REPORT NO.
UCB/EERC-88/10
JUNE 1988

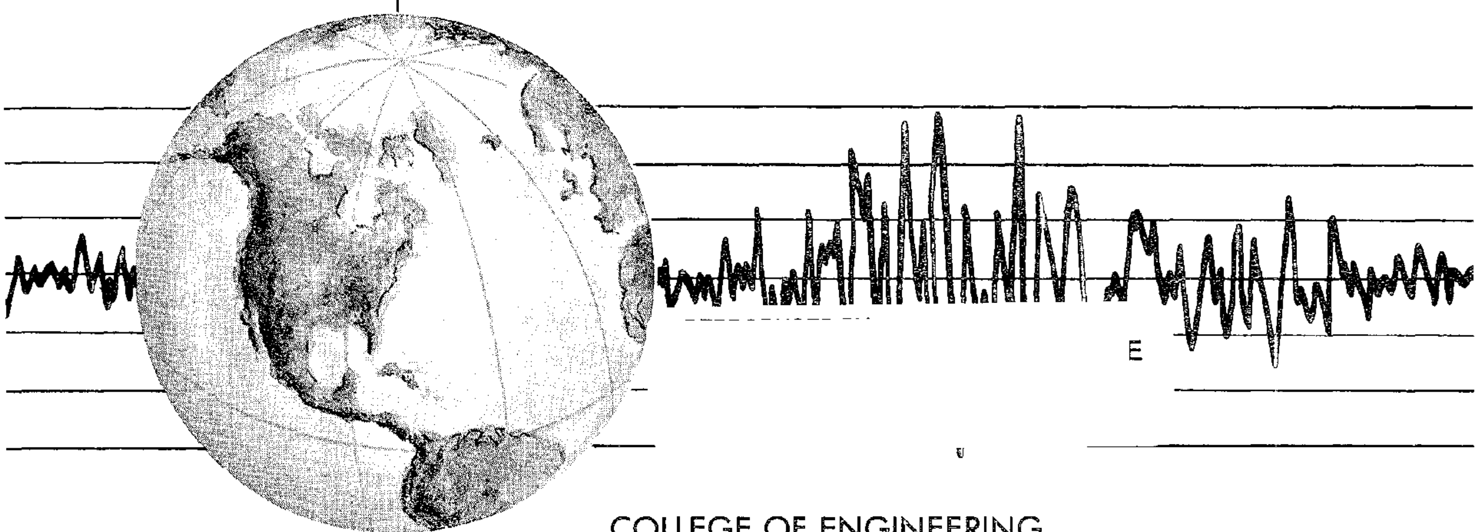
EARTHQUAKE ENGINEERING RESEARCH CENTER

THE LANDSLIDE AT THE PORT OF NICE ON OCTOBER 16, 1979

by

H. BOLTON SEED
RAYMOND B. SEED
F. SCHLOSSER
F. BLONDEAU
I. JURAN

A report on research sponsored by
the National Science Foundation



COLLEGE OF ENGINEERING

UNIVERSITY OF CALIFORNIA · Berkeley, California

See back of report for up to date listing of
EERC reports.

50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF/ENG-88046	2.	3. PB91-210914

4. Title and Subtitle The Landslide at the Port of Nice on October 16, 1979	5. Report Date June 1988
	6.
7. Author(s) H.B. Seed, R.B. Seed, F. Schlosser, F. Blondeau and I. Juran	8. Performing Organization Rept. No. UCB/EERC-88/10
9. Performing Organization Name and Address Earthquake Engineering Research Center University of California, Berkeley 1301 S 46th St. Richmond, CA 94804	10. Project/Task/Work Unit No. 11. Contract(C) or Grant(G) No. (C) (G) CES84-11912
12. Sponsoring Organization Name and Address National Science Foundation 1800 G. St. NW Washington, DC 20550	13. Type of Report & Period Covered 14.

15. Supplementary Notes

16. Abstract (Limit: 200 words)

On the afternoon of October 16, 1979 a major slide occurred in the fill which had been placed to construct the new port at Nice. The early interpretation of the associated events led to the conclusion that the slide in the port fill had triggered a massive underwater landslide which, in turn, caused the tidal wave, but did not explain the occurrence of the slide in the port fill in the first place. A comprehensive investigation was initiated by the French authorities. This report presents a review of the studies performed to evaluate the stability of the construction and to investigate the probable cause of this major slide.

17. Document Analysis a. Descriptors

b. Identifiers/Open-Ended Terms

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
Release Unlimited	unclassified	76
	20. Security Class (This Page) unclassified	22. Price

(See ANSI–Z39.18) See Instructions on Reverse

OPTIONAL FORM 272 (4–77)
(Formerly NTIS–35)
Department of Commerce

EARTHQUAKE ENGINEERING RESEARCH CENTER

THE LANDSLIDE AT THE PORT OF NICE ON OCTOBER 16, 1979

by

H. Bolton Seed

Raymond B. Seed

F. Schlosser

F. Blondeau

I. Juran

Report No. UCB/EERC-88/10

June 1988

College of Engineering
Department of Civil Engineering
University of California
Berkeley, California

The Landslide at the Port of Nice on October 16, 1979

by

H. Bolton Seed,[1] Raymond B. Seed,[2] F. Schlosser,[3]
F. Blondeau,[4] and I. Juran[5]

INTRODUCTION

At about 13.58 hrs. on the afternoon of October 16, 1979 a major slide occurred in the fill which had been placed to construct the new port at Nice on the south coast of France (see Fig. 1). The slide involved about 2 to 3 million cu. meters of fill and also about 7 million cu. meters of the underlying clayey silt and silty sandy deposits on which the fill had been placed. The time required for sliding of the approximately 10 million cu. meters of soil was about 4 minutes.

The slide debris from the port moved out to sea, first down the sloping face of the delta deposit on which it was constructed and then along an off-shore canyon and finally along the sea floor, eventually rupturing two sets of cables located at distances of about 90 and 120 kms off-shore from Nice. Both sets of cables were moved about 15 kms from their original positions. Cable 1, 90 kms off-shore, broke at 18:45 (4-3/4 hours after the slide at the port) and Cable 2, 120 kms off-shore broke at 2300 hrs. The velocity of flow on the almost flat sea bed between the cables was thus about 7 kms/hour.

---

[1]Cahill Prof. of Civil Engrg., University of California, Berkeley, California.

[2]Asst. Prof. of Civil Engrg., University of California, Berkeley, California.

[3]Prof. of Soil Mechanics, Ecole Nationale des Pontes et Chaussées, Paris, France.

[4]Engineer, Terrasol, Puteaux, France.

[5]Assoc. Prof. of Civil Engrg., Louisiana State University, Baton Rouge, Louisiana.

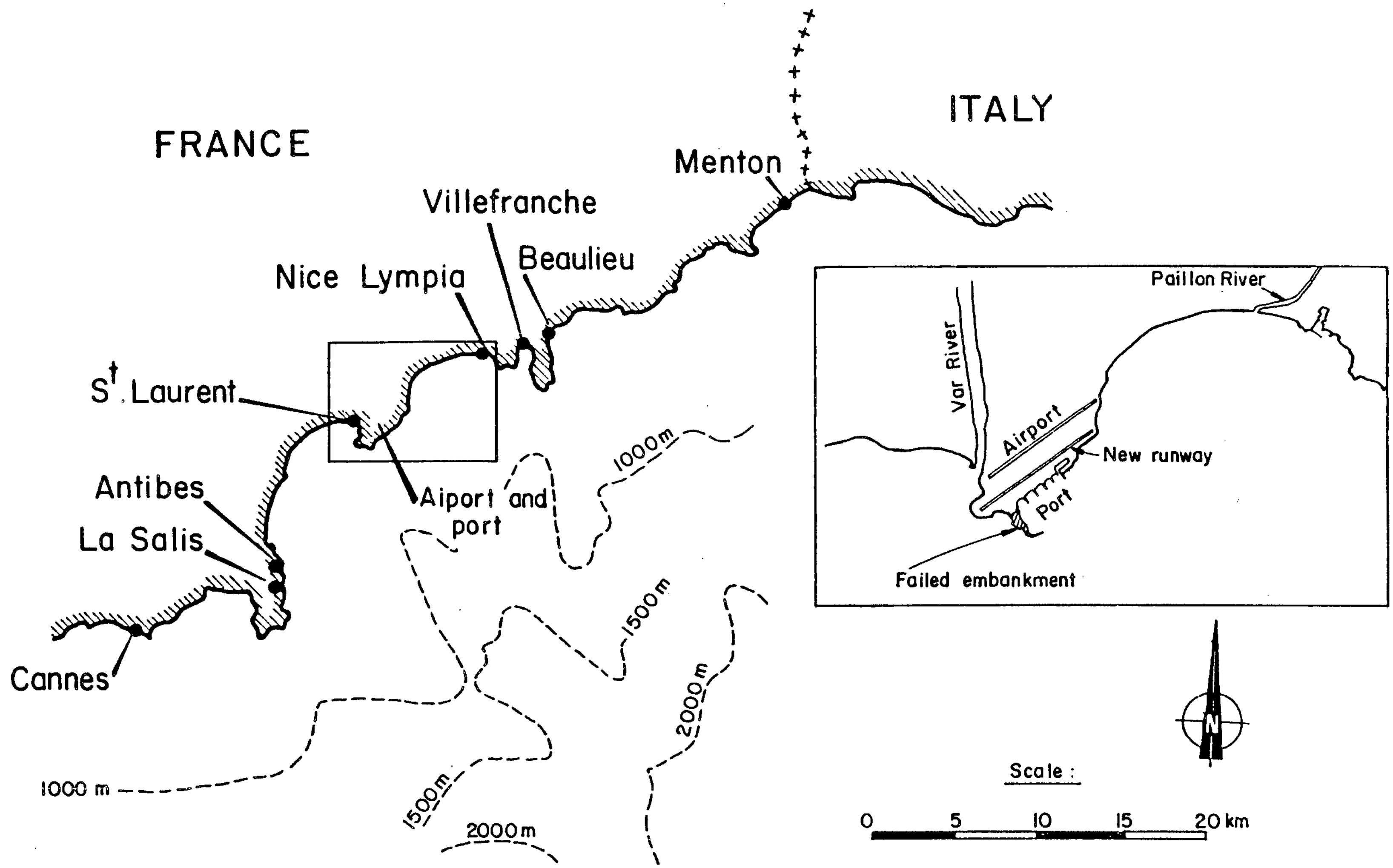


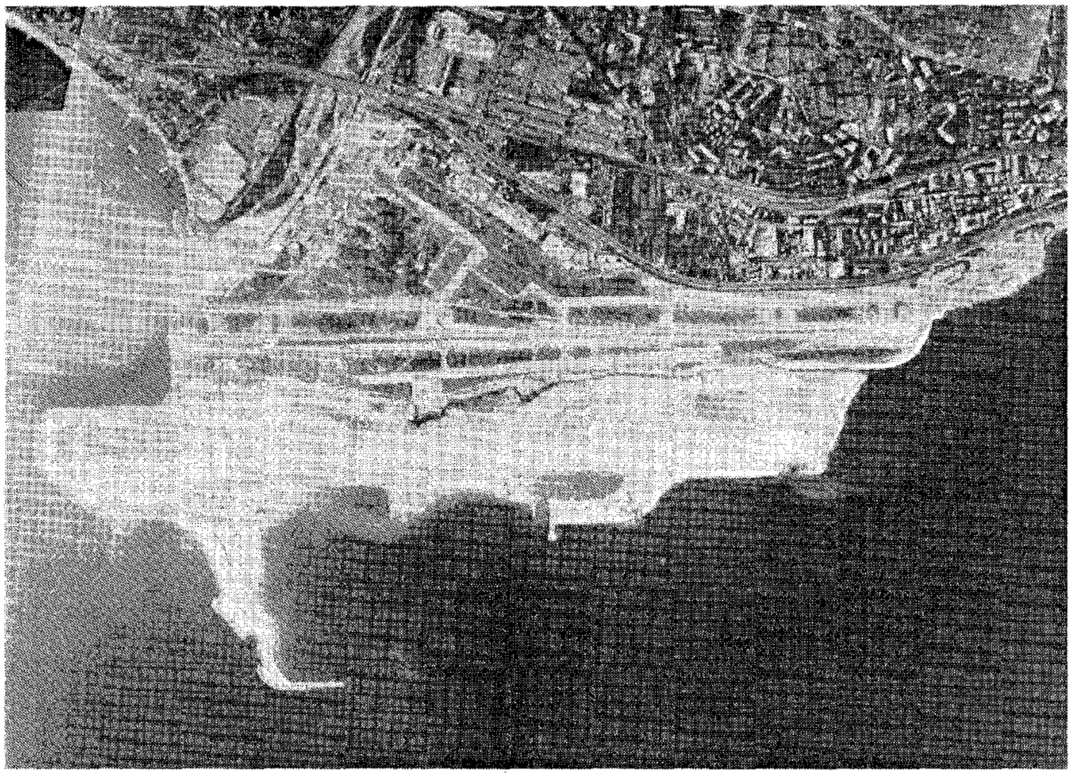
Fig. 1 PROJECT LOCATION ON THE SOUTH COAST OF FRANCE

Studies by Gennesseaux et al. (1980) at the University of Paris, together with off-shore borings in the area where the slide mass came to rest, show that a flow of several hundred million cu. meters of soil was probably involved in causing the cable breaks. Since the port slide involved only about 10 million cu. meters, a very large volume of supplementary slide debris would have to have been generated from some submarine location off-shore to account for this very large volume of sediment which finally came to rest about 150 kms off-shore.

At about the same time as the sliding of the port fill occurred, a tidal wave was observed along a section of about 120 kms of the coast-line of southern France. This tidal wave had a maximum amplitude of about 3 m near Nice and La Salis (about 12 kms. from Nice) and decreased in amplitude to about 0.25 at Ile du Levant (about 90 kms. west of Nice). The flooding associated with this wave caused the loss of several lives and did considerable damage to local communities and harbors.

Aerial photographs of the fill placed to construct both a new airport and a new port at Nice, showing the conditions just before and just after the slide on October 16, 1979 are shown in Fig. 2. It will be seen that only the outer portion of the fill, shown in the photograph in Fig. 3, which had been placed to construct a port facility was involved in the slide. A view of the portion of the fill at the zone where slide movements stopped is shown in Fig. 4. The fill, much of which was deposited through or in water, was constructed on a deltaic deposit of stratified clayey silt and silty sand, the outer boundary of which sloped off-shore at about 15° to the horizontal.

A typical soil profile through the delta in the location where the slide movements started is shown in Fig. 5. The delta is cut by two submarine canyons, the Var and Paillon canyons, which extend to a distance of about



(a) Air-Photo of Fill Area on 8 October, 1979

(b) Air-Photo of Fill Area After Slide on October 16, 1979

Fig. 2 AERIAL PHOTOGRAPHS OF FILL AREA

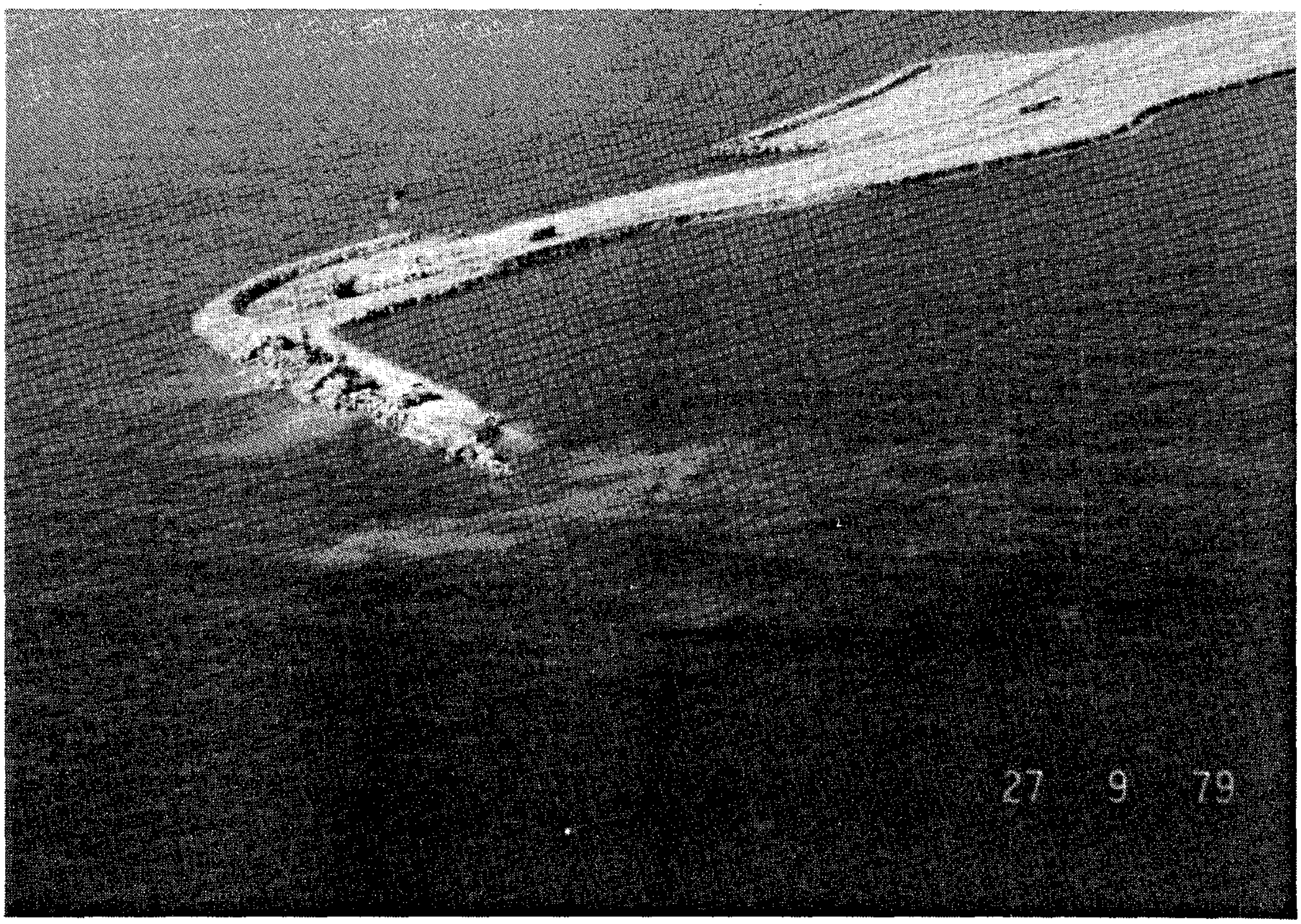


Fig. 3 PORTION OF FILL INVOLVED IN SLIDE OF OCT. 16, 1979

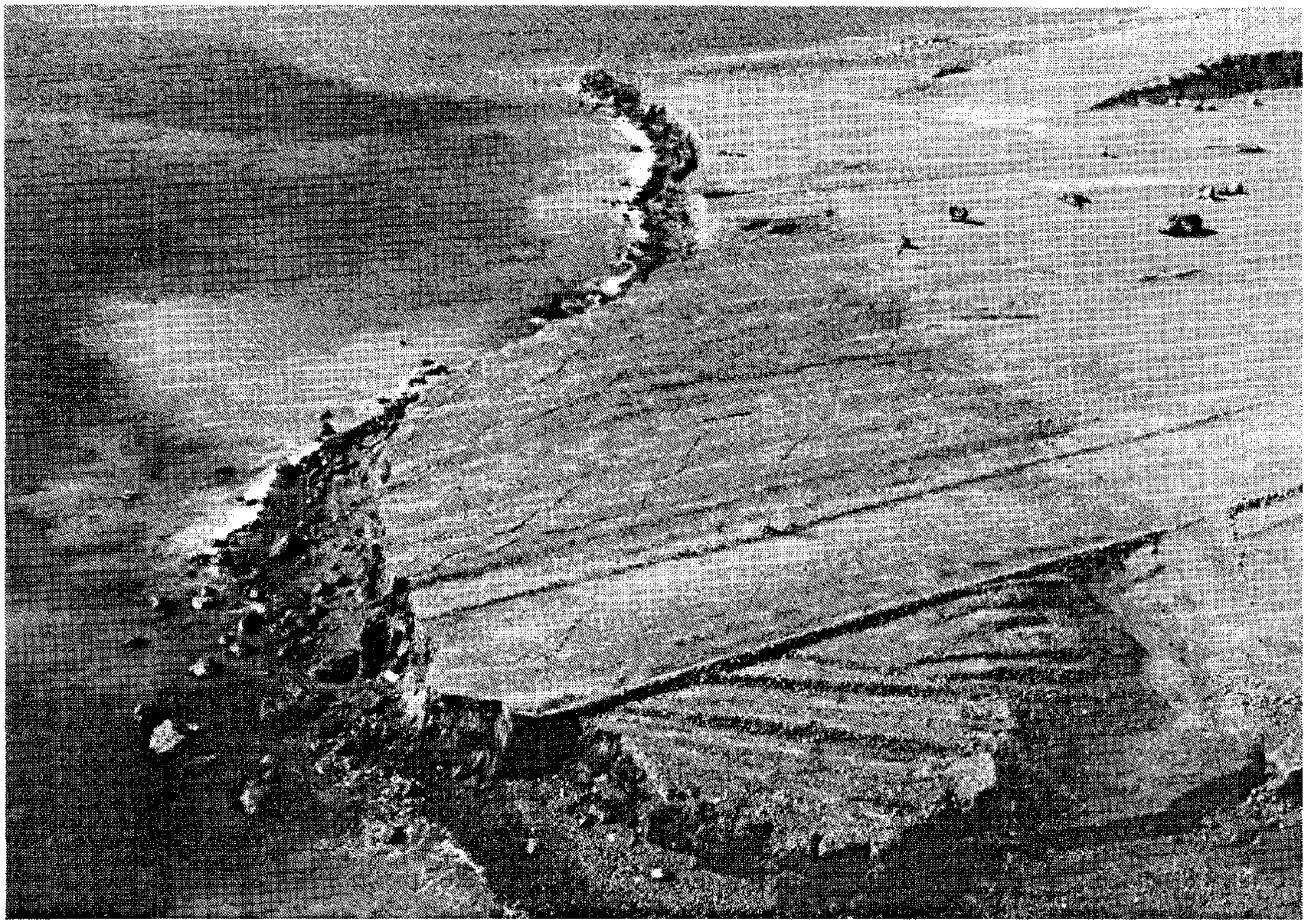


Fig. 4 PORT FILL AT POINT WHERE SLIDE MOVEMENTS STOPPED

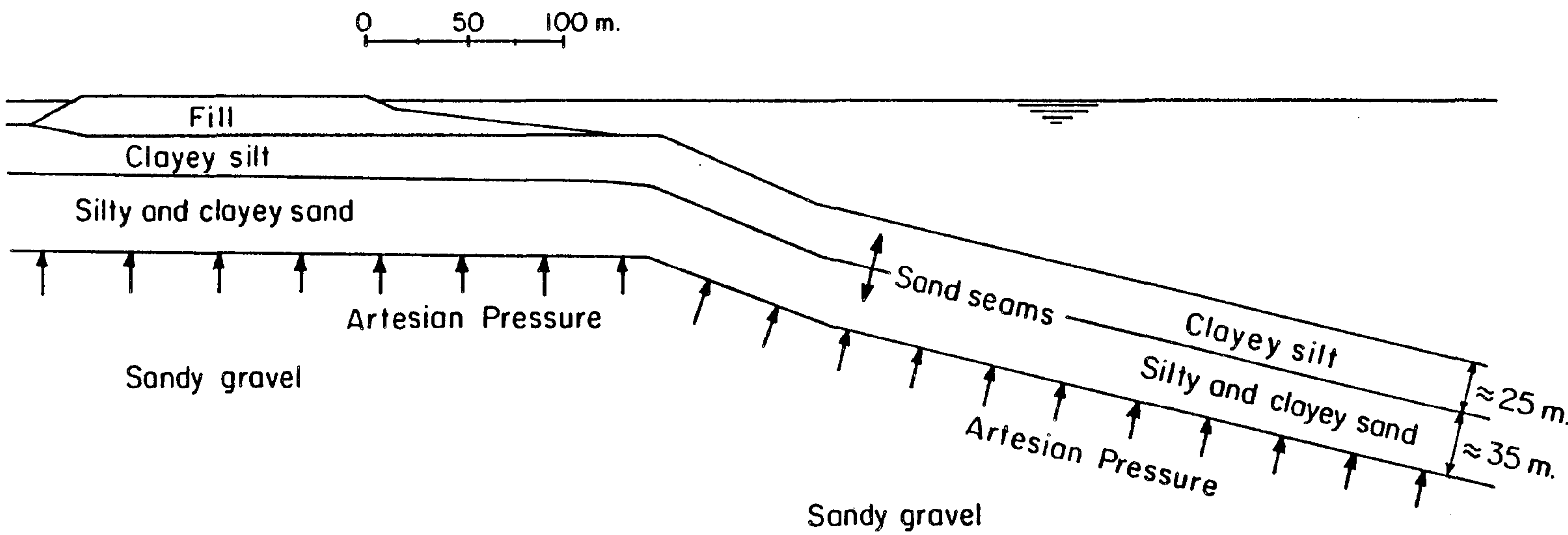


Fig. 5 SCHEMATIC PROFILE OF SOIL CONDITIONS IN SLIDE AREA

15 kms off-shore where they merge together. These are the underwater extensions of the Var and Paillon rivers on land. The main fill was kept at least about 150 meters from the crest of the outer slope of the delta deposits in order that it would not have any significant effect on the stability of the outer slopes (see Fig. 5).

It had long been recognized that pervious layers of soil existed in the delta deposits, which connected with water levels on land and created an artesian pressure condition at depths greater than about 40 meters in the delta deposits. This condition was recognized and taken into account during the planning of the fills.

There were a number of unusual circumstances associated with the slide. They included the following:

1. The port fill and supporting soils were generally similar to those involved in the construction of an extension to the adjacent airport where they had proved to be adequately stable over a period of several years.
2. The fill and underlying soils were indicated to be stable on the basis of geotechnical studies made for the airport fill. While the factor of safety of the outer parts of the delta slopes was somewhat less than that normally required for permanent construction, it was consistent with that generally accepted as adequate for the period of construction.
3. The soil fill proved to be adequately stable over a period of about 8 months during which time very little fill was placed in the port area. Since virtually no fill had been placed for a period of about eight months, piezometers showed, as anticipated, a reduction in the pore water pressures induced by the new fill in the delta deposits and a corresponding improvement in the stability of the fill. This improvement would be expected to continue as the pore pressures dissipated with time.

4. There was no prior indication, from surface observations or from the piezometer readings, of instability of the port fill. In fact observations by divers of the condition of the outer slopes of the delta and the submerged fill, and surface observations on the day of the slide, revealed no unusual movements indicative of any stability problems.
5. The slide was preceded by several days of very heavy rainfall (about 25 cms in 4 days) causing the Var and Paillon rivers to be running full and with greater than usual capacity. The rainfall increased the artesian pressure in the delta deposits by about 1 meter of head, as evidenced by piezometer readings in the delta deposits underlying the adjacent airport area.
6. Although there was no warning of an impending slide, the slide in the port fill occurred rapidly in a period of about 4 minutes.
7. The effects of the tidal wave were recorded on maregraphs installed along the coast-line at Port Lympia, Villefranche and Ille du Levant. These records show clear effects of tidal wave movements although they require careful interpretation to determine the actual changes in sea level associated with the tidal wave effects.
8. Bathymetry and submarine explorations of the underwater Var and Paillon canyons made after the slide occurred indicated that substantial movements of soil from the walls of the canyon had occurred since the last previous bathymetric studies in 1973.
9. Sensitive seismographs at La Salis (about 10 kms from Nice) showed a marked change in both the frequency and amplitude of background seismic waves starting just before the port slide occurred, but they showed no earthquake activity on the day of the slide. In fact the most recent

earthquake was a Magnitude 2 event occurring about 20 kms from Nice on Oct. 1, about 16 days before the slide.

The early interpretation of these events led to the conclusion that the slide in the port fill had triggered a massive under-water landslide, which in turn had caused the tidal wave. However this hypothesis left unanswered the question of what caused the occurrence of the slide in the port fill in the first place.

Since the tidal wave and the failure of the port fill resulted in the loss of lives and property, a comprehensive investigation was initiated by the French authorities. This report presents a review of the studies performed to evaluate the stability of the construction and to investigate the probable cause of this major slide.

TIDAL WAVE EFFECTS

Testimony of Witnesses

Because of the legal implications of the losses associated with the slide a major investigation was made to collect evidence from witnesses of both the slide and the tidal wave. The main items of importance in this connection were considered to be the timing and nature of the port slide and the timing and nature of the tidal wave. Fortunately there was general agreement on some of these items, such as:

Time slide movements started in port fill:	:	$13.57\frac{3}{4}$
Time slide movements ended in port fill	:	$14.01\frac{1}{2}$
Amplitude of wave motions at Nice	:	$\approx$ 3 m.
Period of tidal wave at all locations along coast	:	$\approx$ 8 min.

The time of occurrence of the slide was well-established by observers in the airport observation tower but the times of sea level changes were initially very confusing (ranging from an observed time of 13.55 to 14.20) and thus they required careful interpretation to provide a constant basis for comparison, since some observers of water level changes noted a lowering of the sea level while others noted a rising of the water level.

The changes in water level due to a tidal wave are a series of fluctuations and the development of different phases (first crest, first trough, etc.) will necessarily occur at different times. As shown in Fig. 6, a tidal wave has a starting point, followed by a series of crests and troughs. The period of the wave motions is the elapsed time between two successive crests or two successive troughs. The maximum amplitude is the maximum rise of any crest above normal sea level or the maximum fall of any trough below normal sea level. The times of events reported by witnesses will thus depend on which phase of wave activity they happened to observe.

In this connection it is important to note that the maragraph records of sea level movements showed clearly that the first manifestation of the tidal wave was a lowering of the sea level followed by a long sequence of rising and lowering fluctuations. Unfortunately these records do not provide an accurate time base to establish the times at which the various fluctuations occurred (i.e. with an accuracy of ±5 mins. approx.).

A careful review of the testimony of witnesses to the tidal wave activity also showed clearly that different witnesses saw different phases of the sequence of tidal wave effects, which diminished only slowly over the first half hour and did not fully decay until almost 12 hours after the start. At many locations there can be little doubt that for one reason or another, the beginning phases of the tidal wave effects escaped the attention of potential

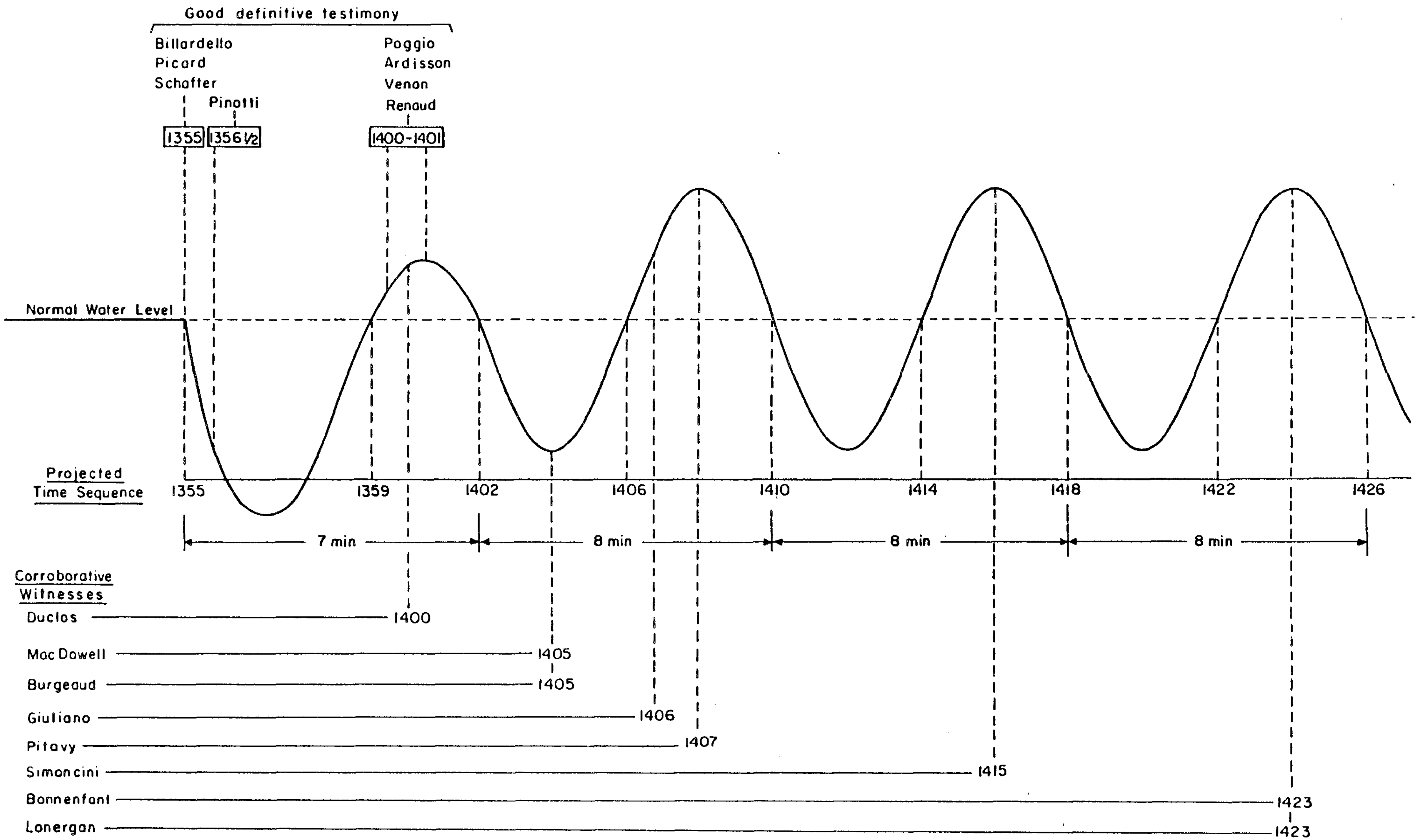


Fig. 6 TIMING OF SEQUENCE OF WATER LEVEL FLUCTUATIONS BASED ON OBSERVATIONS OF WITNESSES

observers. However a careful review showed that the witnesses fell into four main groups, as illustrated in Fig. 6.

In the first group were witnesses at three different locations along the coast, separated by about 26 kms, who noted the start of tidal wave activity (water lowering) at 13.55. These independent observations seem to provide a strong confirmation of this fact.

In the second group was one witness, located on the coast only 2 km from Nice, who provided very strong evidence of a lowering of the sea-level by about 2.5 m at his observation point at a confirmed time of 13.56-1/2.

In the third group were five independent witnesses, at four different locations, who observed the first significant rise in the sea-level at very close to 14.00 hrs. For a wave period of about 8 mins., this would correspond to an initial lowering of 13.55 or 13.56 and a maximum lowering (first trough) at about 13.57-1/2.

In the fourth group were 7 other corroborative witnesses whose observations of subsequent crests or troughs of the tidal wave coincided remarkably with the above schedule and a wave period of 8 mins.

Thus there is strong evidence that tidal wave effects, involving a lowering of the sea level, were already in progress at 13.56, i.e. about 2 minutes before the start of the port slide. In view of this fact, it is clear that the port slide could not have been the cause of the tidal wave, although the reverse may well be true; that is, the sea-level lowering of about 3 m at Nice may well have triggered the slide in the port fill. Tidal waves with sea-level lowering have been a major cause of coastal landslides, and there seems to be a high probability, based on the circumstantial evidence described above, that the slide at the port of Nice was such an occurrence.

A key argument in favor of this point of view is the fact that analyses of the amplitudes of tidal wave movements caused by the port slide based on the assumption that the tidal wave was originated by the slide of the port fill, are completely unable to correctly predict the amplitudes of tidal movements observed at different locations along the coast from Cannes to Menton (see Fig. 1). In the port slide, about 10 million cu m. of soil slipped into the sea in a period of about 4 mins. This displacement of water would undoubtedly cause a tidal wave which would travel from the area of the new port to other points along the coast. Analyses of these effects using model studies show that in general the computed amplitudes of the resulting waves were only about 10% of the observed amplitudes and that their computed arrival times were not in agreement with the observed arrival times of tidal wave activity, further indicating that the tidal wave movements were not caused by the port slide but must have had some other source, probably a major off-shore landslide. Interest thus was directed to this possibility, leading to a study of the off-shore bathymetry in the vicinity of Nice.

Bathymetry and Submarine Observations

Fortunately, excellent studies of submarine canyon bathymetry before and after the port slide were available to provide a good indication of soil movements between 1973 and 1979. In this period there were clear areas of soil deposition near the bottom of the Var Canyon but many more areas of significant soil loss on the steeper canyon slopes. Bathymetry surveys in 1973 and 1979 indicate that the main area of loss of material occurred about 15 km off-shore from Nice in the marl walls of the canyon. The estimated volume of material missing was between 100 million and 300 million cu. meters.

These movements of marl from the canyon walls were also confirmed by submarine observations which photographed large cavities in the marl walls of the canyon and large blocks (up to 100 tons) in the bottom of the Var Canyon. The blocks of marl in the bottom of the Canyon were not observed in the last submarine photographs taken in 1965. They were presumably the result of wall collapses since that time.

There is thus good evidence that a large submarine slide occurred in the walls of the off-shore canyons after 1973. This slide would presumably have generated a significant tidal wave whenever it occurred. Since there are no reports at Nice or other local coastal areas of any other unusual tidal waves in the period 1973 to 1979 except that on Oct. 16, 1979, it is reasonable to believe that it occurred in association with the tidal wave on this date, and that the port failure occurred in association with both this wave and the avalanche in the Canyon. The masses of soil and rock involved in the two failures were:

From the port slide	:	About 10 million cu. meters
From the Canyon avalanche	:	More than 100 million cu. meters

Thus it seems highly probable that it was a canyon slide in the marl, which caused the tidal wave activity starting just before the port slide on Oct. 16, 1979. In the light of previous experience of coastal landslides, this clearly raises the possibility that it was the drawdown of the sea level at about 14.57-1/2 which in all probability provided the trigger mechanism for the slide which then occurred in the port fill.

PREVIOUS CASES OF COASTAL LANDSLIDES

There are numerous examples of coastal landslides, often associated with soil liquefaction, which have been induced by earthquake shaking in off-shore, coastal and sub-marine deposits (see for example, the listing for the period up to 1965 in Table 1). However there have also been landslides induced by very low-level vibrations, such as those caused by trains, pile driving, construction equipment and blasting, while a number of major landslides have occurred in so-called "quick-clays" in Scandinavia due to only minor stress changes due to construction activity or unknown causes. A list of a number of quick-clay slides (after Aas, 1981) is presented in Table 2.

Of special interest in connection with the landslide at Nice are the number of landslides in coastal areas which have occurred without any related earthquake excitation. A list of these landslides, (Edgers and Karlsrud, 1982) is presented in Table 3.

It may be seen from these tables that the primary causes of previous coastal landslides, such as that which occurred at Nice, are

1. Earthquake shaking
2. Low level vibrations from non-seismic sources
3. Tidal fluctuations
4. Construction operations
5. Erosion or other sources of small stress changes.

In addition it may be noted that a number of the slides listed in Tables 2 and 3 involve striking similarities to the conditions at the port of Nice, as evidenced by the following examples:

(a) <u>Orkdals Fjord Slide, Norway</u>

This coastal and submarine slide occurred on May 2, 1930 (Terzaghi, 1956). The soils in the area of the slide were very loose and soft non-plastic silts with a water content of about 33%. It is also believed that the soil was under artesian pressure (note the great similarity to the conditions near the new port at Nice).

Table 1

Landslides During Earthquakes Due to Soil Liquefaction, 373 BC to 1965
(after Seed, 1968)

Date	Earthquake	Magnitude	Location	Type of Structure	Soil Type
373 BC	Helice	-	Helice	Coastal delta	-
1755	Lisbon	8.7	Fez		-
1783	Calabrian	-	Soriano	River banks	Clays with sand seams
			Laureau	River banks	Fluvial deposits
			Terramuova	River banks	Fluvial deposits
1811	New Madrid	8	Many	River banks & islands	Fluvial deposits, sands to muds
1869	Cachar		Silchar	River banks	Fluvial sand to clay
1886	Charleston	7.5	Ashley River	River banks	Fluvial & deltaic sands & silts
1897	Assam	8.7	Many	Canal banks	-
1899	Alaska	-	Valdez	Submarine deposit	Deltaic & marine sediments
1901	St. Vincent	-	St. Vincent	Coastal delta	-
1906	San Francisco	8.2	San Francisco area	Hillsides	-
1907	Karatag	-	-	Loess slopes	Loess
1907	Chuyanchinsk	-	-	Loess slopes	Loess
1908	Alaska	-	Valdez	Submarine deposit	Deltaic and marine sediments
1911	Alaska	7.0	Valdez	" "	" " "
1912	Alaska	7.2	Valdez	" "	" " "
1920	Kansu	-	Kansu Province	Loess slopes	Loess
1923	Kwanto	8.2	Tokyo area	Coastal hillsides	-
1925	Santa Barbara	6.3	Santa Barbara	Eart dam	Silty sand
1928	Chile	8.3	El Terriente	Tailings dam	Mining waste
1933	Long Beach	6.3	Long Beach	Highway fills	Fills over marshland
1934	Nepal	8.4	Motihari	Lake banks	Alluvium - sand lenses
1935	India	7.6	Quetta	River banks	-
1940	El Centro	7.0	Imperial Valley	Canal banks	Fills on deltaic sands
1941	Garm	-	-	Loess slopes	Loess
1943	Faizabad	-	-	Loess slopes	Loess
1948	Fukui	7.2	Fukui plain banks	Levees, river	Fluvial sands and silts
1949	Chait	7.5	Surchob and Yasman river valleys	Loess slopes	Loess
1950	Imperial Valley	5.4	Calipatria	Canal banks	Deltaic sands
1954	Anchorage	6.7	Rabitt Creek	Embankment	Fill on sand
1957	San Francisco	5.3	Lake Merced	Lake banks	Beach sands
1959	Jaltipan	6.5	Coatzacoalcos	Waterfront fill	Fine sandy silt
1960	Chile	8.4	Rinihue	River banks, coastal fills	Fluvial sands & silts
1964	Alaska	8.3	Valdez	Coastal delta	Silty sands & gravels
			Seward	Coastal delta	
1965	Chile	7.2	Several locations	Tailings dams	Sandy silt & silty sand
1965	Seattle	6.7	Port Orchard	Waterfront fill	Sand and marine clay
			Duwamisa	River terrace	Fluvial sands & silts

Table 2

Flow Slides in Highly Sensitive (Quick) Clays of Low Plasticity
(after Aas, 1981)

	Event	Soil Type	Cause
1.	Furre Landslide, April 14, 1959	Quick clay* $\frac{s_u}{p_o'} \approx 0.16$	Slope undercut by river
2.	Bastad Landslide, December 5, 1974	Quick clay* $\frac{s_u}{p_o'} \approx 0.18$	Earthwork construction
3.	Sem Landslide, April 17, 1974	Quick clay* $\frac{s_u}{p_o'} \approx 0.21$	Small earth fill
4.	Bekkelaget Landslide, October 7, 1953	Quick clay* $\frac{s_u}{p_o'} \approx 0.22$	Small earth fill
5.	Rissa Landslide, April 29, 1978	Quick clay* $\frac{s_u}{p_o'} \approx 0.21$	Small earth fill
6.	Vaerdalen Landslide, 1894	Quick clay	Unknown
7.	Kenogami Landslide, Quebec, 1924	Quick clay	Unknown

*Typically LL = 25 to 32, PI = 8 to 12 and w = 6 to 10% > LL.

Table 3

Liquefaction Landslides in Coastal Areas - Not Earthquake Related
(after Edgers and Karlsrud, 1982)

	Event	Soil Type	Cause
1.	229 flow slides in Province of Zealand, Holland, between 1881 and 1946	Fine sand	Seepage forces and erosion associated with large tidal fluctuations. Slides commonly occur at extremely low tide after exceptionally high spring tides.
2.	Many slides along banks of Mississippi River, U.S.A.	Sand	Undercutting of river banks.
3.	Slides in Trondheim Harbor, 1888	Silty sand	Tidal waves - siding when wave receded.
4.	Slide in Trondheim Harbor, 1930	Silty sand	Not known
5.	Slide in Trondheim Harbor, 1942	Silty sand	Not known
6.	Slide in Trondheim Harbor, 1950	Silty sand	Not known
7.	Slide in Orkdals Fjord, Norway, 1930	Loose sand & soft non plastic silt	Occurred at exceptionally low tide and preceded by small tidal wave
8.	Slide in Helsinki Harbor, Finland, 1936	Sand	During fill construction
9.	Kitimat, British Columbia, 1974	Fill on clay	Just after low tide
10.	Kitimat, British Columbia, 1975	Fill on clay	No fill being placed; extreme low tide for tidal range of 20 ft.
11.	Slide at Howe Sound, B.C., 1955	Silty sand	Extreme low tide
12.	Slide in Folla Fjord, 1952	Sand	Unknown, possibly wave-induced
13.	Rockall (Ancient)	Unknown	Rapid sedimentation
14.	Spanish Sahara (Ancient)	Gravelly clayey sand	Rapid sedimentation
15.	Wahro Bay, Africa (Ancient)	Unknown	Unknown
16.	Copper River, Alaska (Ancient)	Silt/sand	Rapid sedimentation
17.	Wil. Canyon (Ancient)	Silty clay and silt	Rapid sedimentation
18.	Mid Atl. Cont. Slope (Ancient)	Silty clay	Rapid sedimentation
19.	Magdalena River, 1935	Unknown	Rapid sedimentation
20.	Sokkelvik, 1959	Quick clay and sand	Unknown

The slide occurred in a period of 7 minutes at exceptionally low tide and was preceded by a small tidal wave which developed some distance out in the fjord. The total volume of soil involved was about 25 million cubic meters.

The flowing soil, moving down a slope of a few percent, broke two cables, one located 3 km off-shore and the other about 18 km off-shore. The velocity of flow between the cables was about 10 km per hour.

(b) Trondheim Harbor Slide, Norway

This coastal and sub-marine slide occurred at low tide on April 23, 1888 in a silty sand deposit (Andresen and Bjerrum, 1967). It is believed that the first slide occurred off-shore (out in the fjord) and caused a tidal wave. The reports state that "at the instant the wave reached Bratloza, or more accurately just when it receded, the railway embankment together with the jetty (up to 7 m high) slid out." The stone jetty rested on a sand layer sloping gently at 8 to 15°.

This is the most clearly defined case of several which are reported to have occurred at the low point of a tidal wave. This same condition could have been the case in the port slide at Nice.

(c) Sub-marine Slide at Kitimat, British Columbia

This slide occurred on April 27, 1975 during placement of fill for construction of a break-water (Wiegel, 1980). However, on the day of failure no fill had been placed. The slide occurred in a soft sensitive marine clay deposit on which the break-water fill had been placed. The time of the slide was about 2 hours after an extreme low tide with a tidal range of about 20 ft.

Investigators attributed the slide to "excess pore-water pressures due to extreme low tides" and concluded that because of this cause, the slides probably could not have been avoided. The slide reportedly caused a tidal wave 25 ft. high.

A similar slide six months earlier (October 17, 1974) also occurred after a low tide condition.

Of special interest in Table 3 is the number of slides in coastal areas which have been caused by tidal waves. However it should be noted that where this has been the case, the failure has been triggered by the fall in water level associated with the wave and not by the rise in water level. This is consistent with the principles of soil mechanics which show that a sudden lowering in water level (drawdown) has a de-stabilizing effect on the stability of submerged slopes. It is for this reason that it is important to study carefully the tidal wave effects at Nice and especially the times of water lowering since these can clearly be triggering mechanisms leading to major landslides.

PRELIMINARY EVALUATION OF FAILURE MECHANISM

Following the port slide, detailed surveys were made to determine the extent of the failure. It was found that the slide mass had moved away from the zone of failure leaving the failure surface exposed with the configuration shown in Fig. 7. It may be noted that the slip surface is slightly concave upwards, very flat, and very long. Simple slope stability calculations for a failure on this surface indicate a factor of safety of about 2.5 to 3.0. These facts suggest that the mechanism of sliding is different from that normally encountered.

Discussions were also held with a workman who was on the slide area at the time the failure developed. He described seeing slide movements beginning in an area about 200 m away from where he was standing and then, seeing the

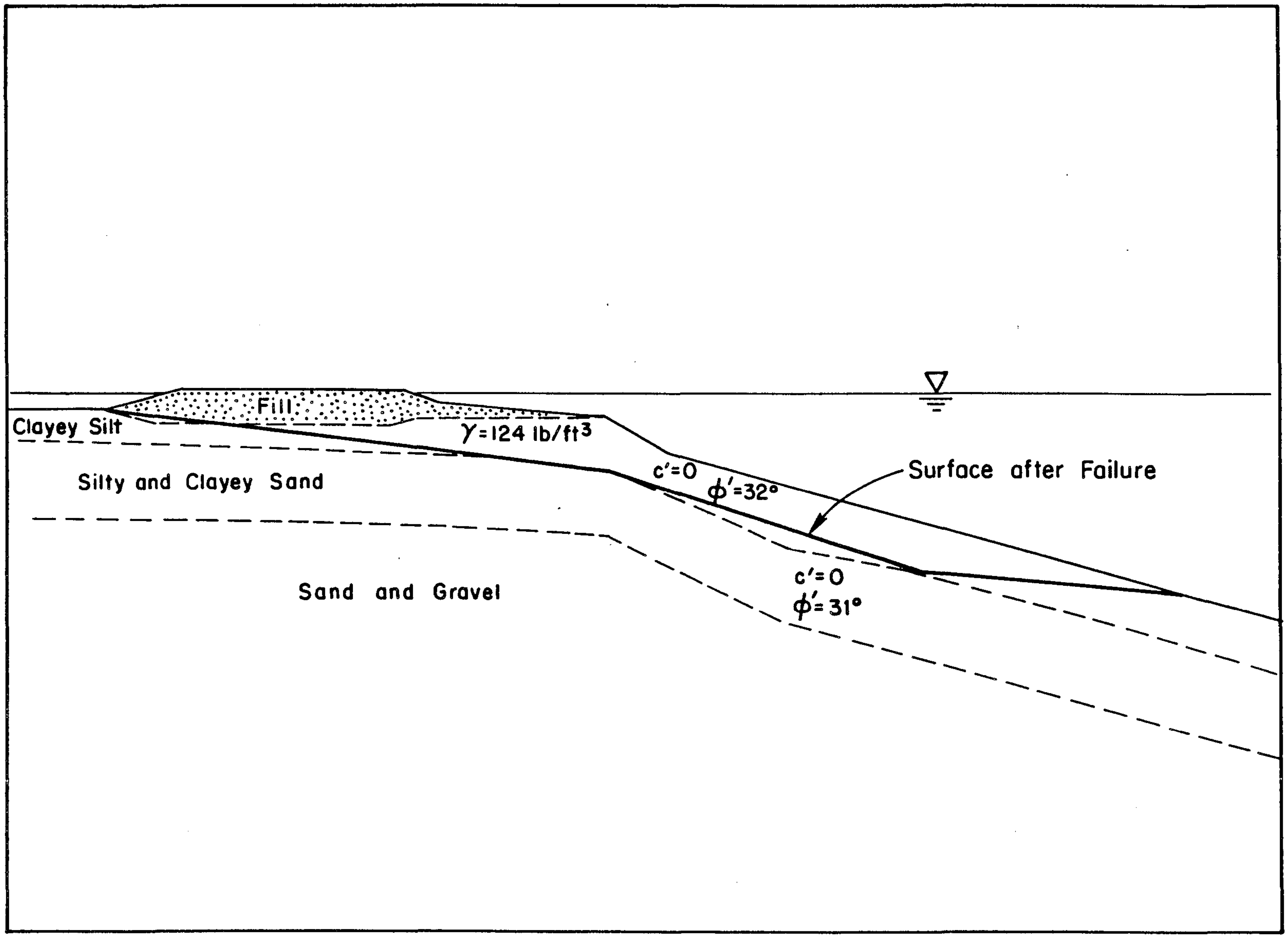


Fig. 7 SLOPE CONFIGURATION BEFORE AND AFTER FAILURE

slide developing towards him, he began to run to escape the failure. As he ran, the unstable area caught up with him but eventually he was able to outdistance the movements and reach a stable area of the fill. This description made clear the progressive nature of the sliding. Thus there are a number of features of the slide which suggest it was probably due to liquefaction of some zone of soil within the delta deposits, including:

(1) The rapid speed of occurrence and complete absence of any earlier indication of impending failure.

(2) The progressive nature of the development of the slide.

(3) The very large distance of movement of the soil, indicating its ability to flow after failure.

and (4) the relatively flat and unusual configuration of the failure surface, which never-the-less conformed closely with the probable configuration of bedding layers in the delta deposits.

While landslides due to liquefaction are most commonly induced by earthquakes, there have been many such slides induced by non-seismic forces and in many cases they seem to have been characterized by the conditions listed above. They have also been frequently induced in coastal deposits associated with tidal wave effects. Accordingly the soil conditions in the area of construction were carefully examined for further evidence of conditions amenable to the development of a liquefaction slide.

SOIL CONDITIONS IN THE SLIDE AREA

Field Investigations

Before construction and following the failure, numerous borings and cone penetration tests were performed in the area of construction to determine the soil conditions underlying the fill. In addition a large number of piezometers were installed in the general area of construction to provide a continuous record of the changes in pore water pressure in the delta deposits resulting from placement of the fill and subsequent consolidation or pore pressure dissipation effects.

It was found that the delta deposits consist typically of about 25 m of soft clayey silt and silty clay, underlain by about 35 m of silty and clayey sand and then by a deposit of sand and gravel. Within the upper layer of clayey silt and near the boundary with the clayey sand are a series of seams (about 0.5 m thick) of fine sand, silty sand and silt. These seams were readily identified by piezocone tests. The general profile in the area where sliding was initiated is shown in Fig. 4. Artesian pressures in the sand and gravel layer at a depth of about 60 m below the ground surface are normally about 5.2 m and at the time of the slide they had built up due to the heavy rainfall to about 6.2 m. Since the permeability of the silty and clayey sand is much higher than that of the overlying clayey silt, most of this artesian pressure was also developed near the base of the clayey silt.

A careful examination of the boring logs showed that there were a number of soil layers in the delta deposits which have characteristics similar to those which are known, on the basis of past performance, to have the potential to liquefy under appropriate loading conditions. These included:

(1) Silt and sand layers with a water content of 33 to 43%.

(2) Sand layers with a relative density, indicated by cone penetration tests, of about 30%.

and (3) Some clay and clayey silt layers with Atterberg limits and water contents similar to those of quick clays.

A detailed examination of the clays and clayey silts which make up much of the delta deposits, however, did not indicate characteristics of the type exhibited by quick-clays and it was considered highly unlikely therefore that they could have been the source of a liquefaction-type slide in the delta deposits. Furthermore laboratory tests on samples of the clay which did have water contents and Atterberg limits similar to quick-clays showed that their stress-strain relationships did not have the pronounced strain-softening behavior associated with quick clays.

Thus it appears that it was the thin (about 0.5 m) layers of sand, silty sand and silt which are most likely to be the source of any liquefaction type failure in the delta deposits, and supplementary borings, CPT tests and laboratory tests were performed to further determine their characteristics.

Because of the inevitable disturbance which occurs in sands during sampling, transportation, and handling in the laboratory it was considered desirable to first investigate the in-situ characteristics of the silty sand seams by means of field tests using a piezocone and then to recover samples of the sand and re-constitute them to their in-situ condition in the laboratory. Some change in structure of the sand will inevitably occur in this process but it was considered the best means of evaluating the general characteristics of the sands in the deposits.

The results of preconstruction borings and CPT tests show that the upper 30 m of clayey silt contain a number of silty sand seams. An additional series of piezocone penetration tests made both in the area of the fill adjacent to the point where sliding stopped and also off-shore in areas close to the landslide zone also showed two important results:

1. The sand seams are apparently continuous and could easily be identified in borings made 25 m apart over a length of at least 100 m.
2. The sand seams have low penetration resistance values indicating that they are in a relatively loose condition.

Of particular importance to the present study is the cone penetration resistance (q_c) of the sand seams since the resistance to penetration of the tip of the cone is indicative of the in-place relative density of the silty sands. A boring adjacent to the slide area showed 5 silty sand seams, averaging about 0.5 m thick, located between depths of 27 and 37 m below sea level and having cone tip resistance values between 20 and 60 ksc (or 200 to 600 kPa). On the average the cone tip resistance in this location was about 40 ksc. Other borings on the fill near the slide zone show sand seams in the same depth range with cone resistance values of about 60 ksc. Piezocone penetration tests further off-shore in zones near the slide area showed cone resistance values of the order of 30 ksc. Such values at a depth of about 30 m in a deposit and under an effective overburden pressure of about 2.7 ksc are indicative of a very loose silty sand.

Having determined that the cone tip resistance of the sand layers in the zone of major interest is about 20 to 60 ksc at a depth of about 30 m and under an effective overburden pressure of about 2.7 ksc, there are a number of correlations which may be used to evaluate the relative density of the sand. Correlations for this purpose have been proposed by Douglas and Olsen (1981), Schmertmann (1978), Gibbs and Holtz (1957), and Tokimatsu and Seed (1987) among others. Relative density values for the sands at Nice determined from these various correlations led to the conclusion that the sand is in a metastable condition (Douglas and Olsen) or that it has a relative density of the order of 30 ± 8%. To facilitate the laboratory testing, samples were tested in the relative density range 30 to 38%, but it is likely that some of the insitu materials were somewhat looser than this.

To investigate further the properties of the sand, a series of borings were made to obtain samples of the sand seams in the upper 30 m of the deltaic deposits for laboratory testing purposes. From each sand layer encountered in the silty clay, a sample was taken for determination of grain size distribution and the remainder of the sand was placed together with sand from other depth zones to accumulate a sufficient quantity of sand for preparation of triaxial test specimens for strength determinations. The grain size distributions measured on a series of samples are shown in Fig. 8. It may be seen that in all cases the sands were silty in character. For test purposes, a bulk sample of the composite sand was prepared having a grain size distribution in the middle of the range for the individual samples. This silty sand was used to study the liquefaction potential of the sand seams in the port area.

Evaluation of Liquefaction Potential of Sands

The pioneering work on the development of laboratory test procedures for evaluating the liquefaction characteristics of sands under static loading conditions is mainly that of Casagrande (1975 and 1984), Castro (1969), Castro and Poulos (1977) and Poulos et al. (1985), who have defined the large-strain residual strength of saturated sands as the "steady-state" strength and proposed methods for its evaluation. The same test procedures also provide a basis for evaluating the stress changes required to trigger liquefaction, however, and this aspect is of particular relevance to the evaluation of the cause of the landslide at the port of Nice.

A schematic diagram of the underwater slope of the ground in the area where sliding occurred is shown in Fig. 9. Liquefaction slides are initiated when a block of soil similar to that shown in the figure becomes unstable and triggers a progressive series of slides involving other blocks of soil. An analysis of the stability of the block shown in Fig. 9, for example, shows that prior to sliding the average shear stress on the base of the block was

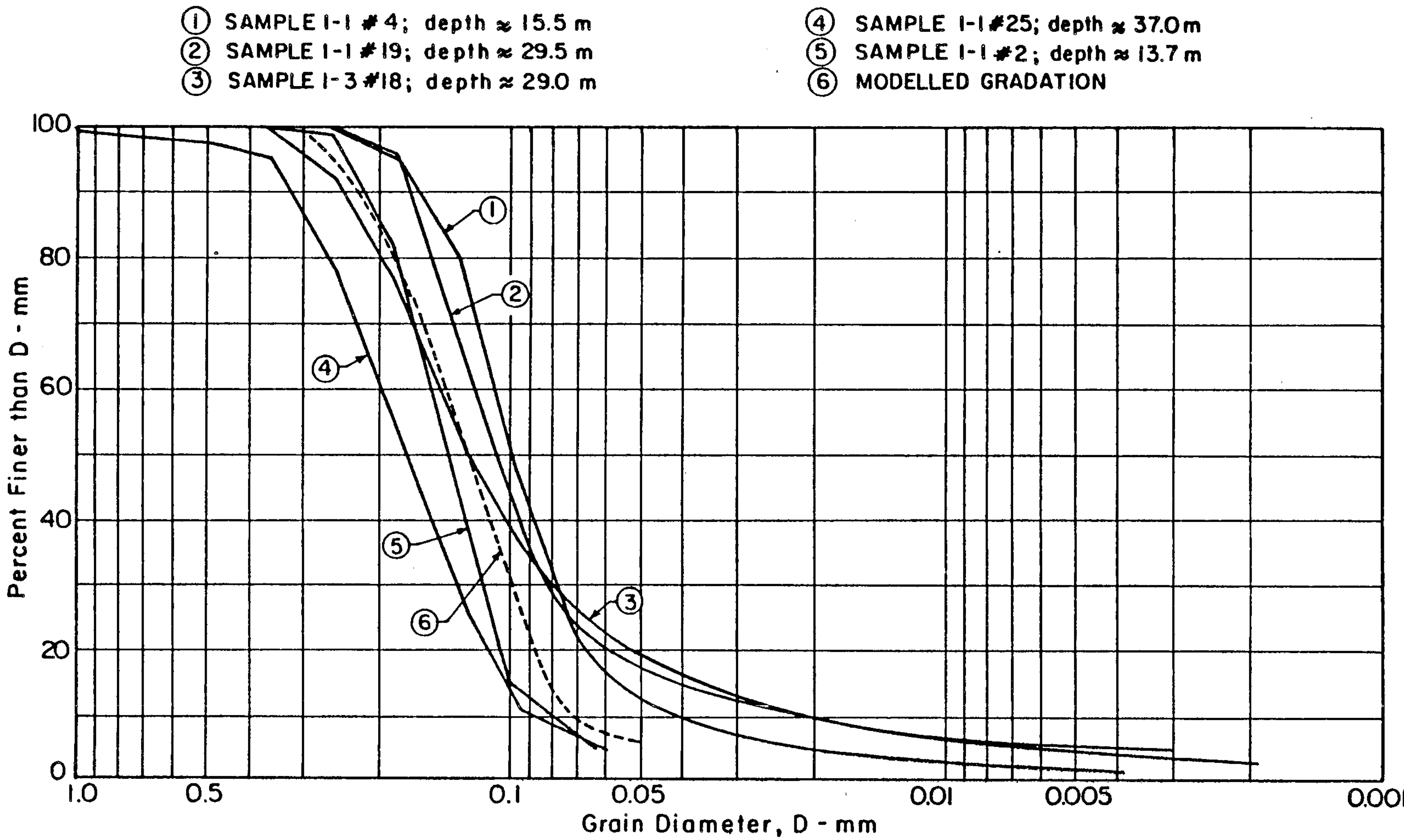


Fig. 8 GRAIN SIZE DISTRIBUTION CURVES FOR SAND SAMPLES FROM NICE HARBOR

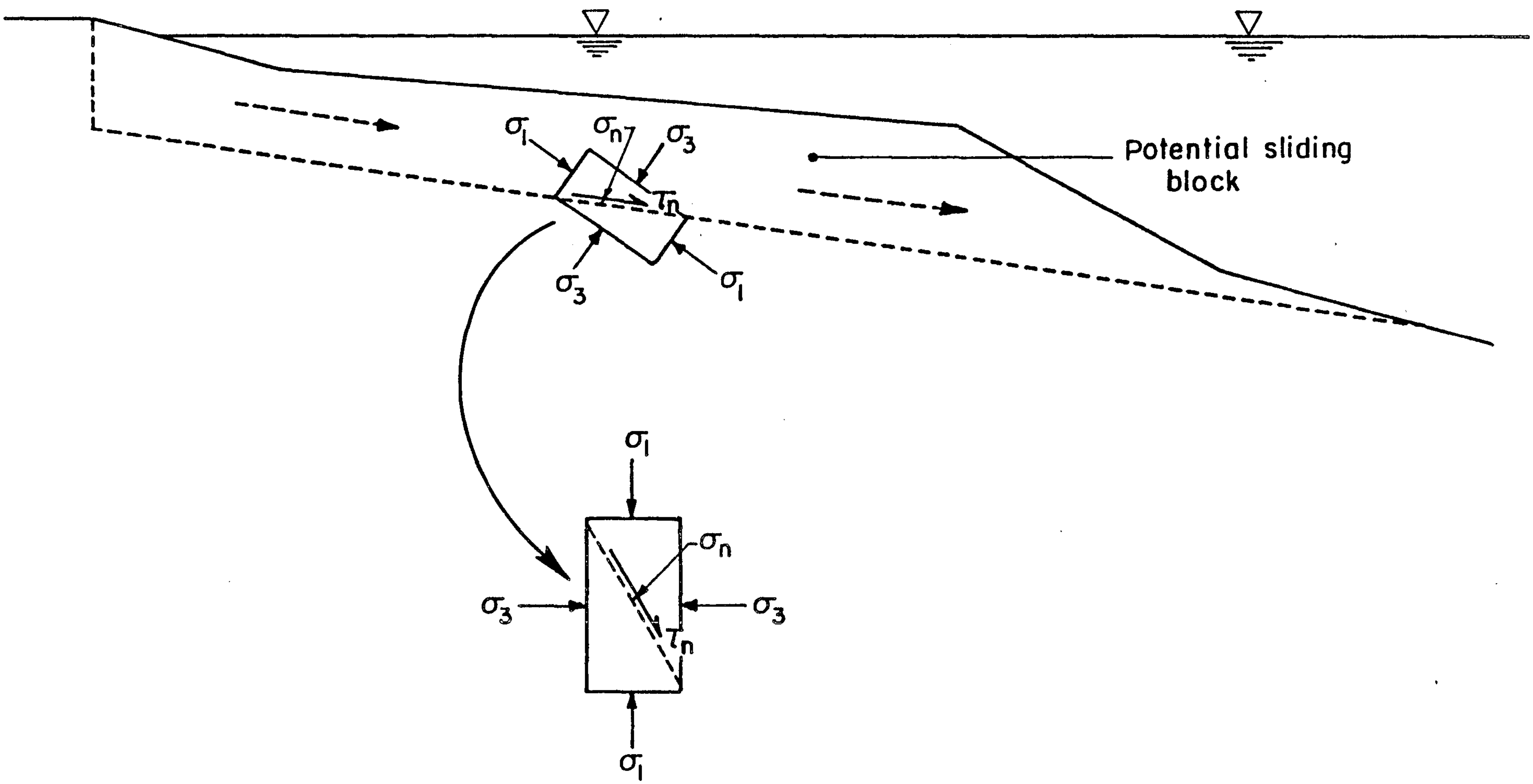


Fig. 9 SCHEMATIC ILLUSTRATION OF STRESSES ON SOIL ELEMENT ON POTENTIAL FAILURE SURFACE AND EQUIVALENT STRESS CONDITION FOR TRIAXIAL TEST SPECIMEN

about 0.48 ksc providing an average shear stress/normal stress ratio of about 0.32. This same stress ratio, representative of stable conditions in the slope on the morning of October 15, can be reproduced in a triaxial test sample by subjecting it to a principal stress ratio of about 2 as indicated in Fig. 9.

Test data for triaxial tests performed on loose fine sands with such initial stress conditions is available from many previous studies to throw light on the stress changes required to cause instability of the sands. Thus, for example, the results of a series of triaxial compression tests on samples of Sacramento River fine sand, initially in equilibrium under an effective principal stress ratio of 2, are shown in Fig. 10. The samples for which data is shown had relatively densities of 32 and 39%.

In the consolidated state the initially applied deviatoric stress on the samples was 3 ksc. If the samples were then loaded to failure by increasing the axial stress slowly, allowing water to drain freely out of or into the samples,the stress built up progressively with increasing strain and at both relative densities the samples showed compressive strengths of the order of 6.8 ksc corresponding to an increase in deviatoric stress required to cause failure of about 125%. On the other hand, when the samples were subjected to loading under undrained conditions (i.e. with no opportunity for movement of water into or out of the samples) the maximum deviatoric stress that could be developed before the samples exhibited a severe reduction in strength, representative of liquefaction, was only about 3.1 ksc, corresponding to a deviatoric stress increase to cause failure of only about 3%.

These results provide a striking illustration of the great difference in strength characteristics of saturated loose sands subjected to drained and undrained loading. If the stress changes are applied slowly enough to permit

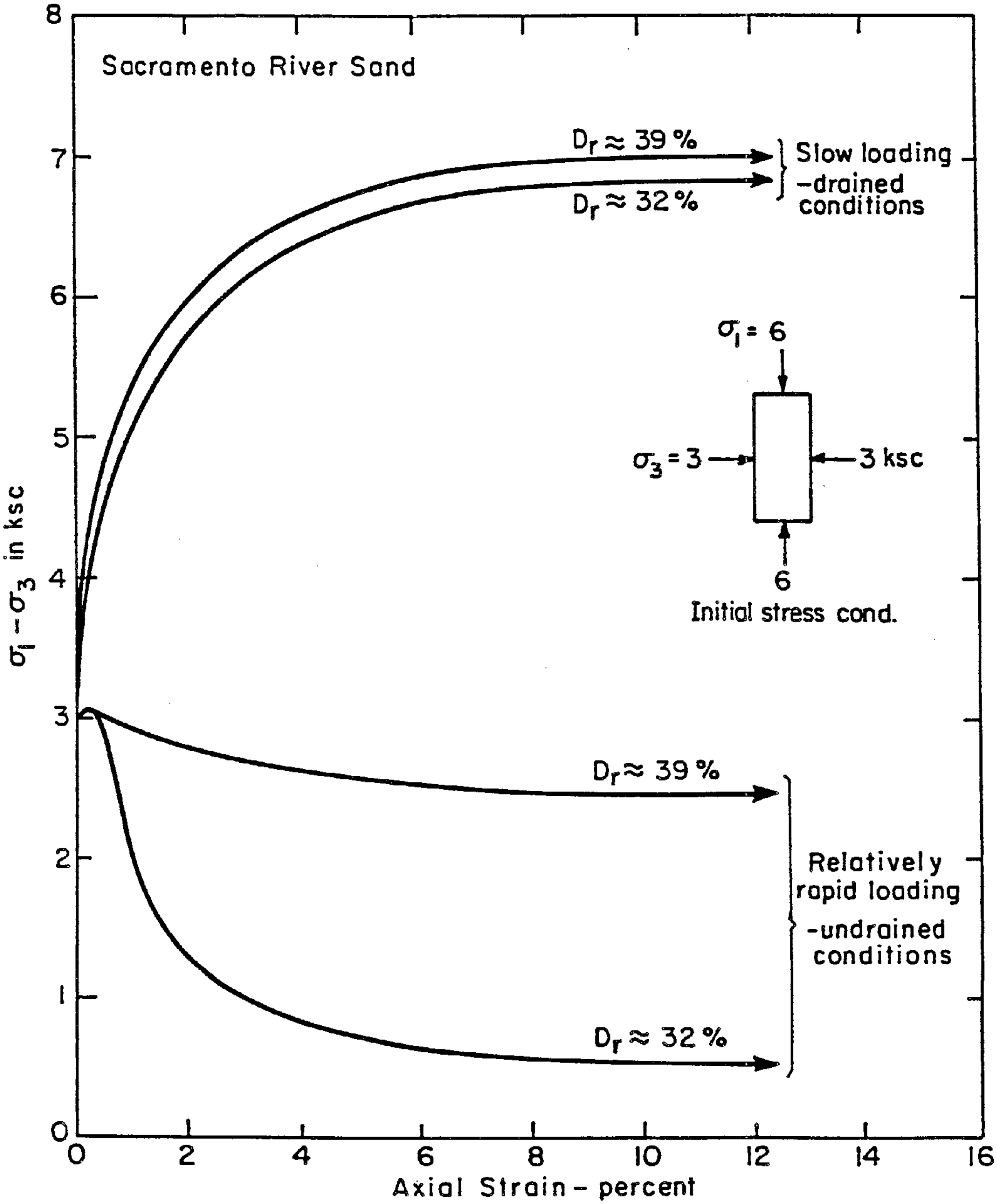


Fig. 10 TYPICAL STRESS-STRAIN RELATIONSHIPS IN DRAINED AND UNDRAINED TESTS ON LOOSE SANDS

drainage the sand has good strength characteristics. If the stress changes are applied too rapidly to permit drainage, a very small deviatoric stress change of the order of 3% can cause a severe loss of resistance and flow failure representative of liquefaction.

It is important to note that the cause of the loss in resistance of a soil leading to liquefaction-type behavior is the progressive, self-induced, build-up of pore water pressure in the soil resulting from the increase in strain as the sample moves towards liquefaction. Thus, for example, the changes in applied load, resistance to deformation and induced pore-water pressure for a typical test sample having a relative density of about 34% are shown in Fig. 11. For this sample the change in applied axial stress was only about 6% (0.25 ksc) and the resulting change in pore-water pressure when this stress was applied was only 0.4 ksc. Nevertheless, the development of this small change in pore-water pressure initiated a progressive increase in strain of the sample, without change in applied load, and this led to a progressive increase in pore-water pressure until it finally attained a value of about 1.75 ksc, by which time the resistance to deformation of the sample had decreased to a relatively small value and the sample had undergone large deformations in a manifestation of the phenomenon of liquefaction. Such behavior could not have occurred, however, without the triggering effect of the initial small stress application.

Clearly not all sand deposits are vulnerable to this type of behavior. As the relative density of a sand deposit increases, its vulnerability to strength loss (and liquefaction) under undrained loading decreases. Thus for Sacramento River fine sand, as the relative density approaches 45 to 50% it ceases to exhibit any strength loss on loading so that dramatic flow slides due to small stress changes can no longer occur. However it is clear that

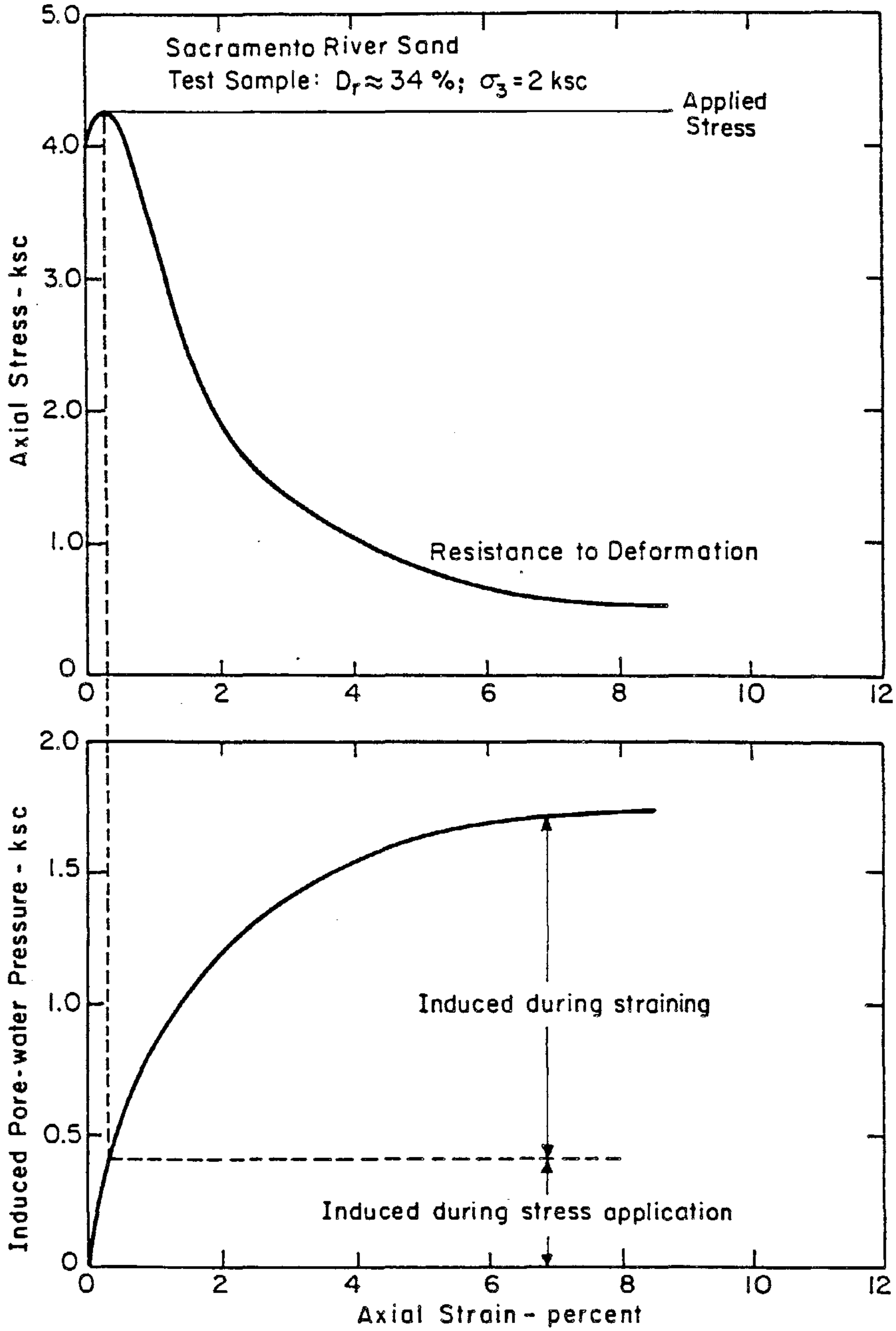


Fig. 11 PORE WATER PRESSURE GENERATION DURING UNDRAINED TEST ON SAMPLE OF LOOSE SAND

substantial reductions in strength occur for undrained loading of this and other sands with relative densities in the range of 20 to 36%.

It is also important to note that the stress change required to cause a sand to liquefy or lose strength under undrained loading conditions depends significantly on the initial shear stress/normal stress ratio (or effective principal stress ratio) at which it was in equilibrium before the undrained loading was applied (Castro, 1969, Seed, 1983, and Kramer and Seed, 1988). This is illustrated by the test data for samples of Sacramento River fine sand with a relative density of 33% shown in Fig. 12. For this sand, if the initial effective principal stress ratio is only 1.5 ($\tau/\sigma_o' \approx 0.2$), the increase in deviatoric stress required to cause failure and collapse was about 40% compared with only about 3% for the same sand under an initial effective principal stress ratio of 2 ($\tau/\sigma_o' \approx 0.33$). Thus any change in conditions which tends to increase the in-situ shear stress/normal stress ratio will make a loose sand more vulnerable to liquefaction under undrained loading. Conversely densification of the sand, from a relative density of 35% to a relative density of 55% will effectively eliminate the possibility of liquefaction under static loading conditions.

A final type of laboratory test result of special interest to the Nice slide is that showing the effect of an increase in principal stress ratio produced by increasing the pore water pressure in a test sample subjected to anisotropic stress conditions. This has important implications in understanding the effects of a change in artesian pressure conditions on the sand deposits in the delta region.

Typical results of such a test are shown in Fig. 13. The triaxial test sample was first brought to equilibrium under a principal stress ratio of 2. Under these conditions the sample could sustain a change in principal stress

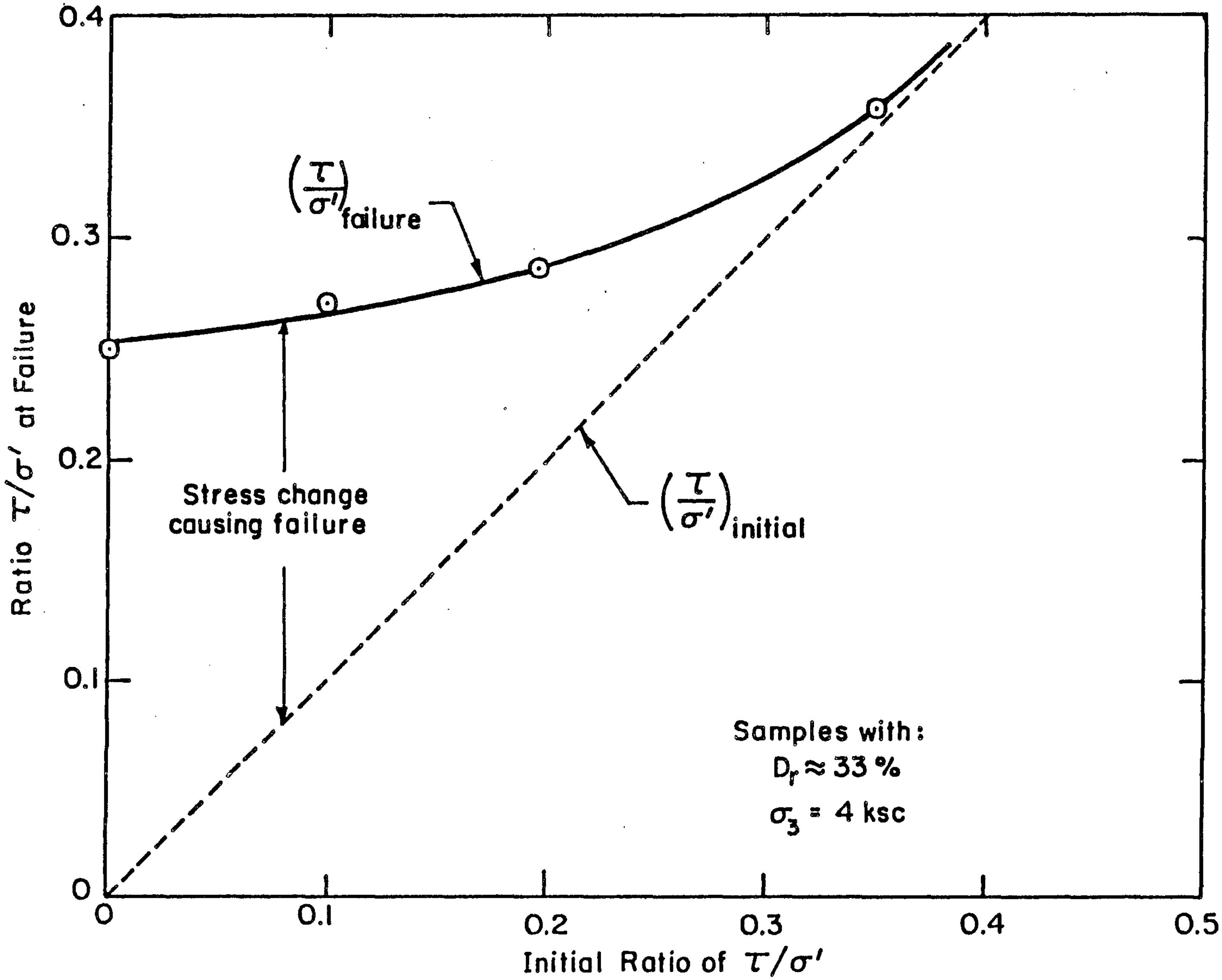


Fig. 12 INFLUENCE OF INITIAL STRESS CONDITIONS ON STRESS INCREASE REQUIRED TO CAUSE LIQUEFACTION FOR LOOSE SAND

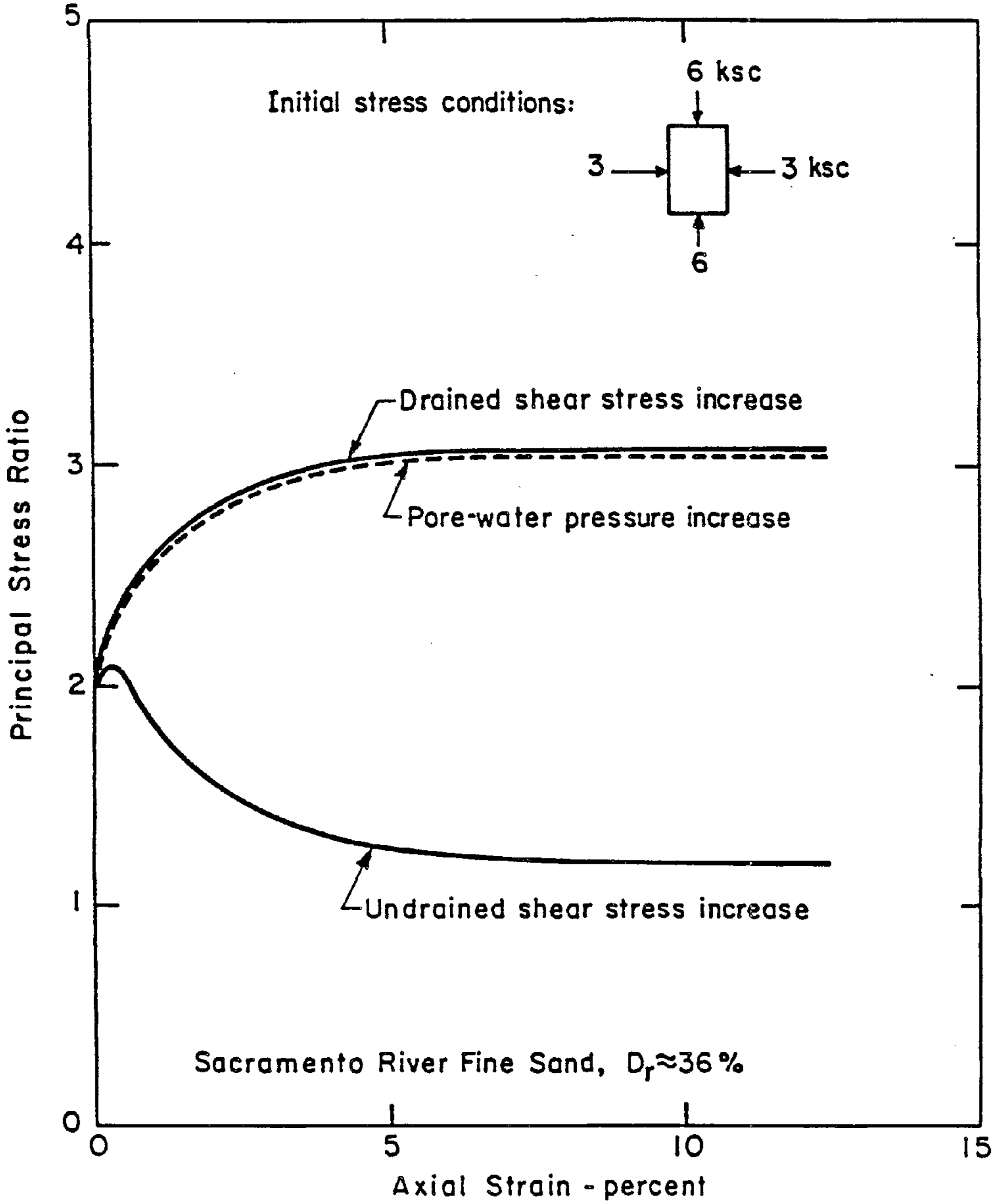


Fig. 13 INFLUENCE OF METHOD OF INDUCING FAILURE ON STRENGTH OF LOOSE SATURATED SAND

ratio of 50% brought about by increasing the axial stress slowly under drained conditions. Conversely it could sustain a change in principal stress ratio of only about 3% brought about by increasing the axial stress reasonably quickly (in a period of several minutes) under undrained conditions. For the test shown with a dashed line in Fig. 13, the sample was brought to failure by increasing the pore-water pressure inside the sample. Such changes also have the effect of increasing the principal stress ratio acting on the sample. However it may be seen that the change in principal stress ratio which must be developed to cause failure in this way is again about 50%, corresponding to the effects of drained axial or shear stress application. Test data of this type indicate that a change in principal stress ratio brought about in the field by an increase in artesian pressure would not cause liquefaction since it is effectively a drained change in stress condition. On the other hand, a small change in principal stress ratio brought about by an increase in shear stress under undrained loading conditions could possibly have caused complete liquefaction and failure of a loose saturated sand.

While such test data seems to eliminate the changes in artesian pressure as being the trigger mechanism for the liquefaction-type slide at the port of Nice, it should be noted that the increase in effective principal stress ratio brought about by a small increase in artesian pressure would make the sand deposits more vulnerable to liquefaction induced by undrained loading than they would otherwise have been.

An important implication of the test data presented in Fig. 10 is that soils which are potentially vulnerable to liquefaction are not necessarily unstable. Under drained loading conditions they can have good strength and stability, as evidenced by the data in Fig. 10. In the absence of any source of undrained loading such soils may remain perfectly stable for very long periods of time. It is only if their liquefaction potential is triggered by a source of undrained loading that they become highly unstable. Thus in dealing with deposits containing or composed of such soils, it is necessary not only to recognize their presence but also to consider possible trigger mechanisms which could lead to realization of their liquefaction potential.

The unfavorable loading conditions which trigger flow-slides in loose sands and silts or sensitive clays are those involving suddenly applied loads without the possibility for pore pressure dissipation from the soils. These loading mechanisms include seismic loading from earthquakes, relative low amplitude vibrations induced by trains, blasting or construction equipment, or changes in stresses due to sudden drawdown of water levels or removal and/or placement of earth on the ground surface. The changes in shear stress required to cause landslides under undrained loading conditions can in some cases be less than about 2% of the existing shear stress.

LIQUEFACTION CHARACTERISTICS OF NICE SAND

The results of drained and undrained tests of the type described above on samples of Nice silty sand at relative densities of 31 and 38% are shown in Fig. 14. It may be seen that the properties of the sand in this relative density range are very similar to those determined in previous investigations for other sands (see Fig. 10 for comparison). For samples consolidated under a principal stress ratio of 2 and tested under drained conditions, the strength builds up progressively with increasing strain, and the increase in deviatoric stress required to cause failure is about 130%. The angle of internal friction of the sand for effective stress conditions was shown by these tests to be about 33°.

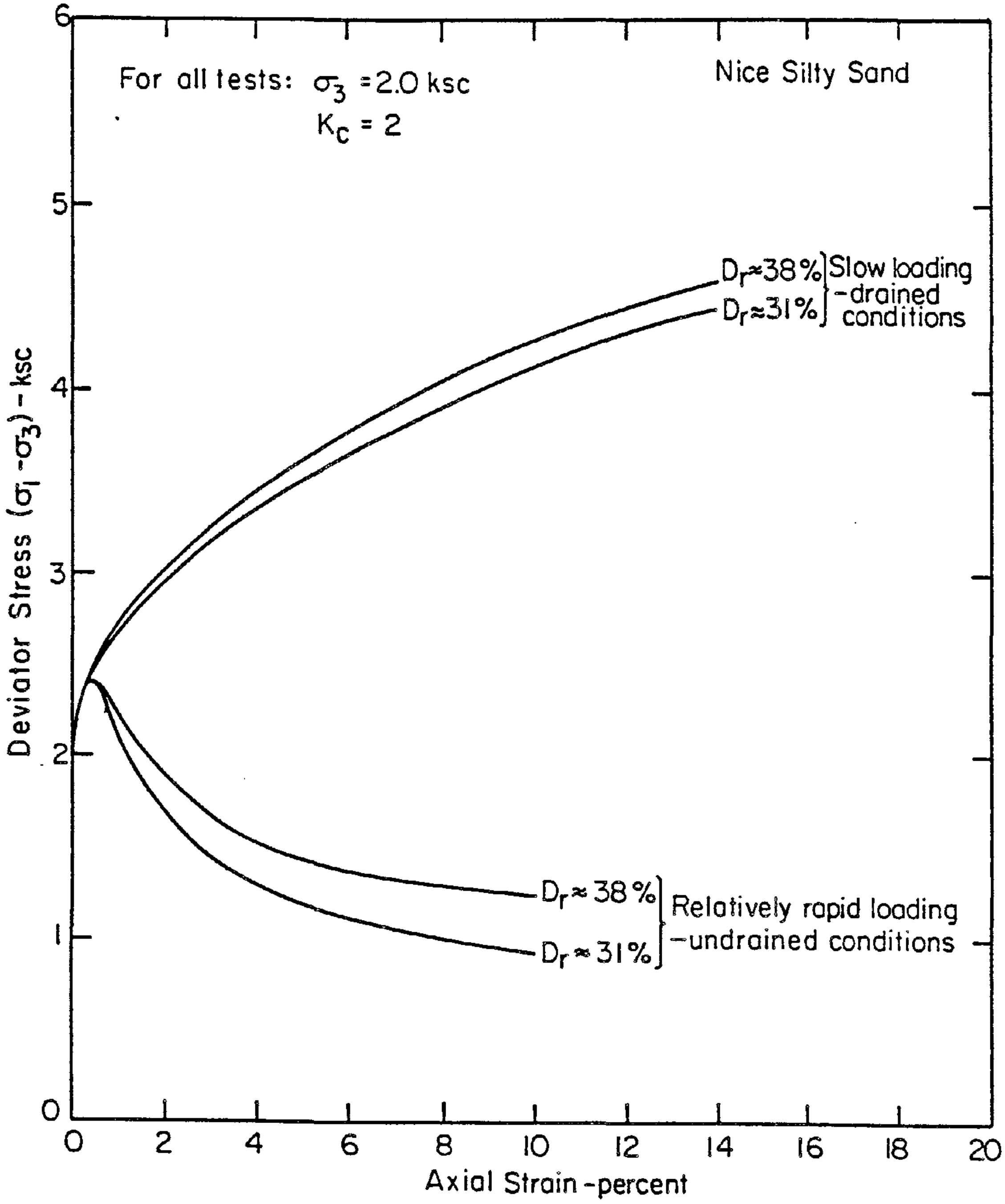


Fig. 14 STRESS-STRAIN RELATIONSHIPS IN DRAINED AND UNDRAINED TESTS ON NICE SILTY SAND

For samples consolidated to the same initial stress conditions ($K_c = 2$) and tested under undrained conditions, however, the stress increase required to cause failure was only about 18 percent, and once this failure condition was reached the samples showed a dramatic loss in resistance to deformation and collapsed in a liquefaction-type failure mode. Similar types of failure were observed in tests performed at a variety of initial stress conditions.

From the results of these tests a relationship between the initial stress conditions in the Nice sands and the stress changes required to produce a liquefaction-type failure under undrained conditions has been developed. This relationship is shown in Fig. 15. For samples with relative densities in the range of 30 to 40%, it may be seen that the increase in shear stress required to cause liquefaction decreases as the initial shear stress/normal stress ratio (τ_{fc}/σ_{fc}) on the failure plane increases, reaching values as low as 2 to 5% for initial shear stress/normal stress ratios in the range of 0.4 to 0.5.

The results of this investigation led to the following conclusions concerning the silty sand layers in the delta deposits in the vicinity of the port slide at Nice:

1. The silty sands which occur as continuous seams near the base of the clayey silt layer (that is, at a depth of about 25 m in the soil profile) have a low relative density of the order of 30 ± 8% and they are vulnerable to liquefaction under undrained loading conditions.
2. The stress increase required to trigger a liquefaction failure in these silty sands depends on the initial ratio of shear stress to effective normal stress on the potential failure plane. However for values of this ratio in the range of 0.4 to 0.5, the stress change required to trigger a

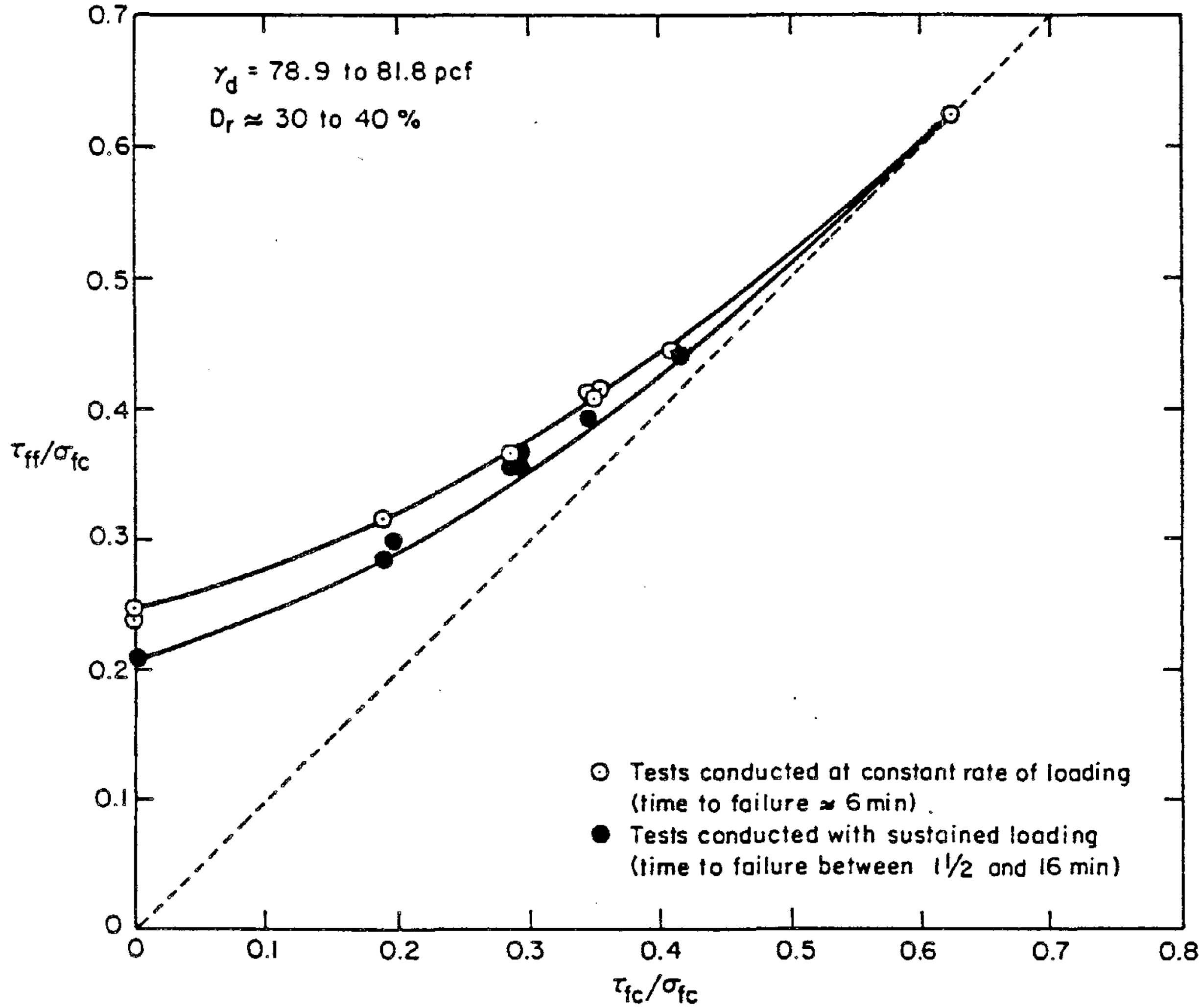


Fig. 15 RELATIONSHIP BETWEEN INITIAL STRESS CONDITIONS AND STRESS CONDITIONS AT FAILURE IN UNDRAINED TESTS ON NICE SILTY SAND

liquefaction failure under undrained (relatively rapid loading) conditions is probably only a few percent.

3. Failure of the sands by liquefaction is associated with a marked reduction in resistance to deformation. As a consequence of this strength loss the sands are also vulnerable to progressive failure, a characteristic of all materials exhibiting major strength losses once the peak strength is exceeded.

4. In the absence of any undrained stress application, the silty sands do not fail with a collapse-type mechanism. Rather they fail with a plastic-type stress-strain relationship corresponding to an effective angle of friction of about 33°.

ANALYSIS OF SLOPE STABILITY IN THE AREA OF THE PORT SLIDE

Analysis for Conditions Prior to October 15, 1979

A detailed study of the stability of the soils in the area of the port slide for conditions existing prior to October 15, 1979 has been made by the Laboratoire Central Des Ponts Et Chaussées (1981). The soil profile used for these analyses is that shown in Fig. 16.

In making these analyses it was considered that the artesian pressures from the sand and gravel deposits were dissipated only slightly in the overlying silty sand layer and were transmitted almost fully to the base of the clayey silt. Under these conditions the hydraulic gradient accompanying the upward flow of ground water through the clayey silt is about 0.2 under normal conditions. However in the few days before the port slide occurred the artesian pressure was observed to increase by about 1 m due to unusually heavy rainfall.

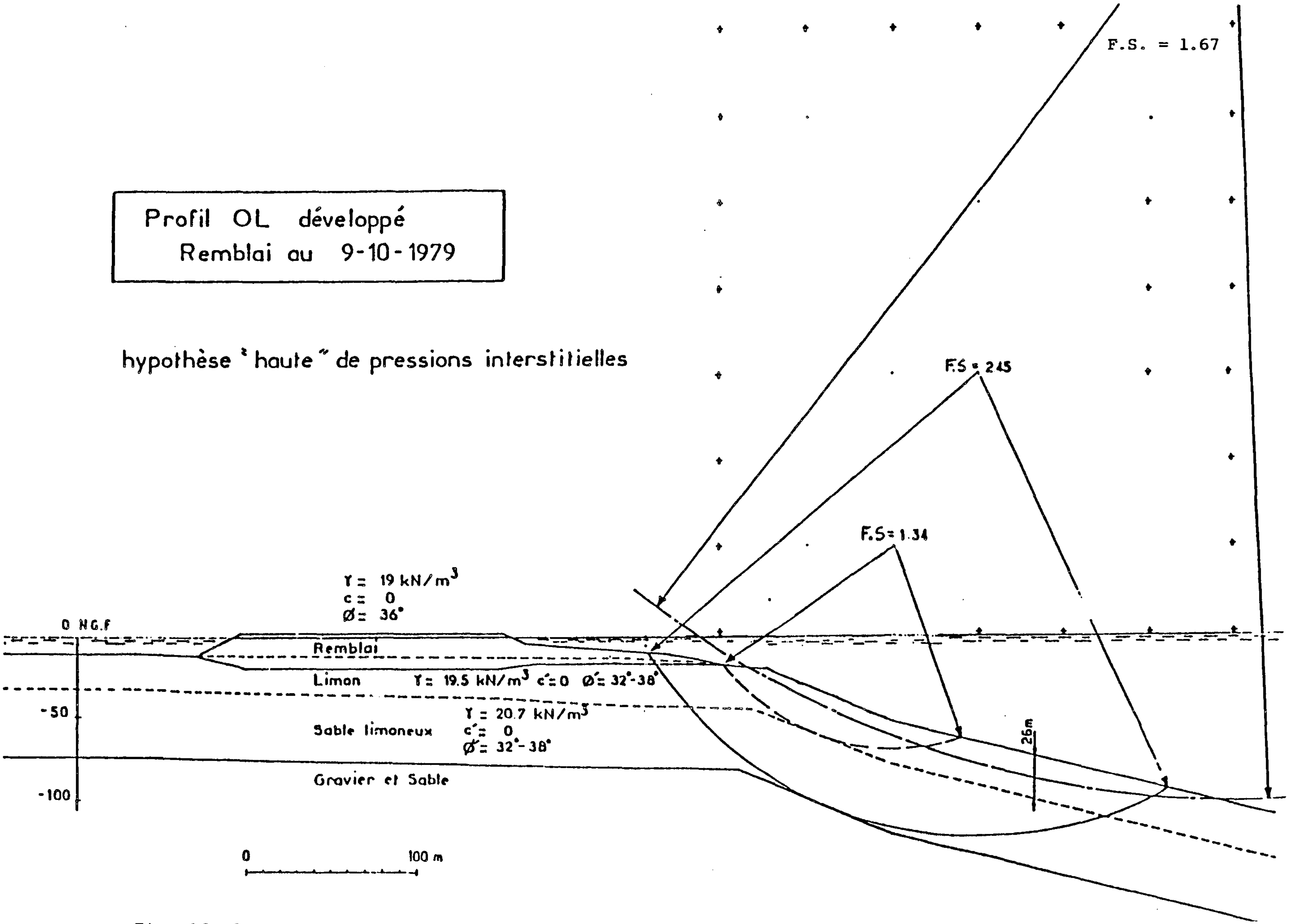


Fig. 16 RESULTS OF STABILITY ANALYSIS FOR SOIL PROFILE IN SLIDE AREA--SLOW LOADING CONDITIONS

Typical results of the analyses, made using the effective stress approach, for the conditions existing on October 9, 1979 and for an upper bound estimate of measured pore pressure conditions in the slope are shown in Fig. 16. It may be seen that the most critical deep-seated slip surface is one extending to the base of the clayey silt layer and that for this surface the computed factor of safety was 1.34. This result was obtained for an effective angle of friction in the silt and sandy silt of 32° and using Bishop's method of slope stability analysis. These latter conditions appear to provide a realistic basis for evaluating the conditions existing in the slope. Analyses performed independently as part of the present study gave generally similar results.

Studies of slope stability conditions at other times were also made by the LCPC. The results of these studies for $\varphi' = 32°$ and using the same effective stress method of stability analysis, but with pore water pressures representative of those at other times in the life of the project are shown in Table 4.

It would appear that before construction the factor of safety of the slope against a major slide was about 1.4. This value decreased to about 1.36 after placement of the fill on February 20, 1979 but then increased to about 1.45 as a result of pore pressure dissipation during the period February 20 to September 10, 1979. Based on this latter value, analyses also indicate that it would be reduced to about 1.35 by the development of the higher artesian pressures in the period October 13-16. Finally it is worthy of note that the computed factor of safety for a failure surface corresponding to the surface on which sliding is believed to have occurred, as shown in Fig. 6, was about 2.5 to 3.0. These factors of safety are indicative of a stable condition, as indeed the slope proved to be in this period of time, so long as there was no

Table 4

Computed Factors of Safety for Slip Surfaces

Extending to Bottom of Clayey Silt Layer

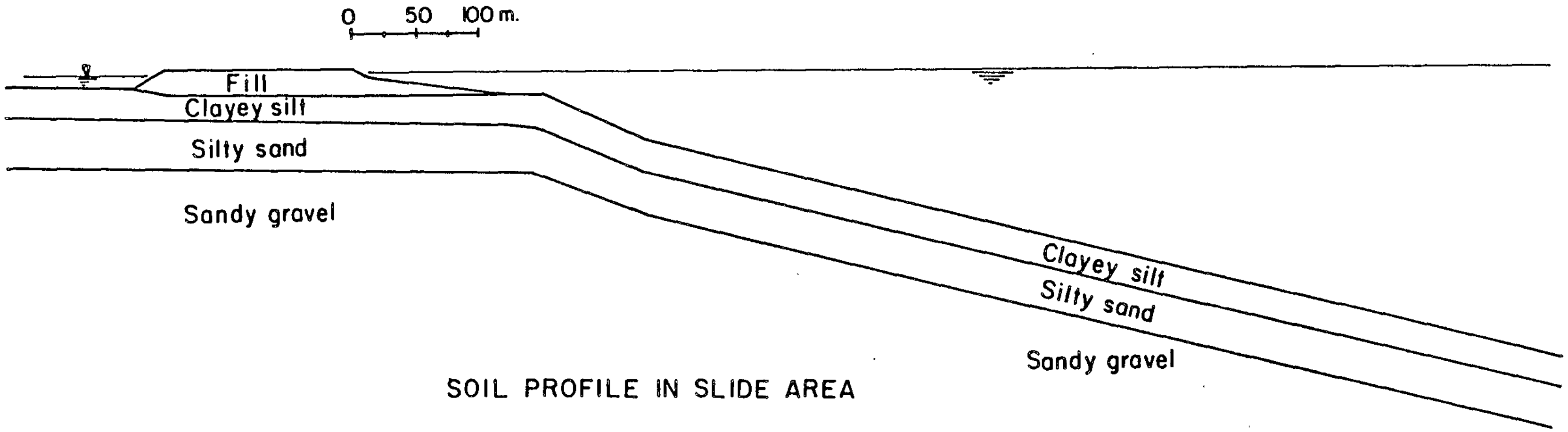
Time	Computed Factor of Safety
Before construction of fill	1.40 (LCPC Report)
After construction of fill (20 Feb. 1979)	1.36 (LCPC Report)
After construction of fill (10 Sept. 1979)	1.45 (LCPC Report)
After construction of fill (16 Oct. 1979) - with artesian pressure increase	≈1.35

significant source of undrained loading applied to the slope. Furthermore the changes in the few days before the slide are not sufficiently large to indicate instability; for practical purposes the computed factor of safety on October 16 was about the same as that existing in February, 1979 or only slightly lower than that existing prior to the start of construction.

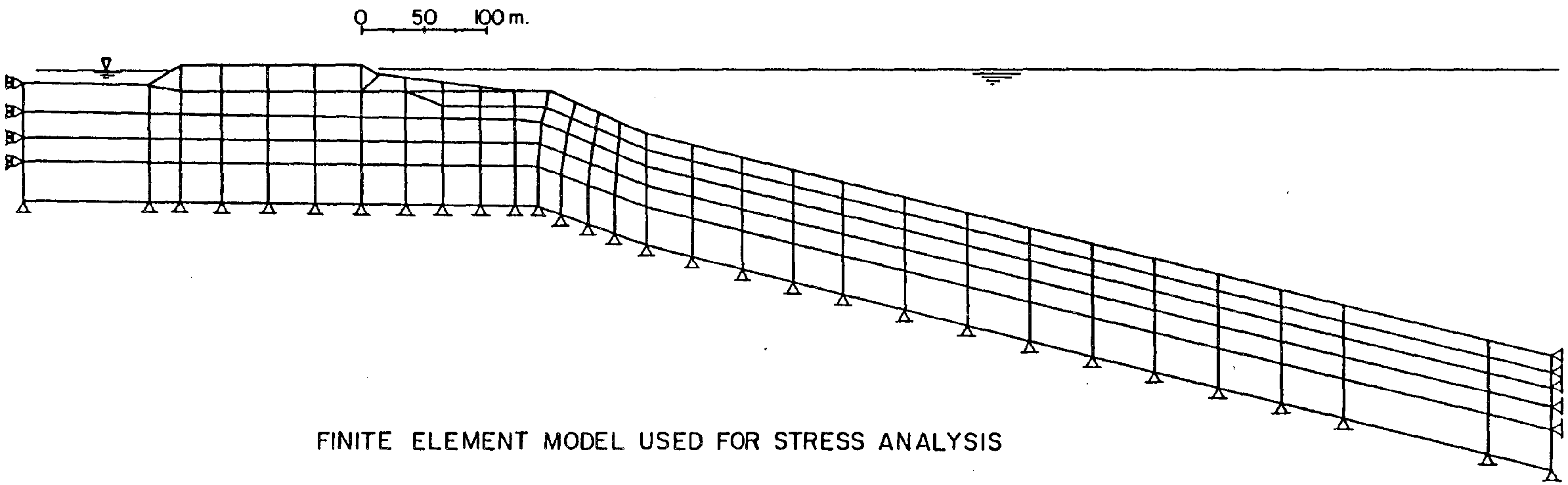
In order to investigate further the stress conditions in the slope, analyses were made using the finite-element program FEADAM (Duncan et al., 1980). This program simulates the deposition sequence of the soils and includes consideration of their non-linear stress-strain and bulk modulus characteristics. The finite-element model of the soil profile in the slide area is shown in Fig. 17. A special boundary was incorporated on the downslope side of the finite element mesh to model the stresses and displacements in this area so that they would be representative of those occurring in an infinite slope.

In making these analyses it was assumed that all construction pore pressures were dissipated and only artesian pressures existed in the soil deposits. It is recognized that this was not the case at the time of the slide but this simply means that conditions at that time were somewhat less favorable than those indicated by the analyses. The artesian pressure conditions modelled were those immediately prior to the observed slide, corresponding to an excess head of approximately 6.2 m at the base of the silty sand layer.

The results of the stress-distribution analyses are presented in Fig. 18, which shows computed values of the shear stress/effective normal stress ratios (τ_{fc}/σ_{fc}) along a surface approximating the ground surface after the slide movements had occurred. Under the sloping portion of the delta deposits it is also close to the boundary between the clayey silt and the



SOIL PROFILE IN SLIDE AREA



FINITE ELEMENT MODEL USED FOR STRESS ANALYSIS

Fig. 17 SOIL PROFILE AND FINITE ELEMENT MODEL FOR CONDITIONS IN SLIDE AREA

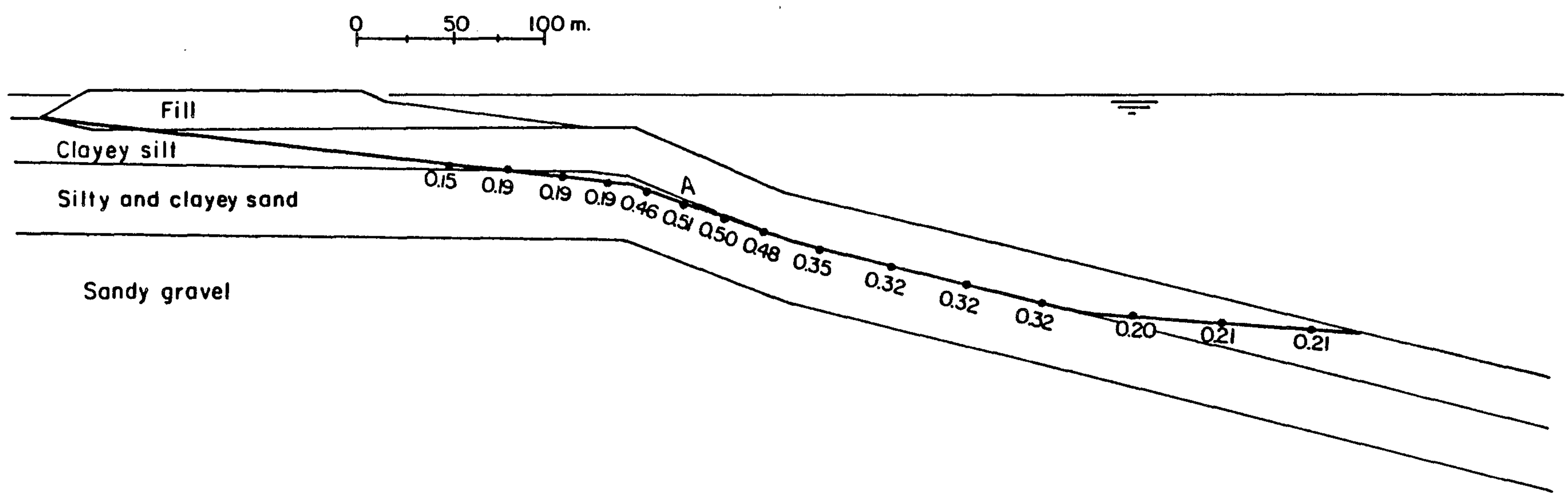


Fig. 18 COMPUTED VALUES OF τ/σ AT POINTS ALONG APPROXIMATE SLIP SURFACE
- Artesian Pressure of 6 m

silty sand layers and in close proximity to the probable configuration of the sand seams near the base of the clayey silt layer.

It may be seen that the shear stress/effective normal stress ratio varies from point to point along this potential slip surface, reaching a maximum value of about 0.51 under the steepest part of the under-water ground surface. At other points it decreases to values of the order of 0.32 to 0.35. The average value along a surface approximating the critical circle in Fig. 16 is consistent with an average factor of safety of about 1.4. These values are also indicative of a stable condition within the slope for a soil with an effective angle of friction of 33°. For such a material, the shear stress/effective normal stress ratio at failure under drained conditions would have to be about 0.63.

From these analyses it seems reasonable to conclude that:

1. Even for the unusually high artesian pressure conditions at the time that failure occurred, the computed factor of safety against a slide extending to the bottom of the clayey silt layer under drained loading conditions was about 1.35.
2. The factor of safety against a failure for a slide mass extending under the fill of the new port and bounded by the apparent failure surface was about 2.4.

These computational results are not indicative of a potentially unstable condition, despite the increase in artesian pressures, in the absence of any sudden and unanticipated loading. However they do serve as a basis for evaluating the stability of the slope against stress changes induced either by a rapid lowering of the sea level or a prolonged series of low-level vibrations.

ANALYSIS OF SLOPE STABILITY UNDER CONDITIONS OF WATER LOWERING DUE TO A TIDAL WAVE

In order to evaluate the potential for slide movements due to liquefaction in a sand seam near the base of the clayey silt layer as a result of sea level lowering it is necessary to determine:

(1) The initial stress conditions at points along a potential failure surface; these results are shown in Fig. 18.

(2) The increase in shear stress at points along the potential failure surface resulting from a sudden lowering of the sea water level. This is a standard drawdown analysis in soil mechanics and the results of such a study can be made either by limit equilibrium methods or by finite element methods. For this study the analysis was made by finite element methods which provide information on the distribution of stress changes rather than a gross average for a potential failure surface. The finite element analyses showed that for the point on the failure surface having the highest ratio of initial shear stress/effective normal stress, (point A in Fig. 18), the increase in shear stress due to a 3 m lowering of the sea level was about 1.5 to 2%.

(3) The stress increases which must be applied to each soil element along the potential failure surface in order to bring it to a condition of incipient liquefaction; this data can only be obtained at the present time by laboratory tests on reasonably representative samples. Such data for the sands in the deltaic deposits at Nice was presented in Fig. 16 and it is reproduced in Fig. 19. Comparison of these required stress increases with those induced by lowering of the sea-level, as discussed in (2) above, will indicate whether liquefaction is likely to be triggered or not at any given point.

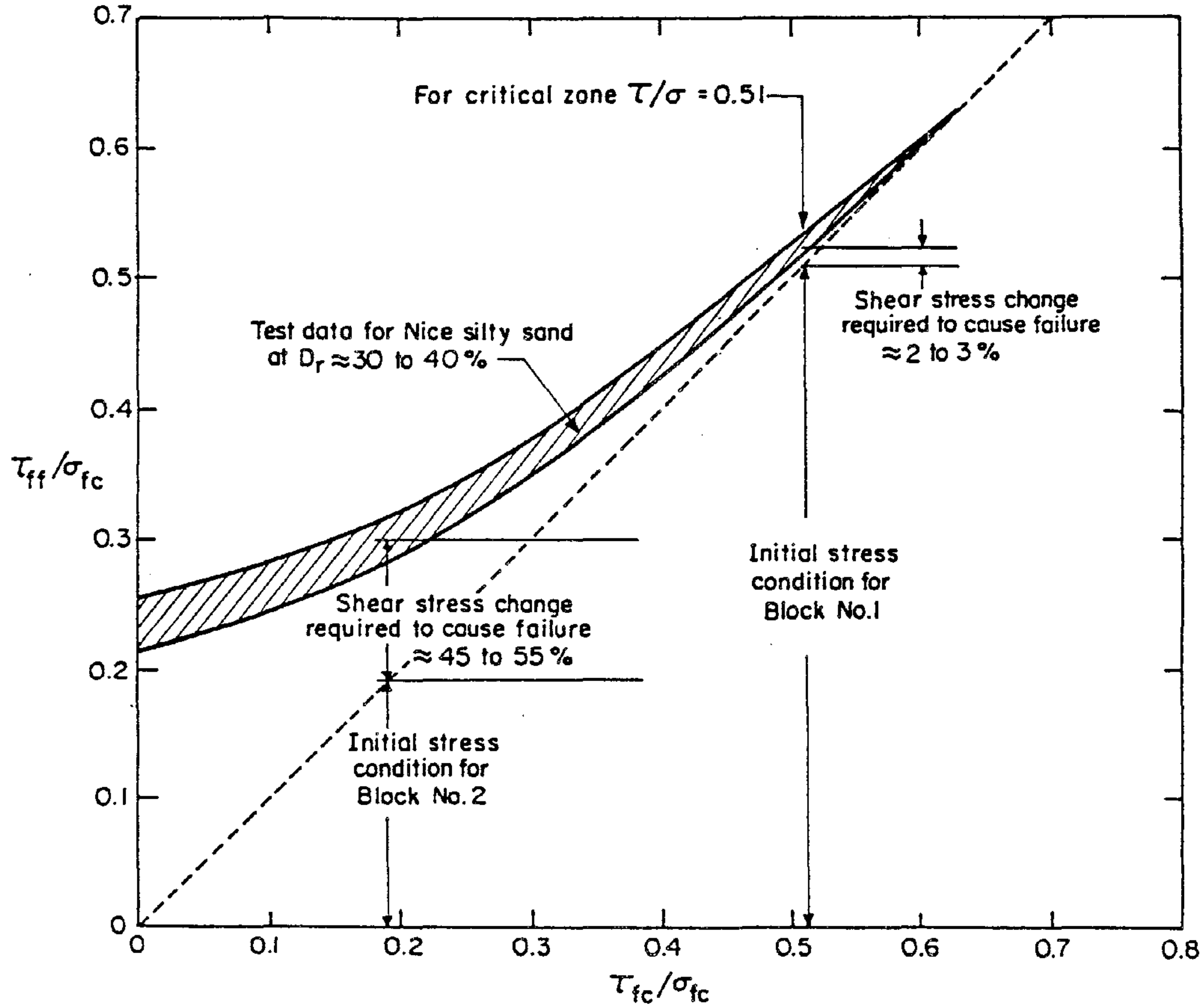


Fig. 19 EVALUATION OF STRESS CHANGES CAUSING FAILURE FOR SOIL BLOCKS IN SLIDE AREA

In making these determinations and in evaluating their significance with regard to the stability of the slope it is necessary to consider the effects of progressive failure mechanisms which can occur in materials which exhibit strong strain-softening or "brittle-type" behavior. There are two types of progressive failure in soils:

1. The progressive development along a single failure surface in a soil which has strong strain-softening characteristics.
2. The progressive development of a slide zone due to a series of sequential slides, each triggered by the occurrence of the preceding slide in the sequence. This type of progressive failure is particularly likely to develop in soils which require only small stress changes to produce a failure condition.

Both of these types of progressive failure are likely to occur in soils which fail by liquefaction as a result of the stress increases indicated by the data in Fig. 19.

Applying these principles to the soil conditions in the slide area at Nice, it may be noted from Fig. 18 that for the most highly stressed elements along the potential failure surface, the shear stress/normal stress ratio is 0.51. For this initial stress ratio (τ_{fc}/σ_{fc}), the data in Fig. 19 indicates that failure and liquefaction of the element will occur for a stress increase of 2 to 3%. The results of the drawdown analysis show that a lowering of the sea level by 3 m would increase the shear stress by about 1.5%. Thus within the limits of geotechnical engineering accuracy, and especially in view of the facts that (1) the in-situ condition of the sand is likely to be looser than that of the samples used in the laboratory testing program, and (2) the initial in-situ stresses are likely to be somewhat higher than those shown in Fig. 18 (see discussion on page 46) these conditions are representative of a potential failure condition in a zone of limited extent along the potential failure surface. This condition is shown schematically in Fig. 20(a). Once liquefaction is induced in this small zone, however, it might be expected that the accompanying loss of shear resistance will result in a transfer of shear stress to the immediately adjacent zone which will then lose shear resistance and, by the repeated development of this type of progressive failure, liquefaction will develop along the boundary ABCD. At this stage the block ABCD will slide out and down the slope, as shown in Fig. 20(b), leaving the soil profile in the configuration shown in Fig. 20(c).

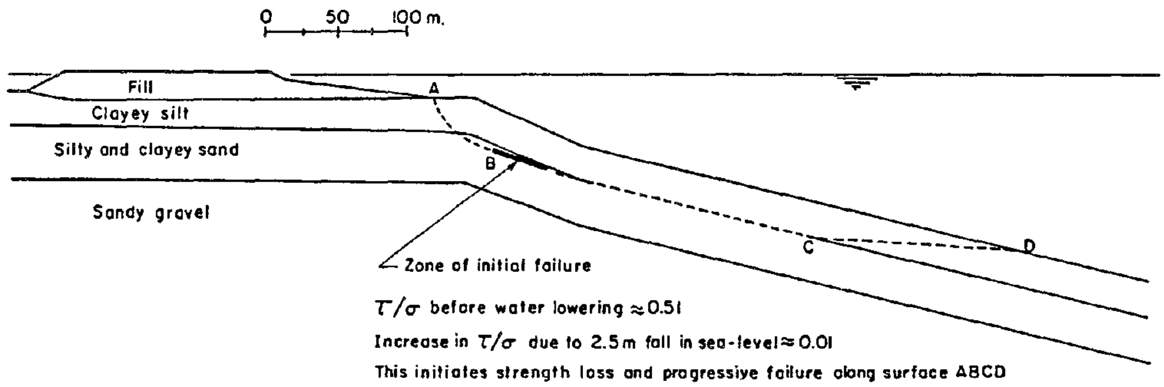
It can readily be shown that the removal of the support provided by the first soil block will produce a sufficient increase in shear stress to initiate failure of a second block, about 50 m wide, as shown in Fig. 20(d); and thus by a progressive sequence of block failures, sliding of the fill for the new port could be expected to occur as slip surfaces continue to develop by a series of regressive slides as shown in Fig. 20(d).

In this way, the analysis and test data for the sands at Nice indicate that a tidal wave producing a sea level lowering of about 3 m could trigger a liquefaction type slide similar to that which occurred on October 16, 1979.

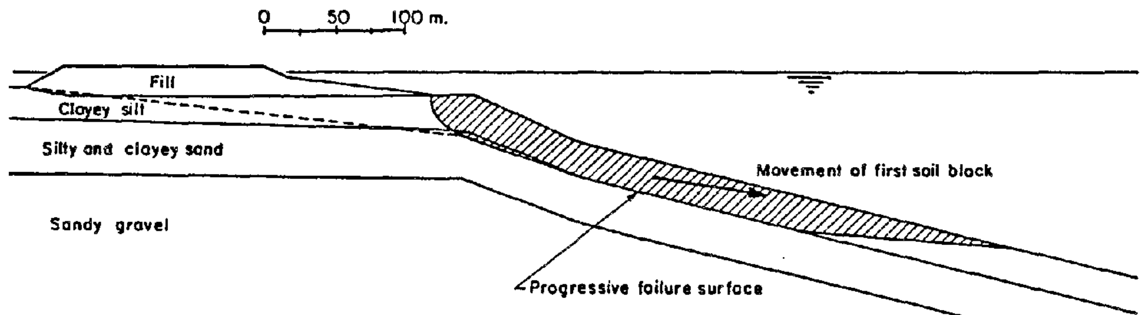
The development of this mode of failure is characteristic of liquefaction-induced landslides and would serve to explain:

1. The speed with which the slide occurred;
2. The extensive movements of the soil in the slide zone;
3. The regressive nature of the slide movements described by the witnesses of the event;

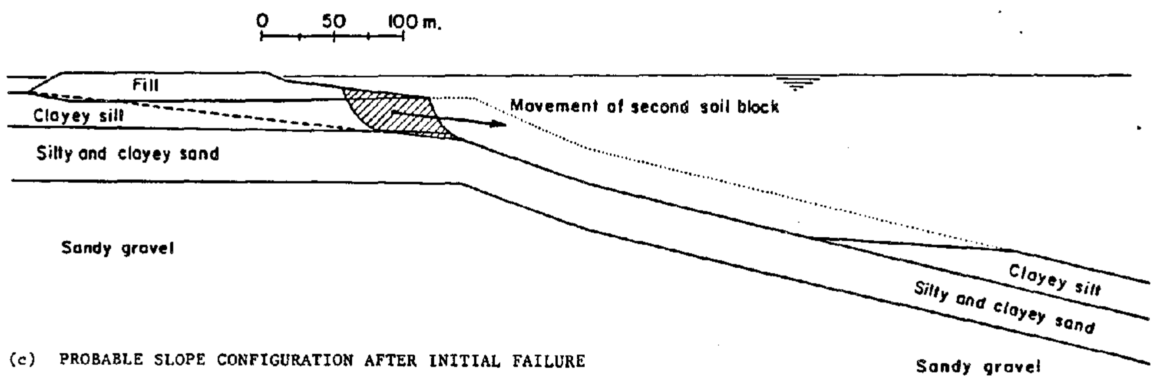
and 4. The unusual configuration of the slip surface (concave upwards) indicated by the soil profile after the sliding stopped.



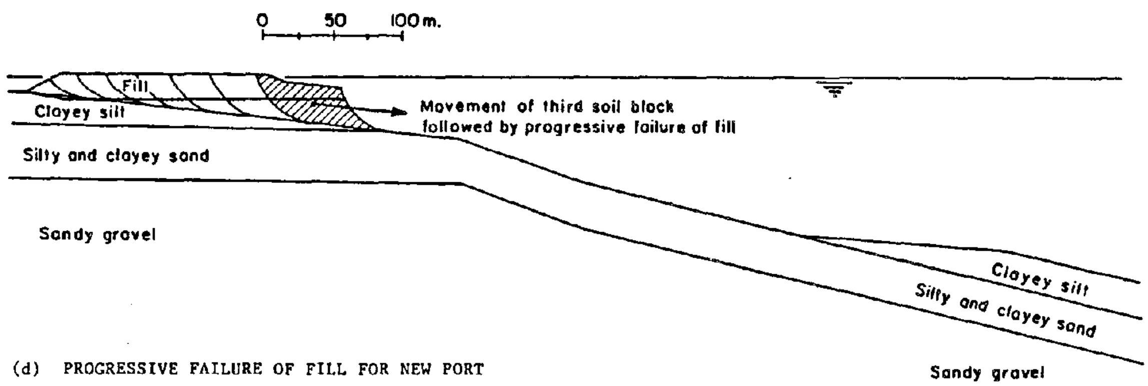
(a) ZONE OF INITIAL LIQUEFACTION INDICATED BY ANALYSES



(b) ZONE OF INITIAL FAILURE INDICATED BY ANALYSES



(c) PROBABLE SLOPE CONFIGURATION AFTER INITIAL FAILURE



(d) PROGRESSIVE FAILURE OF FILL FOR NEW PORT

Fig. 20 HYPOTHETICAL DEVELOPMENT OF SLOPE FAILURE

These aspects of the sliding are not well-explained by conventional types of sliding or slope stability analyses.

It may be noted that a slide induced by a tidal drawdown in accordance with the mechanism described above could only develop in a location where the initial geometric configuration produced a sufficiently high value of the initial shear stress/normal stress ratio and the water lowering produced the required small increase in this ratio. The required small stress increase can only be generated by water lowering if there is an above-water exposure of the soil deposit. If the entire soil deposit were submerged, then simultaneous water lowering over the entire surface area of the deposit would have no effect on the shear stresses in the deposit and there could be no triggering effect to cause liquefaction in this case. Similarly, where fill extends above the water level and a shear stress increase can therefore be induced by water level lowering, it can only trigger a liquefaction failure if it occurs in reasonably close proximity to the steeper portions of the delta deposits where relatively high values of shear stress/normal stress ratios are produced by the steeper surface configuration. If the fill were further removed from the point where the slope increases markedly in Fig. 18, two effects would occur: (1) the average increase in shear stress/normal stress ratio induced by water level lowering would be reduced and (2) the effect would be superimposed on a portion of the soil deposit having relatively low initial shear stress/normal stress ratios. In accordance with the conditions required to trigger liquefaction, as shown in Fig. 19, it would not be possible for the tidal wave to trigger liquefaction in this case.

These effects together with small variations in density of the sand from one zone to another, as evidenced by the results of cone penetration tests made in the soils below the residual portion of the port fill and off-shore from the failure zone, would serve to explain why liquefaction did not occur at other locations around the perimeter of the fill or at other locations in the steeper slopes of the delta deposit on October 16. It seems likely that it was only in the location where the required combination of effects was superimposed that sliding developed.

This would not be the case if sliding were caused, say, by a surficial slide in the steeper slopes of the under-sea delta deposit. Such a slide due to erosion or any other similar cause, could have occurred at any point on the outer slope and then triggered a progressive failure, which would ultimately undermine the fill regardless of the proximity of the fill to the steeper portion of the delta slope. That sliding occurred in only one location, where the necessary critical combination of conditions could exist, would seem to be a strong argument against the possibility of the slide being caused by the chance occurrence of a localized surficial slide.

Consideration was also given to the fact that the slope of the delta deposits near the tip of the slope was slightly steeper than the average and perhaps marginally stable (see Fig. 16). The possibility of a progressive rupture starting in this region was thus investigated. Conventional slope stability surfaces showed that a slice of soil at the surface might fail. However analyses indicated that changes in pore water pressure below the slip surface would take several weeks to months in order to equilibrate. The resulting change in the effective stress conditions could result in progressive rupture, but the time required would be many times greater than that of the observed failure. Furthermore, the other areas of the port having similar soil conditions and geometry have performed well with no observed failures. In this light, the possibility of a rapid progressive rupture developing by this mechanism was considered highly unlikely.

It is also of interest to note the influence of the artesian pressures in the delta deposits on the potential for a liquefaction failure triggered by a tidal wave. In the absence of any artesian pressure in the deposits, the maximum shear stress/normal stress ratio on the potential failure surface shown in Fig. 18 would be such that the shear stress increase required to trigger a liquefaction failure would be about 10%. Such a stress increase could not have been produced by a tidal wave of 3 m amplitude, and failure would not then have been likely to occur.

ANALYSIS OF POTENTIAL FOR SLOPE INSTABILITY DUE TO LIQUEFACTION TRIGGERED BY LOW-LEVEL VIBRATIONS

Since seismographs showed a possible source of low-level vibrations with a frequency of about 5 Hz starting in the vicinity of Nice at about 13.54 on the day of the slide, it is of interest to examine the possibility that low-level vibrations in the vicinity of the new port could have triggered the liquefaction-type slide which occurred at about 13.58.

Important in this respect is the fact that no ground vibrations were felt in this time period by anyone in the Nice area or in the port area. Charts showing human sensitivity to vibrations (e.g. Richart et al., 1970) indicate that if such vibrations were not felt, they would have to be associated with peak accelerations less than about 4×10^{-4} g. It can readily be shown that accelerations of this low magnitude would induce shear strains in the sand seams of the delta deposits less than 50 times smaller than the threshold strain at which pore pressures begin to be developed (Dobry et al., 1982). Thus it is highly improbable that vibrations too small to be noticeable to people could possibly have been responsible for triggering a liquefaction failure in the delta deposits.

SUMMARY OF RESULTS OF SLOPE STABILITY ANALYSES

In summary, it seems reasonable from the results of the analyses described above that:

1. The deltaic deposit on which the fill for the new port was placed consists of about 25 m of soft clayey silt and silty clay underlain by about 35 m of silty and clayey sand and then by a deposit of dense sand and gravel. Within the upper layers of clayey silt and clayey sand are a series of thin seams of fine sand, silty sand and silt. Artesian pressures in the sand and gravel are normally about 5.5 m but at the time of the slide they had built up to about 6.5 m. Most of the deposit, including large zones of the clayey silt, silty clay, clayey sand and the underlying dense sand and gravel does not appear to be vulnerable to liquefaction. However soil layers consisting of sands, silty sands and silt, which could be potentially vulnerable to liquefaction, can be identified within the deposits. Field and laboratory tests confirm that the thin layers of silty sand at depths of 15 to 30 m have the potential to liquefy under undrained loading conditions.
2. Effective stress analyses applicable to drained loading conditions do not indicate any basis for a slide occurring in the new port area, despite the presence of liquefiable silty sand layers and the small increase in artesian pressures which developed due to heavy rainfall in the 3 days preceding the port slide. Thus in the absence of any source of undrained loading it is difficult to see why a slide should occur in the port area. Under normal drained loading conditions, even potentially liquefiable soils may remain

stable for very long periods of time. It is only if their liquefaction potential is triggered by a source of undrained loading (such as tidal waves, earthquakes, etc.) that they become highly unstable.

3. There was no earthquake near Nice at the time of the slide and analyses show that it is very unlikely that low-level vibrations, too weak to be felt by people in the port area, could have been the trigger mechanism causing a liquefaction-type failure in the delta sands.

4. Analyses based on studies of the stress conditions in the soil profile and test data for the silty sands from the delta deposits show that a lowering of the sea level by 3 m due to a tidal wave would probably trigger a liquefaction failure and sliding of the type that occurred at the new port on October 16, 1979. Such a slide could not have occurred due to liquefaction of the silty sands if they had been in a denser or drained condition.

EXAMINATION OF POSSIBLE CAUSES OF THE PORT SLIDE

Factors relevant to a determination of the probable cause of the port slide have been discussed in the preceding sections of this report. It would seem desirable, before drawing final conclusions, to summarize briefly the key pieces of evidence which throw light on this question:

1. The seismograph records show that unusual seismic waves were generated in the area of Nice starting at about 13.54 on October 16, 1979. Since there is no knowledge of any on-shore cause of these observations at this time, it seems highly probable that the seismic waves were produced as a result of some phenomenon occurring off-shore.

2. The maregraph records at Nice (Port Lympia), Villefranche, and Mandelieu la Napoule show clearly that the tidal wave which occurred along a 60 kilometer section of the coastline at about 14.00 on October 16, 1979 involved first a significant lowering of the sea level and then a rise in water level, followed by a prolonged series of fluctuations with a period of about 8 minutes.
3. The testimony of witnesses shows clearly that the port slide began at about 13.58 on October 16, and they provide very good evidence that lowering of the sea level at Nice and a number of other locations along the coast started before the port slide occurred. Thus it appears that the initial water lowering along this section of coast-line was due to some cause other than the port slide and that the port slide occurred when the sea water level was near its lowest point.
4. The large volume of material which caused the cable breaks and was deposited in the area of the cables 90 to 120 kms off-shore (>100 million cu. m) is greater than that provided by the port slide (≈ 10 million cu. m) and indicates that sliding must have occurred in the underwater canyons or on the sea bed.
5. Bathymetry and submarine observations show that a major submarine canyon slide, or several slides involving over 100 million cu. meters of soil, have occurred since 1973 near the confluence of the Var and Paillon canyons. Since slides of this magnitude would be expected to cause significant tidal waves, and none were reported between 1973 and October 16, 1979, a submarine canyon slide associated with the tidal wave on October 16 is clearly the most likely possibility.
6. The observed amplitudes of the waves at many points along the coast are not consistent with those expected and computed to be generated by a

tidal wave caused by the port slide alone. Thus some other source of the tidal wave effects is clearly a strong possibility.

7. It is not likely that a tidal wave due to a canyon slide triggered by the port slide could have reached the coast-line before about 14:10 at the earliest. Before then large tidal wave effects had been noted over a length of 50 km of coast-line.

8. Geotechnical engineering considerations provide no clear explanation why a slide of the port should occur on October 16, 1979 under normal drained loading conditions or due to a 1 m increase in the normal artesian pressure condition. The slide had all the characteristics of a liquefaction slide and this type of failure often results from increases in pore water pressure generated by the strains produced by the slide movements themselves. Nevertheless some triggering mechanism is required to cause the initial strain which leads to progressive pore water pressure generation.

9. Past experiences, analytical considerations, and soil test data for samples of sand taken from the deposits at Nice, all provide a clear rationale for a liquefaction slide occurring due to a relatively rapid loading associated with a 3 m lowering of the sea level due to tidal wave effects. Geotechnical engineering considerations lead to the conclusions that this is the most likely cause of the port slide. Liquefaction induced by a prolonged series of low amplitude vibrations, or a stress redistribution due to a surficial slide on the off-shore slope of the delta deposits, while technically feasible, is not likely to have been the cause of sliding in this case.

10. Consideration of the stress conditions in the critical parts of the delta deposits and the possible stress changes indicate that two types

of progressive failure must have played a key role in the development of this major slide.

1. Progressive failure within elements of soil at very slow strain levels, similar to those which can be observed in laboratory test specimens once a small pore pressure change has been initiated.

and 2. Progressive failure of successive slide masses within the failure zone.

Thus liquefaction-type slides are complex phenomena which require special analytical considerations; they cannot readily be analyzed by conventional effective stress methods of stability analysis.

CONCLUSION

At the beginning of this report it was suggested that there are two possible hypotheses for the events at Nice on the 16th of October, 1979:

Hypothesis No. 1

Starting at 13.58, a failure occurred for no readily explainable reason in the port fill, resulting in a landslide which:

(1) caused a tidal wave which was observed at many points along the coast,

and (2) flowed along the Var Canyon, and either (a) undermined the walls of the canyon causing a series of collapses which increased the volume of slide debris (10 million cu. meters from the port slide) to something of the order of 200 million cu. meters which flowed out to sea and broke cables 90 and 120 kms off-shore or (b) picked up material from the bottom of the canyon and the sea bed as it flowed until the volume was of the order of 200 million cu. meters which flowed out to sea and broke the cables.

Hypothesis No. 2

Starting at about 13.54, a submarine slope failure occurred for no readily explainable reason in the walls of the Var Canyon about 15 kms off-shore from Nice. This submarine slope collapse involved over 100 million cu. meters of marl which fell in an underwater avalanche and

(1) caused a tidal wave which was observed at many points along the coast and at Nice resulted in a major slide in a fill which had recently been placed on a deposit of clayey silt containing seams of fine sand for construction of a new port.

and (2) broke down as the movement progressed and moved out to sea as a flow slide and a turbidity current causing cable breaks 90 and 120 kms off-shore.

On the basis of the key pieces of evidence discussed above, it would appear that a tidal drawdown associated with a tidal wave resulting from a submarine slide in the Var Canyon about 15 kms off-shore, is by far the most probable cause of the slide of the port fill (Hypothesis No. 2). There is little possibility that low level vibrations could have been a triggering mechanism for the slide and there is no apparent reason why a surficial slide on the outer face of the delta deposits, which could possibly have served as a triggering mechanism, should have occurred. Thus there is a very high probability that liquefaction triggered by a tidal wave, a phenomenon which is consistent with past experiences with similar deposits, with the observations of the majority of the witnesses at Nice and along a 60 km length of coastline, with records of the event, and with a geotechnical evaluation of the mechanics of failure, was the cause of the slide on October 16, 1979.

It is pertinent to question, of course: What caused the off-shore slide that produced the tidal wave? Most tidal waves are due either to strong earthquake shaking, which may induce fault movements on the ocean floor or submarine landslides which displace water leading to tidal fluctuations, or simply to landslides which occur off-shore for no known cause. Examples of the latter include:

Date	Location	Tidal Wave Height	Cause
1888	Trondheim Harbor	5 to 7 m.	Presumed due to submarine slide
5/2/1930	Orkdals Fjord	Small	Presumed due to submarine slide
11/17/1972	Washington Coast	2 m	Presumed due to submarine slide
10/24/1974	Kitimat, Canada	6 m	Attributed to submarine slide
7/23/1979	Jakarta	9 m	Collapse of rock into sea
7/2/1981	Palavas (Coast of France)	1 m	Presumed due to submarine slide but may be due to atmospheric effects

There are probably many others. However it may be noted that four significant tidal waves, three of which are attributed to submarine slides or rock-falls, are known to have occurred in the period 1972-1981. Presumably similar numbers of events have occurred in earlier times but complete records have not been kept.

The evidence indicates that tidal waves apparently occur randomly due to submarine slides not related to seismic activity. There are also records that tidal waves of lesser heights than 3 m have occurred in previous centuries along the coast of France near Nice. It is reasonable to believe, therefore, that an off-shore slide occurred in the Var Canyon off the coast of Nice on October 16, 1979 and produced the observed tidal wave effects which in turn, triggered the failure of the port fill and the underlying deposits.

The large blocks of marl from the off-shore canyon walls, which were observed by submarine surveys to be located now on the bottom of the canyon, together with the disappearance of about 100 million cu. yds. of materials from the canyon walls some 15 km offshore would seem to provide convincing evidence of the fact that a major off-shore slide occurred in the canyon walls sometime since the last previous survey in 1973. Since such a slide would inevitably have caused a substantial tidal wave, whenever it occurred, and the only such wave observed between 1973 and 1979 was that occurring in association with the slide of the port fill on Oct. 16, 1979, it seems highly probable that the two events were related and the timing of events clearly demonstrates that the tidal wave effects preceded the onset of sliding. The 100-ton blocks of marl now standing on the bottom of the canyon would thus seem to provide silent but eloquent testimony that such an off-shore event must in fact have occurred and caused the wave which in turn led to the failure of the port fill at about 1358 in the afternoon when the water level was abruptly lowered from its normal elevation.

The most surprising aspect of this event is not that the tidal wave was the cause of the slide but the fact that the shear stress change which apparently triggered the slide was only about 1.5 to 2%. This fact reinforces previous observations that flow-type slides due to liquefaction are sometimes induced by both seismic and non-seismic events causing very small undrained stress changes in soil deposits which are especially vulnerable to such effects.

It also has three very important engineering implications: (1) that very loose sand deposits with high initial shear stress/normal stress conditions are very vulnerable to very small changes in shear stress which may occur under undrained loading conditions, even though they may sustain significant changes in shear stress applied under drained stress conditions; (2) that it is important to take into account such high initial shear stress/normal stress ratios in evaluating the stability of deposits containing sand seams or layers or consisting entirely of sand; and (3) that progressive failure in different forms apparently plays a major role during liquefaction-type slides of the type which occurred at the Port of Nice.

ACKNOWLEDGEMENTS

The studies described in this report were sponsored by the Direction Departementale de L'Equipement des Alpes-Maritimes of France and the U.S. National Science Foundation. This support is gratefully acknowledged. Special thanks are also due to M. A. Gautier, whose assistance in providing data was invaluable to the conduct of the study.

REFERENCES

Aas, G. (1981), "Stability of Natural Slopes in Quick Clays," Procs., 10th ICSMFE, Stockholm, Sweden, 1981, pp. 333-338

Andresen, A. and Bjerrum, L. (1967), "Slides in Subaqueous Slopes in Loose Sand and Silt," Marine Geotechnique, Ed. by A. F. Richards, Urbana, University of Illinois Press, pp. 221-239. Also publ. in Norwegian Geotechnical Institute, Publ. 81.

Casagrande, A. (1975), "Liquefaction and Cyclic Deformation of Sands, A Critical Review," Procs., 5th Pan-American Conference on Soil Mechanics and Foundation Engineering, Vol. 5, Buenos Aires, Argentina, 1975, pp. 79-133.

Casagrande, A. (1984), "Reflections on Some Unfinished Tasks," First Nabor Carrillo Lecture of the Mexican Society for Soil Mechanics presented at the 6th National Meeting of the Society in November, 1972 and published in 1984.

Castro, G. (1969), "Liquefaction of Sands," Thesis presented to Harvard University at Cambridge, Mass., in 1969, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

Castro, G. and Poulos, S. J. (1977), "Factors Affecting Liquefaction and Cyclic Mobility," Journal of the Geotechnical Engineering Division, ASCE, Vol. 103, No. GT6, 1977, pp. 501-516.

Dobry, R., Ladd, R. S., Yokel, F. Y., Chung, R. M. and Powell, D. (1982), "Prediction of Pore Water Pressure Buildup and Liquefaction of Sands During Earthquakes by the Cyclic Strain Method," Building Science Series 138, National Bureau of Standards, U.S. Dept. of Commerce, U.S. Govt. Printing Office, Washington, D.C., 150 pages.

Douglas, Bruce J. and Olsen, Richard S. (1981), "Soil Classification Using Electric Cone Penetrometer," Procs., Cone Penetration Testing and Experience, ASCE National Convention, St. Louis, Missouri, Oct. 26-30, 1981, G. M. Norris and R. D. Holtz editors, pp. 209-227.

Duncan, J. M., Wong, K. S. and Ozawa, Y. (1980), "FEADAM: A Computer Program for Finite Element Analysis of Dams," Geotechnical Engineering Report No. UCB/GT/80-02, University of California, Berkeley, December, 64 pages.

Edgers, Lewis and Karlsrud, Kjell (1982), "Soil Flows Generated by Submarine Slides--Case Studies and Consequences," Procs., Intl. Conf. on Behavior of Offshore Structures, Vol. 2, Cambridge, Mass., pp. 425-437.

Gennesseaux, de Maurice, Mauffret, Alain and Pautotet, Guy (1980), "Geologie Marine," C. R. Acad. Sc., Paris, t. 290, 14 April 1980.

Gibbs, H. J. and Holtz, W. G. (1957), "Research on Determining the Density of Sands by Spoon Penetration Testing," Procs., 4th Intl. Conf. on Soil Mechanics and Foundation Engineering, London, England, Butterworth, Inc., Vol. 1, 1957.

Kramer, Steven L. and Seed, H. Bolton (1988) "Initiation of Soil Liquefaction Under Static Loading Conditions," Journal of the Geotechnical Engineering Division, ASCE, Vol. 114, No. 4, April 1988.

Laboratoire Central Des Ponts Et Chaussées (1981), "Essai de Reconstitution des Condition Geotechniques et Hydrauliques a l'Emplacement du Port," Mission D'Inspection Pluridisciplinaire (M.I.P.) Pour le Sinistre du 16 Octobre 1979 au Noweau Port de Nice, March, 32 p.

Poulos, Steve J., Castro, Gonzalo and France, John W. (1985), "Liquefaction Evaluation Procedure," Journal of the Geotechnical Engineering Division, ASCE, Vol. 111, No. 6, June, 1985, pp. 772-792.

Richart, F. E., Jr., Hall, J. R., Jr. and Woods, R. D., (1970), Vibrations of Soils and Foundations," Prentice-Hall International Series in Theoretical and Applied Mechanics, N. M. Newmark and W. J. Hall, editors, 414 pages.

Schmertmann, J. H. (1978), "Cone Penetration Test Performance and Design," Report No. FHWA-TS-78-209, U.S. Department of Transportation, 1978.

Seed, H. Bolton (1968), "Landslides During Earthquakes Due to Liquefaction," The Fourth Terzaghi Lecture, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 94, No. SM5, Proc. Paper 6110, September, 1968, pp. 1053-1122.

Seed, H. Bolton (1983), "Stability of Port Fills and Coastal Deposits," Procs., Seventh Asian Regional Conference on Soil Mechanics and Foundation Engineering, Technion-Israel Institute of Technology, Haifa, Israel, Aug. 14-19, 1983, 11 pages.

Terzaghi, Karl (1956), "Varieties of Submarine Slope Failures," Harvard Soil Mechanics Series No. 52, Harvard University, Cambridge, Mass.

Tokimatsu, Kohji and Seed, H. Bolton (1987) "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of the Geotechnical Engineering Division, ASCE, Vol. 113, No. 8, August, 1987, pp. 861-878.

Wiegel, Robert L. (1980), "Engineering Aspects of Beach and Nearshore Bottom Changes," Procs., 4th Conf. on Ocean Engineering in Republic of China, Sept. 1980, 24 pages.

EARTHQUAKE ENGINEERING RESEARCH CENTER REPORT SERIES

EERC reports are available from the National Information Service for Earthquake Engineering(NISEE) and from the National Technical Information Service(NTIS). Numbers in parentheses are Accession Numbers assigned by the National Technical Information Service; these are followed by a price code. Contact NTIS, 5285 Port Royal Road, Springfield Virginia, 22161 for more information. Reports without Accession Numbers were not available from NTIS at the time of printing. For a current complete list of EERC reports (from EERC 67-1) and availablity information, please contact University of California, EERC, NISEE, 1301 South 46th Street, Richmond, California 94804.

UCB/EERC-80/01 "Earthquake Response of Concrete Gravity Dams Including Hydrodynamic and Foundation Interaction Effects," by Chopra, A.K., Chakrabarti, P. and Gupta, S., January 1980, (AD-A087297)A10.

UCB/EERC-80/02 "Rocking Response of Rigid Blocks to Earthquakes," by Yim, C.S., Chopra, A.K. and Penzien, J., January 1980, (PB80 166 002)A04.

UCB/EERC-80/03 "Optimum Inelastic Design of Seismic-Resistant Reinforced Concrete Frame Structures," by Zagajeski, S.W. and Bertero, V.V., January 1980, (PB80 164 635)A06.

UCB/EERC-80/04 "Effects of Amount and Arrangement of Wall-Panel Reinforcement on Hysteretic Behavior of Reinforced Concrete Walls," by Iliya, R. and Bertero, V.V., February 1980, (PB81 122 525)A09.

UCB/EERC-80/05 "Shaking Table Research on Concrete Dam Models," by Niwa, A. and Clough, R.W., September 1980, (PB81 122 368)A06.

UCB/EERC-80/06 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 1a): Piping with Energy Absorbing Restrainers: Parameter Study on Small Systems," by Powell, G.H., Oughourlian, C. and Simons, J., June 1980.

UCB/EERC-80/07 "Inelastic Torsional Response of Structures Subjected to Earthquake Ground Motions," by Yamazaki, Y., April 1980, (PB81 122 327)A08.

UCB/EERC-80/08 "Study of X-Braced Steel Frame Structures under Earthquake Simulation," by Ghanaat, Y., April 1980, (PB81 122 335)A11.

UCB/EERC-80/09 "Hybrid Modelling of Soil-Structure Interaction," by Gupta, S., Lin, T.W. and Penzien, J., May 1980, (PB81 122 319)A07.

UCB/EERC-80/10 "General Applicability of a Nonlinear Model of a One Story Steel Frame," by Sveinsson, B.I. and McNiven, H.D., May 1980, (PB81 124 877)A06.

UCB/EERC-80/11 "A Green-Function Method for Wave Interaction with a Submerged Body," by Kioka, W., April 1980, (PB81 122 269)A07.

UCB/EERC-80/12 "Hydrodynamic Pressure and Added Mass for Axisymmetric Bodies.," by Nilrat, F., May 1980, (PB81 122 343)A08.

UCB/EERC-80/13 "Treatment of Non-Linear Drag Forces Acting on Offshore Platforms," by Dao, B.V. and Penzien, J., May 1980, (PB81 153 413)A07.

UCB/EERC-80/14 "2D Plane/Axisymmetric Solid Element (Type 3-Elastic or Elastic-Perfectly Plastic)for the ANSR-II Program," by Mondkar, D.P. and Powell, G.H., July 1980, (PB81 122 350)A03.

UCB/EERC-80/15 "A Response Spectrum Method for Random Vibrations," by Der Kiureghian, A., June 1981, (PB81 122 301)A03.

UCB/EERC-80/16 "Cyclic Inelastic Buckling of Tubular Steel Braces," by Zayas, V.A., Popov, E.P. and Martin, S.A., June 1981, (PB81 124 885)A10.

UCB/EERC-80/17 "Dynamic Response of Simple Arch Dams Including Hydrodynamic Interaction," by Porter, C.S. and Chopra, A.K., July 1981, (PB81 124 000)A13.

UCB/EERC-80/18 "Experimental Testing of a Friction Damped Aseismic Base Isolation System with Fail-Safe Characteristics," by Kelly, J.M., Beucke, K.E. and Skinner, M.S., July 1980, (PB81 148 595)A04.

UCB/EERC-80/19 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol.1B): Stochastic Seismic Analyses of Nuclear Power Plant Structures and Piping Systems Subjected to Multiple Supported Excitations," by Lee, M.C. and Penzien, J., June 1980, (PB82 201 872)A08.

UCB/EERC-80/20 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 1C): Numerical Method for Dynamic Substructure Analysis," by Dickens, J.M. and Wilson, E.L., June 1980.

UCB/EERC-80/21 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 2): Development and Testing of Restraints for Nuclear Piping Systems," by Kelly, J.M. and Skinner, M.S., June 1980.

UCB/EERC-80/22 "3D Solid Element (Type 4-Elastic or Elastic-Perfectly-Plastic) for the ANSR-II Program," by Mondkar, D.P. and Powell, G.H., July 1980, (PB81 123 242)A03.

UCB/EERC-80/23 "Gap-Friction Element (Type 5) for the Ansr-II Program," by Mondkar, D.P. and Powell, G.H., July 1980, (PB81 122 285)A03.

UCB/EERC-80/24 "U-Bar Restraint Element (Type 11) for the ANSR-II Program," by Oughourlian, C. and Powell, G.H., July 1980, (PB81 122 293)A03.

UCB/EERC-80/25 "Testing of a Natural Rubber Base Isolation System by an Explosively Simulated Earthquake," by Kelly, J.M., August 1980, (PB81 201 360)A04.

UCB/EERC-80/26 "Input Identification from Structural Vibrational Response," by Hu, Y., August 1980, (PB81 152 308)A05.

UCB/EERC-80/27 "Cyclic Inelastic Behavior of Steel Offshore Structures," by Zayas, V.A., Mahin, S.A. and Popov, E.P., August 1980, (PB81 196 180)A15.

UCB/EERC-80/28 "Shaking Table Testing of a Reinforced Concrete Frame with Biaxial Response," by Oliva, M.G., October 1980, (PB81 154 304)A10.

UCB/EERC-80/29 "Dynamic Properties of a Twelve-Story Prefabricated Panel Building," by Bouwkamp, J.G., Kollegger, J.P. and Stephen, R.M., October 1980, (PB82 138 777)A07.

UCB/EERC-80/30 "Dynamic Properties of an Eight-Story Prefabricated Panel Building," by Bouwkamp, J.G., Kollegger, J.P. and Stephen, R.M., October 1980, (PB81 200 313)A05.

UCB/EERC-80/31 "Predictive Dynamic Response of Panel Type Structures under Earthquakes," by Kollegger, J.P. and Bouwkamp, J.G., October 1980, (PB81 152 316)A04.

UCB/EERC-80/32 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 3): Testing of Commercial Steels in Low-Cycle Torsional Fatigue," by Spanner, P., Parker, E.R., Jongewaard, E. and Dory, M., 1980.

UCB/EERC-80/33 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 4): Shaking Table Tests of Piping Systems with Energy-Absorbing Restrainers," by Stiemer, S.F. and Godden, W.G., September 1980, (PB82 201 880)A05.

UCB/EERC-80/34 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 5): Summary Report," by Spencer, P., 1980.

UCB/EERC-80/35 "Experimental Testing of an Energy-Absorbing Base Isolation System," by Kelly, J.M., Skinner, M.S. and Beucke, K.E., October 1980, (PB81 154 072)A04.

UCB/EERC-80/36 "Simulating and Analyzing Artificial Non-Stationary Earth Ground Motions," by Nau, R.F., Oliver, R.M. and Pister, K.S., October 1980, (PB81 153 397)A04.

UCB/EERC-80/37 "Earthquake Engineering at Berkeley - 1980," by , September 1980, (PB81 205 674)A09.

UCB/EERC-80/38 "Inelastic Seismic Analysis of Large Panel Buildings," by Schricker, V. and Powell, G.H., September 1980, (PB81 154 338)A13.

UCB/EERC-80/39 "Dynamic Response of Embankment, Concrete-Gavity and Arch Dams Including Hydrodynamic Interation," by Hall, J.F. and Chopra, A.K., October 1980, (PB81 152 324)A11.

UCB/EERC-80/40 "Inelastic Buckling of Steel Struts under Cyclic Load Reversal.," by Black, R.G., Wenger, W.A. and Popov, E.P., October 1980, (PB81 154 312)A08.

UCB/EERC-80/41 "Influence of Site Characteristics on Buildings Damage during the October 3,1974 Lima Earthquake," by Repetto, P., Arango, I. and Seed, H.B., September 1980, (PB81 161 739)A05.

UCB/EERC-80/42 "Evaluation of a Shaking Table Test Program on Response Behavior of a Two Story Reinforced Concrete Frame," by Blondet, J.M., Clough, R.W. and Mahin, S.A., December 1980, (PB82 196 544)A11.

UCB/EERC-80/43 "Modelling of Soil-Structure Interaction by Finite and Infinite Elements," by Medina, F., December 1980, (PB81 229 270)A04.

UCB/EERC-81/01 "Control of Seismic Response of Piping Systems and Other Structures by Base Isolation," by Kelly, J.M., January 1981, (PB81 200 735)A05.

UCB/EERC-81/02 "OPTNSR- An Interactive Software System for Optimal Design of Statically and Dynamically Loaded Structures with Nonlinear Response," by Bhatti, M.A., Ciampi, V. and Pister, K.S., January 1981, (PB81 218 851)A09.

UCB/EERC-81/03 "Analysis of Local Variations in Free Field Seismic Ground Motions," by Chen, J.-C., Lysmer, J. and Seed, H.B., January 1981, (AD-A099508)A13.

UCB/EERC-81/04 "Inelastic Structural Modeling of Braced Offshore Platforms for Seismic Loading," by Zayas, V.A., Shing, P.-S.B., Mahin, S.A. and Popov, E.P., January 1981, (PB82 138 777)A07.

UCB/EERC-81/05 "Dynamic Response of Light Equipment in Structures," by Der Kiureghian, A., Sackman, J.L. and Nour-Omid, B., April 1981, (PB81 218 497)A04.

UCB/EERC-81/06 "Preliminary Experimental Investigation of a Broad Base Liquid Storage Tank," by Bouwkamp, J.G., Kollegger, J.P. and Stephen, R.M., May 1981, (PB82 140 385)A03.

UCB/EERC-81/07 "The Seismic Resistant Design of Reinforced Concrete Coupled Structural Walls," by Aktan, A.E. and Bertero, V.V., June 1981, (PB82 113 358)A11.

UCB/EERC-81/08 "Unassigned," by Unassigned, 1981.

UCB/EERC-81/09 "Experimental Behavior of a Spatial Piping System with Steel Energy Absorbers Subjected to a Simulated Differential Seismic Input," by Stiemer, S.F., Godden, W.G. and Kelly, J.M., July 1981, (PB82 201 898)A04.

UCB/EERC-81/10 "Evaluation of Seismic Design Provisions for Masonry in the United States," by Sveinsson, B.I., Mayes, R.L. and McNiven, H.D., August 1981, (PB82 166 075)A08.

UCB/EERC-81/11 "Two-Dimensional Hybrid Modelling of Soil-Structure Interaction," by Tzong, T.-J., Gupta, S. and Penzien, J., August 1981, (PB82 142 118)A04.

UCB/EERC-81/12 "Studies on Effects of Infills in Seismic Resistant R/C Construction," by Brokken, S. and Bertero, V.V., October 1981, (PB82 166 190)A09.

UCB/EERC-81/13 "Linear Models to Predict the Nonlinear Seismic Behavior of a One-Story Steel Frame," by Valdimarsson, H., Shah, A.H. and McNiven, H.D., September 1981, (PB82 138 793)A07.

UCB/EERC-81/14 "TLUSH: A Computer Program for the Three-Dimensional Dynamic Analysis of Earth Dams," by Kagawa, T., Mejia, L.H., Seed, H.B. and Lysmer, J., September 1981, (PB82 139 940)A06.

UCB/EERC-81/15 "Three Dimensional Dynamic Response Analysis of Earth Dams," by Mejia, L.H. and Seed, H.B., September 1981, (PB82 137 274)A12.

UCB/EERC-81/16 "Experimental Study of Lead and Elastomeric Dampers for Base Isolation Systems," by Kelly, J.M. and Hodder, S.B., October 1981, (PB82 166 182)A05.

UCB/EERC-81/17 "The Influence of Base Isolation on the Seismic Response of Light Secondary Equipment," by Kelly, J.M., April 1981, (PB82 255 266)A04.

UCB/EERC-81/18 "Studies on Evaluation of Shaking Table Response Analysis Procedures," by Blondet, J. M., November 1981, (PB82 197 278)A10.

UCB/EERC-81/19 "DELIGHT.STRUCT: A Computer-Aided Design Environment for Structural Engineering," by Balling, R.J., Pister, K.S. and Polak, E., December 1981, (PB82 218 496)A07.

UCB/EERC-81/20 "Optimal Design of Seismic-Resistant Planar Steel Frames," by Balling, R.J., Ciampi, V. and Pister, K.S., December 1981, (PB82 220 179)A07.

UCB/EERC-82/01 "Dynamic Behavior of Ground for Seismic Analysis of Lifeline Systems," by Sato, T. and Der Kiureghian, A., January 1982, (PB82 218 926)A05.

UCB/EERC-82/02 "Shaking Table Tests of a Tubular Steel Frame Model," by Ghanaat, Y. and Clough, R.W., January 1982, (PB82 220 161)A07.

UCB/EERC-82/03 "Behavior of a Piping System under Seismic Excitation: Experimental Investigations of a Spatial Piping System supported by Mechanical Shock Arrestors," by Schneider, S., Lee, H.-M. and Godden, W. G., May 1982, (PB83 172 544)A09.

UCB/EERC-82/04 "New Approaches for the Dynamic Analysis of Large Structural Systems," by Wilson, E.L., June 1982, (PB83 148 080)A05.

UCB/EERC-82/05 "Model Study of Effects of Damage on the Vibration Properties of Steel Offshore Platforms," by Shahrivar, F. and Bouwkamp, J.G., June 1982, (PB83 148 742)A10.

UCB/EERC-82/06 "States of the Art and Pratice in the Optimum Seismic Design and Analytical Response Prediction of R/C Frame Wall Structures," by Aktan, A.E. and Bertero, V.V., July 1982, (PB83 147 736)A05.

UCB/EERC-82/07 "Further Study of the Earthquake Response of a Broad Cylindrical Liquid-Storage Tank Model," by Manos, G.C. and Clough, R.W., July 1982, (PB83 147 744)A11.

UCB/EERC-82/08 "An Evaluation of the Design and Analytical Seismic Response of a Seven Story Reinforced Concrete Frame," by Charney, F.A. and Bertero, V.V., July 1982, (PB83 157 628)A09.

UCB/EERC-82/09 "Fluid-Structure Interactions: Added Mass Computations for Incompressible Fluid," by Kuo, J.S.-H., August 1982, (PB83 156 281)A07.

UCB/EERC-82/10 "Joint-Opening Nonlinear Mechanism: Interface Smeared Crack Model," by Kuo, J.S.-H., August 1982, (PB83 149 195)A05.

UCB/EERC-82/11 "Dynamic Response Analysis of Techi Dam," by Clough, R.W., Stephen, R.M. and Kuo, J.S.-H., August 1982, (PB83 147 496)A06.

UCB/EERC-82/12 "Prediction of the Seismic Response of R/C Frame-Coupled Wall Structures," by Aktan, A.E., Bertero, V.V. and Piazzo, M., August 1982, (PB83 149 203)A09.

UCB/EERC-82/13 "Preliminary Report on the Smart 1 Strong Motion Array in Taiwan," by Bolt, B.A., Loh, C.H., Penzien, J. and Tsai, Y.B., August 1982, (PB83 159 400)A10.

UCB/EERC-82/14 "Shaking-Table Studies of an Eccentrically X-Braced Steel Structure," by Yang, M.S., September 1982, (PB83 260 778)A12.

UCB/EERC-82/15 "The Performance of Stairways in Earthquakes," by Roha, C., Axley, J.W. and Bertero, V.V., September 1982, (PB83 157 693)A07.

UCB/EERC-82/16 "The Behavior of Submerged Multiple Bodies in Earthquakes," by Liao, W.-G., September 1982, (PB83 158 709)A07.

UCB/EERC-82/17 "Effects of Concrete Types and Loading Conditions on Local Bond-Slip Relationships," by Cowell, A.D., Popov, E.P. and Bertero, V.V., September 1982, (PB83 153 577)A04.

UCB/EERC-82/18 "Mechanical Behavior of Shear Wall Vertical Boundary Members: An Experimental Investigation," by Wagner, M.T. and Bertero, V.V., October 1982, (PB83 159 764)A05.

UCB/EERC-82/19 "Experimental Studies of Multi-support Seismic Loading on Piping Systems," by Kelly, J.M. and Cowell, A.D., November 1982.

UCB/EERC-82/20 "Generalized Plastic Hinge Concepts for 3D Beam-Column Elements," by Chen, P. F.-S. and Powell, G.H., November 1982, (PB83 247 981)A13.

UCB/EERC-82/21 "ANSR-II: General Computer Program for Nonlinear Structural Analysis," by Oughourlian, C.V. and Powell, G.H., November 1982, (PB83 251 330)A12.

UCB/EERC-82/22 "Solution Strategies for Statically Loaded Nonlinear Structures," by Simons, J.W. and Powell, G.H., November 1982, (PB83 197 970)A06.

UCB/EERC-82/23 "Analytical Model of Deformed Bar Anchorages under Generalized Excitations," by Ciampi, V., Eligehausen, R., Bertero, V.V. and Popov, E.P., November 1982, (PB83 169 532)A06.

UCB/EERC-82/24 "A Mathematical Model for the Response of Masonry Walls to Dynamic Excitations," by Sucuoglu, H., Mengi, Y. and McNiven, H.D., November 1982, (PB83 169 011)A07.

UCB/EERC-82/25 "Earthquake Response Considerations of Broad Liquid Storage Tanks," by Cambra, F.J., November 1982, (PB83 251 215)A09.

UCB/EERC-82/26 "Computational Models for Cyclic Plasticity, Rate Dependence and Creep," by Mosaddad, B. and Powell, G.H., November 1982, (PB83 245 829)A08.

UCB/EERC-82/27 "Inelastic Analysis of Piping and Tubular Structures," by Mahasuverachai, M. and Powell, G.H., November 1982, (PB83 249 987)A07.

UCB/EERC-83/01 "The Economic Feasibility of Seismic Rehabilitation of Buildings by Base Isolation," by Kelly, J.M., January 1983, (PB83 197 988)A05.

UCB/EERC-83/02 "Seismic Moment Connections for Moment-Resisting Steel Frames.," by Popov, E.P., January 1983, (PB83 195 412)A04.

UCB/EERC-83/03 "Design of Links and Beam-to-Column Connections for Eccentrically Braced Steel Frames," by Popov, E.P. and Malley, J.O., January 1983, (PB83 194 811)A04.

UCB/EERC-83/04 "Numerical Techniques for the Evaluation of Soil-Structure Interaction Effects in the Time Domain," by Bayo, E. and Wilson, E.L., February 1983, (PB83 245 605)A09.

UCB/EERC-83/05 "A Transducer for Measuring the Internal Forces in the Columns of a Frame-Wall Reinforced Concrete Structure," by Sause, R. and Bertero, V.V., May 1983, (PB84 119 494)A06.

UCB/EERC-83/06 "Dynamic Interactions Between Floating Ice and Offshore Structures," by Croteau, P., May 1983, (PB84 119 486)A16.

UCB/EERC-83/07 "Dynamic Analysis of Multiply Tuned and Arbitrarily Supported Secondary Systems," by Igusa, T. and Der Kiureghian, A., July 1983, (PB84 118 272)A11.

UCB/EERC-83/08 "A Laboratory Study of Submerged Multi-body Systems in Earthquakes," by Ansari, G.R., June 1983, (PB83 261 842)A17.

UCB/EERC-83/09 "Effects of Transient Foundation Uplift on Earthquake Response of Structures," by Yim, C.-S. and Chopra, A.K., June 1983, (PB83 261 396)A07.

UCB/EERC-83/10 "Optimal Design of Friction-Braced Frames under Seismic Loading," by Austin, M.A. and Pister, K.S., June 1983, (PB84 119 288)A06.

UCB/EERC-83/11 "Shaking Table Study of Single-Story Masonry Houses: Dynamic Performance under Three Component Seismic Input and Recommendations," by Manos, G.C., Clough, R.W. and Mayes, R.L., July 1983, (UCB/EERC-83/11)A08.

UCB/EERC-83/12 "Experimental Error Propagation in Pseudodynamic Testing," by Shiing, P.B. and Mahin, S.A., June 1983, (PB84 119 270)A09.

UCB/EERC-83/13 "Experimental and Analytical Predictions of the Mechanical Characteristics of a 1/5-scale Model of a 7-story R/C Frame-Wall Building Structure," by Aktan, A.E., Bertero, V.V., Chowdhury, A.A. and Nagashima, T., June 1983, (PB84 119 213)A07.

UCB/EERC-83/14 "Shaking Table Tests of Large-Panel Precast Concrete Building System Assemblages," by Oliva, M.G. and Clough, R.W., June 1983, (PB86 110 210/AS)A11.

UCB/EERC-83/15 "Seismic Behavior of Active Beam Links in Eccentrically Braced Frames," by Hjelmstad, K.D. and Popov, E.P., July 1983, (PB84 119 676)A09.

UCB/EERC-83/16 "System Identification of Structures with Joint Rotation," by Dimsdale, J.S., July 1983, (PB84 192 210)A06.

UCB/EERC-83/17 "Construction of Inelastic Response Spectra for Single-Degree-of-Freedom Systems," by Mahin, S. and Lin, J., June 1983, (PB84 208 834)A05.

UCB/EERC-83/18 "Interactive Computer Analysis Methods for Predicting the Inelastic Cyclic Behaviour of Structural Sections," by Kaba, S. and Mahin, S., July 1983, (PB84 192 012)A06.

UCB/EERC-83/19 "Effects of Bond Deterioration on Hysteretic Behavior of Reinforced Concrete Joints," by Filippou, F.C., Popov, E.P. and Bertero, V.V., August 1983, (PB84 192 020)A10.

UCB/EERC-83/20 "Analytical and Experimental Correlation of Large-Panel Precast Building System Performance," by Oliva, M.G., Clough, R.W., Velkov, M. and Gavrilovic, P., November 1983.

UCB/EERC-83/21 "Mechanical Characteristics of Materials Used in a 1/5 Scale Model of a 7-Story Reinforced Concrete Test Structure," by Bertero, V.V., Aktan, A.E., Harris, H.G. and Chowdhury, A.A., October 1983, (PB84 193 697)A05.

UCB/EERC-83/22 "Hybrid Modelling of Soil-Structure Interaction in Layered Media," by Tzong, T.-J. and Penzien, J., October 1983, (PB84 192 178)A08.

UCB/EERC-83/23 "Local Bond Stress-Slip Relationships of Deformed Bars under Generalized Excitations," by Eligehausen, R., Popov, E.P. and Bertero, V.V., October 1983, (PB84 192 848)A09.

UCB/EERC-83/24 "Design Considerations for Shear Links in Eccentrically Braced Frames," by Malley, J.O. and Popov, E.P., November 1983, (PB84 192 186)A07.

UCB/EERC-84/01 "Pseudodynamic Test Method for Seismic Performance Evaluation: Theory and Implementation," by Shing, P.-S.B. and Mahin, S.A., January 1984, (PB84 190 644)A08.

UCB/EERC-84/02 "Dynamic Response Behavior of Kiang Hong Dian Dam," by Clough, R.W., Chang, K.-T., Chen, H.-Q. and Stephen, R.M., April 1984, (PB84 209 402)A08.

UCB/EERC-84/03 "Refined Modelling of Reinforced Concrete Columns for Seismic Analysis," by Kaba, S.A. and Mahin, S.A., April 1984, (PB84 234 384)A06.

UCB/EERC-84/04 "A New Floor Response Spectrum Method for Seismic Analysis of Multiply Supported Secondary Systems," by Asfura, A. and Der Kiureghian, A., June 1984, (PB84 239 417)A06.

UCB/EERC-84/05 "Earthquake Simulation Tests and Associated Studies of a 1/5th-scale Model of a 7-Story R/C Frame-Wall Test Structure," by Bertero, V.V., Aktan, A.E., Charney, F.A. and Sause, R., June 1984, (PB84 239 409)A09.

UCB/EERC-84/06 "R/C Structural Walls: Seismic Design for Shear," by Aktan, A.E. and Bertero, V.V., 1984.

UCB/EERC-84/07 "Behavior of Interior and Exterior Flat-Plate Connections subjected to Inelastic Load Reversals," by Zee, H.L. and Moehle, J.P., August 1984, (PB86 117 629/AS)A07.

UCB/EERC-84/08 "Experimental Study of the Seismic Behavior of a Two-Story Flat-Plate Structure," by Moehle, J.P. and Diebold, J.W., August 1984, (PB86 122 553/AS)A12.

UCB/EERC-84/09 "Phenomenological Modeling of Steel Braces under Cyclic Loading," by Ikeda, K., Mahin, S.A. and Dermitzakis, S.N., May 1984, (PB86 132 198/AS)A08.

UCB/EERC-84/10 "Earthquake Analysis and Response of Concrete Gravity Dams," by Fenves, G. and Chopra, A.K., August 1984, (PB85 193 902/AS)A11.

UCB/EERC-84/11 "EAGD-84: A Computer Program for Earthquake Analysis of Concrete Gravity Dams," by Fenves, G. and Chopra, A.K., August 1984, (PB85 193 613/AS)A05.

UCB/EERC-84/12 "A Refined Physical Theory Model for Predicting the Seismic Behavior of Braced Steel Frames," by Ikeda, K. and Mahin, S.A., July 1984, (PB85 191 450/AS)A09.

UCB/EERC-84/13 "Earthquake Engineering Research at Berkeley - 1984," by , August 1984, (PB85 197 341/AS)A10.

UCB/EERC-84/14 "Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils," by Seed, H.B., Wong, R.T., Idriss, I.M. and Tokimatsu, K., September 1984, (PB85 191 468/AS)A04.

UCB/EERC-84/15 "The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," by Seed, H.B., Tokimatsu, K., Harder, L.F. and Chung, R.M., October 1984, (PB85 191 732/AS)A04.

UCB/EERC-84/16 "Simplified Procedures for the Evaluation of Settlements in Sands Due to Earthquake Shaking," by Tokimatsu, K. and Seed, H.B., October 1984, (PB85 197 887/AS)A03.

UCB/EERC-84/17 "Evaluation of Energy Absorption Characteristics of Bridges under Seismic Conditions," by Imbsen, R.A. and Penzien, J., November 1984.

UCB/EERC-84/18 "Structure-Foundation Interactions under Dynamic Loads," by Liu, W.D. and Penzien, J., November 1984, (PB87 124 889/AS)A11.

UCB/EERC-84/19 "Seismic Modelling of Deep Foundations," by Chen, C.-H. and Penzien, J., November 1984, (PB87 124 798/AS)A07.

UCB/EERC-84/20 "Dynamic Response Behavior of Quan Shui Dam," by Clough, R.W., Chang, K.-T., Chen, H.-Q., Stephen, R.M., Ghanaat, Y. and Qi, J.-H., November 1984, (PB86 115177/AS)A07.

UCB/EERC-85/01 "Simplified Methods of Analysis for Earthquake Resistant Design of Buildings," by Cruz, E.F. and Chopra, A.K., February 1985, (PB86 112299/AS)A12.

UCB/EERC-85/02 "Estimation of Seismic Wave Coherency and Rupture Velocity using the SMART 1 Strong-Motion Array Recordings," by Abrahamson, N.A., March 1985, (PB86 214 343)A07.

UCB/EERC-85/03 "Dynamic Properties of a Thirty Story Condominium Tower Building," by Stephen, R.M., Wilson, E.L. and Stander, N., April 1985, (PB86 118965/AS)A06.

UCB/EERC-85/04 "Development of Substructuring Techniques for On-Line Computer Controlled Seismic Performance Testing," by Dermitzakis, S. and Mahin, S., February 1985, (PB86 132941/AS)A08.

UCB/EERC-85/05 "A Simple Model for Reinforcing Bar Anchorages under Cyclic Excitations," by Filippou, F.C., March 1985, (PB86 112 919/AS)A05.

UCB/EERC-85/06 "Racking Behavior of Wood-framed Gypsum Panels under Dynamic Load," by Oliva, M.G., June 1985.

UCB/EERC-85/07 "Earthquake Analysis and Response of Concrete Arch Dams," by Fok, K.-L. and Chopra, A.K., June 1985, (PB86 139672/AS)A10.

UCB/EERC-85/08 "Effect of Inelastic Behavior on the Analysis and Design of Earthquake Resistant Structures," by Lin, J.P. and Mahin, S.A., June 1985, (PB86 135340/AS)A08.

UCB/EERC-85/09 "Earthquake Simulator Testing of a Base-Isolated Bridge Deck," by Kelly, J.M., Buckle, I.G. and Tsai, H.-C., January 1986, (PB87 124 152/AS)A06.

UCB/EERC-85/10 "Simplified Analysis for Earthquake Resistant Design of Concrete Gravity Dams," by Fenves, G. and Chopra, A.K., June 1986, (PB87 124 160/AS)A08.

UCB/EERC-85/11 "Dynamic Interaction Effects in Arch Dams," by Clough, R.W., Chang, K.-T., Chen, H.-Q. and Ghanaat, Y., October 1985, (PB86 135027/AS)A05.

UCB/EERC-85/12 "Dynamic Response of Long Valley Dam in the Mammoth Lake Earthquake Series of May 25-27, 1980," by Lai, S. and Seed, H.B., November 1985, (PB86 142304/AS)A05.

UCB/EERC-85/13 "A Methodology for Computer-Aided Design of Earthquake-Resistant Steel Structures," by Austin, M.A., Pister, K.S. and Mahin, S.A., December 1985, (PB86 159480/AS)A10.

UCB/EERC-85/14 "Response of Tension-Leg Platforms to Vertical Seismic Excitations," by Liou, G.-S., Penzien, J. and Yeung, R.W., December 1985, (PB87 124 871/AS)A08.

UCB/EERC-85/15 "Cyclic Loading Tests of Masonry Single Piers: Volume 4 - Additional Tests with Height to Width Ratio of 1," by Sveinsson, B., McNiven, H.D. and Sucuoglu, H., December 1985.

UCB/EERC-85/16 "An Experimental Program for Studying the Dynamic Response of a Steel Frame with a Variety of Infill Partitions," by Yanev, B. and McNiven, H.D., December 1985.

UCB/EERC-86/01 "A Study of Seismically Resistant Eccentrically Braced Steel Frame Systems," by Kasai, K. and Popov, E.P., January 1986, (PB87 124 178/AS)A14.

UCB/EERC-86/02 "Design Problems in Soil Liquefaction," by Seed, H.B., February 1986, (PB87 124 186/AS)A03.

UCB/EERC-86/03 "Implications of Recent Earthquakes and Research on Earthquake-Resistant Design and Construction of Buildings," by Bertero, V.V., March 1986, (PB87 124 194/AS)A05.

UCB/EERC-86/04 "The Use of Load Dependent Vectors for Dynamic and Earthquake Analyses," by Leger, P., Wilson, E.L. and Clough, R.W., March 1986, (PB87 124 202/AS)A12.

UCB/EERC-86/05 "Two Beam-To-Column Web Connections," by Tsai, K.-C. and Popov, E.P., April 1986, (PB87 124 301/AS)A04.

UCB/EERC-86/06 "Determination of Penetration Resistance for Coarse-Grained Soils using the Becker Hammer Drill," by Harder, L.F. and Seed, H.B., May 1986, (PB87 124 210/AS)A07.

UCB/EERC-86/07 "A Mathematical Model for Predicting the Nonlinear Response of Unreinforced Masonry Walls to In-Plane Earthquake Excitations," by Mengi, Y. and McNiven, H.D., May 1986, (PB87 124 780/AS)A06.

UCB/EERC-86/08 "The 19 September 1985 Mexico Earthquake: Building Behavior," by Bertero, V.V., July 1986.

UCB/EERC-86/09 "EACD-3D: A Computer Program for Three-Dimensional Earthquake Analysis of Concrete Dams," by Fok, K.-L., Hall, J.F. and Chopra, A.K., July 1986, (PB87 124 228/AS)A08.

UCB/EERC-86/10 "Earthquake Simulation Tests and Associated Studies of a 0.3-Scale Model of a Six-Story Concentrically Braced Steel Structure," by Uang, C.-M. and Bertero, V.V., December 1986, (PB87 163 564/AS)A17.

UCB/EERC-86/11 "Mechanical Characteristics of Base Isolation Bearings for a Bridge Deck Model Test," by Kelly, J.M., Buckle, I.G. and Koh, C.-G., 1987.

UCB/EERC-86/12 "Effects of Axial Load on Elastomeric Isolation Bearings," by Koh, C.-G. and Kelly, J.M., November 1987.

UCB/EERC-87/01 "The FPS Earthquake Resisting System: Experimental Report," by Zayas, V.A., Low, S.S. and Mahin, S.A., June 1987.

UCB/EERC-87/02 "Earthquake Simulator Tests and Associated Studies of a 0.3-Scale Model of a Six-Story Eccentrically Braced Steel Structure," by Whittaker, A., Uang, C.-M. and Bertero, V.V., July 1987.

UCB/EERC-87/03 "A Displacement Control and Uplift Restraint Device for Base-Isolated Structures," by Kelly, J.M., Griffith, M.C. and Aiken, I.G., April 1987.

UCB/EERC-87/04 "Earthquake Simulator Testing of a Combined Sliding Bearing and Rubber Bearing Isolation System," by Kelly, J.M. and Chalhoub, M.S., 1987.

UCB/EERC-87/05 "Three-Dimensional Inelastic Analysis of Reinforced Concrete Frame-Wall Structures," by Moazzami, S. and Bertero, V.V., May 1987.

UCB/EERC-87/06 "Experiments on Eccentrically Braced Frames with Composite Floors," by Ricles, J. and Popov, E., June 1987.

UCB/EERC-87/07 "Dynamic Analysis of Seismically Resistant Eccentrically Braced Frames," by Ricles, J. and Popov, E., June 1987.

UCB/EERC-87/08 "Undrained Cyclic Triaxial Testing of Gravels-The Effect of Membrane Compliance," by Evans, M.D. and Seed, H.B., July 1987.

UCB/EERC-87/09 "Hybrid Solution Techniques for Generalized Pseudo-Dynamic Testing," by Thewalt, C. and Mahin, S.A., July 1987.

UCB/EERC-87/10 "Investigation of Ultimate Behavior of AISC Group 4 and 5 Heavy Steel Rolled-Section Splices with Full and Partial Penetration Butt Welds," by Bruneau, M. and Mahin, S.A., July 1987.

UCB/EERC-87/11 "Residual Strength of Sand from Dam Failures in the Chilean Earthquake of March 3, 1985," by De Alba, P., Seed, H.B., Retamal, E. and Seed, R.B., September 1987.

UCB/EERC-87/12 "Inelastic Seismic Response of Structures with Mass or Stiffness Eccentricities in Plan," by Bruneau, M. and Mahin, S.A., September 1987.

UCB/EERC-87/13 "CSTRUCT: An Interactive Computer Environment for the Design and Analysis of Earthquake Resistant Steel Structures," by Austin, M.A., Mahin, S.A. and Pister, K.S., September 1987.

UCB/EERC-87/14 "Experimental Study of Reinforced Concrete Columns Subjected to Multi-Axial Loading," by Low, S.S. and Moehle, J.P., September 1987.

UCB/EERC-87/15 "Relationships between Soil Conditions and Earthquake Ground Motions in Mexico City in the Earthquake of Sept. 19, 1985," by Seed, H.B., Romo, M.P., Sun, J., Jaime, A. and Lysmer, J., October 1987.

UCB/EERC-87/16 "Experimental Study of Seismic Response of R. C. Setback Buildings," by Shahrooz, B.M. and Moehle, J.P., October 1987.

UCB/EERC-87/17 "Three Dimensional Aspects of the Behavior of R. C. Structures Subjected to Earthquakes," by Pantazopoulou, S.J. and Moehle, J.P., October 1987.

UCB/EERC-87/18 "Design Procedure for R-FBI Bearings," by Mostaghel, N. and Kelly, J.M., November 1987.

UCB/EERC-87/19 "Analytical Models for Predicting the Lateral Response of R C Shear Walls: Evaluation of their Reliability," by Vulcano, A. and Bertero, V.V., November 1987.

UCB/EERC-87/20 "Earthquake Response of Torsionally-Coupled Buildings," by Hejal, R. and Chopra, A.K., December 1987.

UCB/EERC-87/21 "Dynamic Reservoir Interaction with Monticello Dam," by Clough, R.W., Ghanaat, Y. and Qiu, X-F., December 1987.

UCB/EERC-87/22 "Strength Evaluation of Coarse-Grained Soils," by Siddiqi, F.H., Seed, R.B., Chan, C.K., Seed, H.B. and Pyke, R.M., December 1987.

UCB/EERC-88/01 "Seismic Behavior of Concentrically Braced Steel Frames," by Khatib, I., Mahin, S.A. and Pister, K.S., January 1988.

UCB/EERC-88/02 "Experimental Evaluation of Seismic Isolation of Medium-Rise Structures Subject to Uplift," by Griffith, M.C., Kelly, J.M., Coveney, V.A. and Koh, C.G., January 1988.

UCB/EERC-88/03 "Cyclic Behavior of Steel Double Angle Connections," by Astaneh-Asl, A. and Nader, M.N., January 1988.

UCB/EERC-88/04 "Re-evaluation of the Slide in the Lower San Fernando Dam in the Earthquake of Feb. 9, 1971," by Seed, H.B., Seed, R.B., Harder, L.F. and Jong, H.-L., April 1988.

UCB/EERC-88/05 "Experimental Evaluation of Seismic Isolation of a Nine-Story Braced Steel Frame Subject to Uplift," by Griffith, M.C., Kelly, J.M. and Aiken, I.D., May 1988.

UCB/EERC-88/06 "DRAIN-2DX User Guide.," by Allahabadi, R. and Powell, G.H., March 1988.

UCB/EERC-88/07 "Cylindrical Fluid Containers in Base-Isolated Structures," by Chalhoub, M.S. and Kelly, J.M. , April 1988.

UCB/EERC-88/08 "Analysis of Near-Source Waves: Separation of Wave Types using Strong Motion Array Recordings," by Darragh, R.B., June 1988.

UCB/EERC-88/09 "Alternatives to Standard Mode Superposition for Analysis of Non-Classically Damped Systems," by Kusainov, A.A. and Clough, R.W., June 1988.

UCB/EERC-88/10 "The Landslide at the Port of Nice on October 16, 1979," by Seed, H.B., Seed, R.B., Schlosser, F., Blondeau, F. and Juran, I., June 1988.