POTAMOLOGY INVESTIGATIONS

Report 12-16

METHODS OF PREVENTING FLOW SLIDES

by

D. C. Banks
W. E. Strohm, Jr.

October 1965

Sponsored by

The President, Mississippi River Commission

and

The Division Engineer
Lower Mississippi Valley Division

Conducted by

U. S. Army Engineer Waterways Experiment Station
CORPS OF ENGINEERS
Vicksburg, Mississippi
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* Not of general informational value and hence not distributed.
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FOREWORD

Between the years 1947 and 1952, the U. S. Army Engineer Waterways Experiment Station (WES), under authorization of the Mississippi River Commission (MRC), conducted extensive potamology investigations. A part of these investigations was concerned with a study of revetment failures, their causes, and methods of prevention, development of new methods of riverbank stabilization, and a study of specific troublesome areas of the Mississippi River with a view to their improvement. After 1952 the effort devoted to the potamology program by the Soils and Hydraulics Divisions, WES, was very modest.

However, in 1956 an objective review of the soils phase of the potamology program was made by Dr. M. J. Hvorslev, Consultant, Soils Division. The review, published in Potamology Investigations Report 12-5, was made to evaluate the accomplishments to that date and to provide direction for future studies. Since 1956 very little actual investigation has been conducted at the WES concerning the mechanisms and prevention of flow slides; however, investigations by others have produced considerable added knowledge on the subject and it was therefore deemed advisable to restudy the general subject of flow slides and their prevention. The WES was authorized to make such a study by the President, MRC, in first indorsement, LMVGP, dated 16 September 1963 to WES letter dated 23 August 1964, subject "Minutes of the Twelfth Potamology Conference." The study had a twofold purpose, (a) to update the material and references presented in Dr. Hvorslev's 1956 summary report, and (b) to evaluate methods of stabilizing potentially unstable deposits with the ultimate objective of selecting the most promising method for field testing.

The literature review described in this report was conducted by Messrs. D. C. Banks and W. E. Strohm, Jr., under the direction of
Mr. W. C. Sherman, Jr., Chief, Engineering Studies Section, and Mr. J. R. Compton, Chief, Embankment and Foundation Branch, Soils Division. The study was under the general supervision of Messrs. W. J. Turnbull and A. A. Maxwell, Chief and Assistant Chief, respectively, of the Soils Division, WES.

This report was prepared by Messrs. Strohm and Banks and has been reviewed and approved by the Potamology Board in accordance with LMVD Special Orders No. 20, dated 12 August 1964, as amended by Special Orders No. 29, dated 23 December 1964; members of the Board are:

Mr. A. J. Davis, MRC, Chairman
Mr. R. H. Haas, MRC, Member
Mr. J. W. Gurley, St. Louis District, Member
Mr. R. T. Easley, Memphis District, Member
Mr. J. E. Henley, Vicksburg District, Member
Mr. O. M. Jernigan, New Orleans District, Member
Mr. J. B. Tiffany, WES, Member
Mr. E. B. Lipscomb, MRC, Secretary

Directors of the WES during the preparation and publication of this report were Col. Alex G. Sutton, Jr., CE, and Col. John R. Oswalt, Jr., CE. Technical Director was Mr. J. B. Tiffany.
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This report summarizes a study of investigations of liquefaction of sands accomplished since 1956 with the purposes of (a) updating Potamology Investigations Report 12-5, A Review of the Soils Studies, by Dr. M. J. Hvorslev, which was published in June of 1956, and (b) evaluating methods of stabilizing potentially unstable deposits along revetted reaches of the banks of the Lower Mississippi River. The ultimate objective of the study was to determine the most promising method of stabilization for field testing.

Recent literature describing liquefaction phenomena, occurrence of flow slides, results of laboratory and field tests dealing with liquefaction of sands, and methods for densifying loose sand and silt deposits is summarized in this report. The results of the literature review support and extend the findings of Report 12-5. However, no data were found which provide the information that would be obtained from the additional investigations recommended in Report 12-5 on development and progress of arcuate-shaped liquefaction failures, including information on development of excess pore pressures during such failures, and more detailed information on associated soil conditions. These recommended investigations contained in Report 12-5 are still considered desirable, but field experiments can be undertaken now utilizing currently available information to investigate the feasibility of improving the stability of loose sand deposits with respect to flow failure by densification.

The most suitable procedure is considered to be densification by the vibroflotation process using the results of an extensive soil investigation before and after densification to determine the success of the densification procedure. The use of explosives to densify loose sand is not considered suitable at present because of the large amount of preliminary experimental investigation that would be required to determine optimum charge weights, depths, spacings, and other factors for the range of conditions to be found at potentially unstable sites.

Field experiments to evaluate stabilization by densification are proposed.
PART I: INTRODUCTION

Background

1. By 1956, the date of Potamology Investigations Report 12-5* prepared by Dr. M. J. Hvorslev, a considerable amount of data from field and laboratory investigations had been collected and analyzed to permit delineation of specific types of soil deposits along the banks of the Mississippi River which are susceptible to major flow failures. The results of the investigations also permitted some general conclusions concerning the mechanics of the initiation and progress of major bank failures. On the basis of the investigations discussed in Report 12-5, it was found that in general:

a. All major flow failures had occurred in point-bar deposits.

b. Point bar deposits usually contained three basic soil zones: a somewhat cohesive overburden; underlying fine sands called the "upper sand series"; and deeper underlying coarse sands and gravels called the "lower sand series."

c. The overburden materials were stratified and predominantly cohesive but did vary from fat clays to silty sands. The upper sand series could generally be subdivided into a potentially unstable section of fine and loose sands, called zone A, underlain by an essentially stable section of coarser and denser sands, called zone B.

d. The zone A sands, zone B sands, and lower sand series could be distinguished on the basis of peak grain diameter which could be correlated with natural and relative density.

e. All major flow failures were practically confined to the more or less cohesive overburden materials and zone A sands.

f. Factual data on the initiation of major flow failures were fragmentary, but local scour and oversteepening of the bank were probably the principal causes combined with exposure of potentially unstable strata.

* Superscripts refer to similarly numbered entries in the Literature Cited at the end of this report.
Most, and probably all, of the major flow failures were progressive rather than instantaneous in character. The flat outer slopes of the failure areas could be explained only by partial liquefaction of zone A sands, at least during later stages of a failure cycle.

Excess pore-water pressures were probably developed in the bank prior to failure of individual sections and thus contributed to failure; however, such excess pressures were possibly confined to strata of subcritical density (density at which a particular gradation of sand is subject to more or less spontaneous liquefaction), and partial liquefaction of the remaining parts of zone A sands may have been caused by disturbance after actual failure of a section.

The actual sequence of failure combining the three previous items is believed to be as follows. River scour produces an oversteepening of the underwater toe, resulting in an ordinary shear failure slide. This instantaneously produces a very steep slope, causing vertical and horizontal strains to develop in the sandbank some distance from the face of the slope. In the low-density sands, these strain effects produce a tendency for consolidation, thus suddenly producing a high pore pressure with more or less complete reduction of intergranular contact friction. A sudden flow develops in the block of sand so affected. The preceding results in the low-density sand flowing riverward, with the topstratum in effect riding on this liquefied sand layer. The failure through the topstratum soils may well be combined tension and shear. The preceding cycle is then repeated until eventually stability results, leaving the very flat slope in the bottom of the failure area. The phenomenon of the arcuate failure may be related to arching, but the relation remains relatively obscure at this time.

Available data indicated that conditions may be potentially unstable and a major failure may develop when the ratio \((R)\) of the overburden thickness to the thickness of zone A sands is less than 0.85 and the thickness of zone A sands is greater than 25 ft. The overburden materials in a failed section may block further progress of a failure when the ratio \((R)\) is large.

2. Report 12-5 also contained general comments, as restated below, on the prevention of major flow failures in the banks of the Lower Mississippi River:

Potentially unstable point bar deposits should be located. Their extent should be determined by field explorations and use of the criteria concerning the ratio of the thicknesses of overburden materials and zone A sand. Six possible measures suggested for decreasing the potential danger of flow failure were as follows:
(1) Setback levees  
(2) Grading  
(3) Preventive failure  
(4) Compaction  
(5) Drainage  
(6) Grouting  

b. For an area in great danger of flow failure according to the criteria, setback levees should be constructed to provide protection. After the area has scoured, the bank should be protected by revetment.

c. It has been observed in the laboratory and field that remolded sand samples and disturbed or failed sand deposits (which were in a loose condition initially) are more stable than undisturbed, stratified sand samples and deposits because of densification resulting from remolding. Therefore, it is possible that the stability of point bar deposits in danger of flow failure or containing singular very loose strata may be increased by controlled preventive failure produced by dredging and/or explosives.

d. Stabilization of loose and stratified sand in point bars by compaction, drainage, or grouting would probably be very difficult and expensive. These types of stabilization should not be undertaken before further investigations have been completed.

3. Report 12-5 recommended three groups of investigations for the soils phase of future potamology investigations on the basis of the level of effort possible, as follows:

a. Minimum investigations. Field observations and records of the behavior of the riverbanks, hydrographic surveys, reconnaissance borings with representative sampling, and simple classification tests on the samples should be performed in adequate extent or number to insure that the established empirical criteria for potential bank stability are fully utilized and that new data are obtained for the checking and refining of the criteria.

b. Limited investigations. The minimum investigations should be supplemented by more detailed explorations of point bar deposits through cone penetrometer soundings and undisturbed sample borings, attempts to establish correlations between the cone penetration resistance and relative densities of sands, more or less standard laboratory tests on the samples, model or field tests on the development of arcuate failures, and trials of controlled preventive failure. The objectives of these investigations would be to obtain additional data
on the initiation and mechanics of progressive flow failures, refine criteria for the stability of the riverbanks, and permit a more detailed delineation of potentially unstable deposits.

c. Detailed investigations. The minimum and limited investigations should be supplemented by large-scale field tests with pore-water pressure measurements, improvement of laboratory equipment and methods for determination of critical densities and other basic properties of cohesionless soils, and subsequent detailed tests on sands from representative point-bar deposits. The objectives would be to solve the basic problems connected with flow failures, to obtain truly adequate criteria for potentially unstable soil conditions, and ultimately to develop the most efficient methods for avoiding or preventing flow failures.

4. Since publication of Report 12-5, some of the minimum investigations as outlined above have been continued by the U. S. Army Engineer Waterways Experiment Station (WES) and the Lower Mississippi Valley Division (LMVD) Districts; limited field and laboratory investigations have been made by WES. Development of empirical criteria for predicting potential bank stability has been continued and the criteria have been modified. The modified criteria have proved reliable in predicting susceptibility to flow failure; locations where flow failures have occurred have been predicted to be unstable. Laboratory and field investigations have been conducted with the rotary cone penetrometer, and the results have indicated that the cone thrust is a reliable measure of the combined effects of gradation and density of the sand being penetrated. Criteria based on cone thrust have been developed for predicting stability against flow failure. The validity of the cone criteria is currently being determined on the basis of additional tests.

Purpose of Study

5. The objective of the study reported herein is to review available literature published since 1956 on the general subject of liquefaction of sands, occurrence of flow slides, and methods of stabilizing potentially unstable sand deposits, in order to update the information presented in Report 12-5 and to evaluate possible methods of stabilizing potentially unstable sand deposits, with the ultimate purpose of selecting the most promising method for field testing at a specific site.
Scope of Study

6. A detailed search was made of reference sources in the WES Research Center Library, including references to foreign publications, and the pertinent literature was reviewed.

7. Results of the literature review are presented in this report under the following headings:
   a. Occurrence of flow failures.
   b. Laboratory tests.
   c. Field investigations.
   d. Methods of stabilizing potentially unstable deposits.

The degree to which the information obtained affects the findings and recommendations for future investigations contained in Report 12-5 is discussed. Finally, the selection of a site for a possible future field test for stabilizing is described and a test program, including a recommended procedure and a possible alternate, is outlined.
PART II: LITERATURE REVIEW

Occurrence of Flow Failures

8. Only one paper published since 1956 was found which provides information on occurrences of flow failures in sufficient detail to be useful. This paper contains descriptions of several failures involving liquefaction of fine-grained soils which occurred in other countries and discusses the mechanics of initiation and progress of liquefaction.

Mr. W. J. Turnbull, Chief of the Soils Division, WES, was associated with the Central Nebraska Public Power and Irrigation District during the period 1933-1940. During this association a number of flow slides in disturbed material were encountered. Failures described and discussed in Dr. Terzaghi's paper and those which occurred at Central Nebraska Public Power and Irrigation District projects are summarized in the following paragraphs.

Description of failures

9. Failures described by Terzaghi.

a. Howe Sound, British Columbia. A post-Pleistocene delta formed by a mountain stream and located on the north shore of Howe Sound in southern British Columbia was used as a site for construction of a pulp mill in about 1916. During construction of the mill, the stream was confined to a permanent channel and at a later date the sediment load in the stream was intercepted and directed to another part of the sound before the stream reached the mill. Several warehouses and docks were located at the waterfront on both sides of the mouth of the confined stream. Portions of these warehouses and docks extended several hundred feet seaward of the old shoreline and were supported by piles.

(1) On 22 August 1955, a few minutes after extreme low tide (the maximum tidal range in the sound was about 13 ft), the end of one of the warehouses resting on piles collapsed and fell into the water. Twenty-five minutes later, piles dropped out from under a portion of one of the adjacent docks and reappeared at a distance about 100 ft seaward of their original location. Soundings along two lines which had been taken prior to the failure (probably during construction of the mill) and which had extended 150 to 200 ft seaward of the postconstruction shoreline indicated a slope angle...
of about 28 deg. Soundings taken after the failure and extending more than 600 ft seaward also indicated a slope angle of about 28 deg, but the elevations of the slope at various points to a distance of some 150 to 200 ft seaward of the shoreline were as much as 40 ft lower than they were before the failure. It was surmised that a steeper slope had existed just beyond the extent of the original soundings and that this slope had failed with a resulting flow of material from under the warehouse and dock.

(2) Borings made after the failure indicated that clean sands and gravels existed to depths of about 120 ft in the delta deposits; however, it was believed that deposits consisting primarily of silt had formed on either side of a promontory of sands and gravels at the mouth of the stream prior to construction of the mill. The mantling deposits of silt interfered with the tidal rise and fall of the water table in the more pervious deposits beneath. When the tide ebbed, the downward movement of the water table behind the silt deposits was retarded and the base of the silt deposits was subjected to a hydrostatic pressure great enough to cause liquefaction of the deposit.

b. Swir III Dam, U.S.S.R. The failure of a sand embankment adjacent to the concrete section of the Swir III Dam, a storage dam in U.S.S.R., occurred in the spring of 1935. The embankment contained a till core, and the sand on both sides of the core had been deposited in a moist state by dumping (no compaction effort was indicated in the description). The upstream slope was 1 on 2. The failure occurred shortly after the reservoir was filled for the first time and during blasting operations on partly submerged remnants of a cofferdam some 600 ft upstream from the dam. The failure started by spontaneous liquefaction of the upstream portion of the embankment at the surface of contact between the concrete section and embankment. Within less than 1 min the liquefaction failure had progressed to the abutment end of the embankment, a distance of more than 1000 ft from the contact at the concrete section. The flow failure reduced the slope of the embankment to less than 1 on 10.

c. Folla Fiord, Norway. A dredge operating along the southwest shoreline of Folla Fiord in Norway, about 120 miles northeast of Trondheim, was anchored by three chains and anchors to the sea bottom and by two cables to the shore. On 9 January 1952, one of the anchor chains and one of the cables snapped "on account of a local sand slide." The sea was calm, but a few minutes later, a 5-ft-high wave from the head of the fiord swept past the dredge. About 14 min later, after the sea had calmed, another anchor chain and the remaining shore cable broke; the remaining anchor chain pulled the dredge about 1000 ft out into the fiord at a
speed estimated to be between 8 and 10 mph. It was later learned that a large flow slide had occurred in the upper end of the fiord about 2800 ft from the dredge. The observed phenomena suggested that liquefaction of the sand to a shallow depth at the barge location had spread with increasing depth to the head of the fiord and produced a major slide involving liquefaction of sand to a great depth. With occurrence of the major slide, indicated by the wave, liquefaction proceeded back to the site of the dredge and this time involved a much larger body of sand which carried the embedded anchor, chain, and dredge out in the fiord. Subsequently, an attempt was made to recover the anchor. However, the anchor was so deeply buried that the anchor chain broke and the anchor was not recovered.

d. Orkdals Fiord, Norway. The spreading of liquefaction to include increasingly larger areas is indicated by a series of slides that occurred 2 May 1930, in the Orkdals Fiord, about 18 miles southeast of Trondheim. The first slide occurred in a small fill (about 5000 cu yd) close to the head of the southwestern end of the fiord, and the time of the slide corresponded to that of an earthquake tremor recorded at Bergan which had an epicenter in New Hebrides. A few minutes later a second slide about 2000 ft wide occurred at the head of the southwestern end of the fiord, about 1 mile away from the first slide. About 7 min after the first slide, a third slide occurred across the fiord and facing the first slide. This portion of the fiord was about 1.4 miles wide. Two communication cables on the bottom of the fiord at distances of 2 and 12 miles seaward from the slides broke 7 min and 1 hr, 52 min, respectively, after the time of the first slide. The bottom of the fiord between the slides and cables had a gentle slope, and it was concluded that the liquefaction spread from the slides seaward past the cables and that the cables broke because of temporary loss of support by the sediment underlying the cables.

10. Failures described by Turnbull.

a. Kingsley Hydraulic Fill Dam. Two flow slides occurred during the construction of the Kingsley Hydraulic Fill Dam near Ogallala, Nebraska.

(1) One failure occurred in a gravelly sand foundation material which had been stockpiled and bulldozed into place to form the left slope on which riprap revetment was placed below the stilling basin. The material averaged about 40 ft in thickness, and the lower half of the slope was inundated. The failure occurred upon simultaneous detonation of a number of heavy explosive charges which were placed 300 to 500 ft downstream and detonated to break up a 3-ft-thick ice layer in order
to float a large dredge. The slide took place immediately upon the detonation of the explosive charges and lasted only a few seconds. The bank of bulldozed gravelly sand flowed out over the bottom of the basin, carrying the riprap with it. The riprap surface was about level at the completion of the slide.

(2) The other failure occurred in a gravelly sand material in the outer portion of the dredged shell which was bulldozed into place around the "morning glory" spillway tower. The thickness of the material around the tower was variable but averaged 20 to 30 ft. The material was entirely inundated at the time of failure. The failure occurred during cracking of a heavy ice layer as a result of a February thaw. The only visible effect of the cracking was several cracks extending from the reservoir into and around the tower. The cracking was accompanied by loud explosive noises, and people for several miles around the reservoir were aware of the noise. The gravelly sand material flowed out from around the morning glory spillway, down an approximately 1 on 3 slope for a distance of several hundred feet, and spilled out over the cribbing of the outlet tower and blocked two of the four gates.

b. Water-supply canal. Several flow slides in cast loess material occurred along an 80-mile supply canal furnishing water for three powerhouses. Goodly portions of the supply canal were in sidehill cuts, and the material had been cast to the downhill side of a small compacted core section. This bank of loose material in many places was quite massive. The core section was quite narrow being only about 8 ft across the top surface with the top 1 ft above the maximum water surface in the canal. All of the flow slides in the cast loess banks occurred away from the canal. No water was against the outer slopes. These failures were quite spectacular in that the material would flow down the canyon wall, across the bottom of the canyon, and in some cases pile up against the opposite canyon wall. The soil mass in some of these failures traveled as much as 500 or 600 ft. The liquefaction of the cast loess extended down to but not into either the compacted loess core or the in situ loess foundation beneath the core.

Mechanics of initiation and progress of flow failures

11. General discussion. The discussion of the mechanics of initiation and progress of flow failures in cohesionless material contained in Professor Terzaghi's paper is concerned primarily with submarine slopes.
However, certain aspects are applicable to liquefaction of riverbank deposits. These aspects are described below.

12. In coarse-grained sediments such as coarse sand or gravel subjected to vibrations, excess pore pressures almost never occur because the void ratio is commonly low and remains practically unchanged even by intense vibrations such as might be produced by an earthquake. However, as the grain size decreases, the resulting natural in situ void ratio of the sediment increases and the effect of vibrations and other disturbance on the sediment also increases. This has been demonstrated by using several different gradations of crushed quartz particles and determining the void ratio, first after sedimentation and then after hammering the walls of the vessel containing the sediment. The results are shown below.

<table>
<thead>
<tr>
<th>Range of Gradation (mm)</th>
<th>Void Ratio</th>
<th>After Sedimentation ($e_0$)</th>
<th>After Vibration ($e_1$)</th>
<th>Decrease ($e_0 - e_1$)</th>
<th>Decrease (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.70-0.25</td>
<td>0.67</td>
<td>0.33</td>
<td>33</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.10-0.02</td>
<td>0.80</td>
<td>0.41</td>
<td>34</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.02-0.006</td>
<td>1.13</td>
<td>1.07</td>
<td>51</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.006-0.002</td>
<td>1.50</td>
<td>1.07</td>
<td>42</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Less than 0.002</td>
<td>2.16</td>
<td>0.50</td>
<td>19</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The above tabulation indicates that as the particle sizes decrease the decrease in void ratio becomes greater and then lesser, with the maximum decrease occurring for a gradation range of 0.02 to 0.006 mm. It was theorized that in the sedimentation process, as soon as a sinking particle arrives at the surface of the sediment, it tends to roll or slide into stable position. This tendency is resisted by adhesion which acts at the point of first contact between the particle and the sediment. The resistance produced at the first contact is independent of the particle size, whereas the tendency of the particle to roll or slide decreases with decrease in particle size. Therefore, the probability that the particle will assume an unstable position increases with decrease in particle size. The tabulation above indicates that with void ratios
greater than about 1.0, vibrations produce significant changes in void ratio. However, as soon as the gradation range drops below about 0.006 mm the effect of vibration again decreases. This fact indicates that the adhesion between contact points counteracts perceptibly the effect of vibration. It was hypothesized that the sediments most sensitive to shock, on the basis of the results in the preceding tabulation, are those within a gradation range of about 0.02 to 0.006 mm, which is the gradation range of a fine silt. A particle structure which would exhibit considerable decrease in void ratio or would collapse when vibrated is termed metastable (a metastable structure would have a density at or below the critical density); if the material is saturated, this structure collapse causes water to be forced from the voids.

13. At the instant of collapse of the structure of a saturated fine-grained sediment, the weight of the soil particles is temporarily transferred to the water. The resulting increase in excess hydrostatic pressure at any depth, \( z \), is practically equal to the submerged weight of the soil between the surface and the depth, \( z \). This concept of temporary excess hydrostatic pressure in fine-grained cohesionless sediments is termed spontaneous liquefaction. The liquefaction is followed by an expulsion of excess water; the average degree of consolidation of the soil mass is very low at the instant of particle structure collapse and increases to 100 percent as the particles come to rest in a more stable and denser structure. The time for this readjustment process increases with decrease in permeability of the sediment which is related to decrease in particle size, other factors being equal. Thus, when liquefaction and flow of cohesionless material composing a given slope occur, the duration of the liquefaction period and thereby the distance which the liquefied material will be transported during the transition from the temporary liquefied state to a static state will be inversely proportional to the gradation of the material.

14. In fine-grained uniform cohesionless sediments having a metastable structure and forming an underwater slope which has not been subjected to a disturbance, the excess hydrostatic pressure is normally equal to zero. The effective vertical pressure at any depth, \( z \), below the surface of the sediment is equal to the submerged unit weight, \( \gamma_s \).
times the depth, \( z \), or \( \gamma_s z \), and thus increases linearly with increase in depth. The shearing resistance, \( s_0 \), before any disturbance is a function of effective vertical pressure and also increases linearly with depth. It is equal to \( \gamma_s z \tan \phi \), where \( \phi \) is the true angle of internal friction. However, when the sediment is subjected to a disturbance which causes liquefaction, the excess hydrostatic pressure at the instant of particle structure collapse increases almost instantaneously from zero to a value close to the effective vertical pressure \( \gamma_s z \) and the shearing resistance drops to a value close to zero. At the time of collapse of the sediment, the degree of consolidation, \( U \), decreases from a value of 100 percent to a value close to zero. Thus, the shearing resistance, \( s_u \), at the instant of liquefaction can be expressed by the equation:

\[
s_u = \gamma_s z (U \tan \phi) = \gamma_s z \tan \phi_1
\]

where \( \tan \phi_1 = U \tan \phi \)

15. The loss of shearing resistance during liquefaction caused by vibration can easily be demonstrated by filling a vessel with sand and water. By introducing water at the bottom of the tank under pressure so that water flows up through the sand, an unstable structure is formed. The flow of water is then stopped and a weight is placed gently on the surface of the sand. The shearing resistance of the sand at this stage is unimpaired, and the sand will support the weight. However, if the side of the vessel is tapped with a hammer, the structure of the sand will collapse. The shearing resistance of the sand becomes temporarily almost zero, and the weight will sink to the bottom of the vessel.

16. The steepness of the slope of a submerged metastable deposit influences the nature of the movement of material resulting from liquefaction. For the case of a steep slope subjected to liquefaction, the material adjoining the slope flows rapidly to the bottom of the slope, coming to rest in the form of a fan-shaped deposit unless stream currents carry the material away. For a gentle slope subjected to vibrations causing liquefaction, the resulting slope is flatter, and the center of gravity of the mass is displaced in the direction of the slope. Sedimentary deposits with metastable structure commonly contain layers and lenses of
material having stable structures. Therefore, actual rates of movement in a liquefied sediment are likely to change erratically from point to point in the sediment.

17. Failures described by Terzaghi. On the basis of the breaking of the cables in the bottom of the Orkdals Fiord, it appears that progressive liquefaction can occur over large areas in metastable submarine sediments having a very flat slope. It is believed, but as yet unproven, that the advance of the boundary between liquefied and intact sediment is caused by seepage pressures. On one side of the boundary, in the liquefied portion of the sediment, the excess hydrostatic pressure at any depth, \( z \), is almost equal to the submerged weight of the sediment above. Then, as the liquefied portion consolidates, water is forced to move upward and outward. Some of the water tends to move into the adjacent intact portion of the sediment, causing seepage pressures of sufficient magnitude to unbalance the forces holding the metastable structure intact. This results in the progress of liquefaction to formerly intact portions.

18. Progressive liquefaction after the initiation of liquefaction of a metastable sediment by earthquake vibrations is considered to be the cause of the flow failures and the breaking of the cables in the Orkdals Fiord. Progressive liquefaction is considered to have caused the large flow slide in the Polla Fiord, and failure of the sand embankment in the Swir III storage dam in the U.S.S.R. Development of seepage pressures at the base of a silt deposit of low permeability resting on more permeable deposits is believed to have caused liquefaction of the silt deposit at the pulp mill on Howe Sound, B.C.

19. Failures described by Turnbull. Both failures at the Kingsley Hydraulic Fill Dam involved a reasonably well-graded gravelly sand material bulldozed into place at a measured density which was lower than the critical density determined from laboratory tests. The average gradation of the gravelly sands involved in the flow slides was as follows:

<table>
<thead>
<tr>
<th>Sieve</th>
<th>Particle Size, mm</th>
<th>Percent Finer</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 in.</td>
<td>25.40</td>
<td>97</td>
</tr>
<tr>
<td>1/4 in.</td>
<td>6.35</td>
<td>88</td>
</tr>
<tr>
<td>No. 10</td>
<td>2.00</td>
<td>70-76</td>
</tr>
</tbody>
</table>

(Continued)
20. The cause of the liquefaction failure of the gravelly sand forming the left bank below the stilling basin was probably vibrations from the explosions to break up the ice. The laboratory-determined critical densities (dry densities) of this material (based on results of consolidated-drained direct shear tests on saturated samples using a shear box having an area of 64 sq in. and a total depth of 10 in.) were about 120 to 125 lb per cu ft; whereas the in-place dry densities ranged from 110 to 120 lb per cu ft. It was noteworthy that the in situ bank of gravelly sand immediately adjacent to the area of the explosion did not fail. The in situ density of this gravelly sand material was approximately equal to the so-called critical density determined by laboratory tests.

21. As with the bulldozed material below the stilling basin, the material bulldozed around the morning glory spillway also had a dry density which was significantly lower than the critical density. Undoubtedly, strong vibrations from sound waves of the cracking ice were produced in the water phase of the bulldozed gravelly sand mass resulting in sudden consolidation, high pore pressure, and more or less complete loss of effective stress.

22. An interesting sidelight was the fact that a careful study had been made of the possibility of flow failures occurring in the in situ and disturbed blow-sand material which made up the entire left abutment of the dam. The critical densities of this blow-sand material (using results of consolidated-drained and constant volume triaxial compression tests performed on saturated samples) were determined by Professor Taylor of the Massachusetts Institute of Technology (MIT) and were found to range somewhat below 100 lb per cu ft (dry density), whereas the natural dry density of the blow sand in place probably ranged from 100 to 104 lb per cu ft, averaging about 102 lb per cu ft. Both Professor Taylor and Dr. Gilboy (a private consultant) felt that this was a sufficient margin and that flow slides would not be experienced in the very uniformly graded blow-sand.
material. Actually, no flow failures were experienced either in the in situ or disturbed blow-sand material during construction, and none have been experienced since construction. The average gradation of the blow sand in percent passing the various sieves is as follows:

<table>
<thead>
<tr>
<th>Sieve</th>
<th>Particle Size, mm</th>
<th>Percent Finer</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 20</td>
<td>0.841</td>
<td>100</td>
</tr>
<tr>
<td>No. 50</td>
<td>0.297</td>
<td>75</td>
</tr>
<tr>
<td>No. 100</td>
<td>0.149</td>
<td>35</td>
</tr>
<tr>
<td>No. 200</td>
<td>0.074</td>
<td>1</td>
</tr>
</tbody>
</table>

23. The average gradation of the loess material involved in the flow slides in the water-supply canal was as follows:

<table>
<thead>
<tr>
<th>Sieve</th>
<th>Particle Size, mm</th>
<th>Percent Finer</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 50</td>
<td>0.297</td>
<td>100</td>
</tr>
<tr>
<td>No. 100</td>
<td>0.149</td>
<td>90</td>
</tr>
<tr>
<td>No. 200</td>
<td>0.074</td>
<td>45</td>
</tr>
</tbody>
</table>

The plasticity of this material ranged from nonplastic to a maximum plasticity index of about 10. Standard Proctor dry density on this material averaged about 100 lb per cu ft, and the critical density was 5 to 8 lb per cu ft less. The cast in-place dry density of this material was very low, ranging from 70 to 80 lb per cu ft. These low densities, of course, tended to increase with time through consolidation. Although no water was against the outer slopes, the cause of the numerous failures was the gradual rise of the saturation line through the loess, from water in the canal, resulting in rapid consolidation of the loess beneath the saturation surface under the load above it. This consolidation undoubtedly produced pore pressures resulting in reduction of effective shear stresses and thus in flow failures. The core section was quite narrow, being only about 8 ft across at the surface, with the top 1 ft above the maximum water surface in the canal. It was interesting to note that no failures occurred in the compacted or in situ loess materials, as evidenced by the fact that the core section was not crevassed at any of the sites of liquefaction failures.

**Laboratory Tests**

24. A considerable amount of literature has been published since
1956 on results of laboratory investigations in connection with liquefaction of sand. This section summarizes pertinent results of these investigations.

25. Florin and Ivanov\textsuperscript{5} describe laboratory experiments in which the excess hydrostatic pressure was measured versus time at various depths in a liquefied sand having an initial height of 20 cm (7.9 in.). The sand was placed in a container at a saturated density of about 125 lb per cu ft; the container was subjected to both impact and vibration tests. Although the drainage conditions were not stated, it is probable that the container was sealed at the bottom. Details on the magnitude and direction of impact and the amplitude and frequency of vibrations were not given. The hydrostatic pressure was measured using special membrane-type inductive cells. At the instant of liquefaction, the excess hydrostatic pressure increased linearly with depth from 0 at the surface to about 0.3 psi at a depth of 20 cm for liquefaction induced by both impact and vibration. This value of excess hydrostatic pressure corresponds to the pressure obtained by multiplying the submerged unit weight in pounds per cubic inch by the depth in inches

\[
\left(\frac{62.57}{1728} = 0.0362 \text{ lb per in.}^3 \times 7.9 \text{ in.} = 0.29 \text{ psi}\right)
\]

26. In the impact tests, the measurements of excess hydrostatic pressure with time indicated that the excess pressure dissipated in about 40 sec and that the decrease started at the bottom of the liquefied sand specimen and progressed upward with time. For example, at 12 sec after liquefaction, the excess pressure was at a constant value of about 0.15 psi from the bottom up to a depth of 10 cm, and at 25 sec it was at a constant value of about 0.07 psi from the bottom up to a depth of 5 cm. This indicates that consolidation of the sand began at the bottom and progressed upward. Some tests were conducted with up to 100 impacts being applied to the same specimen. Although first impact caused liquefaction, subsequent impacts had no effect on prolonging the state of liquefaction and high excess pressures.

27. In tests using vibrations, liquefaction appeared to progress from the top downward, and about 5 sec were required for complete liquefaction over the total depth of 20 cm. Immediately after liquefaction
over the total depth, consolidation began at the bottom and progressed upward. The pore pressure decreased in a manner similar to that in the impact tests. However, continued vibration after complete liquefaction caused additional consolidation, and the decrease in excess hydrostatic pressure was slower and more gradual with decrease in depth, as opposed to the impact test in which the excess pressure was a constant value from the bottom up to various depths depending upon elapsed time.

28. Tests using different gradations of sand indicated that the duration of the liquefied state (until the entire specimen had consolidated) decreased rapidly from about 40 sec to less than 1 sec with increase in the $D_{60}$ size (i.e. the maximum diameter of 60 percent of the soil particles) from about 0.2 mm to about 1.13 mm. Tests were also conducted with a pervious surcharge, and the results indicated a rapid decrease in the time of the liquefied state with increase in effective surcharge pressure at the top of the sand.

29. The laboratory tests also indicated that the presence of entrapped air or gas greatly reduced the propagation of dynamic disturbances and thus decreased the zone of liquefaction. Florin and Ivanov\textsuperscript{5} state that loose sands with a relatively small content of entrapped gas may not be liquefied by dynamic disturbances. They also state that their investigations have shown that propagation of liquefaction of a "chain reaction" type does not occur in nature and that if the surface of the soil is horizontal, no liquefaction propagation is likely to occur. It may be noted that the last statement disagrees with the concept described by Professor Terzaghi as summarized in paragraph 17.

30. Haung,\textsuperscript{6} in describing vibration-induced liquefaction of a saturated sand mass, contends that the magnitude of the excess hydrostatic pressures (or pore pressures) developed during vibration is affected not only by the density and other physical properties of the sand, but also by the intensity and other characteristics of the dynamic action and moreover, in a very important way, by the original state of stress in the sand. He suggested that the relation between these factors can be established on the basis of triaxial compression tests using values of major and minor principal stresses, $\sigma_1$ and $\sigma_3$, equal to those at a given point due to in situ static loading and varying these values dynamically.
by increments equal to $\pm \Delta \sigma_1$ and $\pm \Delta \sigma_3$ which are created by the in situ dynamic loading. Tests of this type are described which were performed using a triaxial chamber fastened to a vibrating platform, with the specimen loaded vertically with weights. A lateral pressure equal to $\sigma_3 - \Delta \sigma_3$ was applied and the vibrating table was set into motion producing an acceleration, $a$, in feet per second per second such that the vertical stress on the specimen changed between a specified limit:

$$\sigma_1 \pm \Delta \sigma_1$$

where $\Delta \sigma_1 = \sigma_1 \frac{a}{g}$; $g$ is the gravitational acceleration constant equal to 32.2 ft per sec per sec. Tests were conducted on saturated specimens of a sand having a 50 percent particle size ($D_{50}$) of 0.09 mm and a uniformity coefficient of 1.4, with dry densities ranging from about 88 to 97 lb per cu ft. Several series of tests were performed, and only one parameter at a time was varied. The test results indicated that the maximum pore pressure developed varied inversely with (a) the ratio of major to minor principal stress, (b) the density, and (c) the minor principal stress. However, the maximum pore pressure developed varied directly with magnitude of acceleration.

31. Measures recommended by Haung for prevention of liquefaction of sand slopes included:

a. Increase in degree of compaction of the sand.

b. Decrease in slope.

c. Increase in effective lateral stress.

He stated that since liquefaction will last longer in a fine sand having low permeability than in a coarser sand having a relatively high permeability, the stability of a sand mass against liquefaction will be improved by providing more effective drainage.

32. Bjerrum, Kringstad, and Kummeneje present the results of triaxial tests on a sand having a $D_{60}$ size of about 0.17 mm and a uniformity coefficient of about 2.3 obtained from the site of a slide in the Trondheim Fiord. Drained, consolidated-undrained, consolidated-constant volume, and special triaxial tests at controlled rates of strain were performed on specimens prepared at porosities ranging from 36 to 48 percent (void ratios, $e$, of 0.56 to 0.92). Complete liquefaction was never obtained in the tests on the sand in the loosest state. However, the test results indicated that the angle of shearing resistance varied with porosity over a much wider
range than had been previously supposed and that the angle of shearing resistance decreased very rapidly when the porosity exceeded about 44 percent ($e = 0.78$). Undrained tests on sand in the loosest state resulted in angles of shearing resistance as low as 11 deg. In the consolidated-undrained tests on the loose sand, the maximum value of deviator stress ($\sigma_1 - \sigma_3$) was reached at very low strains, on the order of about 0.5 percent, and long before the maximum principal stress ratio ($\sigma_1/\sigma_3$) was reached. This is explained as resulting from the rapid development of pore pressure with strain. The special tests were conducted on specimens consolidated anisotropically, and a principal stress ratio was used which allowed no change in the diameter of the sample during consolidation. The ratio of the minor to major principal stresses observed during consolidation is a measure of the coefficient of earth pressure at rest, $K_o$. The measured values of $K_o$ ranged from 0.3 at a porosity of 36 percent ($e = 0.56$) to 0.70 at a porosity of 48 percent ($e = 0.92$).

33. Whitman and Healy report results of rapid undrained triaxial tests on relatively loose saturated Ottawa sand specimens prepared at an initial void ratio of 0.65. The results indicated that the deviator stress ($\sigma_1 - \sigma_3$) increased abruptly to a strain of about 0.5 percent, remained constant for a perceptible interval, and then continued to increase at a slower rate than initially. The significant change in the rate of increase of deviator stress at a strain of about 0.5 percent was termed the initial yield point, and it is suggested that this initial yield point may be the critical factor in the behavior of loose sand deposits. This is based on the concept that once a slide begins in a sand mass, the inertia of the mass may cause the slide to continue even though larger shear resistance develops at larger strains.

34. Seed and Lundgren report results of slow and rapid triaxial tests on specimens of fine sand having a $D_{50}$ size of about 0.1 mm and a uniformity coefficient of about 1.8. In the rapid tests, the load was applied by a falling weight; and for a void ratio of 0.83, a noticeable decrease in deviator stress occurred between strains of 3.5 and 10 percent which was considered to have been caused by partial liquefaction. The results of static tests on specimens of the sand at the same void ratio, 0.83, indicated a continued increase in deviator stress with increase in
strain. It was concluded that in the rapid tests the pore pressure developed faster than that which could be relieved by drainage or by volume increase that occurs even in loose sands at large strains, and that considerable deformation or partial liquefaction occurred before the developed pore pressure was relieved.

Field Investigations

35. Two field investigations which have been reported in recent literature involved the use of explosives for determining the degree of susceptibility of saturated sandy soil deposits to liquefaction. The results of these investigations are summarized in the following paragraphs.

36. Florin and Ivanov⁵ describe a method of standard blasting in which tests are made by exploding charges of equal weight at equal depths so that the results for different soil conditions can be more easily evaluated. For investigations of liquefaction of a saturated sand stratum 26 to 33 ft deep, and 11-lb charge of "ammonite" (interpreted here as possibly meaning ammonium nitrate) at a depth of 14.7 ft was used as a standard to ensure that the explosion would not be vented to the surface. This charge weight at this depth in a loose, saturated sandy soil caused surface settlement within a few minutes after detonation. This was followed by expulsion of water at the surface which sometimes occurred as a concentrated stream or fountain and sometimes lasted as long as 15 to 25 min. A 2-in.-diam steel bar weighing about 20 lb was observed to sink as deep as 6 ft into the soil.

37. The criteria used to define susceptibility to liquefaction were the magnitude of the average settlement of the surface of the soil within a 16.4-ft radius around the location of the standard explosion and the ratios of the average settlement within this radius resulting after each of three successive standard explosions. It is stated that if the average settlement (presumably from the first explosion) in the 16.4-ft radius is less than about 3 to 4 in., there is no need to provide measures against liquefaction. However, the greater the difference between the average settlements resulting from successive explosions, the looser the soil was initially, and the greater is the possibility of liquefaction.
For example, if the ratio between settlement values from two successive explosions is greater than 1:0.6, there is danger of undesirable spreading of liquefaction.

38. Florin and Ivanov\(^5\) state that the procedure of standard blasting has been used to study the possibility of liquefaction of natural deposit sand hydraulic fills on certain Russian rivers. It is considered to be reliable and has the advantages of simplicity and low cost.

39. Kummeneje and Eide\(^10\) describe the results of field tests in which explosives were used at several sites to assess the possibility of flow slides occurring in underwater marine deposits of sand and silt along the shores of the Trondheim Fiord. The tests were performed by exploding charges of dynamite having weights ranging from 0.15 to 5.3 lb at depths of 16 to 33 ft below the surface of the deposits and generally at different locations within the test area. In some tests as many as seven individual charges using increasing charge weights were fired at the same location and depth.

40. A comprehensive field investigation was made which included undisturbed sample borings and cone penetration borings before and after the blasting tests to determine changes in properties of the natural soil deposits. Piezometers were installed and readings obtained during the blasting tests were used to determine the excess pore-water pressures developed.

41. For a 2.6-lb charge exploded at a depth of 23 ft, the excess pore-water pressures measured at the same depth and at distances of 16 and 66 ft from the explosion were 45 and 5 percent, respectively, of the effective overburden pressure. For the same charge weight exploded at a depth of 16 ft, the measured excess pore pressure at the same depth and at a distance of 16 ft was about 55 percent of the effective overburden pressure; the excess pore pressure dissipated within about 30 min. For a charge weight of 5.3 lb exploded at a depth of 16 ft, the measured excess pore-water pressure at the same depth and at a distance of 16 ft from the explosion was 70 percent of the effective overburden pressure. Successive explosions of the 5.3-lb charge at the same location and depth produced similar increases in pore-water pressures. After seven consecutive explosions at one location, the maximum surface settlement amounted to about
33 in. It was concluded from the tests that although partial liquefaction had been induced from the explosions, the deposits were not susceptible to flow slides.

42. In situ tension tests using a screw-plate test apparatus* were made before and after a total of seven charges had been exploded at a depth of 16 ft (a total of 15 lb of explosive) to determine the change in soil strength in the vicinity of the detonations. The results indicated that the soil had become looser over a zone from a depth of about 5 ft to about 25 ft below the surface with the maximum diameter of the loosened zone being about 20 ft at a depth of about 13 ft. Outside this zone the soil was denser than it was before the blasting.

Methods of Stabilizing Potentially Unstable Deposits

43. Terzaghi and Peck\textsuperscript{11} have stated that flow slides occur only if the sand is very loose and that the tendency toward sliding can be reduced by increasing the density of the sand. Measures recommended by Haung\textsuperscript{6} for prevention of flow slides include increasing the degree of compaction of the sand, flattening the slopes, and increasing the lateral pressure by adding surcharge such as riprap to the slopes. In addition, Haung\textsuperscript{6} suggests that stability against liquefaction can be improved by providing more effective drainage.

44. The literature review did not reveal any information on application of the above-mentioned measures for prevention of flow failures. However, a considerable amount of literature was found which describes detailed methods for increasing the density of loose sand deposits. The methods described involve the use of the vibroflotation process, explosives, electrical discharges, and piles. These methods are summarized in the following paragraphs.

Vibroflotation

45. Vibroflotation is a licensed process which employs mechanical

* An apparatus, similar to an earth anchor, which is screwed into the ground then pulled up. The force required to pull the screw plate up a specified distance is an indication of the shear strength of the soil.
vibrations, jetting, and saturation if necessary to move, shake, and "float" the loose sand particles into a denser state. A device known commercially as a "vibroflot" is used to produce vibrations in and saturation, if necessary, of the sand. The vibroflot is a metal cylinder about 15 in. in diameter and 6 ft long. It contains two compartments and has water passageways in the wall. The upper compartment contains a 30-hp, water-cooled, electric motor which drives a 200-lb eccentric weight located in the lower compartment. The weight is rotated at a speed of 1800 rpm and produces a centrifugal force of 10 tons which causes the bottom of the vibroflot to vibrate horizontally, with the maximum movement being 3/4 in.

46. The vibroflot is suspended vertically from the boom of a crane and is guided by wooden rods secured to the base of the crane. The vibroflot is positioned over the location to be compacted and the motor is started. Water under pressure is allowed to flow out of the bottom of the vibroflot; this causes a jetting action which liquefies the soil below and adjacent to the vibroflot so that the vibroflot sinks into the soil under its own weight. When the vibroflot reaches the desired depth of compaction, the flow of water is transferred by closing and opening certain valves so that the water flows in a downward direction from the top of the vibroflot. This aids in compaction of the sand. Sand, previously stockpiled on the surface, is shoveled into the crater formed about the vibroflot to replace the in situ sand that is compacted. The vibroflot is then raised at the rate of about 1 ft per min under normal conditions, and shoveling of sand into the hole is continued until the vibroflot reaches the surface. In free-draining soils about 5 cu ft of sand are added per foot of depth compacted. The vibroflotation process produces a cylindrical column of compacted material of approximately 8 to 10 ft in diameter. By placing the vibroflot in successive positions so that overlapping of compacted areas is produced, an area may be compacted to almost any desired relative density.

47. The density of the soil at any time during compaction with the vibroflot is related to the power input to the motor. As compaction proceeds, the resistance to movement of the vibroflot increases and greater power input is required. A recording ammeter measures the
power input, and the ammeter readings can be correlated with the density of the soil. Therefore, the ammeter provides a means of achieving a predetermined soil density.

48. A study of the effectiveness of the vibroflotation process has shown a general applicability to clean, free-draining, granular soils, i.e. material between 60- and 0.06-mm grain size (2 in. to approximately the No. 200 sieve). The vibroflot can be inserted into the ground to almost any desired depth. However, the maximum practical depth is limited to the depth to which the vibroflot will sink under the combined action of jetting and its own weight. In past applications of this method, the maximum depth has been 90 ft. Field experience has indicated that the effectiveness of the vibroflotation process decreases with the distance from the center of compaction, with increase in density being almost zero at a radial distance of about 6 ft; therefore, the zone in which the liquefied or quick condition is developed at one time during the operation is small, and the operation is not likely to have any undesirable effect on the entire deposit. Another feature of the process is that the loose sand deposits need not be naturally saturated. Numerous articles and papers have appeared in the literature describing the application and results of the vibroflotation process in densifying loose sand deposits for a variety of purposes.

Explosives

49. A bibliography on the use of explosives was compiled by WES in connection with studies on compaction of sands for structural sites at Cape Kennedy, Fla. Buried explosive charges have been successfully to compact saturated, loose cohesionless soils in foundations of dams and foundations for houses on fills. The charges are placed in the saturated sand mass at preselected vertical and horizontal spacings. Usually each charge is inserted into the sand mass through a hollow pipe which has been jetted to the correct position. The pipe is removed prior to detonation and the hole stemmed with tamped sand. Upon detonation the explosive energy is transferred to the surrounding medium causing liquefaction of the sand and allowing the sand grains to fall into a much denser arrangement. The process is aided by the pressures of the overburden.

50. In general, the soil limitations imposed upon the explosive
method of densification are that the soil should not contain more than 20 percent silt (nonplastic material finer than 0.074 mm) or 5 percent clay (plastic material finer than 0.074 mm). The sand to be densified should be below the water table (saturated) for best results, although some degree of compaction by the explosive method has been reported in partially saturated conditions. In practice, no limitation is imposed by depth; however, optimum vertical and horizontal spacing must be determined by trial tests. For compaction to less than 30 ft, one tier of charges placed at the two-thirds depth and fired simultaneously will usually be sufficient, and for compaction deeper than 30 ft, it appears that two or more tiers of charges should be used. Horizontal spacing should be planned to produce an overlapping effect. The spacing will depend on several factors including the depth and the size of the charge. In general, the spacing varies from 10 to 25 ft, since closer spacing is considered uneconomical. Investigations reported by Kummeneje and Eide have shown, for example, an increase in pore pressure at distances up to approximately 65 ft from a 2.64-lb explosive charge although complete liquefaction was not obtained over this entire region. The values of relative density obtained by means of explosive shocks also depend largely upon the size and spacing of charges and the degree of saturation of the sand, but values in excess of 65 percent relative density can easily be obtained at economical costs.

51. Several other articles and papers have appeared in the literature describing the application and results of the explosive process in densifying loose sand deposits; however, they do not give any additional information and are therefore not specifically cited. The most comprehensive bibliographies are found in references 17 and 22.

52. Recently reports have been made of compaction of loose saturated sands by the use of high-voltage discharges within the sand deposit. The soil is compacted by the combined action of the excess pore-water pressure caused by the impact of the high-voltage discharges and the soil weight.

53. Electrodes are lowered into position by water jetting. High-voltage discharges are created by an impulse generator which supplies the
necessary quantities of electrical power at predetermined time intervals. The shock wave produced is similar to that produced by buried explosives in that both create a zone of liquefied material and thereby increase the density of the loose sand. Increases of as much as 10 percent in density were reported.

54. No cost figures are available, but on a Russian project about 1300 cu yd of sand was compacted to a depth of about 33 ft with one working shift. The effectiveness and general applicability of the electrical method of densifying loose sand deposits are as yet unproven.

Piles

55. The vibrations caused by driving piles into a loose sand deposit can produce densification. Conceivably, piling driven by vibratory hammers would be particularly suitable for this purpose. Best compaction would occur for saturated conditions. Spacing and depth requirements would need to be established by means of test sections. One possible advantage of using piles to achieve densification is the column of somewhat looser material left behind as the pile is extracted. The sand columns of somewhat looser material would not be subject to liquefaction and yet would serve to relieve excess water pressure within the soil mass.

56. Literature concerning the use of piles for this purpose is limited. Terzaghi and Peck\textsuperscript{11} describe briefly, without reference to the original papers or projects, two instances of piles being used for compaction purposes only. In one instance, concrete piles 45 ft long were driven on 3-ft centers into fine sand below the water table to cause a surface subsidence of as much as 3 ft and a decrease in porosity from about 44 to about 38 percent (void ratios of 0.89 to 0.61). In the second instance, steel pipe piles filled with ballast were driven to compact a layer of sand extending to a depth of 50 ft below the water table. During the driving, the surface settled about 1.5 ft, and the average porosity of the sand decreased from 42 to 35 percent (void ratios of 0.72 to 0.54). The piles were extracted after they had been driven.

57. Reference \textsuperscript{24} presents the following general statements regarding the use of pile driving to attain soil compaction:
a. The radius of influence of compaction is about five times the pile diameter.

b. For loose sand, a typical increase in relative density would be about 30 percent.
PART III: DISCUSSION

Results of Literature Review

58. Although the literature that was reviewed does not provide information on techniques and procedures specifically for prevention of flow slides, it does provide a considerable amount of useful information on liquefaction phenomena, the occurrence of flow slides, laboratory and field tests which aid in determining susceptibility to liquefaction, and field procedures for increasing the density of loose sandy materials. The investigations summarized in the literature review add valuable information to that contained in Report 12-5.

59. Excess hydrostatic or pore pressures can cause liquefaction and can also result from liquefaction. Terzaghi has indicated that excess hydrostatic pressures resulting from seepage pressures at the base of a silt deposit of relatively low permeability caused liquefaction of the silt deposit. Excess pore pressures can also develop in saturated sands and silts where drainage is prevented and stresses are applied which cause only very small strains, as indicated by laboratory triaxial test results described by Bjerrum, Kringstad, and Kummeneje and Whitman and Healy. The magnitude of pore pressures developed is greater in loose sands and silts than in denser ones. On the other hand, liquefaction or collapse of particle structure induced by vibration or impact causes a temporary increase in pore pressure. Florin and Ivanov have measured excess pore pressures developed during liquefaction initiated by impact and vibration and have shown that the duration of liquefaction is dependent on the particle size and drainage conditions.

60. The natural flow failures described by Terzaghi and the data and discussion provided by Turnbull furnish valuable information on the possible causes and areal extent of flow slides. However, no information was found in the literature on the magnitude of pore pressures developed during initiation and progress of natural flow slides or other changes in conditions of the material involved. This is understandable, since it is not known with any degree of certainty when or where flow slides will occur and thus the need for the prior installation of piezometers or a detailed
investigation of initial soil conditions prior to failure could not be anticipated.

61. The results of field tests described by Kummeneje and Eide\textsuperscript{10} have shown that explosives can be used to assess the possibility of flow slides and that excess pore pressures developed from explosive shocks amount to a significant percentage of the effective overburden pressure. The excess pore pressures developed were dependent on the distance from the explosion and also on the charge weight and depth of explosion. The results of this investigation and the results of field tests with explosives described by Florin and Ivanov\textsuperscript{5} provide valuable guidance in the use of explosives in saturated sand deposits to assess the possibility of flow slides or undesirable spreading of liquefaction.

62. After review of the current literature on methods of densification of loose sand deposits, it is concluded that the vibroflotation process offers the highest degree of control in obtaining desired densities. Although explosives have been used for densification, a number of trials would be required to establish specific criteria for charge weights, depths of emplacement, spacing, number of charges at a particular depth, and sequence of firing to obtain desired densities at a given area. The use of electrical discharges and the use of piles have not been employed extensively and would probably require a considerable amount of experimental development to provide a reliable procedure for field use.

Effect of Literature Review on Findings of Report 12-5

63. The results of the literature review do not change any of the findings contained in Report 12-5. The descriptions of the occurrence of flow failures by Terzaghi\textsuperscript{4} tend to strengthen the discussions contained in Report 12-5 on the progressive development of flow failures. The results of triaxial shear tests described by Kummeneje and Eide\textsuperscript{10} provide additional information which supports the discussion of shear strength and volume change characteristics in Report 12-5.

64. The description and discussion of the Central Nebraska Public Power and Irrigation District failures reemphasize the importance of critical density (discussed in detail in Report 12-5) in determining the
susceptibility to flow failure and also indicate that deposits even as coarse as gravelly sand materials when the in situ densities are lower than the range of critical densities are subject to flow failure. However, since this report is concerned primarily with methods for increasing the stability of deposits susceptible to flow failure rather than methods of determining susceptibility, critical density is not further discussed.

65. No information was found in the literature on model or field investigations of the development of arcuate failures as recommended under "Limited Investigations" in Report 12-5. Nor was information found in the literature pertinent to detailed field and laboratory investigations of point bar sands as recommended in Report 12-5 and summarized in paragraph 3c of this report. It is apparent that these areas still need to be investigated.

66. Information was found which pertains to several of the six measures suggested in Report 12-5 for decreasing the potential danger of flow slides. The literature on the use of explosives to assess the possibility of liquefaction and to densify loose sands provides information which could be used as a basis for the development of detailed field procedures for accomplishing controlled preventive failures as suggested in Report 12-5 or for densification of loose sand strata. Although no new information was found on the use of setback levees, this measure is still considered suitable for protecting mainline levees against possible crevassing along reaches of riverbank susceptible to flow failure where the land required can be obtained without difficulty. Although a large amount of information exists on the general use of grouting and on improvement of drainage conditions for increasing the stability of soil deposits, no specific information was found on the application of these measures to riverbank deposits susceptible to flow failure.

Prevention of Flow Failures

67. It is considered desirable that laboratory and field investigations be conducted to obtain additional information on the development and progress of arcuate-shaped liquefaction failures, including information on the development of excess pore pressures during such failures and more
detailed information on the associated soil conditions prior to any large-scale field experiments. However, even without these studies, field experiments can be conducted to determine the feasibility of preventing failures at potentially unstable sites by (a) inducing preventive failures using explosives or over steepening of banks by dredging at unrevetted locations, or (b) by increasing the density of loose sand strata using the vibroflotation process or explosives at unrevetted or at revetted sites. Field experiments to deliberately induce failures at unrevetted sites in order to stabilize loose deposits would have to be very carefully planned and executed since controlling the sizes of the induced failures to tolerable limits might be difficult. Densification of loose sand strata appears to be the best approach and is considered superior to other methods of increasing stability such as flattening slopes or providing more effective drainage. The latter two methods would be limited to unrevetted locations, and it would be rather difficult to ensure satisfactory results since much of the work would have to be performed under water.

68. Of the four previously mentioned methods that might be used to densify the loose zone A sands, and thereby increase the stability of point bar riverbank slopes against liquefaction failures, the vibroflotation process appears at present to be the most suitable. Based on past experience, the success and degree of control offered by the vibroflotation process are well established. Although the use of explosives might be more economical, the results to be obtained would be quite uncertain. For the purpose of a field study to evaluate the feasibility of densification for minimizing or eliminating the occurrence of flow slides, densification should be accomplished using the vibroflotation process.

69. One possible experiment would be to attempt to initiate a small flow failure in a portion of riverbank by small explosive charges and/or excavation of the toe of the bank at one location in a potentially unstable site to verify that the location was unstable. The loose sand at another location in the area would then be densified; small explosive charges and/or excavation of the toe of bank would again be used to determine whether the densification procedure would prevent an additional failure. This procedure would require an unrevetted site and a comprehensive soils investigation to determine detailed soil conditions over an extensive area to ensure that
creation of a small failure would not progress into an intolerably large failure. This procedure would also require preliminary trial tests to determine the charge weight, depth of charge, and location of charge with reference to the riverbank required to cause a small flow failure. The information contained in reference 10 would provide guidance in determining charge weight necessary to cause various excess pore pressures, and the information contained in table 5 of Report 12-5 would provide guidance as to the excess pore pressure required to cause instability of various slopes.

70. As an alternative, field experiments could be conducted by excavating a trench having slopes flat enough to be stable in a saturated, loose sand deposit having a relatively thin, cohesive overburden. Explosive charges and excavation of the toe of the slope would then be used in an attempt to cause a flow failure of one of the slopes to verify instability against liquefaction. Provided a flow failure was created, one of the remaining slopes or the slopes subsequently excavated in a nearby identical area could then be densified to various degrees, depths, and distances back from the slope. Additional explosive charges and excavation of the toe of the slope could then be used to determine the effect of the degree and extent of densification on the stability of the slope against liquefaction. This procedure would also require a rather extensive field exploration program to locate a suitable deposit and to determine characteristics of the deposit before, during, and after the experiment. In addition, experimental field work would be required to develop proper procedures for using explosives as mentioned in the preceding paragraph. Such experiments appear to be more suitable than those involving riverbanks, since the latter introduce the danger of creating an intolerably large failure. Therefore, the following section on Proposed Field Experiments refers to field experiments using excavated ditches.

Proposed Field Experiments

71. The feasibility of achieving densification to increase the stability of loose zone A sands against liquefaction, as outlined in the preceding section, can best be determined by field experiments at an accessible site where suitable soil conditions exist. The purpose of
these experiments would be to evaluate the feasibility of the densification process and to develop suitable techniques required to accomplish the desired results. A field site for the tests should:

a. Contain relatively extensive and thick deposits of saturated, loose zone A sands with a minimum thickness of cohesive overburden soils.

b. Be located in an area along the Mississippi River having relatively large, open areas of flat ground surface adjacent to the riverbank.

c. Be accessible with heavy equipment using existing roads.

A proposed field test for the procedure using densification is described in the following paragraphs.

72. Preliminary selection of potentially suitable sites would be based on an office study of available revetment boring data and geological information. Accessible sites which appeared to be most promising would then be investigated in the field using the rotary cone penetrometer to determine the characteristics of the zone A sands. The selected site must be sufficiently distant from the riverbank so that field tests would not endanger the stability of the riverbank or levees in the area.

73. After selection of the most suitable site, undisturbed sample borings would be made to verify the thickness of cohesive topstratum and thickness of the zone A sands. Undisturbed samples of the zone A sand obtained would be tested to determine their relative density and to verify that the range of relative densities over the area and with depth was low enough to ensure that flow failures of excavated slopes would be possible. Based on previous potamology studies, it is considered that relative densities on the order of 40 percent are necessary for liquefaction to occur in zone A sands.

74. After selecting the site, a ditch would be excavated having a depth and steepness of slope likely to result in a flow slide when subjected to small explosive charges, perhaps accompanied by excavation of the toe of the slope. Several trials might be necessary using several different explosive weights and placement locations before a flow failure was produced.

75. After creation of a flow failure, the remaining excavated slopes of the ditch, or the slopes of another ditch excavated in the same area as the first ditch, would be densified using the vibroflotation method.
Various sections of the ditch slopes would be densified to various degrees, and explosive charges and toe excavation would be used to determine the stability.

76. The field exploration for the experiments would include undisturbed sample borings and cone penetration borings before and after initiation of failure and densification. Laboratory tests would be performed on undisturbed zone A sand samples to determine the relative density of the sand before and after failure and densification. Piezometers (closed-system type) would be installed with some type of automatic recording apparatus incorporated to determine the excess pore pressures developed during the time of failure.
PART IV: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

77. The results of the literature review provide additional useful information and support in principle the findings of Report 12-5 which indicate that very loose, saturated zone A sand strata, exposed by scour and subjected to excess pore pressures, are the principal cause of liquefaction failures along the Mississippi River. However, no data were found which provide the information that would be obtained from the additional investigations recommended in Report 12-5 on development and progress of arcuate-shaped flow failures, including information on development of excess pore pressures during such failures, and more detailed information on associated soil conditions. While such additional investigations are still considered desirable, it is concluded that sufficient information is presently available to undertake field experiments to investigate the feasibility of improving the stability of loose sand deposits with respect to flow failure by densification.

78. Field experiments at riverbanks to develop controlled preventive failure to stabilize loose deposits would have to be carefully planned because of the possibility of inducing large-scale failures. Other procedures, such as reducing bank slopes or improving drainage, are not considered feasible.

79. It appears that the most promising means of stabilizing loose sands is by densification. Of the several possible methods for densifying loose sands, the vibroflotation process, although expensive, appears to be the most suitable at present since field procedures for specific degrees of densification with this technique are well established. The use of explosives would require a considerable amount of preliminary field experimentation to develop procedures for obtaining specific degrees of densification. In addition, the use of explosives would not provide the degree of control that has been established for the vibroflotation process and it is possible that the use of explosives could cause liquefaction failures during the experimental phase. This would be highly undesirable, especially at sites already revetted. It is believed that the use of electrical
discharges or piles to accomplish densification has not been sufficiently developed to be considered economically useful at the present time.

80. It is considered that field experiments on the feasibility of stabilizing loose sand slopes by densification using the vibroflotation process can best be accomplished by utilizing excavated trenches and checking the stability of the trench slopes before and after densification with explosive charges. If densification is successful in preventing flow failures in an excavated ditch, then additional field testing may be warranted to develop more economical procedures for densification of loose sands susceptible to flow failure for ultimate use along the banks of the Mississippi River.

Recommendations

81. Based on the results of the literature review, the following recommendations appear to be warranted.

a. It is recommended that field experiments be undertaken to investigate the feasibility of improving the stability of loose sand deposits with respect to liquefaction failure by densification.

b. It is recommended that additional investigations outlined in Report 12-5 to obtain information on the development and progress of arcuate-shaped liquefaction failures, including information on development of excess pore pressures and more detailed information on associated soil conditions, be considered.
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