AUTHORIZATION

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Present tailings disposal methods are subject to limitations that warrant the development of alternative disposal techniques. A recently proposed method is the thickened discharge disposal method, which involves sloping tailings towards a downstream embankment, thereby reducing the height of the embankment required to store a given volume of waste material. The seismic stability of such slopes is a concern and model studies were performed to investigate their stability.

The model was composed of a sloped deposit of fine sand 81 cm long, 20 cm wide and a downstream barrier 14 cm high. Slopes ranging from 4 to 14 percent were subjected to base accelerations ranging from .025 g to .10 g. Test deposits subjected to accelerations above a critical acceleration, dependent on the slope angle, were observed to liquefy and flow. These test deposits came to rest at a final slope of approximately one percent.

Model deformations were recorded and liquefied deposits were observed to behave similarly to a viscous fluid. A viscous fluid model was found to predict actual partial displacements reasonably well. Although application of test results may be limited to size effects, it seems appropriate to analyze liquified cohesionless material as a viscous fluid.
It is suggested that a statically stable sloped tailings deposit, upon liquefaction to a significant depth, may become unstable. The resulting flow, governed by post-liquefaction mechanical properties, could overtop a downstream embankment.
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CHAPTER 1
INTRODUCTION

The disposal of mine waste material in a safe and cost effective manner is a subject that has received considerable attention in recent years. Present commonly used disposal techniques are subject to limitations, as discussed in Chapter 2, such that the development of alternative techniques is warranted. Any such proposed alternative methods must be critically evaluated to assess their viability as an economical, safe solution to the tailings disposal problem.

One such alternative is the thickened discharge disposal method (Robinski, 1975, Robinski, 1978). This method involves creating a mildly sloped conically shaped tailings deposit, thereby eliminating the need for a high and costly embankment as required for conventional tailings disposal systems. The thickened discharge method, as discussed in Chapter 2, has proven to be an economical solution to the tailings disposal problem. However, the stability of a sloped deposit during and after an earthquake is a design consideration that must be assessed to determine if the thickened discharge method is a viable alternative appropriate for widespread use.

A shaking table model testing program was developed to help to assess the seismic stability of sloped tailings deposits. Due to the wide degree of variability of tailings material produced in the mining industry, and problems associated with the uniform deposition of fine-grained materials in the laboratory,
a uniform fine-grained sand was selected to model tailings behaviour. The test material is described in Chapter 3, where it is compared to a variety of typical tailings materials.

Of interest in this study is the response of a soil deposit to cyclic loading. Not only is the response of the material up to the point of liquefaction of interest, even more pertinent is the post-liquefaction response of the deposit. Fundamentals of liquefaction and post-liquefaction material behaviour are presented in Chapter 4. Possible ramifications of material behavior fundamentals with regard to seismic stability of a sloped tailings deposit are discussed in Chapter 5.

The development of model test equipment and procedures included a review of previous model studies, discussed in Chapter 6. The testing program, including equipment and procedures, is discussed in Chapter 7. Test results are presented and discussed in Chapter 8.

The results obtained led to the empirical modification of a fluid analysis describing the particle movements within a viscous fluid body. The fluid analysis is described in Chapter 9, and predicted particle displacements are presented in Chapter 10, along with measured particle displacements for comparison.

The consequences of a tailings disposal facility failure can be enormous, and several case histories are briefly discussed in Chapter 11. A particularly relevant case history, resulting in limited damage, involved the liquefaction and
post-liquefaction flow of sloped tailings material, as discussed in Chapter 11.

Conclusions drawn are presented in Chapter 12.
CHAPTER 2

METHODS OF DISPOSAL

2-1 Introduction

Tailings are waste material resulting from the grinding and mineral extraction processes of a mining operation. A flow chart for a typical tailings production and disposal system is shown in Fig. 1. The disposal of tailings is generally considered to be a capital expenditure resulting in no economic gain. The interest of the mining company is therefore to minimize the capital outlay for tailings disposal. The viability of marginal mining properties can depend upon economical tailings disposal.

The economic feasibility of mining low grade ore has resulted in a large increase in the amount of waste material to be disposed of (Wahler and Schlick, 1976). Compounded with the fact that many open pit mining operations are situated in mountainous terrain with narrow valleys, higher retaining embankments were required using the present tailings disposal methods. The construction of tailings dams has traditionally been empirical and the mining industry in general did not recognize the inherent stability problems in the disposal methods in use until several catastrophic failures occurred. Several of these failures are briefly discussed in Chapter 11.

Hoare (1974) estimated that 90% of Canada's large mining operations have suffered instability of some kind, while only 26% have performed stability analyses (Mittal, 1974). As a
Ore Body

Crushing

Grinding

Concentrating

Tailings (about 99%)

Mineral Concentrate (about 1%)

Spigotted Tailings

Cycloned Tailings

Underflow

Overflow

Natural Earth (imported)

Natural Earth (local)

Waste Rock

Sands

Slimes

Stored Tailings

Tailings Dam

Tailings Pond

FLOW CHART FOR TYPICAL DISPOSAL SYSTEM
(After Jeyapalan, 1980)

FIGURE 1
result governmental regulatory agencies have begun to take a much more active role in governing the disposal of tailings. The monetary, environmental and social costs of failures as experienced at Buffalo Creek, Aberfan and during the Chilean earthquake of 1965 were enormous and strict regulations are now being enforced throughout North America and in many parts of the world. These restrictions govern stability and control of contaminants during the mine's life as well as for the abandonment of the facility. Abandonment alone poses major design problems (D'Appolonia et al., 1972).

2-2 Present Disposal Methods

There are several tailings disposal methods in use. The most common procedure involves the construction of an embankment behind which the waste material is stored. The performance of the embankment is greatly affected by the method of construction. There are typically three methods of embankment construction. They are the upstream, downstream and centreline construction methods. In all three the dam is initiated by the construction of a pervious toe, or starter, dam.

2-2-1 Upstream Construction Method

The upstream construction method is depicted in Fig. 2. This method was widely used prior to the aforementioned catastrophic failures. It was considered to be the most economical storage facility. For an incremental height increase required for tailings disposal a relatively small dyke, which can be raised quickly using a low volume of construction material, is
Dyke raised by scooping Coarse Tailings from beach

UPSTREAM CONSTRUCTION

FIGURE 2
required. The dyke is generally constructed using the coarse tailings, which separate hydraulically from the fines near the point of discharge. The progression of the dam's centreline is in the upstream direction.

The stability of dams constructed by this method has been found to be inadequate in many instances. As the embankment progresses upstream, subsequent dykes must be constructed over fine waste material. These fines are often underconsolidated and exhibit very low shear strength. For this reason, there is a limiting height to which the structure can be built for static stability to be satisfied.

The fine material is also highly susceptible to liquefaction owing to its loose, saturated state. Liquefaction can be induced by earthquake, blasting or construction vibrations. It can also occur if the slimes are subjected to relatively rapid shear strain rates resulting in sufficient cumulative pore pressure buildup. Upon liquefaction, the outer shell is incapable of supporting the liquefied mass and failure would result.

A safe upstream dam can be constructed if proper monitoring (Nyren et al., 1978), basic earth dam engineering and proper material handling is employed (Mittal, 1974). A safe upstream tailings dam is depicted in Fig. 3.

2-2-2 Downstream Construction Method

The downstream method, Fig. 4, is considered to result in a more stable structure than the upstream method. This gain is balanced by the increased expenditure required to achieve this
SAFE DAM USING UPSTREAM METHOD
(After Casagrande and Maclver, 1971)

FIGURE 3
Cyclone

Starter dam

DOWNSTREAM METHOD

FIGURE 4
The downstream method requires a much larger volume of construction material. This material is generally obtained by cycloning the tailings, thus separating the coarser and finer fractions of the material, the coarse fraction, or sands being used for construction. The cyclone operation must be carefully controlled to ensure the proper gradation is obtained for construction material. The sand yield is a critical factor in the downstream method, as a low yield results in a slow rising crest combined with a larger volume of material or be stored. If cycloning does not yield sufficient volumes of sand, borrow material must be used.

The fines are usually spigotted off the upstream face of the dam, while the sands are spigotted off the downstream face, resulting in the centreline progressively moving downstream. The possibility of liquefaction of the sands can be eliminated through compaction. Drainage facilities can also be incorporated in the design to limit the degree of saturation, and hence, the deposit's susceptibility to liquefaction. Either or both methods can be used to guard against liquefaction and the needs must be determined for individual operations. In situ testing at Brenda mines (Mittal, 1974) indicates that compaction is not necessary if proper drainage is ensured.

Besides the increased cost of this method, another major disadvantage exists. The downstream face of the structure changes as the crest elevation is raised. Construction takes place over many years, and no erosion protection can be applied
until the final crest elevation has been reached. The down­stream face is therefore subjected to surface erosion for long periods of time, and erosion channels can develop.

Another construction method commonly described in the literature is the centreline method. This method, Fig. 5, is essentially a modified downstream construction technique. The crest of the structure rises vertically as it is raised.

2-2-3 Comparison with Conventional Earth Dam

Analytical and design procedures for a tailings dam are similar to those used for conventional water retention earth dams. There are, however, inherent differences due to the construction method and nature of the material being stored that preclude the structure from being considered a conventional earth dam, depicted in Fig. 6.

Under static loading conditions the slimes have a low shear strength that contributes to the static stability of the structure. This material is highly susceptible to liquefaction and, for design purposes, it is usually considered the entire deposit is in a liquefied state (Klohn, 1979, Finn and Byrne, 1976). Under these conditions the specific weight of the fluid is considerably greater than that of water and there is an increased hydrostatic design load. The fines are likely to liquefy quickly and result in a rapid shear stress application that could be considered undrained (Klohn and Maartman, 1972). This could result in increased pore pressures and corresponding decreased strengths within the embankment.
CENTRELINE CONSTRUCTION

FIGURE 5
CONVENTIONAL EARTH DAM

FIGURE 6
The tailings material, when spigotted from the crest of the dam, separate hydraulically resulting in a beach made up of coarser material near the dam. This has the effect of lowering the phreatic surface within the dam. The analysis of flow through a tailings dam is much more complex due to the excess pore pressures in the underconsolidated slimes. Mittal (1974) provides a detailed seepage analysis and design criteria for a tailings structure. The dam itself is usually homogeneous, with the exception of possibly blanket and toe drains. Vertical drains are generally not installed and therefore control over the phreatic surface is not as effective as in a conventional structure.

Another major distinction lies in the fact that construction of a tailings dam takes place over a much longer period of time. The construction is performed at a rate that suits the requirements of the mining operation. Also, construction is not as well controlled and the construction material is obtained from the tailings. This material is defined by the milling process, rate of feed, grade of ore and separation technique. The construction material can change considerably over the mine's life.

2-3 Recently Proposed Method of Disposal—Thickened Discharge

The thickened discharge method has been proposed as an alternative to the construction of high, and costly, embankments (Robinski, 1975, Robinski, 1978). This method was becoming operational at 13 mines as of 1978, and implementation was being considered at several others.
Essentially, the method involves the dewatering, or thickening, of the total tailings to a pre-determined water content that will result in a conical deposit with an outer slope of approximately 6%. A typical deposit is depicted in Fig. 7. It is also possible to implement the thickened discharge method at a site that has been using conventional disposal techniques, as shown in Fig. 8. The system can be adapted to almost any terrain.

2-3-1 Advantages of Thickened Discharge

The use of thickened discharge results in considerable capital savings, both initially and over the mine's life. A much smaller impoundment facility is required to store a given volume of waste material. The elimination of large tailings dams eliminates the inherent costs and problems in the design, construction and maintenance of such structures.

The facility, as proposed, eliminates the need for a decant system. A relatively small pond at the toe of the tailings deposit is required. This pond is designed to receive natural and tailings runoff. It has been found that only a very small amount of fine tailings ever reach the pond area. The pond is situated in the topographical low by design, and all runoff reaches it without constructing special drainage systems.

Because the tailing is deposited continuously over the entire deposit, the surface is constantly wetted. This eliminates wind erosion and environmental problems associated with dusting.
THICKENED DISCHARGE METHOD
(Robinski, 1978)

FIGURE 7
Existing Pond
-90 foot dam

Thickened Discharge System
-additional 20 year capacity
-6% tailings slopes
-Fixed discharge points

Conventional Expansion
-additional 20 year capacity
-requires 225 foot dam

EXPANSION OF STORAGE FACILITY
(Robinski, 1978)

FIGURE 8
Although the thickened material is more difficult to pump, costs for energy used for pumping is reduced from that of a conventional system because of the reduced volume to be pumped. It has also been found that smaller pipelines are required for both the tailings and the return line for recycled water.

A major advantage of this system is the inherent properties of the deposit that facilitate abandonment. Appropriate chemical neutralizers and fertilizers can be added to the waste material to provide a uniformly treated surface layer. The surface slope also provides good surface drainage.

Robinski also states that the stability, both static and under earthquake conditions, is superior to that of conventional systems. This statement is questionable and is discussed in the next section.

2-3-2 Limitations of Thickened Discharge

The deposit formed by the thickened discharge method is considered to be statically safe. One must be skeptical, however, of the reasoning behind assessing the deposits as safe under earthquake conditions. Robinski (1978) states that the slope is "determined by its angle of internal friction (viscosity)." Consolidation will then result in a two fold increase in viscosity, which provides a large reserve angle of friction to oppose movement during earthquakes.

Although an increase in the angle of internal friction will result from consolidation, the increase may not be sufficient to
resist flow upon reliquefaction. The above reasoning does not account for the fact that an earthquake will likely result in a fluid mass of much greater depth than present during deposition. The shear stresses at the base of the fluid increase with fluid depth, and if the increased shear stresses exceed the increased angle of internal friction angle, flow will result.

Tailings are generally a fine, cohesionless material in a loose, saturated state and are considered to be highly susceptible to liquefaction, as discussed in Section 3-2-4. Robinski (1978) states that the low profile of the hill ensures stability and reduces the dangers of liquefaction. Although the static shear stresses will likely increase the deposit's resistance to liquefaction (Vaid and Finn, 1979), Chern, 1981), considering the nature of the material it is likely that liquefaction would occur during a significant seismic event. It would be prudent to assume that the deposit could liquefy to a considerable depth. Due to the low profile of the hill, shear stress reversal would be likely, and large deformations could result if the gradient were large enough to cause shear stresses greater than the shear strength of the liquefied material.

A detailed discussion of the conditions for flow failure is presented in Chapter 5. However at this point, it should be mentioned that the geometry of the deposit as determined by the depositional behavior of the material is not necessarily a stable geometry for a considerable depth of liquefied material that could result from an earthquake. The shear stresses at the
base of the liquefied material would likely be much greater than those present during deposition. If they are greater than the shear strength of the liquefied mass, flow would result.

In summary, existing methods of tailings disposal are subject to limitations. The downstream method of construction is very expensive, however the resultant stability of the deposit is maximized. The stability of the upstream method has been questioned as embankment failures, and resulting flow failures, have resulted in catastrophic damage. The thickened discharge method, although very cost effective, may also be of questionable seismic stability, and the possibility of significant damage does exist. The model testing program discussed in Chapter 7 was designed to assess the response of statically stable sloped material under cyclic loading conditions. Test results, presented in Chapter 8, indicate that final slope angles are one percent or less, and therefore concern with the seismic stability of thickened discharge deposits may be warranted.
CHAPTER 3

PRELIMINARY TESTING

Preliminary testing was performed to define the model test material, and provide a basis for comparison with typical tailings material.

3-1 Test Material

The material used in this study was a uniform fine-grained Ottawa silica sand. This type of sand is a subrounded, quartz sand with a specific gravity, $G_s = 2.67$.

Several standard tests were performed according to procedures outlined by Lambe (1951). A standard sieve analysis was performed and the resultant grain size distribution curve is shown in Fig. 9. The soil has a coefficient of uniformity, $Cu = \frac{D_{60}}{D_{10}} = 1.8$.

Variable head permeability tests were performed at a variety of void ratios to produce the $e$ vs. log($k$) plot shown in Fig. 10. For the hydraulically deposited material used in the model test, void ratios ranging from .75 to .78 were obtained, corresponding to a range in permeability from .0105 to .0145 cm/s. Successive tests were performed at each void ratio to ensure data reliability. The permeability compares well with that predicted by Hazen's empirical formula, $K = 100 \times D_{10}^2$.

Tests were run for the determination of minimum and maximum void ratios. The maximum void ratio was obtained using the standard dry method and was compared to the relatively quick
GRAIN SIZE DISTRIBUTION

FIGURE 9
Figure 10: Void Ratio \( \varepsilon \) vs. Log(K)

Coefficient of Hydraulic Conductivity \( K \) (cm/s)
and simple methods proposed by Burmister (1970) and Yemington (1970). Agreeable results were obtained, yielding $e_{\text{max}} = 0.86$. A minimum void ratio, $e_{\text{min}} = 0.56$, was obtained. The relative density, $Dr$, for this sand is therefore found by:

$$Dr = (0.86-e)/0.3.$$  

Cyclic triaxial tests were performed using the equipment and procedures described in detail by Chern (1981). The system is shown schematically in Fig. 11. A liquefaction resistance curve was established for isotropic consolidation conditions and is shown in Fig. 12. Relative densities for each test are shown on the plot. Load, pore pressure and strains were measured simultaneously on a strip chart recorder. A typical record is shown in Fig. 13. The range of void ratios used was representative of model test conditions. In all tests, care was taken to ensure sample saturation ($B = 1.00$) and tests were performed undrained at 1 Hz. Note that very large strains resulted after initial liquefaction. Liquefied samples exhibited unlimited strain potential associated with a contractive flow structure. This is reasonable for the low relative densities used ($Dr = 30\%$).

A series of standard triaxial tests were performed to investigate the shear strength characteristics of the material. Both drained and undrained tests were performed on loose samples. After the application of Bishop's energy corrections to the drained test results, to correct to constant volume conditions, the results yield $\phi' = 30.5^\circ$. 
CYCLIC TRIAXIAL TEST APPARATUS (After Chern, 1981)

FIGURE 11
LIQUEFACTION RESISTANCE CURVE

FIGURE 12
TYPICAL CYCLIC TRIAXIAL TEST RECORD

FIGURE 13
In both cyclic and static triaxial testings, all samples were consolidated to an isotropic effective stress of .5 kg/cm^2. Ideally, testing should have been performed at lower stresses, corresponding to those found in the model test, however experience has shown that lower confining pressures may not yield reliable results (Vaid, 1981). The results of the tests at the confining pressure used are felt to be reasonably representative of the material response under model test conditions.

The coefficient of consolidation was estimated from cyclic triaxial test initial consolidation data. The values obtained showed considerable scatter, but were reasonably close to those estimated using results from Yoshimi (1975), who studied the compressibility of a very similar sand at very low confining pressures. The average \( \text{C}_v \) value, \( \text{C}_v = 3 \text{ ft}^2/\text{s} \), also agreed fairly well with typical values from Lambe and Whitman (1969) and Byrne (1980), using test sand permeability, and is felt to be appropriate for test conditions.

3-2 Comparison of Test Sand with Typical Tailings Material

The material used in the testing program was chosen by its grain size to simulate tailings material as closely as possible. The main constraint on the material beyond resembling tailings was that it be practical from a testing point of view. The sand was deposited hydraulically through water, and was chosen to do so in a reasonable length of time, and be reproducible over a number of tests. A uniform fine sand was chosen to satisfy testing requirements. It is recognized that tailings material is generally a more well graded material with a high
degree of silt size particles but the deposition of such a material in the laboratory was deemed impractical.

The characteristics of tailings materials are highly variable, depending on the milling process, host rock and type of ore being mined. Due to the gravitational separation inherent in many disposal schemes, the material characteristics can be highly variable within a given deposit. The thickened discharge method, however, results in a relatively uniform deposit as there is little separation during deposition (Robinski, 1978). The model deposit was also uniform. The following information regarding various tailings is for the total tailings produced by the milling operation. Corresponding properties of the test sand are included for comparison.

3-2-1 Grain Size Distribution

Typical grain size distributions are shown in Figures 14 and 15. Most mining operations result in a silt to fine sand tailing. For coal tailings, coarse particles in the coarse sand to gravel sizes are produced by crushing operation, while clay size to fine sand result from the flotation process.

It is often considered that the susceptibility to liquefaction is dependent on the grain size distribution. Typical bounds are shown in Fig. 15(d) (Ishihara, 1980). In the case of some tailings, many cohesionless clay size particles are present. If one were to use Fig. 15(d) as a strict guide, error could result in the assessment of the deposit. The cohesionless fines are in a loose, saturated state highly susceptible to
TAILINGS GRAIN SIZE DISTRIBUTIONS

FIGURE 15
liquefaction. If the fines are moderately plastic, they tend to be less susceptible to liquefaction.

The sand used in the model study has a D$_{50}$ that is fairly representative for most of the tailings considered. It is considerably more uniform than most typical tailings.

3-2-2 Permeability and Compressibility

The permeability of several tailings is shown in Fig. 16. Note that they fall reasonably close to the line determined by Hazen's empirical formula, $K = 100 D_{10}^2$, for granular soils. The test sand is also shown plotted and is comparable to some copper tailings, but for the most part has a permeability much higher than most total tailings.

This high permeability, coupled with a relatively low compressibility, results in a $C_v$ value considerably higher than most tailings material. As mentioned in Section 3-1, $C_v$ was estimated to be $3 \text{ ft}^2/\text{s}$ for the test sand under test conditions. Typical values of $C_v$ for tailings are in the order of $0.0001 \text{ ft}^2/\text{s}$. The relatively high $C_v$ value of the test sand is indicative of a rapidly draining material. This implies that most cohesionless tailings would be more easily liquefied during cyclic loading in which drainage is allowed at its boundaries, and would remain liquefied for a longer period of time. The $C_v$ value has been found to be the most important parameter in delaying pore pressure response leading to liquefaction (Seed et al., 1976).
VARIATION OF PERMEABILITY WITH $D_{10}$

**FIGURE 16**
3-2-3 Relative Density

A relative density of approximately 30% was obtained for the model tests. A relative density of 25-45% is representative of most tailings deposits (Finn & Byrne, 1976). Void ratio, rather than relative density, may serve as a more useful parameter to describe the denseness or looseness of a tailings deposit because of the wide range in grain sizes, from clay to coarse sand in a typical tailings deposit (Ishihara, 1980). Typical values for a deposit formed by the thickened discharge method (Robinski, 1978) are compared to that of the test sand in Table 1. Note the similarity in solids fraction, Cs, between the test sand and a typical thickened discharge deposit. This parameter exhibits considerable control over the post-liquefaction behavior of a material, as discussed in Section 4.2. The test sand should therefore be reasonably representative of post-liquefaction behaviour.

TABLE 1

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Test Sand Deposit</th>
<th>Thickened Discharge Deposit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity, Gs</td>
<td>2.67</td>
<td>2.9</td>
</tr>
<tr>
<td>Void Ratio, e</td>
<td>.77</td>
<td>.95</td>
</tr>
<tr>
<td>Percent by Weight</td>
<td>77</td>
<td>75</td>
</tr>
<tr>
<td>Solids Fraction, Cs</td>
<td>.56</td>
<td>.54</td>
</tr>
<tr>
<td>Water Content, w</td>
<td>.29</td>
<td>.32</td>
</tr>
</tbody>
</table>
Liquefaction Resistance

The interest in the liquefaction resistance of tailings materials has been largely confined to the material used in embankment construction. This is generally the coarse fraction of the tailings and behaves much like a natural sand of similar gradation (Byrne, 1980).

Of particular interest in the present study is the cyclic response of tailings material prior to separation of the coarse and fine components. Unfortunately, no published data exists. The response of fine grained tailings is assumed to be more representative of the total tailings than the response of the coarse grained discard.

It is generally assumed that the fine material stored behind an embankment is very susceptible to liquefaction (Byrne, 1980, Klohn, 1979, Ishihara, 1980, Wahler and Schlick, 1976, Guerra, 1972, Taylor and Morrell, 1979). Its loose, saturated, cohesionless, underconsolidated state is the basis of this conclusion, however, very little published data is available to confirm this notion.

Taylor and Morrell (1979) performed cyclic triaxial tests on fine coal-mine discard. These tailings had similar grain-size characteristics to most typical total tailings from other mining operations. Some of the tailings exhibited plasticity, and their resistance to liquefaction was found to be dependent on the plasticity index (Fig. 17). The pore pressure buildup in plastic specimens was much more gradual than in non-plastic
EFFECT OF PLASTICITY ON LIQUEFACTION RESISTANCE
(Taylor & Morrel, 1979)

FIGURE 17
samples. Tests were carried out both on remolded specimens and reasonably undisturbed ones. Those non-plastic specimens prepared in the laboratory were found to be more susceptible to liquefaction than undisturbed samples. Most of the non-plastic specimens liquefied in less than 20 cycles at a stress ratio, $\frac{\tau}{\sigma_0'} = .15$, which is similar to the resistance of the test sand.

Taylor, Kennedy and MacMillan (1979) performed a study on the susceptibility of coarse grained colliery discard to liquefaction. The material tested was considerably coarser than typical tailings discussed in Section 3-2-1. The material was generally found to be more resistant to liquefaction, this being attributed, by the authors, to its high shear strength and rapid equalization of pore pressure throughout the sample. Although these findings agree with the concepts of liquefaction and appear conclusive, the disastrous tip failure at Aberfan is believed to have resulted from the liquefaction of similar coarse discard.

Ishihara (1980) performed a series of cyclic triaxial tests on tailings materials. The slimes tested were considerably finer than typical total tailings. The liquefaction resistance curves were similar to that obtained for the test sand, Fig. 18. The effect of void ratio on the liquefaction resistance is shown in Fig. 19. The test sand is shown plotted, and compares fairly well to a variety of tailings. From this plot, one can see that deposits at void ratios similar to that obtained in the thickened discharge method of disposal ($e = .95$)
Cyclic Stress Ratio causing 5% D.A. Strain in 20 Cycles
would be at least as susceptible to liquefaction as the test sand.

Neither of the above studies accounted for the possible increase in resistance to liquefaction due to static shear. Nor did they account for any degree of underconsolidation that exists in the deposit due to relatively rapid rates of deposition. It is believed that this factor could substantially reduce the liquefaction resistance of a non-plastic tailings deposit.

It is felt that the test sand is practical from a model testing standpoint, and at the same time reasonably represents the cyclic response, as well as the post-liquefaction behavior, of tailings material.
CHAPTER 4
MATERIAL BEHAVIOUR

4-1 Liquefaction

Extensive research has been performed in the last 15 years to understand the phenomenon of liquefaction. The state-of-the-art has progressed considerably in that time, and the mechanisms leading to liquefaction are well understood (Seed, 1979). However, varying terminology exists in the literature and ambiguities may arise if the terminology used is inconsistent.

The definition of liquefaction adhered to in this thesis is consistent with that put forward by Youd (1973). "Liquefaction is the transformation of a granular material from a solid state into a liquefied state as a consequence of increased pore pressures." This is equivalent to initial liquefaction as defined by Lee and Seed (1967).

Upon reaching liquefaction, a reasonably dense sand exhibits dilative behaviour, and further straining results in reduced pore pressures and increased strength and resistance to shear deformation. This strain hardening type of material exhibits limited flow potential upon liquefaction. Typical behaviour under monotonic loading is shown in Fig. 20 (Castro, 1969).

Typically, a sufficiently loose material, contractive by nature, will exhibit unlimited flow deformation. In the liquefied condition, dilative tendencies to reduce pore pressures are insufficient and deformation continues until the applied shear
RESPONSE OF MEDIUM DENSE SAND (After Castro)

FIGURE 20
stresses are reduced to a level compatible with the low shear strength of the liquefied material (Youd, 1973). Typical behaviour under monotonic loading is shown in Fig. 21 (Castro, 1969). The response of garnet tailings under monotonic loading is shown in Fig. 22 for comparison (Highter and Tobin, 1980). The response is very similar to that of sand.

The response of saturated cohesionless material to undrained cyclic loading was also studied by Castro (1969). A recent study (Chern, 1981) investigates the effective stress conditions within each cycle, and liquefaction was found to coincide with a critical effective stress ratio, \( \sigma_1' / \sigma_3' \), independent of cyclic stress ratio and anisotropic consolidation ratio. Typical deformation and stress path characteristics for loose, isotropically consolidated material are shown in Fig. 23 (Chern, 1981). Note that large flow deformations occurred in loose samples. Large (> 20%) deformations occurred in cyclic triaxial tests performed in the present study, and any indications of significant dilative behaviour at large strains were not evident. Unlimited flow at low relative densities is consistent with other work (Castro, 1969, De Alba et al., 1976).

Although there are many variables affecting the potential for liquefaction of a given deposit, the most important are generally regarded to be the relative density, or void ratio, angularity of the particles, the compressibility and drainage characteristics, the initial stress state of the material and the nature and duration of the stresses to which it is subjected.
RESPONSE OF LOOSE SAND (After Castro)

FIGURE 21
**FIGURE 22**

RESPONSE OF GARNET TAILINGS (AFTER HIGHTER AND TOBIN, 1980)

![Graph showing deviator stress and induced pore pressure over axial strain. The graph includes labels and a peak at 98.1 kN/m².](image-url)
RESPONSE OF LOOSE SAND TO CYCLIC LOADING

(AFTER CHERN, 1981)

FIGURE 23A
Effective Stress Path of Cyclic Loading Test on Isotropically Consolidated Loose Sand.

RESPONSE OF LOOSE SAND TO CYCLIC LOADING (After Chern, 1981)

FIGURE 23 B
Of particular interest in this study is the initial stress state of the material. The effect of static shear on liquefaction resistance has been found to be dependent on the relative density, level of initial shear stresses and the degree of shear stress reversal (Vaid & Finn, 1979, Chern, 1981). Loose samples were found to be both more and less resistant to liquefaction depending on the magnitude of the static shear stress ratio. Chern (1981) found that high static shear stress levels resulted in a less resistant material, and postulated that the reason lies in the fact that the initial state of stress lies closer to the critical effective stress ratio line at higher initial static stress levels. Vaid and Finn (1979) performed a similar study on a uniform Ottawa sand of different gradation than the present test sand. Results from their study are shown in Fig. 24. Correction factors reflecting the effect of static shear on liquefaction resistance were obtained from Fig. 24 for slopes used in the model study and are tabulated in Table II. The liquefaction resistance curves adjusted for static shear are shown in Fig. 25. The predicted increase in resistance with increasing slope is consistent with results obtained in the model test, as discussed in Chapter 8, however these results must be considered qualitative as the relative densities used by Vaid and Finn (50%) were considerably higher than those used in the model test. The results of Vaid and Finn were used for lack of more appropriate available data. The corrected curves are used in an analysis in Appendix 1.
CYCLIC SHEAR STRESS REQUIRED TO DEVELOP 10% SHEAR STRAIN

a) 10 Cycles (Loose Ottawa Sand)
b) 30 Cycles (Loose Ottawa Sand)

**FIGURE 24**

<table>
<thead>
<tr>
<th>Slope Angle</th>
<th>$\tau_s/\sigma_{vo}'$ (10 cycles)</th>
<th>$F$ (10 cycles)</th>
<th>$F$ (30 cycles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2°</td>
<td>0.07</td>
<td>1.1</td>
<td>1.1</td>
</tr>
<tr>
<td>4°</td>
<td>0.14</td>
<td>1.4</td>
<td>1.4</td>
</tr>
<tr>
<td>6°</td>
<td>0.21</td>
<td>1.7</td>
<td>2.0</td>
</tr>
<tr>
<td>8°</td>
<td>0.28</td>
<td>2.0</td>
<td>2.3</td>
</tr>
</tbody>
</table>

**TABLE II**

CORRECTION FACTORS FOR INITIAL SLOPE
LIQUEFACTION RESISTANCE CURVES
CORRECTED FOR STATIC SHEAR

FIGURE 25
Initial static shear results in a terminal value of residual pore pressure less than the initial effective confining pressure. The maximum residual pore pressure is independent of the relative density and cyclic stress ratio, and can be shown theoretically to vary linearly with static shear stress level, as shown in Fig. 26 (Chern, 1981). Chern's data is shown to compare favorably with that theoretically predicted. The affect of a residual pore pressure ratio less than unity on the post-liquefaction flow characteristics has not been quantified in the present study, other than to state that the material will exhibit enchanced viscosity and yield shear strength in the liquefied state over that present for residual pore pressure ratios of unity.

The material used in this study was a uniform, fine sand. Tailings material is generally more well-graded with the presence of cohesionless fines. Yoshimi (1977) reports conflicting results regarding the affect of these fines on the liquefaction resistance of the material. Field studies indicate that the resistance is decreased, while laboratory investigations indicate the opposite. Yoshimi feels that the discrepency is a function of the depositional environment. It is also likely that the effect of fines in the field is to suppress drainage significantly. In the laboratory, triaxial testing was conducted undrained and the suppression of drainage is not a factor. Because the fine material is cohesionless it is considered that it will have a detrimental effect on the liquefaction resistance of a deposit where drainage is allowed.
TERMİNAL RESİDUAL PORE WATER PRESSURE (After Chern,1981)

FIGURE 26
The presence of plastic fines affects the liquefaction resistance, and the nature of the affect is dependent upon the plasticity index. If the plasticity index is high the resistance to liquefaction increases significantly. The pore pressure increase is more gradual for the highly plastic slimes. This is illustrated by Ishihara (1980), who tested a variety of tailings material. Note that at high void ratios, or low relative densities, sand and low plasticity slimes have similar cyclic strengths, while the highly plastic slimes are significantly stronger. The post-liquefaction behaviour of these materials is assumed to be the same. For a liquefied mass, "there is not tangible evidence suggesting that the non-plastic materials should move faster as grain flows than the plastic fine discards" (Taylor and Morrell, 1979).

4-2 Post Liquefaction

Due to the loose, saturated state of most tailings deposits, they are susceptible to liquefaction. Many deposits have liquefied, several of which are discussed in Chapter 11. In all cases where failures occurred, the tailings were observed to behave as a viscous fluid. It is therefore considered appropriate to discuss the post-liquefaction behavior in terms of fluid behavior. To facilitate the discussion, it will be made within the framework of a rheological model generally accepted as describing the behaviour of liquefied tailings.
Liquefied tailings behave in a non-Newtonian fashion. There are a large number of models available to describe non-Newtonian fluid behaviour. The most common models used to describe liquefied tailings are the Bingham plastic model and the power law fluid model (Wasp et al., 1977). Of these, the Bingham model is the simplest and has been used in many previous studies. It is believed that the Bingham model accurately describes the flow characteristics of homogenous tailings flow (Jeyapalan, 1980, Wasp et al., 1977) and natural mudflows (Enos, 1977). If the solids concentration is fairly constant throughout the body of the fluid, it can be considered homogenous (Wasp et al., 1977). The liquefied test sand is considered as a homogenous slurry, and can therefore be described by the Bingham model.

The Bingham model is shown schematically in Fig. 27 and a typical flow curve is shown in Fig. 28. The two parameter model can be expressed as:

\[ \tau = \tau_y + \eta_p \dot{\varepsilon} \]  
for \( \tau > \tau_y \)

and \( \dot{\varepsilon} = 0 \)  
for \( \tau < \tau_y \)

where \( \tau_y \) represents the threshold shear stress required to initiate movement and \( \eta_p \) is the plastic viscosity, or coefficient of rigidity, which is fully analogous to the Newtonian viscosity above the threshold shear stress.

The model can be extended to three parameters to include the effect of water content, as shown in Fig. 29. This could
BINGHAM RHEOLOGICAL MODEL

FIGURE 27

BINGHAM MODEL FLOW CURVE

FIGURE 28

Shear stress $\tau$

Bingham fluid

Newtonian fluid

Strain rate, $\dot{\varepsilon}$
A GENERAL PORE PRESSURE (WATER CONTENT) DEPENDENT BINGHAM PLASTIC MODEL

FIGURE 29 (After Jeyapalan, 1980)
be used to account for consolidation effects. Also, the quantitative impact of the maximum pore pressure ratio being less than unity during cyclic loading, for anisotropically consolidated material, is difficult to ascertain and is beyond the scope of the present work, other than to say that for pore pressures less than the initial confining pressure the material would have higher shear strength and viscosity than if the pore pressure equalled the confining pressure.

4-2-1 **Viscosity**

The viscosity is defined as the ratio of the shear stress to the rate of shear strain produced by a given shear stress. The viscosity depends on the nature and extent of mechanical interaction between the particles. The principal factors that govern the viscosity of a slurry are the particle size, distribution and shape, and the solids concentration. The solids concentration, used by Robinski (1978) to describe the fluid behaviour, determines the degree of mechanical interaction. Fig. 30 shows data for a variety of materials, indicating the dependence of viscosity upon the water content, or solids concentration. Using a water content, \( w = 0.66 \) at the time of disposal, and \( w = 0.32 \) in the deposit (from Robinski (1978), one could expect an increase in the viscosity of roughly 10 times due to consolidation. This neglects excess pore pressure due to overburden.

Chong (1971) performed a study on silicon beads, and showed that the relative viscosity, \( \eta_r = \eta / \eta_0 \), is independent of particle size, shape and distribution. The results are shown in
VARIATION OF PLASTIC VISCOSITY WITH WATER CONTENT

FIGURE 30 (After Jeyapalan)
Fig. 31. Note that the absolute viscosity is determined by $\eta_0$, the viscosity for no particle interaction, and $\phi_\infty$, the asymptotic solids concentration, which is dependent on the nature of the material. In Fig. 32, results from Robinski (1978) are provided for comparison. Note the similarity in shape of results presented in Figures 31 and 32, indicating the control of solids concentration, via the viscosity, on the slope angle. This control is dependent on the depositional characteristics discussed earlier.

Although cohesive clay particles are generally thought to increase viscosity, Taylor and Morrell (1979) caution that it is the effective stress condition that controls the viscosity and the effect of plasticity may not be significant. Chong's (1971) results indicate that non-plastic fines may result in a decrease in viscosity.

Extensive research of flow slides, and subsequent back-calculations allowed the determination of a range of viscosities for liquefied tailings (Jeyapalan, 1980), as shown in Fig. 33. The viscosity of the test sand, as determined from Fig. 30, and that corresponding to a typical thickened discharge deposit ($w = 29\%$ and $32\%$ respectively) is approximately $5 \times 10^4$ cps, which is slightly lower than the lower bound suggested in Fig. 33. The value used in the analysis described in Chapter 6 was chosen to be $1 \times 10^5$ cps for the liquefied test sand, and provided reasonable predictions of fluid behaviour. This value provided the best correlation between predicted and observed model test deformations.
DEPENDENCE OF VISCOSITY ON SOLIDS CONCENTRATION

FIGURE 31 (After Chong, 1971)

Empirical Equation

\[
\frac{\eta}{\eta_0} = \left[1 + 0.75 \frac{\phi}{\phi_\infty} \right]^2
\]

Relative viscosity \( \eta/\eta_0 \)

Relative solids volume \( \%, \phi/\phi_\infty \)

Dependence of Laboratory Slope on Solids Concentration

FIGURE 32 (After Robinski)

Mixture behavior (increasing segregation)

Slurry behavior (no segregation)

Slope of Deposit - %

Percent Solids by weight

45 50 55 60 65 70
Probable Range for Liquefied Tailings

Viscosity (cps)

$1 \text{cps} = 2 \times 10^{-5} \text{ lb-sec/ft}^2$

$1 \text{cps} = 0.001 \text{ Pascal-sec}$

- Dow-corning grease
- Aberfan tailings
- Viscosity used in analysis
- Natural mudflow
- Tailings sand
- Tailings deposit

As per Fig. 30

Wet cement mortar

Machine oil

Water @ 20°C

VISCOSITY SPECTRUM (After Jeypalan)

FIGURE 33
4-2-2 Yield Shear Strength

The yield shear strength, $\tau_y$, is the shear stress required to initiate movement of a Bingham fluid. As a material approaches liquefaction, the yield strength decreases considerably, and after liquefaction $\tau_y$ remains a constant finite value. $\tau_y$ increases with solids concentration as a result of increased particle interaction, as can be seen in Fig. 34 (Govier and Aziz, 1972). As a result, consolidation after deposition can be expected to increase $\tau_y$. However, little quantitative data of this nature exists.

During flow $\tau_y$ is constant. Upon cessation of flow, a theoretical Bingham fluid's yield structure reforms (Wasp et al., 1977). Note that without a finite yield strength, flow would continue indefinitely.

The yield stress during liquefaction is a difficult parameter to assess. Jeyapalan (1980) estimates the range shown in Fig. 35, based on Castro's (1969) data and the behaviour of soft marine sediments. Ponce and Bell (1971) performed triaxial tests at very low confining pressures, over a wide range of relative densities (5-95%) and obtained values for the apparent cohesion of 0.14 to 0.22 psi. This corresponds to the lower end of data presented by Jeyapalan and is shown in Fig. 34. Extrapolating the results of Fig. 34 to solids concentrations of interest in this work ($C_s = 0.55$) results in a yield strength of approximately 10 psf, which is slightly lower than the lower bound suggested by Jeyapalan. The material used to produce Fig.
a) Rheogram for Water Suspension of Finely Divided Galena (D₅₀=50 microns)
(After Govier & Aziz, 1972)
Probable Range for Liquefied Tailings

- 8000 psf: Hard clay
- 4000 psf: Very stiff clay
- 2000 psf: Stiff clay
- 1000 psf: Medium clay
- 500 psf: Soft clay
- 250 psf: Very soft clay
- 150 psf
- 20 psf: Range from Ponce & Bell, 1971
- 10 psf: Extrapolation of Figure 34

YIELD SHEAR STRENGTH SPECTRUM

FIGURE 35
34 was similar in grain size to typical tailings, however extrapolation involved only two known points, and may not provide reliable results at high solids concentrations.

The Bingham rheological model reasonably describes the flow of liquefied, loose, saturated material. It is useful in the discussion of the depositional behaviour of tailings, as compared to the post-liquefaction behaviour of the resulting statically stable deposit, as is discussed in the following chapter.
CHAPTER 5

DEPOSITIONAL BEHAVIOUR OF TAILINGS COMPARED TO
THE RESPONSE OF A LIQUEFIED DEPOSIT

In order to assess the stability of a liquefied, sloped deposit, one must first analyze the depositional characteristics of the fluid. It is believed by the present author that the slope angle, and length of slope, of a thickened discharge deposit is determined by the rheological properties of the fluid as well as by the pattern of flow during deposition, whereas Robinski (1978) discusses only the fluid properties, neglecting the depositional pattern inherent in the disposal method.

As discussed in Section 4-2, the flow of tailings can be adequately described using the Bingham fluid rheological model. It is a two parameter model, the flow being described by a threshold yield shear stress, $\tau_y$, and a plastic viscosity, $\eta_p$. These properties and the fluid characteristics on which they depend are discussed in Sections 4-2-1 and 4-2-2. An applied shear stress must be greater than $\tau_y$ for flow to occur. The flow is then described by the plastic viscosity.

When the tailings are deposited, they flow down the slope, and the flow is unsteady, as it eventually comes to rest. The driving forces are the initial velocity and gravity. The fluid has a boundary layer, which is a critical aspect of the depositional behaviour. The boundary layer is the region next to the base of the flow in which the fluid has had its velocity diminished because of shearing resistance created at the boundary due.
to a no-slip condition. The boundary layer thickness is in the order of one foot for a typical liquefied tailings, as determined using formulae presented by Schlicting, 1955.

As the material flows from the point of discharge, it then spreads out due to lack of lateral confinement. For conservation of mass, the fluid layer must therefore become thinner, and the boundary layer effect becomes increasingly dominant, in accordance with boundary layer theory for laminar flow (Schlicting, 1955, Roberson and Crowe, 1975). Most tailings and debris flows are characterized by laminar flow (Enos, 1977, Jeyapalan, 1980).

The shear stress within the boundary layer varies from a maximum at the base of the fluid to zero at the surface. The maximum shear stress is proportional to the thickness of the fluid, and therefore decreases from the point of deposition. This is one of the mechanisms leading to cessation, or "freezing", of the flow.

The viscosity manifests itself in the development of the boundary layer. The boundary layer dissipates the fluid momentum and hence retards the flow considerably. The amount of momentum decay is a function of the viscosity. Momentum decay translates to a reduction in velocity, with which the shear stress at the base also varies. Therefore the shear stress decreases in the downstream direction due to the viscous nature of the material. Both the effect of viscosity and the fluid spreading result in decreased shear stresses downstream. When
the shear stress at the base equals the threshold shear stress, motion ceases. Also, any pore pressure dissipation during flow would result in a higher yield stress and viscosity and flow would cease sooner. If the fluid did not exhibit a yield stress, flow would continue indefinitely.

If a greater depth, or volume, of material is deposited, it would flow further for a given slope, as the driving shear forces due to gravity would be increased. The pipe exit velocity, if increased, would also result in higher base shear stresses. A given fluid will come to rest at a variety of slopes, with the initial depth, velocity and slope determining the extent of flow, while the final depth of the material depends on the slope.

Having established the mechanisms that result in the flow "freezing", one must now account for changes that occur after deposition. Robinski states that typically 2 to 4 feet of material is deposited uniformly per year. When the fluid ceases motion, it is assumed to have very high excess pore pressures. These pore pressures will dissipate, with the material consolidating, resulting in a higher solids content.

Upon reliquefaction the material would exhibit an increased viscosity over that present during deposition. The yield strength upon liquefaction may increase, however no quantitative study has been performed to determine the extent of change. Robinski (1978) feels that the enhanced fluid properties result in a reserve angle of friction available to oppose
movement during an earthquake. Although the fluid itself has improved, the shear stress conditions of the base of the fluid have changed drastically, having a detrimental effect on the stability of the deposit.

In the event of a significant seismic event, it is considered that the deposit is likely to liquefy to a considerable depth. Although the rheological properties are indicative of a fluid more able to resist a given shear stress than the fluid present at the time of freezing, the boundary shear stress conditions of the fluid are much different than at the time of freezing. The depth of the fluid, and hence the shear stresses at the base of the fluid will have increased dramatically and will likely exceed the yield shear strength of the liquefied material. The boundary layer is only a fraction of the fluid depth, and hence exhibits much less control over the flow of the entire fluid layer. The fluid outside of the boundary layer behaves like an inviscid fluid. Flow is likely to result in the down slope direction and the distance of flow is a function of depth of liquefaction. It is suspected that any downstream embankment would be overtopped under these conditions, likely resulting in failure. The results of the model study, discussed in Chapter 8, indicate that the viscous fluid resulting from liquefaction acts as a standing wave, overtopping the downstream barrier consistently and resulting in a final slope of approximately one percent.

The depositional flow structure and boundary shear stresses cannot be considered to govern the flow of a sloped deposit
liquefied to any significant depth. It is doubtful that any enhanced fluid properties resulting from consolidation after deposition would be able to resist the much larger shear stresses that would exist at the base of the liquefied layer. For this reason the stability of a deposit formed by the thickened discharge method is questionable during a significant seismic event.
CHAPTER 6
REVIEW OF PREVIOUS MODEL STUDIES

Model studies have been performed by many authors on the cyclic response of soil deposits and earth embankments (Yoshimi, 1977; Prakash, 1977). The majority of these were designed to study the structural response of embankments and retaining walls for purposes of design (Prakash, 1977). Much of the model testing performed has been done with either dry sand, at extremely high accelerations or with imposed boundary conditions that render the results inapplicable to this study.

Other studies observed the pore pressure response within a soil deposit subjected to cyclic loading (Finn et al., 1970, Ishihara, 1967, De Alba et al., 1976, Kawakami, 1966, Goto, 1966, Tanimoto, 1967). These studies provide both qualitative and quantitative descriptions of the response leading the liquefaction. Similitude requirements for a shaking table model study are outlined by Ishihara. He concluded that strict similitude requirements cannot, and need not, be satisfied as partial similitude should be sufficient. Yoshimi (1977) suggests that model studies be used to verify analytical tools, which in turn can be used to predict field behaviour, thereby enabling one to neglect similitude requirements.

Results of studies using pore pressure monitoring at various depths indicate that liquefaction occurs simultaneously over the depth of the deposit (Finn et al., 1970, De Alba et al., 1976, Yoshimi, 1967). No such monitoring was used in the
present study and it is assumed that these results are applicable. This assumption was confirmed by visual observation.

The previous studies were also used to select test parameters in the present study. Accelerations, frequency and soil depths were selected with due regard to previous studies. Finn et al., showed that for a sample of roughly the same depth there existed a frequency at which the acceleration distribution was not uniform over the depth of the sample. For frequencies greater than 20 Hz, this behaviour became more pronounced. There also existed a natural frequency at which acceleration response was greatly enhanced. The implications of this are discussed further in Appendix 1. Most previous studies were performed at similar accelerations, if slightly higher, than the present program. Threshold accelerations were not being sought in previous studies.

The pore pressure response of liquefiable shaking table specimens has been found to be very similar to that in cyclic triaxial and simple shear tests (Finn, 1970, Yoshimi, 1967), Fig. 36. The pore pressure typically rose to approximately 60% of the effective confining pressure, then exhibited a rapid pore pressure increase leading to liquefaction. These studies were designed to observe the response within a horizontal deposit. Anisotropic pore pressure response within sloping deposits has not been studied, but is assumed to be similar to that found in cyclic triaxial or simple shear tests conducted on specimens exhibiting a static bias.
Zircon Sand

$P = 150 \text{ cm H}_2\text{O}$

$a = 440 \text{ cm/sec}^2$

OBSERVED PORE PRESSURES FOR A SHAKING TABLE TEST

(After Yoshimi, 1967)

<table>
<thead>
<tr>
<th>Transducer</th>
<th>Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC1</td>
<td>40 cm</td>
</tr>
<tr>
<td>PE2</td>
<td>140 cm</td>
</tr>
<tr>
<td>PC3</td>
<td>240 cm</td>
</tr>
</tbody>
</table>
No tests have been performed to describe the post-liquefaction behaviour in a sense that would be applicable to the present model study. Most of the work on slurry flow has been with regard to assessing pipe flow and are therefore not considered here. Jeyapalan (1980) performed an inundation study of liquefied tailings using oil as the viscous material. This work produced results that support the concept of "freezing" flow discussed in Chapter 5. Robinski (1978) performed laboratory studies to determine the residual angle of tailings at various solids concentration, but provided no information on the test equipment or depositional features that would be of interest here. No model study has been performed to observe threshold accelerations and subsequent behaviour of a liquefied deposit.
CHAPTER 7
TESTING PROGRAM

The testing program comprised 18 model tests and was designed to determine the response of sloped, cohesionless, saturated deposits subjected to horizontal cyclic accelerations. Horizontal accelerations simulate vertically propagating horizontal shear waves produced during an earthquake. This assumption is common in earthquake related analyses.

The sloped deposit was used to simulate a sloped tailings deposit as obtained using the thickened discharge method of disposal. Fine sand, simulating tailings, was deposited hydraulically into a plexiglass container to obtain a loose, uniform deposit. The relative density was essentially the same for all tests, and equal to 30% ± 5%. Model dimensions are given in Figure 37. Steady state seepage conditions were induced by a constant head upstream boundary condition. The initial pore pressures were governed by the boundary conditions shown in Figure 38, and a flow net showing equipotential values is shown in Fig. 39.

Slopes of 2°, 4° and 8° were subjected to base accelerations ranging from .03 g to .10 g. Initially, varying downstream boundary heights were used to produce the desired slope, and in the latter portion of the program a constant downstream boundary height was used, while varying the upstream boundary height. A number of tests were performed using a sloped base to simulate a sloping valley floor. All tests were performed at a
BOUNDARY CONDITIONS FOR STEADY STATE SEEPAGE

FIGURE 37

MODEL TEST DEPOSIT DIMENSIONS

FIGURE 37

BOUNDARY CONDITIONS FOR STEADY STATE SEEPAGE

FIGURE 38
Seepage Analysis
Slope=6°
frequency of 5 Hz, with steady state seepage and similar relative densities. A discussion of the selection of test parameters can be found in Appendix 1.

7-1 Model Test Equipment

An MTS Earthquake Simulator console was used to provide the base accelerations. The system is a closed-loop servo-controlled system. This type of system involves the generation of the desired motion and monitoring of the response of the table. Any difference between the input and the output is relayed as a corresponding voltage to the servo valve, which adjusts the hydraulic flow accordingly. Monitoring at the table is by way of an LVDT located at the piston that moves the table. A schematic of the loop is shown in Fig. 40.

The command signal was chosen to be a sine wave, provided by an Exact 340 function generator. This function generator was found to have less distortion at the peak of the sine wave than other function generators tested. This reduced a spike in the acceleration response of the table at its maximum displacement.

The spike is an inherent feature of the hydraulic system. Pressure accumulators did not eliminate the occurrence of the spike. This undesirable feature has been obtained in similar test equipment (Ramsay, 1981). Although a spike was unavoidable, it could be reduced substantially by operating the hydraulics at low pressure (150-250 psi). This adjustment did not affect the frequency response of the system.
Function Generator

Servo Controller

Oil Supply

150–250 psi

Servo Valve

Hydraulic Piston

LVDT

Table

Scope

Acceleration Record

Displacement Feedback

SERVO LOOP

SCHEMATIC OF TABLE LOOP

FIGURE 40
Several frequencies were used in preliminary tests. It was found that a frequency of 5 Hz led to reasonable results, imparting virtually undrained conditions, and provided reproducible response over the range of accelerations used. Typical table response is shown in Fig. 41.

The table itself is 4 feet by 9 feet and it weighs approximately 1000 lbs. It is made of cast aluminum with bracing to provide the necessary rigidity. It moves in one horizontal direction and is supported by two v-slotted needle bearings and two flat bearings. In this way proper alignment is maintained without obtaining unevenly distributed loads on the bearings due to bending or temperature changes. The table response was monitored at its surface by a Kistler Model 305 accelerometer. The accelerations were recorded by a Visigraph cathode ray recorder.

The model container was constructed using 1/2 inch plexiglass. The container is shown in Fig. 42. The box was designed to provide a constant head boundary condition at the upstream end and a variable downstream boundary height to simulate a dam at the toe of the deposit. It is assumed in this study that the dam provides a relatively rigid boundary in the field. The upstream constant head boundary condition was maintained by a drainage line fixed at the height required for a given slope and downstream boundary height. The water was separated from the deposit by a baffle and a pervious plastic wall. This eliminated erosion due to splashing during shaking. The base of the
Inlet

Overflow

Height to Bottom of Inlet = 16.3 cm
Height to Overflow = 16.6 cm

PLAN VIEW

Variable Height Of Bottom Weir

20.4 cm (top)
20.35 cm (bottom)
20.1 cm (top)
20.35 cm (bottom)
20.3 cm (top)
20.4 cm (bottom)

81.0 cm

Hopper

Slope Adjustment

Scraper Guide

Pervious Plastic

Outlet

Inlet

MODEL CONTAINER

SIDE VIEW

FIGURE 42
box was sanded to provide frictional resistance. The container was designed so that it could be tilted to simulate a sloped base boundary condition.

A hopper was designed to deposit the test sand evenly over the width of the box. Smooth runners, a smooth plexiglass surface and lubricants were used to minimize disturbance during deposition. The hopper outlet could be closed during filling and upon opening had a reasonably even flow over its width. The outlet space was 1/8" and the volume of each pass resulted in a sand layer approximately 1 cm thick.

When the desired slope was approximated, a scraper was used to provide an even slope. The scraper was cut to just over half the width of the box to prevent scour at its base and edges during scraping. The scraper was braced for stiffness and was adjustable to different slopes. The runners were smooth to reduce disturbance during scraping.

Particle movements were monitored with silica beads placed in the sand against the plexiglass wall. Particle movements were recorded on plastic sheets attached to the plexiglass. It was determined that side effects were minimal by observing transverse lines on the surface of the deposit. The glass beads were placed in layers on a grid pattern. The placement of layers, and subsequent scraping, were observed to have no noticeable effect on layers that had been previously placed.
7-2 Test Procedures

The model container was cleaned prior to each test. It was then blocked in place to prevent movement relative to the table surface. The downstream boundary was placed, using silicone grease to seal its edges. A grid was placed on the outside to facilitate placement of the silica beds and to record their displacements. All electronic equipment was carefully calibrated prior to the first test.

The box was then filled with de-aired water with inlets and outlets closed. Sand was then deposited in 1 cm lifts through approximately 6" of water. The height of water was maintained by draining water intermittently as necessary. Silica beads were placed according to a pre-determined grid pattern. The oven dried sand was weighed prior to placement for the void ratio determination. When the slope was approximated, the scraper was used to remove the excess sand, which was dried and weighed. The downstream outlet was then opened to drain off excess water. With the upstream overflow and inlet opened, steady stage seepage was established. No piping as observed at the downstream boundary, as predicted in preliminary calculations.

The hydraulics were then switched on, the inlet and outlets closed, and the test would be run for 20 cycles using a stop watch. Table accelerations, particle displacements, final slope configuration and overtopping volume were recorded.
CHAPTER 8
TEST RESULTS

The testing program was designed to study the response of various test model initial surface slopes to a range of base accelerations. The primary purpose was to determine the final slope to which a liquefied deposit would come to rest. Of secondary interest was the determination of a critical acceleration required to induce liquefaction throughout a given test model configuration. The effect of a sloping base on test results was also studied. Several tests were duplicated to establish reproducibility of results.

A total of 18 shaking table tests were performed. Figure 43 provides a breakdown of the tests in terms of degree of liquefaction throughout the sample, base slope and acquisition of particle displacements. Table III provides a summary of individual test details.

Figures 44 through 49 depict the particle displacements that resulted in tests for which a significant portion of the deposit was assumed to have liquefied. Shown in each of these figures are the initial and final surface configuration, as well as the displacement of vertical lines over the length of the sample. Note that in the center of the sample, where the end boundary effects are minimized, initially vertical lines assume a parabolic shape, as might be expected from a viscous fluid.

The final slopes obtained are very shallow, all being less than 1%. There appears to be a trend, as shown in Figure 50,
Tests Exhibited

Complete Liquefaction
(used in particle displacement prediction comparisons and final slope determinations)

- 6-Obtained Particle Displacements
- 2-Did Not Obtain Particle Displacements

- 8-Horizontal Base

1-Sloped Base-Obtained Particle Displacements

- 8-Horizontal Base

9 Tests Exhibited
Partial Liquefaction
(used in shakedown and for establishing threshold accelerations)

- 9 Tests Exhibited
- 1-Sloped Base-Obtained Particle Displacements
- 1-Sloped Base

TOTAL OF 18 TESTS

BREAKDOWN OF TESTING PROGRAM

FIGURE 43
<table>
<thead>
<tr>
<th>TEST</th>
<th>INITIAL SLOPE</th>
<th>AMAX</th>
<th>RELATIVE DENSITY</th>
<th>CYCLES</th>
<th>FREQUENCY</th>
<th>LIQUEFACTION</th>
<th>FINAL SLOPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.2°</td>
<td>.05g</td>
<td>28%</td>
<td>28</td>
<td>5Hz</td>
<td>X</td>
<td>--</td>
</tr>
<tr>
<td>2</td>
<td>4°</td>
<td>.05g</td>
<td>29%</td>
<td>20</td>
<td>5Hz</td>
<td>X</td>
<td>--</td>
</tr>
<tr>
<td>3</td>
<td>8°</td>
<td>.05g</td>
<td>31%</td>
<td>18</td>
<td>5Hz</td>
<td>X</td>
<td>--</td>
</tr>
<tr>
<td>4</td>
<td>8°</td>
<td>.05g</td>
<td>27%</td>
<td>21</td>
<td>5Hz</td>
<td>X</td>
<td>--</td>
</tr>
<tr>
<td>5</td>
<td>10°</td>
<td>.05g</td>
<td>35%</td>
<td>22</td>
<td>5Hz</td>
<td>X</td>
<td>--</td>
</tr>
<tr>
<td>6</td>
<td>8°</td>
<td>.05g</td>
<td>25%</td>
<td>33</td>
<td>20Hz</td>
<td>X</td>
<td>--</td>
</tr>
<tr>
<td>7</td>
<td>4.8°</td>
<td>.05g</td>
<td>26%</td>
<td>20</td>
<td>5Hz</td>
<td>X</td>
<td>.5%</td>
</tr>
<tr>
<td>8</td>
<td>8°</td>
<td>.05g</td>
<td>38%</td>
<td>22</td>
<td>5Hz</td>
<td>X</td>
<td>--</td>
</tr>
<tr>
<td>9</td>
<td>8°</td>
<td>.025g</td>
<td>33%</td>
<td>20</td>
<td>5Hz</td>
<td>--</td>
<td>--</td>
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<td>10</td>
<td>4°</td>
<td>.05g</td>
<td>29%</td>
<td>21</td>
<td>5Hz</td>
<td>X</td>
<td>Sloped Base</td>
</tr>
<tr>
<td>11</td>
<td>8°</td>
<td>.05g</td>
<td>29%</td>
<td>20</td>
<td>5Hz</td>
<td>X</td>
<td>Sloped Base</td>
</tr>
<tr>
<td>12</td>
<td>8°</td>
<td>.10g</td>
<td>30%</td>
<td>20</td>
<td>5Hz</td>
<td>X</td>
<td>.7%</td>
</tr>
<tr>
<td>13</td>
<td>8°</td>
<td>.08g</td>
<td>37%</td>
<td>20</td>
<td>5Hz</td>
<td>X</td>
<td>.6%</td>
</tr>
<tr>
<td>14</td>
<td>8°</td>
<td>.06g</td>
<td>31%</td>
<td>21</td>
<td>5Hz</td>
<td>X</td>
<td>3.0%</td>
</tr>
<tr>
<td>15</td>
<td>4°</td>
<td>.04g</td>
<td>31%</td>
<td>21</td>
<td>5Hz</td>
<td>X</td>
<td>--</td>
</tr>
<tr>
<td>16</td>
<td>4°</td>
<td>.045g</td>
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<td>5Hz</td>
<td>X</td>
<td>.5%</td>
</tr>
<tr>
<td>17</td>
<td>2°</td>
<td>.04g</td>
<td>28</td>
<td>20</td>
<td>5Hz</td>
<td>X</td>
<td>.3%</td>
</tr>
<tr>
<td>18</td>
<td>2°</td>
<td>.03g</td>
<td>35</td>
<td>20</td>
<td>5Hz</td>
<td>X</td>
<td>--</td>
</tr>
</tbody>
</table>

**SUMMARY OF TESTING PROGRAM**

**TABLE III**
RESULTS OF TEST 7

FIGURE 44

SCALE: 1 inch = 10 cm
RESULTS OF TEST 12

FIGURE 45

SCALE: 1 inch
RESULTS OF TEST 13

FIGURE 46

SCALE: 1 inch = 10 cm
RESULTS OF TEST 14

FIGURE 47
RESULTS OF TEST 16

FIGURE 48

SCALE: 1 inch = 10 cm
RESULTS OF TEST 17

FIGURE 49

SCALE: 1 inch = 10 cm
whereby increasing the initial surface slope results in an increased final slope of the liquefied material. This could be the result of lower induced maximum pore pressures due to higher initial static shear induced in the steeper samples, as discussed in Section 4-1. The lower maximum pore pressures would result in a higher viscosity and higher threshold shear stress, as discussed in Section 4-2. The net result would be increasing final slope angle for increasing initial surface slopes. Another contributing factor could be the affect of dilation, and a resulting increase in shear strength of the material. Increasing the surface slope implies that higher shear strains were imposed, and the effect of dilation increases with increasing strain. Although cyclic triaxial tests indicate no significant dilation up to 10% axial strain, it is felt that a small degree of dilation could account for some of the increase in final slope obtained. It should be noted that in all model tests, movement ceased prior to eliminating the base accelerations.

Several samples at a given initial surface slope angle were subjected to a range of base accelerations to determine the relationship between the base acceleration and the final slope angle. It was determined that a critical acceleration level existed, above which the liquefied deposit assumed a constant final slope angle. Figure 51 shows the effect of increasing acceleration on final slope angle. The critical acceleration level was obtained for 2°, 4° and 8° and Figure 52 depicts the results. Increasing the initial surface slope angle results in
EFFECT OF INITIAL SLOPE ON FINAL SLOPE

FIGURE 50
ACCELERATION LEVEL VERSUS FINAL SLOPE ANGLE

FIGURE 51
THRESHOLD ACCELERATION VERSUS INITIAL SLOPE ANGLE

FIGURE 52
an increased critical acceleration level. This is consistent with the effect of static shear stress on liquefaction resistance as discussed in Section 4-1.

The effect of a sloping base (simulating a sloping valley floor) was studied to determine the applicability of the horizontal base test results for sloped base conditions. Figure 53 shows the results for Test 10, in which a sloped base was imposed. Also shown for comparison are the deformations obtained for test 16, in which a horizontal base was used. Both tests, had identical initial surface slopes (4°) and similar base accelerations (.05g and 0.45g, respectively). It was determined from Test 4 (horizontal base) and Test 11 (sloped base), that a sloping base had little effect on the threshold acceleration and final slope angle for a given initial surface slope.

In general, test observations, both visual and measured, indicated that the liquefied deposits were behaving similar to what might be expected from a highly viscous fluid. For this reason, a viscous fluid model was used to predict the particle displacements observed. This model is discussed in the following chapter.
LEGEND

--- Horizontal Base
- - - Sloped Base

EFFECT OF SLOPED BASE ON MODEL RESPONSE
CHAPTER 9

FLUID ANALYSIS

An analysis was performed assuming liquefied test sand acts as a viscous fluid, consistent with test observations. For liquefied test deposits that experienced a marked reduction in the surface slope, it is assumed that the static shear stress at the base due to the slope exceeded the threshold shear stress required to cause flow. This has been the assumed mechanism in flow failures in nature (Castro, 1969; Casagrande, 1971). Upon exceeding this shear stress, the material is assumed to behave as a viscous Newtonian fluid until such time as the driving shear stress equals the threshold shear stress, at which point the fluid ceases motion, as depicted in Figure 28. Over the period of motion it is assumed that a unique plastic viscosity governs the deformation of the material.

The material appeared to liquefy simultaneously throughout. It then appeared to behave as a viscous fluid, and rather than the surface material flowing downstream, it exhibited motion similar to a standing wave. A preliminary analysis treated the problem as a standing wave in water. Surface displacements were close to those obtained experimentally but horizontal displacements near the base were grossly over-predicted, due to the neglect of viscosity in water wave theory.

Introduction of viscosity imposes a no-slip base boundary condition. Lambe (1945) provides a qualitative description of the motion of a vertical line due to a standing wave in a
viscous fluid. Motions of the boundary layer are very similar to those obtained in the model test. No rigorous, closed form solution could be found for a standing wave in a highly viscous fluid.

Liu and Davis (1977) study a standing wave in water and account for the viscosity of the water. Their model essentially involves two boundary layers capable of transferring shear, bounding an inviscid interior fluid. The lower and upper boundary layers are required to satisfy the constraints of no-slip at the base and zero shear at the free surface.

It is assumed that this theoretical model of the boundary layer could be representative of the liquefied test model. The thickness of the lower boundary layer was determined to be very close to the thickness of the test deposit. The theory calls for the use of a transformed, or "stretched", coordinate system in the boundary layer. Vertical displacements at the edge of the boundary layer, in the interior fluid, are assumed to be small. The "stretched" normal coordinate is required to obtain correct displacements relative to the interior fluid. Having no interior fluid, the transformation became unnecessary. All predictions were made using "unstretched" coordinates and the accuracy of the predictions empirically substantiate neglecting the transformation. The primary objective of this analysis was to show that the model test material could be described as a viscous fluid, not to propose that this model be used to describe field behavior.
The theory, as proposed by Liu and Davis, enforces the no-slip condition of the base through the insertion of a boundary layer of thickness:

\[ \delta = (2\gamma/w)^{1/2} \]

where \( \gamma = \eta/\sigma \) = kinematic viscosity

\( \sigma = 1950 \text{ g/cm}^3 \)

\( w = k \cdot \omega = \text{angular frequency} \)

\( k = 2\pi/\text{wave length} = \text{wave number} \)

\( \omega = \sqrt{g \tan h(kd)/k} \)

Using the length of the model container as 1/2 the wave length and \( \gamma = .1 \text{ m}^2/\text{s} \) (\( \eta = 2 \times 10^5 \text{ cps} \)), one obtains a boundary layer thickness of 20 cm, which is slightly greater than the mean depth of the model deposit. The boundary thickness is shown in Fig. 54 as a function of viscosity over the range of viscosities for liquefied tailings determined in Section 3-2-1.

In solving the boundary value problem of the bottom boundary layer, using unstretched coordinates, one obtains dimensionless velocities as:

\[ u = \frac{-A(\xi)}{\sin h_B} \sin(t) - e^{-\psi} \sin(t-\psi) \]

and \( w = \frac{A(\xi)\cos(x)}{\sin h_B} \left[ \psi \sin(t) + \frac{e^{-\psi}}{2} \sin(t-\psi) - \right. \]

\[ \left. \frac{e^{-\psi}}{2} \cos(t-\psi) + 1/2 \cos(t) - 1/2 \sin(t) \right] \]
EFFECT OF VISCOSITY ON BOUNDARY LAYER THICKNESS.

FIGURE 54
where \( A(t) \) accounts for the viscous decay of the wave amplitude, \( A \), with slow time, \( \tau = kC_0t \)

\[
B = k^2d^2
\]

\[
J = kz + \sqrt{B}
\]

Integrating the velocities with respect to time, one obtains particle displacements, expressed dimensionally as:

\[
\Delta x = C_0A_0 \frac{\sin(kx)}{\sin \sqrt{J}} e^{at} \left( -\frac{b^2}{a^2+b^2} \right) \left[ -\frac{1}{b} \cos(bt) + \frac{a}{b^2} \sin(bt) \right]
\]

\[
- e^{-J} \left( -\frac{1}{b} \cos(bt-J) + \frac{a}{b^2} \sin(bt-J) \right)
\]

\[
\Delta y = C_0A_0 \frac{\cos(kx)}{\sin \sqrt{J}} e^{at} \left( -\frac{b^2}{a^2+b^2} \right) \left[ J \left( -\frac{1}{b} \cos(bt) + \frac{a}{b^2} \sin(bt) \right) \right]
\]

\[
+ \frac{e^{-J}}{2} \left( -\frac{1}{b} \cos(bt-J) + \frac{a}{b^2} \sin(bt-J) \right) + \frac{e^{-J}}{2}
\]

\[
\left( \frac{1}{b} \sin(bt-J) + \frac{a}{b^2} \cos(bt-J) + \frac{1}{b} \frac{1}{2} \frac{1}{b} \sin(bt) \right)
\]

\[
+ \frac{a}{b^2} \cos(bt) \right) - \frac{1}{b^2} \left( -\frac{1}{b} \cos(bt) + \frac{a}{b^2} \sin(bt) \right)
\]

where \( a = -2k^2 \sqrt{d} \)

\( b = kC_0 \)
For prediction of particle displacements in the model test, it is necessary to provide corrections to the theoretical displacements. The first factor accounts for settlement due to cyclic loading. It was assumed that the settlement at any point is related to the settlement at the node in the centre of the deposit. This could be determined simply by observing the post-testing position of the surface point. The surface settlement at any other point was assumed to be directly proportional to the original depth at that point. On a vertical line, the settlement was assumed linear from zero to the base to the surface. Surface settlements at the node were found to be 1-2%, which agrees well with the 1-3% estimated by Yoshimi et al. (1975).

Another correction was imposed to account for the necessary assumption of a sinusoidal surface. The correction factor was determined at the surface and decreased linearly with depth. Due to the shallow slopes involved in the study, the assumed sinusoidal surface was reasonable and the correction factors relatively small.

The initial control particle positions were in vertical lines. The coordinates used in the above analysis are the mean particle positions. The initial vertical lines correspond to the maximum particle displacements. It was therefore necessary to assume a mean particle position and solve for the particle displacements iteratively. Particle displacements were solved for incrementally over one-quarter of a cycle of wave motion.
The analysis does not account for the frictional aspect of the Bingham fluid. The actual material ceased motion slightly before 1/4 of a cycle had been completed. It was at this point that driving shear stresses equalled the threshold shear stress of the material. The particle displacements used in the predictions in Chapter 10 were those corresponding to the time at which the corrected vertical displacement at the upstream antinode equalled the final displacement in the model test.

The viscosity used in the analysis was selected as being representative for liquefied tailings. This analysis is not intended to provide quantitative results in terms of viscosity, but is designed to investigate the observed phenomenon. The observed characteristic response of the liquefied model deposit was found to be in agreement with the selected viscous fluid model.

The successful application of the analytical model to predict model test behaviour does not necessarily imply that extrapolation can be made to field behaviour. Liquefied field deposits would likely be of much greater depth and more complex geometry than present in the model test.
The fluid analysis presented in Chapter 9 can be used to predict particle displacements obtained in the model tests. Predictions are made only for tests in which the entire deposit was observed to liquefy. Due to the complex nature of the boundary value problem involving a sloping base, predictions are made only for deposits with horizontal bases.

The particle displacements for initial surface slopes of 4° and 8° were obtained using a single value of kinematic viscosity. This value, $\nu = 0.1 \text{ m}^2/\text{s}$, is within the range of probable viscosities for liquefied tailings, as was pointed out in Figure 47. Accurate predictions are not meant to imply that the viscosity can be quantified. It is the nature of the fluid movement that is the interest, not absolute quantities. Treating the liquefied test material as a viscous fluid appears to give reasonable results for a first approximation. Although only one viscosity was employed, viscosities of one test relative to another can be qualitatively discussed with regard to pore pressure effects and solids concentration.

Predicted particle displacements near the downstream boundary are not made due to distorted deformations created by overtopping of the boundary. A finite boundary height was not considered in the analysis.

Predicted displacements for an initial slope of 8° are shown in Figure 55. Actual displacements obtained in Test #13,
Legends:
- Actual Displacements
- Predicted Displacements

Comparison of predicted and observed slope movements for 8° slope

Figure 55
where $a_{\text{max}} = 0.08g > (a_{\text{max}})_{\text{crit}}$, are also shown plotted for comparison. Excellent agreement was obtained. The particle motion appears to be predicted well using the viscous fluid standing wave model.

Predicted displacements for an initial slope of $4^\circ$ are shown in Figure 56. Actual displacements obtained in Test #7, where $a_{\text{max}} = 0.05g > (a_{\text{max}})_{\text{crit}}$, are also shown plotted for comparison. Predictions compare very well with actual particle motions.

The fact that the viscous fluid model accurately predicts the particle motions lends credibility to the treatment of the liquefied material as a viscous fluid. The analysis used employed a boundary layer, essential in the analyses of viscous fluids. Whether the flow is in the realm of a standing wave, as is the case of a liquefied sloped model test deposit, or in the case of unrestrained flow as occurs during deposition in the thickened discharge method, the effect of the boundary layer must be considered. This point was discussed in Chapter 5.

In summary, the predicted displacements, using a viscous fluid stand wave model, accurately describe the shape of actual displaced vertical lines as obtained in the model tests. The test results imply that a flow failure in the field could result from liquefaction of a sloped non-plastic tailings deposit.
Comparison of Predicted and Observed Slope Movements for 4° Slope

Figure 56
In the event of failure of a tailings facility, large quantities of material can flow downstream, resulting in an extensive inundation zone. Several catastrophic failures have considerable attention in the literature. The most prominent of these are the Aberfan disaster (Jeyapalan, 1980), the Buffalo Creek failure (Wahler and Schlick, 1976), and the El Cobre dam failure (Dobry and Alvarez, 1967). These failures all involved the liquefaction and flow of mine tailings. A detailed discussion of these failures is not considered necessary here, and it suffices to report that hundreds of lives were lost and hundreds of millions of dollars damage resulted.

Jeyapalan (1980) used a viscous fluid model to accurately predict the flow characteristics and extent of flow involved in these, and other, failures. The consideration of a boundary layer was fundamental in determining the extent of flow. It was determined that a given fluid could come to rest on a variety of slopes, the flow distance being longer for steeper slopes.

Of particular interest in this study are flow failures occurring at mild slopes. Casagrande (1971), Youd (1973) and Castro (1969) provide limited descriptions of such failures. A failure discussed by Williams (1978) is the most applicable case history to the present study. The tailings disposal scheme consisted of depositing coarse tailings at slopes of 3 to 4 degrees, similar to slopes used in the present study. The
material has similar grain size characteristics to that of the test sand, as shown in Figure 57. The tailings sand is slightly coarser and less uniform. The deposit configuration is shown in Figure 58. Note the similarity to the test sand deposit also shown in Figure 58 (as discussed in Chapter 8, model test results for sloped base deposits exhibited no noticeable difference in either final slope angle or threshold acceleration).

The tailings deposit had exhibited previous instability due to excess pore pressures generated during construction, and remedial measures were employed to obtain a statically stable deposit. The apparently stable deposit then experienced liquefaction due to an unknown source, as evidenced by sand boils and the extent of failure, and mass movement occurred downslope. The material, although statically stable at 3 to 4 degrees, flowed and produced upthrusting at the toe of the slope. The model test deposit exhibited very similar behaviour, and would therefore appear to be reasonably representative of field behaviour for this particular case history.
TAILINGS FLOW FAILURE CASE HISTORY GRAIN SIZE DISTRIBUTION

FIGURE 57
TAILINGS FLOW FAILURE CASE HISTORY DEPOSIT CONFIGURATION

FIGURE 58
A shaking table model study has been performed to investigate the stability of sloped, cohesionless, saturated deposits during and after cyclic-loading. Of particular interest is the stability of sloped tailings deposits created using the thickened discharge disposal method. In addition to model testing, a literature review of conventional disposal techniques, typical tailings material properties and the behaviour of viscous fluids was performed. Based on the results obtained in this study, the following conclusions are made:

1. Conventional tailings disposal techniques are subject to limitations such that the development of alternative methods of disposal is warranted. The upstream construction method does not, in general, meet stability requirements, while the downstream construction method results in extremely high costs for design, construction and abandonment.

2. The thickened discharge method of disposal has proven to be an economic alternative of questionable stability.

3. The model test sand used has a liquefaction resistance curve similar to that of typical cohesionless tailings material.

4. The post-liquefaction response of the model test sand is reasonably representative of cohesionless tailings material under model test conditions.
5. When subjected to horizontal base accelerations sufficient to cause complete liquefaction of the model test deposit a significant reduction in the deposit slope was obtained. The final slope was observed to be approximately one percent. All model test deposits were formed at a Relative Density of approximately 30 per cent, which translates to a solids fraction similar to that obtained in a typical thickened discharge deposit.

6. A critical acceleration was observed for each initial slope, above which complete liquefaction was obtained. The critical acceleration was found to increase with initial slope angle, and ranged from approximately .035g, at 2 degrees, to .060g, at 8 degrees.

7. The model test results cannot be reliably extrapolated to the field until further studies are made to quantitatively assess model geometry effects.

8. Model test deformations can be empirically predicted using a viscous fluid model that accounts for boundary layer effects. Based on the success of the viscous model to predict model test results, combined with a review of pertinent literature, it is suggested that liquefied tailings can be viewed as a viscous fluid.

9. A sloped deposit liquefied to a considerable depth is not necessarily stable just because it is statically stable prior to liquefaction. Post-liquefaction stability is
dependent on the depth of liquefaction as well as the properties of the liquefied material.
REFERENCES


APPENDIX 1

Parameter Selection
The test parameters considered are the slope, downstream boundary height, acceleration and frequency. In selecting these parameters use was made of similitude requirements, previous model studies, available analytical tools and preliminary tests in the present study.

As mentioned previously, similitude requirements need not strictly be met, however they can be used as a useful guideline in parameter selection. Similitude requires that both geometric and dynamic similarity be satisfied (Ishihara, 1967).

For complete geometric similitude, both deposit dimensions and surface displacements must have similarity, Fig. A1(a). However, Ishihara shows that very large accelerations are required to satisfy complete geometric similitude. It is therefore practical to satisfy only incomplete similitude. Where \( \lambda \) denotes the scale ratio; \( \lambda = \frac{l_m}{l_p} = \frac{h_m}{h_p} \) must be satisfied for incomplete geometric similitude, \( l \) being length and \( h \) being height. Incomplete similitude, for a model to field acceleration ratio of 1.0, was satisfied for this study. The model test therefore does not strictly represent the prototype behaviour of the deposit prior to liquefaction as model surface displacements required for complete similitude are extremely high.

Dynamic similitude must satisfy elastic and inertial forces in this analysis.
El Centro

$A_p = 15 \text{ cm}$

$T_p = 4.0 \text{ sec}$

$\omega_p = 2g$

a) Deposit dimensions

SIMILITUDE DATA

FIGURE A 1
\[ \lambda_i = \lambda^3 \frac{Y_m}{Y_P} \frac{a_m}{a_p} \]

\[ \lambda_e = \lambda^2 \frac{\varepsilon_m}{\varepsilon_P} \frac{G_m}{G_P} \]

Similitude requires that \( \lambda_i = \lambda_e \), therefore

\[ \frac{a_m}{a_p} = \frac{1}{\lambda} \frac{G_m}{G_P} \frac{Y_P}{Y_m} \frac{\varepsilon_m}{\varepsilon_P} \]

Satisfying incomplete geometric similitude, and assuming;

\[ \varepsilon \sim A/H, \quad G_m = G_P \quad \text{and} \quad Y_m = Y_P \quad \text{implies} \]

\[ \frac{a_m}{a_p} = \frac{1}{\lambda^2} \frac{A_m}{A_P} \]

\[ T_m = \lambda T_P \quad (T = \text{period}) \]

Ishihara provides prototype data to use in a similitude study, Fig. A1(b). This data implies incomplete similitude has been satisfied for \( \lambda = 1/25 \) in this study. For \( \frac{a_m}{a_p} = 1 \), one obtains \( \frac{A_m}{A_P} = 1/750 \). This surface deflection corresponds to \( \varepsilon_m = .1\% \), which is the approximate level of strain prior to liquefaction in the cyclic triaxial tests. The above analysis implies that the parameters used in the model test appear reasonable, and the predicted strains of .1% implied by incomplete similitude is reasonable for a typical earthquake record.

For the geometric configuration of the model test the probability of piping at the downstream boundary was assessed. The program "SEEPAGE", developed at UBC using finite element techniques, was used to obtain a flow net, providing a maximum hydraulic gradient of \( i = .11 \), well below the value of 1.0.
required for piping. No piping occurred during any of the tests.

Early tests were performed with a constant upstream soil depth and variable downstream depths to obtain the desired slope angle. It was found that there existed, for a given acceleration and slope, a critical depth at which complete liquefaction did not occur. This phenomenon was investigated using "STAB.W" (Siddharthan, 1981). This program calculates the incremental pore pressure increase due to cyclic shearing and allows for dissipation of incremental excess pore pressures, while changing $M_v$ to account for the reduced effective stress. Note that this analysis is necessary to analyze the drained condition of the model test, whereas a typical tailings deposit could be treated as undrained without serious error.

Typical initial test results are shown in Fig. A2. The abrupt change in slope was a consistent feature in tests where only a portion of the soil mass liquefied. It is assumed that soil upstream from the change had totally liquefied, and large particle displacements were recorded. Relatively small particle displacements occurred in the nonliquefied material.

In order to perform the analysis, the liquefaction resistance curve, adjusted for static shear, was used to determine the numbers of cycles to liquefaction under a $\tau/\sigma' = .18$, corresponding to $a_{\text{max}} = .05$ g. The isotropic pore pressure generation curve shown in Fig. A3(a) was used, and triaxial test data implied using a value of $\Theta = .7$. A back analysis of test
Initial Slope=8°

Final Configuration

$H_{critical}$

TYPICAL PARTIAL LIQUEFACTION RESULTS,

FIGURE A2
DC

Pore Pressure Generation Curves

Cycle Ratio $N/N_L$.

- $D_r$
  - $31.5$
  - $34.8$
  - $28.5$

a) Theoretical and Experimental Pore Pressure Generation Curves

b) Analytical Pore Pressure Generation

PORE PRESSURE GENERATION.

FIGURE A3
results provided a compressibility slightly lower than that determined in Section 5-2-2 ($C_v = .4 \text{ ft}^2/\text{s}$). This could be expected using the isotropic pore pressure generation curve, as the pore pressure generation is more rapid than if anisotropy were accounted for.

It should be noted that the results of the analysis confirm qualitatively the observations made during testing. Typical maximum pore pressure ratios obtained for varying depths are shown in Fig. A3(b). This curve implies that there is indeed a depth of soil at which the response of the deposit changes dramatically. This depth corresponds to the abrupt change in the slope obtained in model tests.

Although the results of the model test can be explained by the analysis, parameters, such as compressibility, used may not be correct. The pore pressure generation curve applies to isotropically consolidated samples, while anisotropic conditions exist in the model. This has the affect of "stretching" the pore pressure generation curve, but would not eliminate the existence of a critical height at which liquefaction would occur, it would merely increase $m_v$ required to provide a given critical height.

The use of consistent input parameters ($\Theta = .7, \; m_v = .4 \text{ psf}^{-1}$) successfully predicted the appropriate critical height for various slopes, which have different liquefaction potential curves. The critical heights determined experimentally, coupled with analytical results, were used to establish a downstream
boundary height for which the entire soil mass would liquefy under the excitations used in the latter portion of the testing program. The selected height proved to be satisfactory for this purpose.

Another analysis was performed to assess the choice of the depth of the deposit. This elastic analysis involved estimating the frequencies of the first 5 modes of vibration. The analysis requires the selection of a shear modulus and the empirical relation proposed by Seed and Idriss (1970) was used, where $G_{\text{max}} = 1000(K_2)_{\text{max}}(\gamma' )^{1/2}$ psf. Appropriate values, corresponding to test conditions and various depths were used to obtain $G_{\text{max}}$. The value of $G_{\text{max}} = 1 \times 10^5$ psf is supported by the equation proposed by Richart (1977). It was established that the table did not impart accelerations near the natural frequency of the deposit. Confidence in the procedure was established by predicting the natural frequency obtained experimentally by Finn et al. (1969) to within 10%.

The accelerations used (0.03 g - 0.10 g) are felt to be reasonably representative of field accelerations that might be expected. The sinusoidal frequency of 5 Hz was chosen for aforementioned practical considerations and is felt to be representative of earthquake motion. De Alba et al. (1976) consider a frequency of 4 Hz to be a representative frequency.

All tests were run for the arbitrary number of 20 cycles, which is considered to be a realistic number of uniform cycles for a seismic event. Seed et al. (1976) suggest that 20 uniform
cycles are appropriate for an earthquake of Richter magnitude of 7-1/2.

In the determination of the appropriate design earthquake for a given site the site geology, areal attenuation characteristics, distance from causitive fault and extent and depth of rupture should be estimated for a realistic evaluation of the soil response to be determined. The accelerations, frequencies and durations of the model test discussed herein are not representative of any particular seismic event, rather they are considered to be reasonably representative of possible conditions and were chosen to produce liquefaction of the model deposit in order to observe the post-liquefaction behaviour of the material.