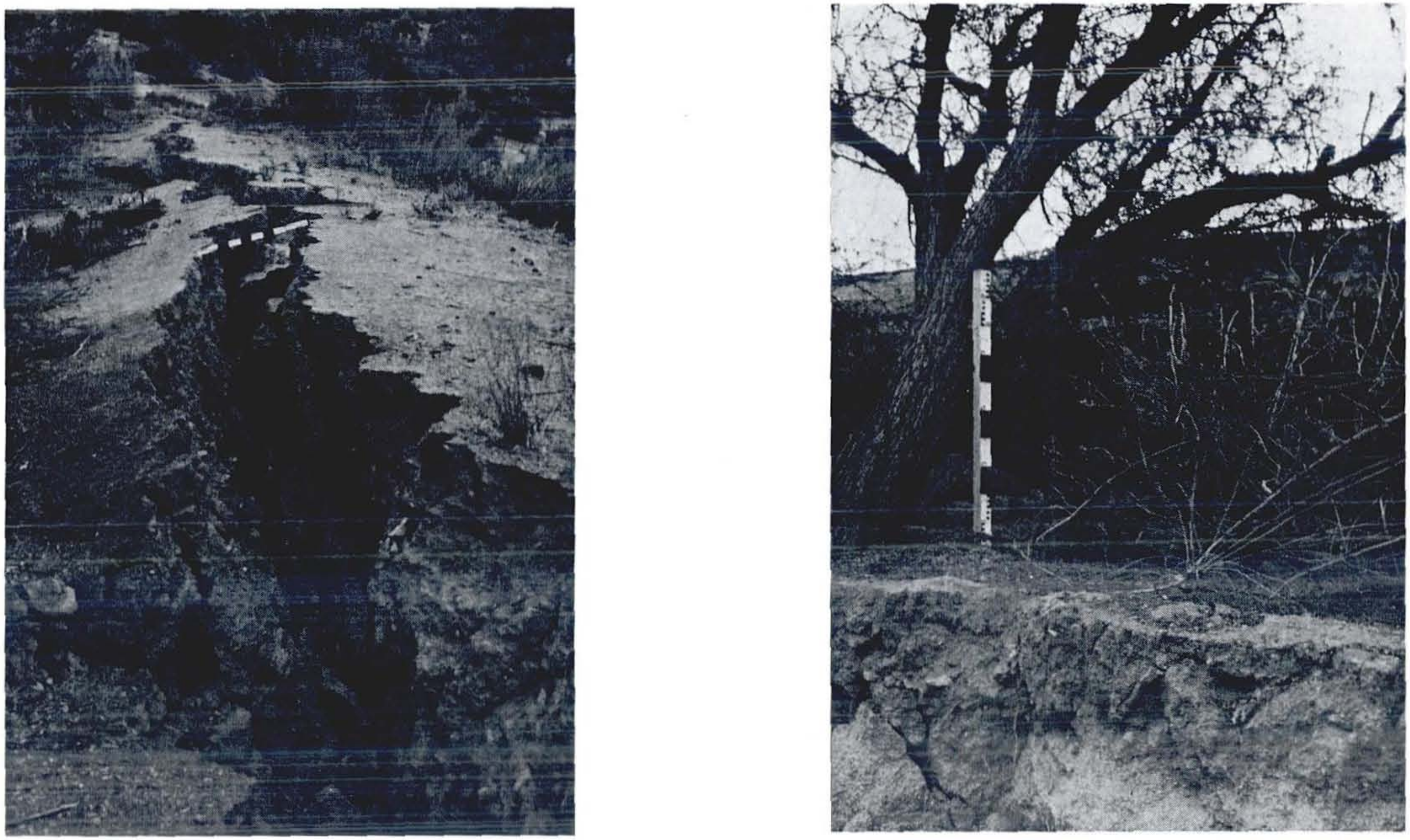


Fig. 9 OVERALL VIEW OF LA PALMA DAM SHORTLY AFTER EARTHQUAKE



(a) Cracking in Slope of Dam

(b) Slumping of Crest of Dam--tree was formerly on crest of embankment

Fig. 10 CRACKING AND SLUMPING IN UPSTREAM SLOPE OF LA PALMA DAM earthquake, the embankment was breached shortly thereafter to prevent any sudden release.

Field Exploration

Exploration at La Palma Dam consisted of five SPT borings made in 1986; Borings B-1 to B-4 were carried out along cross-section BB in the major damage zone of the dam, as shown in Fig. 11. Judging from the topography, this section is in the old streambed, where major damage was concentrated. The last boring, B-5, was made at a distance of about 35 m (115 ft) towards the right abutment, in a zone where damage was minor. Borings were carried out by the same crew, and using the same procedures, as at La Marquesa Dam. The water level in the reservoir at the time the borings were made was only 50 cms (1.6 ft) below that at the time of the earthquake.

The soil profile and the results of SPT tests for Borings B-1, B-2, B-3 and B-4 along section BB are summarized in Fig. 12. Measured N-values were normalized as for La Marquesa Dam and the results are presented in terms of $(N_1)_{60}$. Plasticity index and fines content were determined for each split-spoon sample.

From these results, it was concluded that the contact between the embankment and the original ground surface was defined by the black and grey silts in Borings B-3 and B-4, and by the brown sandy clay in Boring B-1. No clearly defined contact could be found in Boring B2; this was taken to indicate stripping to an impervious base for the core in this area. This belief is supported by the water levels observed in the borings during exploration, which are indicated in Fig. 12. The water levels in Borings B-2 to B-4 are approximately the same as the pool level, while in boring B-1 the water is

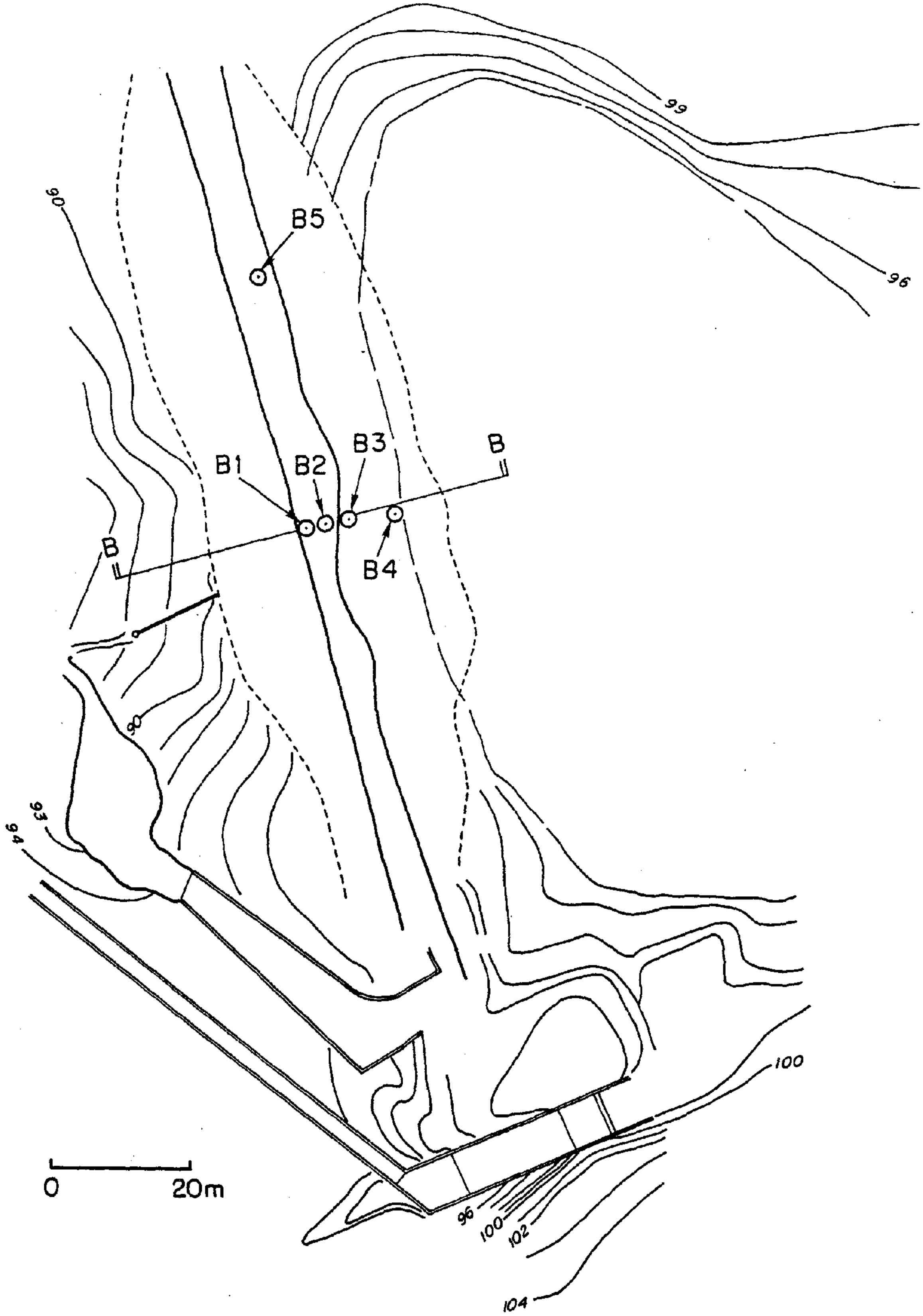


Fig. 11 PLAN OF LA PALMA DAM SHOWING BORING LOCATIONS

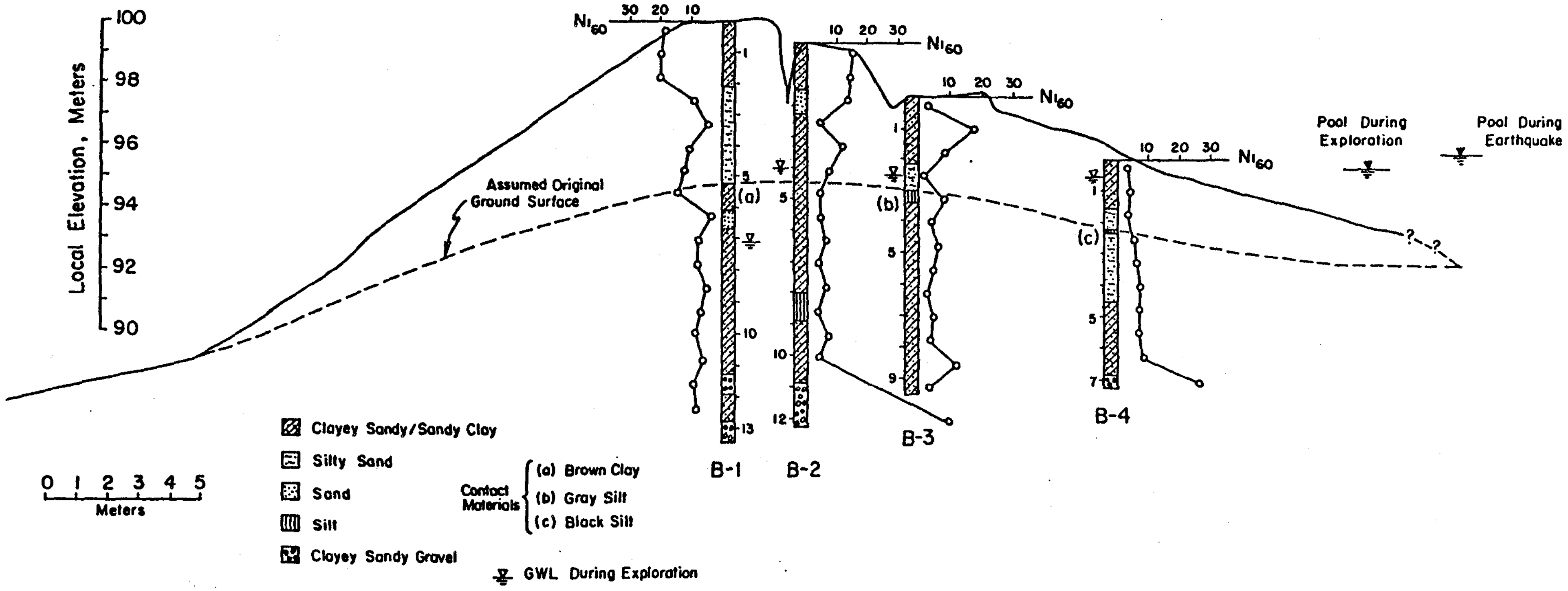


Fig. 12 CROSS-SECTION THROUGH FAILURE ZONE OF LA PALMA EMBANKMENT

considerably lower, indicating the effectiveness of the core and the foundation contact in preventing seepage.

A comparison of the results of Borings B-3 and B-4 in the slide area with Boring B-5 outside the slide area, shown in Fig. 13, indicates what is apparently the same loose silty sand layer forming the base of the embankment. This layer is somewhat denser at B-5 where $(N_1)_{60}$ = 5 bpf, as compared with 2 bpf at B-3 and 3 bpf at B-4, and it was not submerged at B-5 during the earthquake, the base of the embankment being at a considerably higher elevation at this location. Since it was not saturated, the silty sand at B-5 could not have liquefied during the earthquake. It should also be noted that the silty sands at B-5 and B-4 have practically the same fines content, and thus the N_1 $_{60}$-values are directly comparable; it may be argued on the basis of this comparison of the liquefied and non-liquefied zones that the penetration resistance of the silty sand did not change significantly as a result of liquefaction.

Based on the field studies it was concluded that the saturated loose silty sand at the base of the upstream shell had a very low penetration resistance $(N_1)_{60}$ of about 3 and a fines content of about 15%, giving an equivalent clean sand value of $(N_1)_{60} \approx 4$.

Analyses of Failure

Figure 14 shows a reconstructed section of the embankment based on the results of the exploration program. Basically, the dam is rather similar in section to La Marquesa, with a sandy clay core supported by clayey and silty sand shells. The upstream shell and part of the downstream shell are underlain by a layer of loose silty sand. This layer is significantly looser than the underlying silts and silty sands, and it is highly probable that this is

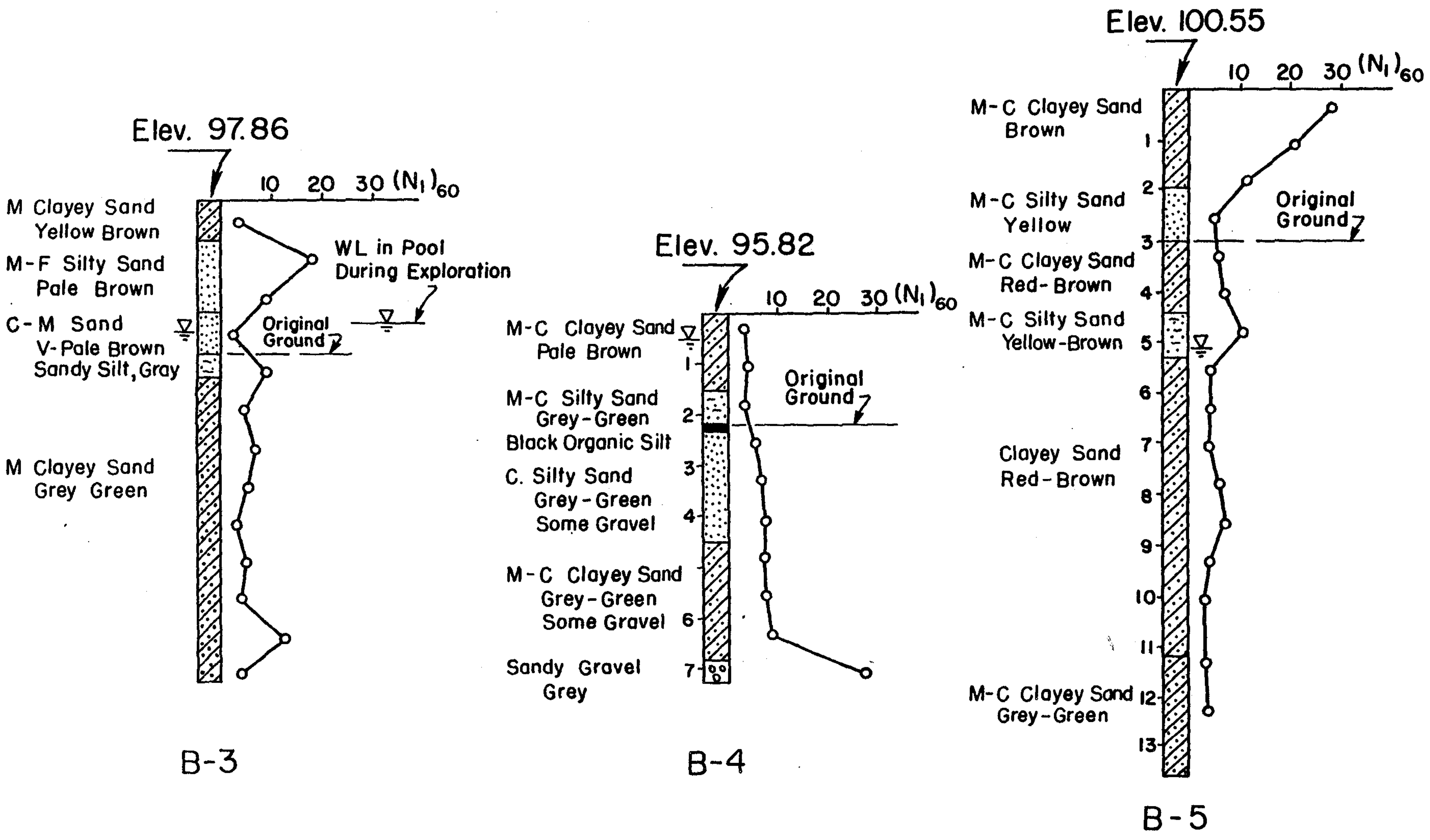


Fig. 13 LOGS OF BORINGS AT LA PALMA DAM

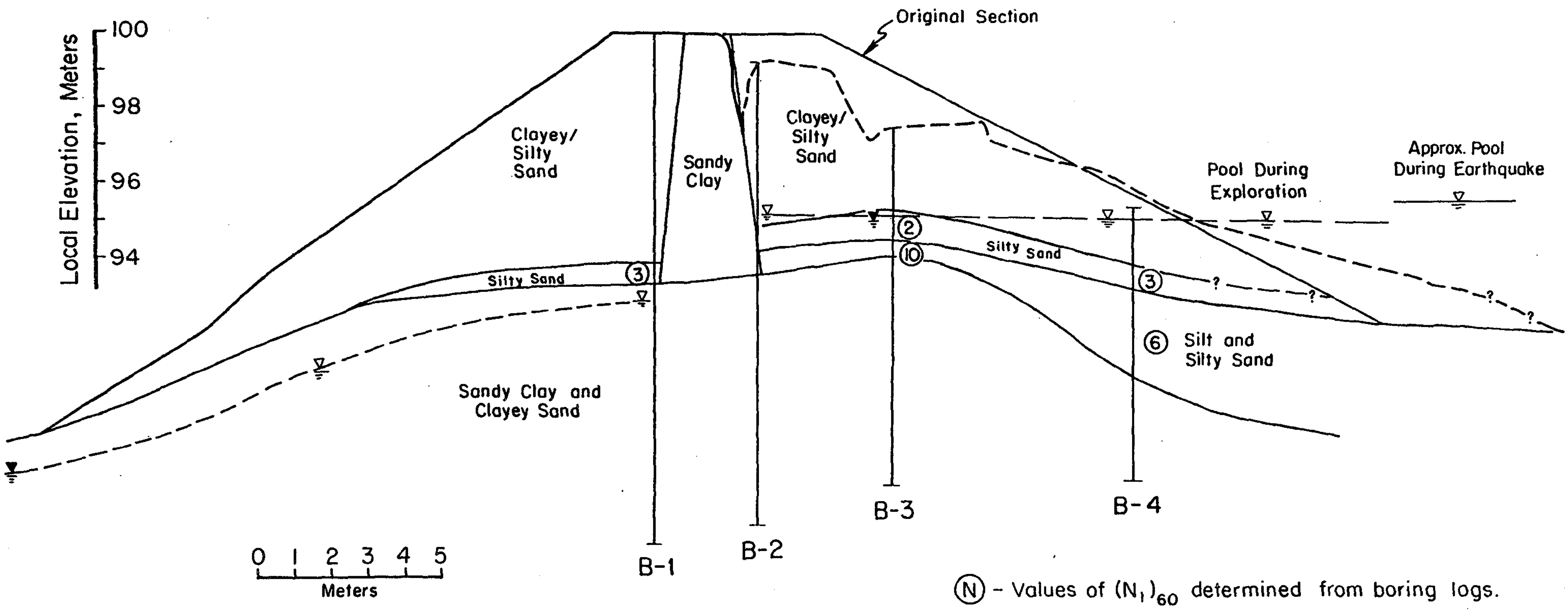


Fig. 14 RECONSTRUCTED CROSS-SECTION THROUGH FAILED PORTION OF LA PALMA EMBANKMENT

the layer that liquefied during the earthquake, resulting in the failure of the upstream slope. Liquefaction probably did not occur under the downstream shell because the loose silty sand was above the water table in this area.

Stability analyses analogous to those at La Marquesa were carried out for the upstream shell of La Palma Dam: (a) considering the original section of the dam and including earthquake-induced inertia forces, and (b) considering the final configuration of the damaged section and no inertia forces.

Fig. 15 shows the potential failure surfaces chosen for the case (a) analyses; as at La Marquesa Dam, wedge ABC was considered to have full effective at-rest earth pressure acting along AB with resistance provided by the residual strength of the liquefied sand acting along BC. For a horizontal toe displacement of 5 m and a free-field acceleration of 0.46 g a yield acceleration k_y = 0.027 g, was determined from the Makdisi-Seed deformation charts and a residual strength value of 295 psf was calculated for this case. An alternate surface suggested by the surface damage pattern, ADEC, was also studied using a sliding-wedge analysis procedure and assuming a uniform strength along the surface. A similar residual strength value, of 300 psf, was obtained in this case. Again, this would be an upper bound for the section ADEC since, if a higher value of undrained strength is assumed along DE in the embankment material, the residual strength along EC will be significantly lower than the computed value.

Case (b) analyses for the upstream slope at La Palma are summarized in Fig. 16 and Table II; lower bound values of residual strengths on the order of 120 psf were obtained, as shown.

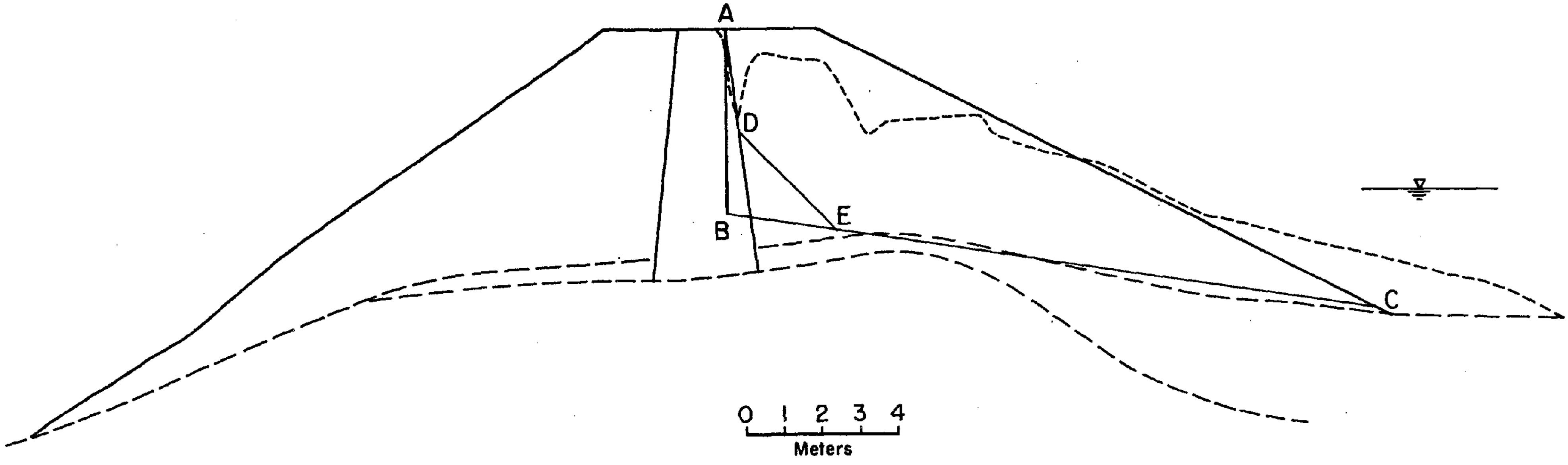


Fig. 15 POTENTIAL SLIDING SURFACES IN UPSTREAM SHELL OF LA PALMA DAM BASED ON ORIGINAL CROSS-SECTION

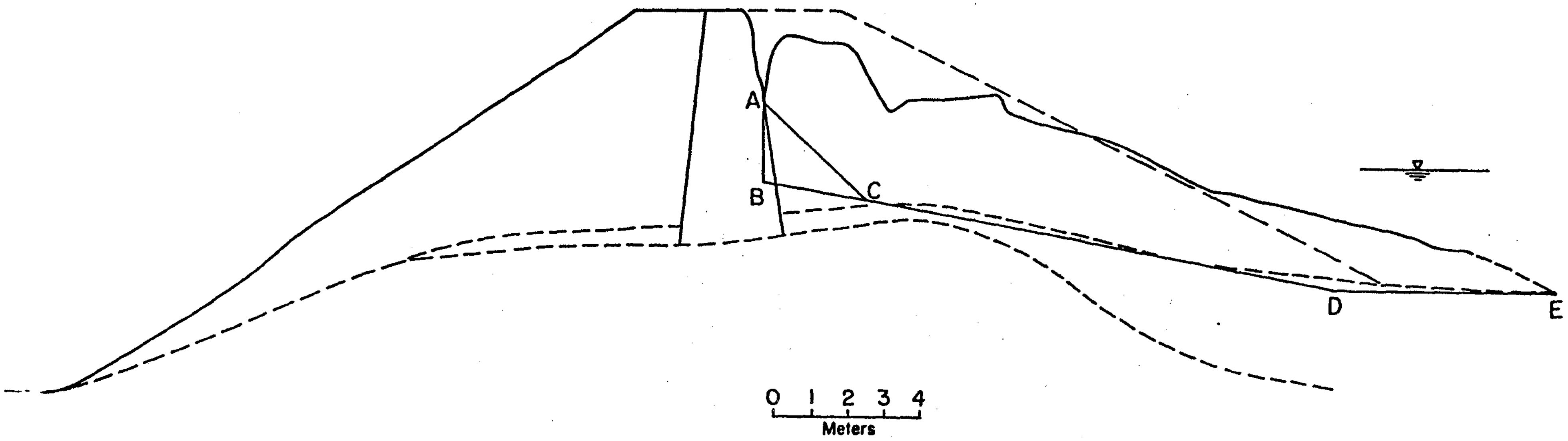


Fig. 16 POTENTIAL SLIDING SURFACES FOR POST-EARTHQUAKE STABILITY ANALYSIS OF UPSTREAM SHELL OF LA PALMA DAM

Table II

Residual Strength Calculations for La Palma Dam, Upstream Slope --Post Earthquake Section

Surface	S_{res} (psf)
ABDE	115
ACDE	127

Based on both sets of analyses it was concluded that the residual strength of the liquefied sand at the base of the upstream shell could be expected to lie within the range of 120 to 300 psf.

5. Conclusions

The slope failures at La Marquesa and La Palma Dams in the Chilean earthquake of March 3, 1985 were apparently due to liquefaction of loose sand layers near the base of the embankments. The dams were low structures (4 to 10 m high) and the horizontal movements were substantial. However the configuration of the embankments after the failures suggests that the liquefied soil retained a small but significant strength after liquefaction ($r_u \approx 100\%$) occurred.

Field explorations provided a basis for assessing the characteristics of the liquefied sands before the earthquake occurred and analyses of the configuration of the slide material provided a basis for evaluating the probable ranges of residual strengths for the liquefied soils. The highest values for residual strength were obtained by assuming the driving forces to be the initial at-rest earth pressure of the core and the development of small inertia forces produced by the earthquake; similar, but generally slightly lower values, were obtained considering other failure surfaces suggested by the surface cracking and deformation patterns. For the latter analyses, it was necessary to assume a uniform residual strength along the failure surface, resulting in upper-bound strengths for these surfaces. Lower bounds to the residual strength were obtained considering the stability of the failed section; again, a variety of reasonable surfaces were considered, all of which gave values of the same order of magnitude.

The results of these analyses led to the following results:

The results of these analyses led to the following results:

Sand Layer	$(N_1)_{60}$	Fines Content	Equivalent Clean Sand $(N_1)_{60}$	Residual Strength
Base of upstream shell of La Marquesa Dam	≈4	≈30	≈6	76 to 340 psf
Base of downstream shell of La Marquesa Dam	≈9	≈20	≈11	266 to 580 psf
Base of upstream shell of La Palma Dam	≈3	≈15	≈4	120 to 300 psf

It may be noted that other alternatives (not shown) for the inclination of the core and the position of the base sliding surface were considered, as well as different reasonable undrained strength values for the somewhat more plastic embankment soils; in all cases, the residual strengths obtained fell within the range of values reported above.

The two cases of dam failure described are of special interest because they provide examples of the performance of very low dams, in which static stresses are quite low, under strong earthquake conditions and they also provide two additional case studies to add to the relatively sparse record of case studies from which residual strengths of liquefied soil can be determined based on field performance (Seed, 1987). Thus the results of these studies can be added to the data previously compiled, as shown in Fig. 17. It may be seen that they are generally consistent with values obtained from previous studies.

Acknowledgements

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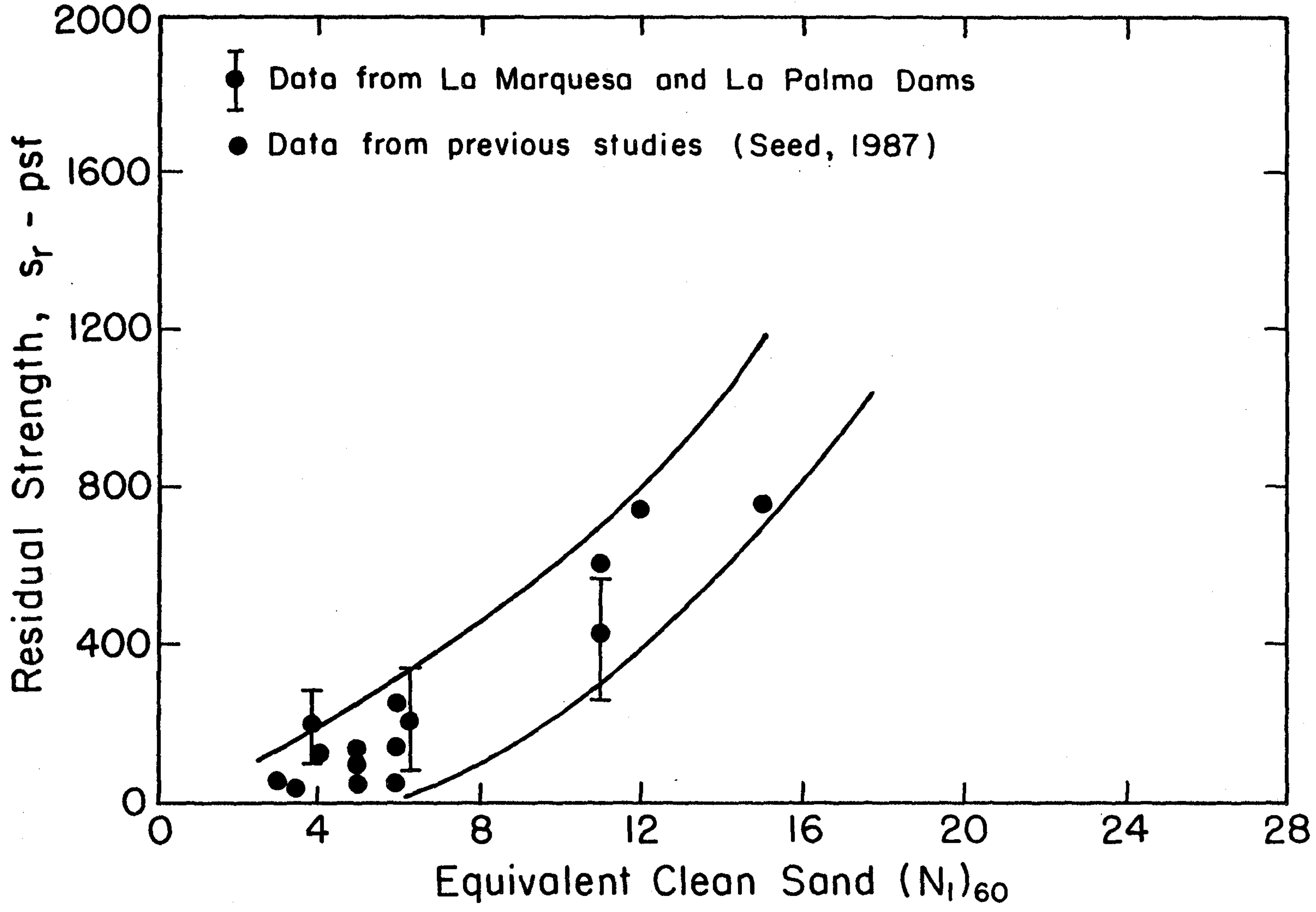

Fig. 17 RELATIONSHIP BETWEEN RESIDUAL STRENGTH AND EQUIVALENT CLEAN SAND VALUE OF $(N_1)_{60}$

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