11 May 1984, 8:00 am - 10:30 am

The Brenda Mines' Cycloned- Sand Tailings Dam

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SYNOPSIS For the benefit of those readers not fully conversant with mine tailings and the methods commonly used to store them, the paper presents a brief overview covering the composition of tailings and the procedures commonly used to design and construct the required tailings dams. The remainder of the paper presents a detailed description of the design, construction, and performance of the Brenda Cycloned-Sand Tailings Dam. This includes a summary and assessment of the large volume of data available from extensive field, laboratory, and office studies that have been ongoing since the start of construction in 1968. The most recent of these studies (1980-83) involved "state-of-the-art" seismic analyses to determine if the original dam could be raised to an ultimate height of 530 ft and remain stable under the maximum credible earthquake for the site.

INTRODUCTION

Tailings

Tailings are a waste product of the mining industry. They consist of the ground-up rock that remains after the mineral values have been removed from the ore. The grain size distribution of tailings depends upon the characteristics of the ore and the mill processes used to concentrate and extract the metal values. The wide range of gradation curves that may be encountered is illustrated in Figure 1.

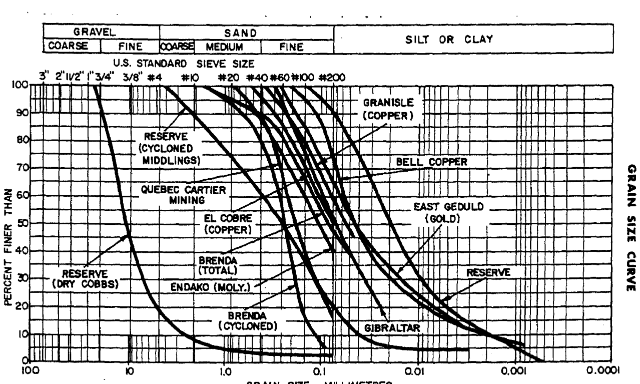

Figure 1

Typical Grain Size Curves for Tailings

Tailings normally are transported to the disposal area hydraulically at concentrations ranging between 30% and 50% by weight of solids to liquid. The potential pollution hazards associated with storage of the tailings slurry vary with different mining operations, and range from very severe for the radioactive wastes associated with uranium mining, to none for mining processes that merely grind up an inert ore without the addition of toxic chemicals during processing. In between these two extremes are a wide range of conditions that present either short or long term potential pollution problems.

The recent trend towards mining very low-grade ores has required the development of large-scale mining and milling operations which produce huge quantities of waste tailings. Safely storing these waste products of the mining operation requires the construction of extremely large tailings dams. Currently, dams are under construction which will have ultimate heights in the order of 600 feet and will retain billions of tons of tailings and lesser amounts of fluids. These dams are critically important hydraulic structures whose structural stability must be maintained under all possible design loads, including earthquake forces. Failure of a large tailings dam is completely unacceptable as it would cause the release of very large volumes of water and/or semi-fluid tailings. Such an event would pose a serious threat to life and property downstream of the dam as well as causing extensive pollution. In addition to being structurally sound, these large tailings storage facilities must also be relatively watertight to prevent the escape of polluted effluent into either the groundwater or surface water downstream of the structure.

Tailings Dam Design

In recent years tailings dam design and construction procedures have evolved from a largely "trial and error" approach to the current methods which are based on sound engineering principles. These current methods of tailings dam design are founded mainly on the engineering knowledge and experience available from conventional water storage dam designs, suitably modified to satisfy the special requirements of the mining industry. In this regard, it should be noted that tailings dam design differs from conventional dam design in three important aspects:

(1) The bulk of material stored behind the dam is soft, loose, relatively impervious tailings rather than water. The consistency of these tailings may range between the solid state and the semi-fluid state, depending on their fineness, their age, and the location of the water table. However, under severe seismic shock, all saturated tailings are likely to liquefy, becoming a fluid of high unit weight and exerting an additional thrust which the tailings dam must resist.

(2) A large part of the dam is usually constructed using the coarser sand fraction of the tailings.

(3) Most of the dam construction is carried out by the mining operators, as part of the tailings disposal operation, with the dam being continuously raised as required to stay ahead of the rising tailings pond.

The first factor affects the forces assumed to act on the dam, especially under seismic loading. The second two factors strongly influence the design section finally selected for the dam. Because tailings dams usually are constructed slowly over a period of many years, the designer is able to select a design and then check its performance, making modifications as required throughout the long construction period. This is a critically important aspect of tailings dam design since it allows far more flexibility than is available for design of conventional water retention dams.

Except for the above qualifications, the basic design requirements for tailings dams are very similar to those for water storage dams.

Although tailings are far from being ideal dam-building materials, they are used in most tailings dam designs for the obvious reason that they are the cheapest available construction material. Some of the disadvantages of tailings as a dam-building material are: they are highly susceptible to internal piping; they present highly erodible surfaces; and loose and saturated tailings are subject to liquefaction under earthquake shocks. Obviously, if tailings are to be used as the main dam-building materials, the tailings dam design must take into account these undesirable physical properties. This is usually accomplished by incorporating into the design such considerations as:

- Separation of the tailings into sands and slimes, with only the sands being used for dam building.
- Control of the sand separation procedures to ensure that the sand produced meets specific gradation and permeability requirements.
- Installation of internal filters and drains to prevent piping and to lower the phreatic surface within the sand dam.
- Compaction and/or drainage of the tailings sand to increase resistance to liquefaction under earthquake shocks.
- Protection of highly erodible surfaces with vegetation, coarse gravel, or waste rock. (The tailings sands are subject to both wind and water erosion.)

There are a wide variety of tailings dam designs used throughout the world; however, the three most common types of structures currently used to retain tailings are:

(1) "Upstream Tailings Dams"
(2) "Downstream Tailings Dams"
(3) "Conventional Water Storage Dams"

Many variations and combinations of the above three commonly used types of tailings retention structures are encountered in actual practice. However, for purposes of this brief overview our comments are limited to these three types.

Upstream Tailings Dams - The oldest method of tailings dam design is the "upstream method" of dam building, which is illustrated in Figure 2. This method evolved as the natural development of the original mining procedures for disposing of the tailings as cheaply as possible. There are many variations of this method but they all involved constructing a small starter dam and then depositing the total tailings upstream of the dam. Subsequent raising of the starter dam is done in stages by constructing in the upstream direction, over the top of the previously deposited, loose, saturated tailings.

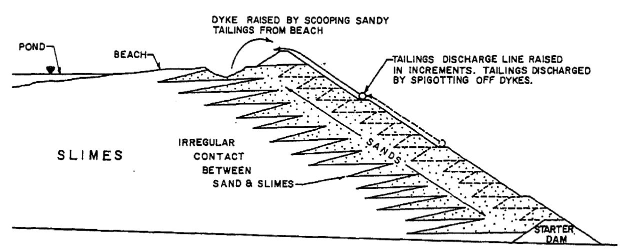

Figure 2
Tailings Dam Construction by Upstream Method

Under static loading conditions there is a limiting height to which such a dam can be safely built. This height depends on the location of the phreatic line, the shear strength parameters of the dam and tailings, and the geometry of the section. Under earthquake loading, this type of dam may be subject to failure by liquefaction at any height and the author considers that the "upstream method" of tailings dam design should not be used for seismically active sites.

Downstream Sand Tailings Dam - The "downstream methods" of sand tailings dam design have evolved as acceptable alternatives to the generally unsatisfactory "upstream methods". The sand required for dam construction is usually produced by cycloning the total waste tailings. The cycloned sand thus produced is a very cheap construction material that utilizes up to 50% of the total tailings as dam building material. A commonly used method of downstream, sand tailings dam construction (centreline method) is illustrated in Figure 3. All of the downstream sand tailings dams have two features in common; the sand dam is raised in a downstream direction and consequently is not underlain by previously deposited tailings; and the fine tailings are spigotted off the upstream face of the dam to provide a low permeability beach between the sand dam and the free water in the tailings pond. The major advantages of this method of construction are: the sand dam is constructed over prepared foundations; placement and compaction control can be exercised as required; underdrainage facilities can be installed as required; and the dam can usually be raised above its originally planned design height with a minimum of problems and design modifications. Consequently, sand tailings dams designed using the "downstream method" can be constructed to whatever degree of competency required, including the ability to safely withstand major earthquakes.

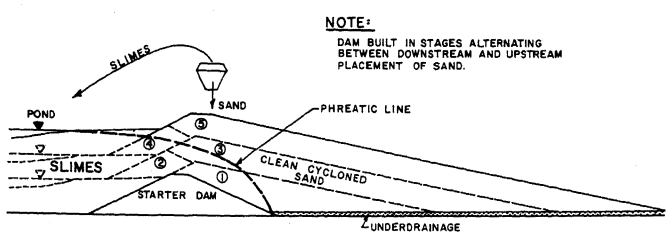

Figure 3
Sand Tailings Dam Constructed by
Downstream (Centreline) Method

Conventional Water Storage Dams - In areas of very high seismicity and/or where large volumes of water must be stored along with the tailings, construction of a conventional water storage type dam to retain the tailings often provides the most satisfactory solution. Where waste rock or overburden materials are available from the open-pit, stripping operations, these are usually used for construction. Figure 4 illustrates a conventional water storage dam that might be constructed for storing tailings.

The advantages of using a conventional water storage dam for the retainment of tailings are obvious. The structure can be designed and constructed to safely resist any desired earthquake event, and, in addition, adds a great deal of flexibility to the operation of the tailings pond. Unfortunately, this method of tailings dam construction is usually the most expensive and for this reason, is the least common type of tailings dam in current use.

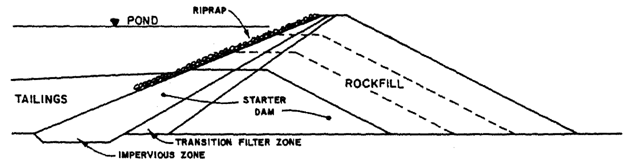

Figure 4
Tailings Dam Constructed as
Conventional Water Storage Dam

The Brenda Mines cycloned sand tailings dam, which is reviewed in detail in the following sections of this paper, was designed using the "centreline method" of downstream construction.

SITE CONDITIONS

Location

Brenda Mines is a low-grade open pit, mining operation that mills approximately 32,000 tons of ore per day to produce copper and molybdenum concentrates. The mine is located approximately 11 miles west of Okanagan Lake in south central British Columbia, Canada. The tailings storage area is located at the headwaters of a small stream that flows eastwards into Okanagan Lake. The stream has a steep gradient, falling from elevation ±4100 ft at the toe of the tailings dam to elevation ±1120 ft at Okanagan Lake. A general location plan is presented in Figure 5.

Figure 5
General Site Location Plan

The Okanagan valley which is an important fruit producing area, is also one of the major tourist and recreational areas in British Columbia. Consequently, the safety and pollution aspects of the tailings storage facilities have always been of prime importance to both the mining company and the Governmental Regulatory Agencies. From the outset, Brenda Mines was required to design the tailings storage facilities as a closed-circuit system with zero discharge downstream of the tailings dam. In view of the critical location of the tailings dam above the inhabited Okanagan Valley, the mining company has fully supported the need to construct an "engineered" tailings dam at this site. The detailed field and laboratory testing programs that have been ongoing at this site over the past 15 years attest to their continued dedication to building a safe tailings dam.

Climate

Climatic conditions at Brenda Mines are fairly typical for this latitude and altitude. Total annual precipitation is in the order of 55 cm. Approximately 60% of this falls as snow during the period November to April. Spring runoff, owing to the snowmelt, is estimated to contribute about 80% of the total runoff. Mean daily temperatures at the site range from -10°C in January to +22°C in July.

Geotechnical Features

Geology - Brenda Mines is located in an area known as the Thompson Plateau. This is a late Tertiary erosion surface of gently rolling uplands (elevation 4000 to 5000 feet). In the vicinity of the minesite this old erosional surface has been deeply dissected by streams flowing into nearby Okanagan Lake.

No known faults have been mapped near the Brenda Mines site. However, some faulting, both known and inferred, has been mapped in the adjacent Okanagan Valley, as indicated in Special Volume #15, published by The Canadian Institute of Mining and Metallurgy (1976). Moreover, a well-defined known fault, the Louis Creek fault, is located approximately 80 miles north of the Brenda Mines site. The closest distance from the tailings dam site to the Okanagan Valley faulting is 11 miles (18 km).

During the last glacial period, the entire area was covered by an extensive ice sheet having an estimated thickness in excess of 2500 ft at the tailings dam site. The major soil deposits directly attributable to the glacial period consist of dense, basal, glacial tills, glacial lake silts and clays, and meltwater deposits of sand and gravel. At the tailings damsite for example, the centre of the valley is underlain by over 100 ft of dense, basal, glacial till. The glacial till deposit thins out on the valley walls and bedrock outcrops at some locations on the right abutment. The basal till is a dense, impervious, relatively incompressible, soil deposit having high shear strength and providing an excellent foundation for the tailings dam.

Seismicity - Figure 6 shows the epicentres of earthquakes located within 150 miles of the Brenda tailings dam. Also shown on this figure are: the location of the Louis Creek fault, the locations of the known and inferred Okanagan Valley faults; and the location of the author's postulated Okanagan Valley branch of the Louis Creek fault. This postulated fault, which connects the Louis Creek faulting with the Okanagan Valley faulting, possibly extends south past Grand Coulee in the U.S.A.

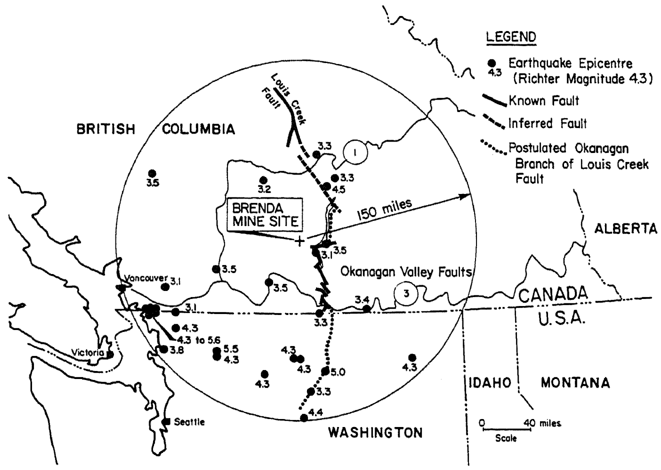

Figure 6
Earthquake Epicentres Within 150 Mile Radius of Brenda

As shown on Figure 6, only a small number of epicentres have been located in the Brenda Mines area. This is partly due to the low level of seismic activity in the area and partly due to the restricted ability of the seismic instrument network to locate small earthquakes with a Richter magnitude of 4 or less in this region prior to 1960. Also to be kept in mind is the relatively short historical period over which records have been kept of seismic activity in British Columbia (since 1899).

The location of many of the epicentres cannot be determined exactly, owing to the low density of the monitoring network and the simplified assumptions required for the computations. However, an examination of Figure 6 indicates that nine of the recorded earthquakes appear to occur either along or close to the postulated Okanagan Valley branch of the Louis Creek fault. Two of these epicentres are located immediately east of the Brenda Mines site. The largest recorded earthquake which may be associated with this postulated fault system is an earthquake of magnitude 5.0 near Grand Coulee in the U.S.A. No earthquakes have been recorded within the ancient highlands where Brenda Mine is located.

Studies currently underway by Canada Department of Energy, Mines and Resources, Basham et al (1982) suggest that the maximum earthquake that might occur in this general region is Richter magnitude 6.5. This value has been arrived at by considering both the tectonic and geological setting and the recorded seismic history for the region. The value is considered to be a very conservative figure that defines the upper boundary of any future seismic activity in this region.

The Brenda seismic stability analyses were carried out assuming that a maximum credible earthquake of Richter magnitude 6.5 could occur at the nearest point where the postulated Okanagan

Valley branch of the Louis Creek fault passes the site. This places the maximum credible earthquake approximately 11 miles (18 km) from the damsite and results in an estimated bedrock acceleration at the tailings damsite of 0.38 g, using the attenuation curves of Hasegawa et al (1981).

Subsoil Profile - The sand dam foundation consists of a thin surficial mantle of ablation till underlain by very dense, basal, glacial till deposits, overlying bedrock. The bedrock is exposed on the abutments but the depth to bedrock exceeds 100 ft in the centre of the valley. The bedrock at this site is generally classified as a granodiorite. The surficial, ablation till deposits contain discontinuous lenses of silt and clean sand, reworked by fluvial processes. The basal, glacial till deposits comprise a very dense mixture of gravel, sand, and silt, having 30% to 40% of particles passing a #200 sieve. Boulders occur randomly throughout the glacial till deposit. The glacial till deposit is relatively impervious, having a coefficient of permeability in the order of 10^{-6} cm per sec. However, the glacial till does contain several thin lenses of sand and some layers of gravel.

Soft or loose surface soils encountered under the drainage blanket were stripped down to the dense basal till. Beneath the Starter Dams, any soft or loose surface soils were either removed or compacted by the heavy equipment used for handling the rockfill. Thus, it is considered that both the drainage blanket and the Starter Dams are founded on dense, competent foundation materials. Over the remaining plan area of the sand dam the amount of stripping carried out was minimal. For purposes of analyses all foundation soils overlying the dense glacial till in this area were treated as tailings sand, which is considered a conservative assumption.

Groundwater - Generally, the natural groundwater table is within the ablation till deposits, above the top surface of the dense glacial till that forms an impervious boundary on which runoff from rainfall and snowmelt flow down the valley slopes to the base of the valley. The glacial till deposit is saturated throughout its entire depth. In the bottom of the valley, the inclusions of pervious material within the basal till deposit are usually under some artesian pressure, being fed by the runoff from higher elevations on both abutments. This condition is normal for such valleys and although these pervious zones are unlikely to have any direct connections to the tailings pond, they must be monitored to ensure that excessive water pressures do not develop. Piezometers were installed to monitor these zones.

DESIGN AND CONSTRUCTION

General

The valley in which the tailings dam and pond are located has a steep gradient, is relatively narrow, and requires a high dam to provide the necessary storage. At the time (1968) the proposed dam was the highest tailings dam designed to that date in British Columbia. Consequently, it was considered by all involved parties to be a critically important structure. Moreover, its location a short 11 miles west and 3000 ft above the populated Okanagan Lake resort area placed additional emphasis on the importance of achieving a safe design.

Since the date of the original design, several modifications have been made to both the design section and the construction procedures used. These changes were made primarily to increase the volume of tailings that could be stored behind the dam to accommodate the larger orebody that has since been defined. Justification for the changes was based on both the experience gained during the early years of dam construction and further technical data obtained from detailed field and laboratory investigation programs.

Original Design Concepts

The original design concepts for the Brenda Mines' cycloned-sand tailings dam were developed in 1967-68. At that time, the proposed volume of tailings to be stored required a dam having a maximum height of approximately 400 ft above the downstream toe. In 1967 our knowledge concerning the liquefaction of sands under earthquake forces was much less than it is today. Moreover, the analytical tools required to assess the susceptibility of a sand dam to liquefaction under a given earthquake force were not available to practising engineers.

The concept of a critical void ratio above which a sand, when strained would contract and below which it would dilate, had been addressed by Casagrande (1936) many years earlier. More recent work by Seed and Lee (1966), Lee and Seed (1967a) and Lee and Seed (1967b) had demonstrated that cyclic loading of sands caused their pore pressures to increase, until ultimately they failed by liquefaction. Field observations providing qualitative information on liquefaction failures were available in the published engineering literature (Dobry and Alvarez, 1967). Collectively, all of these data indicated that loose, saturated sands were subject to liquefaction under earthquake forces. On the basis of these available data, the author judged that provided the proposed Brenda sand dam was kept relatively dry, was designed with flat downstream slopes, and was constructed to a medium density, the structure should be able to safely resist a major earthquake.

To achieve this design, the following measures were taken:

(1) The dam was constructed of clean, free-draining, cycloned sand. Double cycloning was required to produce material having a permeability in the order of 10^{-2} cm/sec.

(2) Large foundation drains were constructed to convey construction water and seepage to the toe of the dam where it was collected and recycled.

(3) A wide tailings beach was provided on the upstream side of the sand dam to act as an impervious zone.

(4) The dam was constructed in the downstream direction, over previously prepared foundations and underdrains.

(5) The average downstream slope of the sand portion of the dam was set at 4 horizontal: 1 vertical.

(6) A rockfill toe dam was constructed across the steep-sided bottom portion of the valley to reduce the width of the sand dam required at this location and also to serve as a free-draining buttress fill.

Construction of the cycloned-sand tailings dam started in March 1970. The mining operators experimented with several methods of dam construction during the first two years of operation. These included: first-stage cycloning on the abutment and second-stage cycloning directly off the crest of the upstream starter dam; two-stage cycloning on the abutment and discharging the sand onto a free slope within the dam; and two-stage cycloning on the abutment and discharging the sand into cells, created by bulldozing sand dykes. The latter method was finally adopted and has been used with some variations ever since.

Current Design

The construction procedures currently being used to raise the dam are similar to those used for conventional, hydraulic, sand fill operations. The sand is produced by double cycloning the tailings in a cyclone house located high on the left abutment and is transported to the dam in a polyethylene pipeline. Cells or paddies, 6 to 8 ft deep, are constructed by pushing up dykes using bulldozers and these cells are then filled hydraulically with sand. The sand and water enters the cell at one end and the water is discharged at the other end of the cell. The sand deposits as a beach, on a slope of approximately 25:1. Filling starts at one end of the cell and continues across the entire length of the dam. Figure 7 is a recent photograph (1980) taken from the right abutment of the dam, showing sand cell construction in progress. The transportation water (approximately 1500 - 2000 gpm) used to convey the sand to the dam seeps downwards through the pervious sand fill and is carried away by the underdrains. Some of this percolation occurs within the sand cell under construction and the rest in previously constructed areas lower in the dam.

The original design for the Brenda Sand Tailings Dam called for a structure approximately 400 ft high with a crest elevation of 4510 ft. Since then two separate requests have been made by the mine owners to provide more tailings storage by raising the dam crest. The first request for a revision was reviewed by the designers in 1973. This involved raising the dam crest 60 ft to elevation 4570. An assessment of the stability of the raised dam under seismic loading was made using the simplified method of seismic analysis described by Klohn et al (1978). These simplified analyses indicated that the raised structure, with a dam crest elevation of 4570 ft was safe under the anticipated earthquake loadings. The second request, made in 1980, was to raise the dam an additional 80 ft to a final crest elevation of 4650 ft. This proposed second revision to the original design would make the Brenda sand tailings dam one of the highest tailings dams in the world. For such a major sand dam, the designers considered it essential that the seismic stability of the structure be evaluated using "state-of-the-art" finite element dynamic methods. The mine owners supported this approach, as the relatively high costs of making the necessary field, laboratory, and office studies would be more than offset by the savings achieved if it were determined that the existing dam could be safely raised to the ultimate crest elevation of 4650 ft.

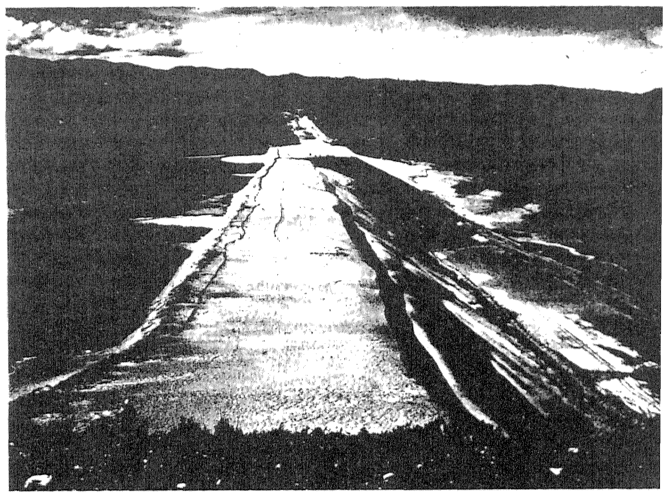

Figure 7
Hydraulic Sand Cell Construction

Consequently, as described in later sections of this paper, extensive field and laboratory programs were carried out to provide the parameters needed for making the desired finite element, dynamic analysis.

The recently completed (1982) studies, using the latest data acquired from the field and laboratory studies and "state-of-the-art" finite element methods of analysis confirm that the original 400 ft high dam (1967-68 design) could safely resist a maximum credible earthquake of 6.5 Richter magnitude. These studies also indicate that, with some minor design modifications the original dam may be safely raised to an ultimate height of 530 ft above the downstream toe. As currently proposed, the ultimate dam will have a crest length of approximately 6500 ft and a maximum base width of approximately 1500 ft as measured from the centreline of the sand dam to the downstream toe. (The dam is being raised by the centreline method of construction which produces a vertical upstream face of interfingered cycloned sand and slimes). The final downstream sand slope will be approximately 3.0 horizontal to 1 vertical. Total sand requirements will be approximately 32,500,000 cubic yards.

Figure 8 presents a plan and section through the Brenda Mines cycloned-sand tailings dam for both the original and current designs. Also indicated on this figure is the present (1983) sand dam which is now slightly higher than the original design.

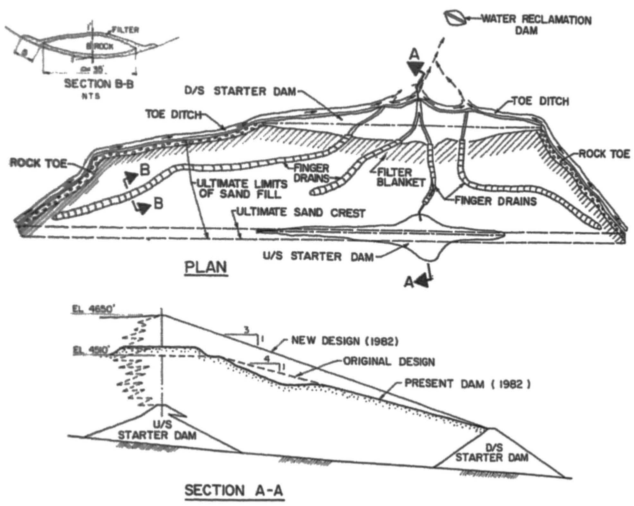

Figure 8
Plan and Section Through Brenda Sand Tailings Dam

CONSTRUCTION AND OPERATIONAL PROBLEMS

General

The construction and operation problems that usually develop throughout the life of a tailings dam are many and varied and the Brenda Mines tailings dam has been no exception. Fortunately, each of the prôblems that has arisen to date has been immediately identified and treated in a satisfactory manner by the mine operating staff, and no serious repercussions have developed. A brief review of some of the major problems that have occurred during the 14 year life of the Brenda Mines operations is quite instructive and is presented following.

Pond Levels

In the initial few years of tailings dam construction, maintaining adequate freeboard between the crest of the dam and the tailings pond levels is a critical item. Normally, for economic reasons, the height of the starter dam is only adequate to provide approximately one year's storage of tailings. During that first year the construction of the tailings dam must progress fast enough to provide the required freeboard for the second year's operation. At the Brenda Mines' tailings dam, two problems arose at the outset of operations.

(1) Hydrological conditions differed from those originally given to the designers, resulting in more water entering the tailings pond than had been anticipated.

(2) The clay mineral content of the ore prevented production of the necessary volumes of clean sand needed to keep the dam above the rising pond. This resulted in production of only about 50% of the required sand during the first year of operation.

As a result of these problems, not only did the dam have to be raised faster than originally planned, but there was also a shortage of suitable cycloned sand to achieve the raising. Consequently, it was necessary to twice raise the starter dam, each time in increments of 15 ft, using costly borrow materials. To obtain the required volumes of suitable sand, two-stage cycloning of the tailings was necessary. The two-stage cycloning has proven to be most successful with about 40% of the total tailings production becoming dam-building sand.

Seepage Problems

Two types of seepage problems can easily develop with sand tailings dams. One of these problems can occur when the tailings pond level rises quickly and floods the tailings beach which normally provides the sand dam with its upstream impervious zone. Flooding the beach places the freewater in the pond in direct contact with the pervious sand and can result in large seepage flows through the sand fill, creating a high phreatic surface which may cause slumping and/or piping on the downstream face of the sand dam.

The second problem involves the piping of tailings sand into rockfill zones, through fine rockfill transition and/or faulty filter zones.

Rising Pond Levels - During the second year of operation of the Brenda Mines' tailings pond, the spring runoff caused the tailings pond levels to quickly rise. The rising pond levels overtopped a short, below-design-elevation section of the impervious zone that formed the top of the starter dam, placing pond levels in direct contact with the sand dam. Appreciable seepage began to occur and within a very short period, large volumes of sand were carried away by the emerging seepage water. Figure 9 presents a photograph of the area in question and clearly indicates the extent of the problem. An alert tailings dam operating crew were quickly aware of the problem and repairs were made by dumping impervious fill over the upstream face of the dam and gravel filter material over the exit area of the piping. In this example, the

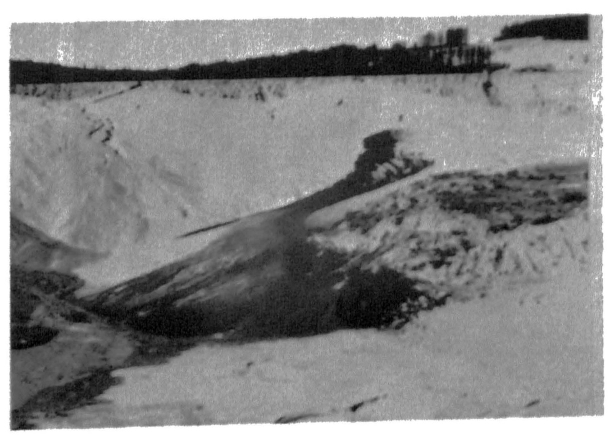

Figure 9
Photo Showing Piping Near Sand Dam Abutment

seepage and resultant piping of the downstream sand fill posed no real threat to the dam as pond levels were well below the top of the compacted, impervious starter dam, except for the very short, below-grade section where the seepage occurred. However, should such a seepage problem develop after the sand dam has risen well above the original starter dam it could pose a serious risk to the safety of the dam unless treated immediately.

Rockfill Transition and/or Faulty Filter Zones - In the early years of the operation, ponding of the sand transportation water frequently occurred immediately upstream of the rockfill toe dam. This ponding was not contemplated in the original design. Consequently, although a fine rockfill transition zone was provided between the rockfill toe dam and the cycloned sand, this transition zone was not considered a critical item and did not meet filter design requirements. At an early stage in the dam construction, when the pond was close to the rockfill and high seepage gradients existed, piping suddenly occurred through a window in the rockfill transition zone. This quickly drained the ponded, excess transportation water, and some sand, through the rockfill and into the downstream seepage recovery pond. Although this incident caused no structural damage to the tailings dam, it did partially fill up the recovery pond with sand, causing a very small volume of effluent to be discharged over the recovery pond emergency spillway.

The water ponding problem developed when the hydraulic sand fill construction procedures that evolved on the Brenda tailings dam required more construction water to transport the sand to the dam than originally contemplated. In the early years of construction, the sand surface area available for downward percolation of the construction water was limited and the above noted ponding occurred. As the dam has been raised, the surface area of sand available to accept downward percolation of construction water has been greatly increased and ponding is no longer a problem.

A second instance of piping has apparently occurred into one of the large, rockfilled, finger drains. A sinkhole, circular in shape, and having a diameter of approximately 14 ft suddenly appeared in 1983. The sinkhole initially was 40 ft deep and had vertical sides. The sinkhole has occurred directly above a major rockfill drain and there appears to be little doubt but that for some unknown reason a window exists in the filter zone which was supposed to cover the rockfill drain. This has allowed the fine sand to slowly pipe into the drain under the downward seepage of the construction water used to place the sand. All drains were adequately filtered and carefully inspected at the time of their construction. However, the downstream portion of the drain in question remained uncovered for a short period of time after its original construction and the author assumes that at some time during this period, the filter blanket was damaged at the point where the sinkhole formed.

The sinkhole was filled with 30 cubic yards of coarse filter material, followed by 300 cubic yards of fine filter material. Water was added throughout filling on a continuous basis. At the completion of filling, the sinkhole was estimated to have sunk a further 18 ft, based on the total volume of gravel required to fill the hole. In the six months that have elapsed since the sinkhole initially developed, the infilled granular materials have settled a further 25 to 30 ft. Additional filter gravel is being added and water is continuing to be poured into the sinkhole.

This localized sinkhole is not considered to materially affect the integrity of the sand dam and eventually the author anticipates that the filter materials being dumped into the sinkhole will reach and cover the window in the filter. However, there is concern about the potential risk that may exist in this local area for the formation of additional sinkholes. In an effort to reduce this risk the mining operators have been requested to surface flood the area above the finger drain at least 30 days prior to carrying out any construction activities. The downward seepage associated with such flooding is expected to hasten the movement of sands towards any undetected voids that may exist in the area. In addition, some wash bore soundings will be made above the buried drain to determine if any other voids exist. Piezometers will also be installed to determine the effectiveness of the partially plugged drain.

Figure 10 presents a plan and section through the sandfill at the location of the sinkhole. Figure 11 presents a photograph of the sinkhole taken during the backfilling operation.

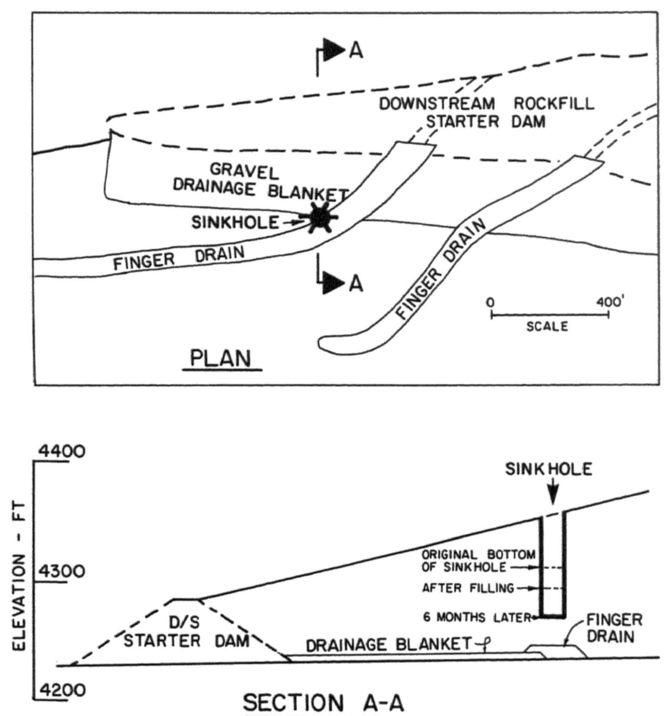

Figure 10
Plan and Section Through Sand Dam at Sinkhole Location

Figure 11

Sinkhole Near Toe of Sand Dam

Subsequent sections of the paper describe the field, laboratory, and office studies carried out to assess the feasibility of raising the tailings dam to its presently projected, ultimate height of 530 ft.

SITE INVESTIGATIONS

General

The original foundation investigations for the Brenda Mines tailings dam were carried out between 1967 and 1969. A conventional foundation investigation, consisting of deep machine drill holes and shallow test pits and trenches was made for the original dam. Laboratory tests were carried out to determine the physical properties of the foundation soils, including their shear strength and consolidation characteristics. Piezometers were installed in the pervious inclusions encountered in the glacial till to measure the effects of raising the tailings pond on foundation piezometric pressures. Once construction of the sand tailings dam was underway, standpipe piezometers were installed in the sand fill to measure piezometric levels.

Since the date of the original site investigations, three investigations of the "as-built" sand tailings dam have been made. The first two "as-built" investigations were made in 1972 and 1977. These investigations were primarily concerned with assessing the insitu densities of the sands placed in the dam to that date. The type of investigations carried out involved: Dutch Cone Penetration Tests, Standard Penetration Tests, downhole nuclear density tests, near-surface density tests made in test pits, and density tests made in deep drill holes using thin-wall Shelby tube samples. Piezometric data for the sand dam also was reviewed and additional piezometers installed as required to replace non-functioning units.

The third series of investigations of the "as-built" sand tailings dam were carried out in 1980 as part of a program to provide data for a "state-of-the-art" seismic stability analysis of the tailings dam. This series of investigations was much more comprehensive than any of the earlier investigations and the bulk of the information presented in subsequent sections of this paper was obtained from the 1980 work. The 1980 investigations included: 5 clusters of bore holes on the dam, 5 cone penetration holes on the dam, and 3 on the beach, and the installation of 15 additional pneumatic piezometers in the dam and 4 piezometers in the tailings beach. The 1980 field program covered the necessary activities to:

- retrieve undisturbed samples at depth, for laboratory testing;
- evaluate insitu densities at depth using undisturbed samples, standard penetration tests, geophysical logging, and static cone penetration tests;
- measure insitu densities of the sand near the dam surface;
- measure seismic wave velocities through the sandfill; and
- confirm the phreatic levels within the dam.

Figure 12 shows the locations of the pertinent test holes and the major test installations constructed in the cycloned sand dam to date (1983). Also shown on this figure are the locations of 4 relief wells and 2 piezometers installed at the beginning of construction, (1969) within pervious zones in the glacial till foundation soils immediately downstream of the dam.

Laboratory testing programs were carried out on samples of the cycloned-sand fill obtained during each of the three investigations made on the "as-built" sand dam. The majority of test results were obtained from samples taken during the detailed 1980 field investigations as part of the "state-of-the-art" seismic evaluation program. These included: index properties, consolidation characteristics, and static and dynamic shear properties.

Field Investigations

Block Samples and Shelby Tube Densities - In all field investigation programs, insitu densities at shallow depth were obtained by test pitting and taking block samples. These samples were subsequently used for the laboratory test programs. Thin-wall Shelby tube samples were also obtained at shallow depths, in test pits.

In the 1980 investigations, thin-wall Shelby tube samples to depths of up to 250 ft were taken in 5 inch diameter, mud-filled drill holes, using a fixed-piston sampler. The samples were taken using 3-inch and 2.5 inch O.D. thin-wall Shelby tubes, with area ratios of 11.9% and 14.0% and inside clearance ratios of 1.0% and 0.8% respectively. Very careful sampling techniques were developed, using a double-rodded, piston sampler. Recovery ratios in excess of 98% were generally achieved. It should be noted that,

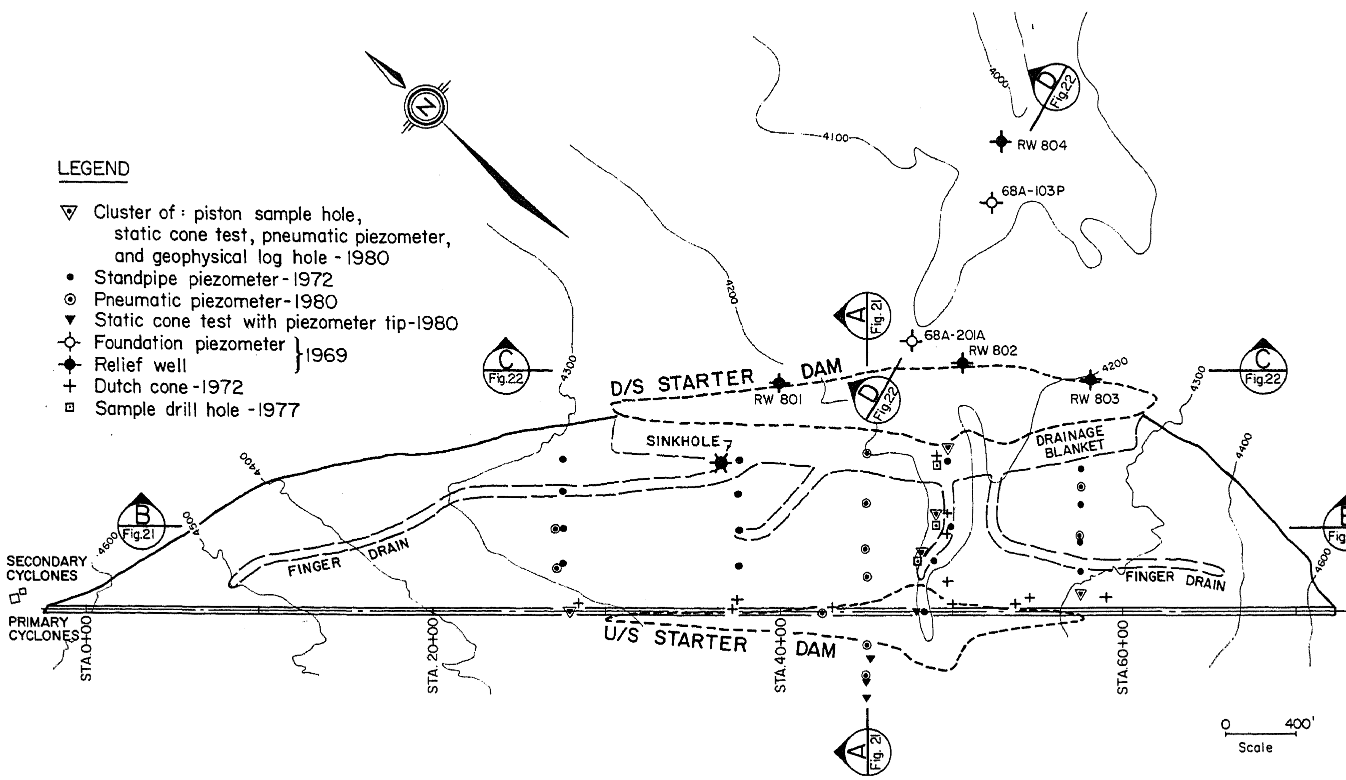

Figure 12
Location of Pertinent Test Holes and Test Installation Within Sand Dam

despite the fact that all sample tubes were carefully lacquered, corrosion still occurred for several soil samples that were not extruded from the tubes within a relatively short time following sampling operations. In future, in those cases where it is not feasible to extrude the soil shortly after sampling, stainless steel, sample tubes may be used with corrosive tailings.

The Shelby tube samples are considered to provide relatively undisturbed samples of the sand to the depths sampled. The insitu dry densities determined from these samples are considered to comprise the base-line density data for this investigation. Figure 13 presents a summary plot of insitu dry density versus depth for all of the data obtained during the 1980 investigations. Superimposed on this plot are two consolidation test curves for reconstituted sand samples. One of these tests had an initial relative density of approximately 30% and the other 50%. The two curves, which are parallel, define a band that appears to bracket the average rate of increase of density with depth. Most of the observed density increase occurs in the upper 50 ft followed by a further slow, approximately linear, increase with depth. At a depth of 250 ft the relative densities computed from the two consolidation curves have reached values of 60% and 70% respectively.

Figure 14 presents a summary plot of relative density versus depth based on the dry density values presented in Figure 13. As would b expected for the normally consolidated sand, th relative density increases slowly with depth. best-fit line drawn through the points suggest an increase in the relative density of the san of about 10% between the 50 ft depth value c ±50% and the 240 ft depth value of ±60%.

Nuclear Probe Densities - Nuclear density probe were used in the 1977 and 1980 site investiga tions. In 1977, a 1 1/2 inch diameter, 12 inc long nuclear probe was used to measure insit densities. The measurements were made inside 1.55 inch diameter access tube which was pushe or driven into the sand in short increments, a a closed tube. A detailed description of th equipment and the test procedure is given b Mittal and Morgenstern (1975). Although th equipment provided both bulk density and wate content data that checked closely with directl measured values, the probe was only partiall successful as the access tube could only b advanced a relatively short distance into th sand before meeting refusal.

In 1980, the 1 1/2 inch probe with its limite penetration capacity was dropped in favour c using large-scale oil well logging equipmer with a calibrated gamma probe. The 1980 prob was carried out in a 5 inch diameter, mud-fille hole. In addition to running a gamma-gamma lc in each drill hole, caliper probes, neutrc probes, and resistivity probes were also run t

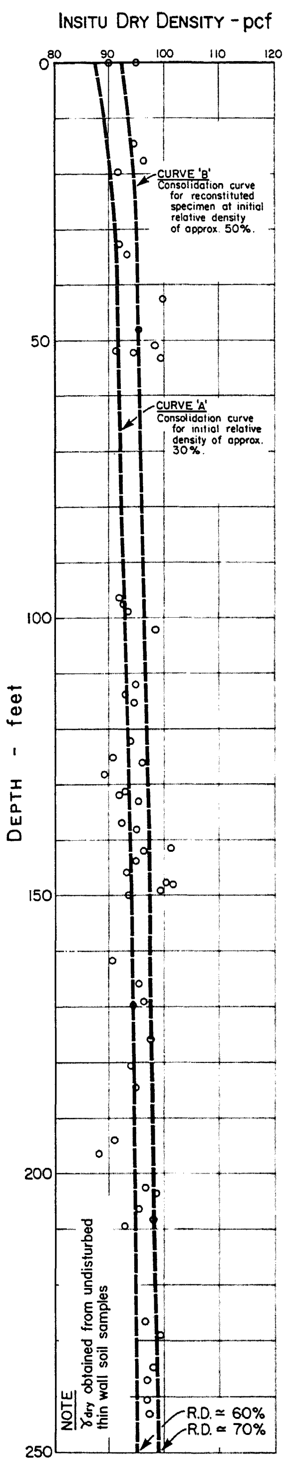

Figure 13
Insitu Dry Densities - From Undisturbed Samples

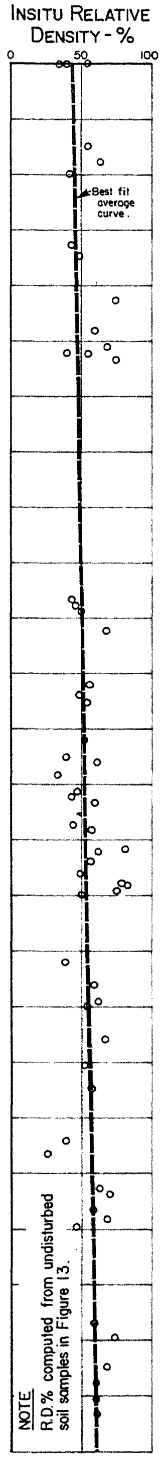

Figure 14
Insitu Relative Densities - From Undisturbed Samples

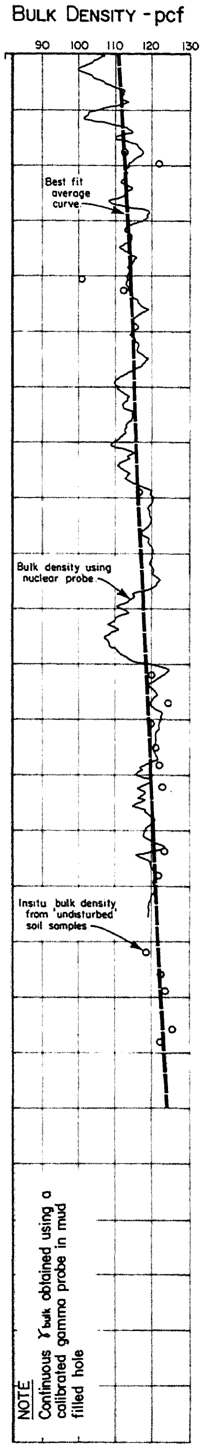

Figure 15
Bulk Density Profile Using Nuclear Probe

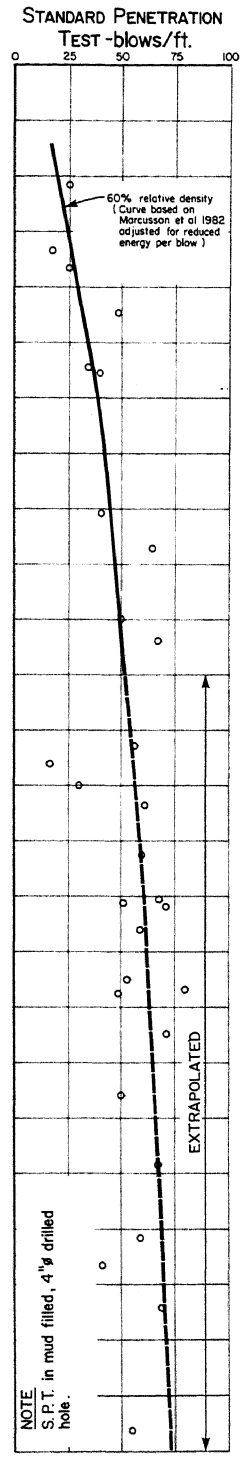

Figure 16
Standard Penetration Test Data

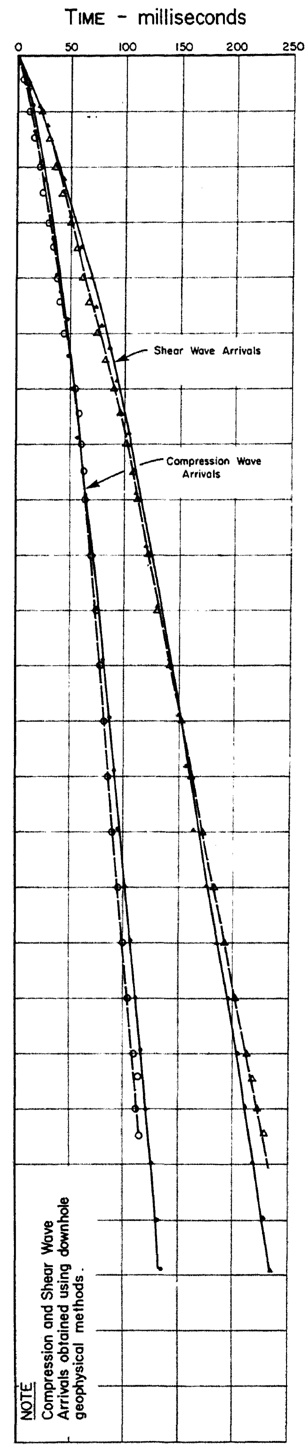

Figure 20
Downhole Shear and Compressive Wave Velocities

assist in interpretation of the results.

The nuclear density probe program was only partially successful. In most instances, the measured bulk densities checked fairly closely with values obtained from undisturbed samples at the same location. However, the water content data produced was erratic and generally not suitable for use. Figure 15 presents a continuous nuclear bulk density profile at one of the test holes where a fairly good correlation was obtained between nuclear determined bulk densities and directly measured bulk densities.

Standard Penetration Test - The procedures used to carry out the standard penetration tests generally conformed to those widely used in North America. The drill hole was advanced using drilling mud to support the sides of the hole. A 2-inch O.D. by 1 3/8 inch I.D. by 18 inches long, standard penetration sampling spoon was used. A 140 lb weight, falling 30 inches was raised and dropped using a rope and cathead system, having two turns of the rope around the cathead. The nominal test hole diameter was 5 inches. B.W.(2 1/8 inch) size drill rods were used to drive the sampling spoon. All penetration resistances were recorded over the range of 6 to 18 inches of penetration. To prevent excessive soil disturbance ahead of the drill hole, side-eject drilling bits were used.

A summary of all standard penetration test data is presented in Figure 16. Correlations between relative density and blow count have been suggested by Gibbs and Holtz (1957), Bazaraa (1967) and most recently by Marcusson et al (1977). The Marcusson correlation used a free-falling trip hammer to correlate the blow count with the relative density. Studies by Kovacs et al (1977) suggest that the conventional method of lifting and dropping the hammer by using two turns of rope on a cathead delivers only about 60% of the energy developed by the free-falling trip hammer. Kovacs further suggests that the blow count correction is inversely proportional to the energy delivered to the sample spoon. For purposes of comparison, the writer has adjusted the Marcusson "N-values" upwards by dividing them by 0.6. In Figure 16, the adjusted Marcusson curve for 60% relative density is plotted over the measured "N" values. An examination of this figure shows that the adjusted Marcusson curve for 60% relative density provides a reasonably good fit to the plotted "N" values.

All of these correlations should be considered approximate at best and should only be used to provide general guidance rather than indicate absolute values. Moreover, at the Brenda site, the various curves have been extrapolated far beyond their original values.

Static Cone - The 1972 mechanical "Dutch Cone" tests were made using a standard Dutch cone. Both the friction sleeve and point resistance were measured and recorded. The 1980 cone measurements were carried out using both a standard Fugro Friction-Cone and a Fugro Piezometer Cone. Continuous static cone logging was carried out for all tests. All procedures and penetration cones used in both series of test generally conformed to ASTM Specification No D-3441-75T. A detailed description of th static cone equipment used for the 1980 Brend program is given by Campanella et al (1981).

Before examining the cone penetration data, brief review of the history of the procedure used to construct the sand dam is desirable.

Initially, the sand was deposited by cyclonin from the dam crest in the downstream direction The sand deposited in this manner was no rehandled by bulldozers. Following the on-da cycloning method, the sand was deposited for short time by discharging off the abutments an flowing onto the dam. Deposition of sand b both these methods produced a sand deposit whic had a relatively low initial density that wa quite uniform over the entire area o deposition.

The above two methods of deposition were use during the initial two years of operation Since this period, the sand has been placed i cells, using hydraulic fill placement tech niques. This process involves the use of bull dozers operating over the deposited sand to pus up retaining dykes. The bulldozer traffi causes compaction of the sand. However, as th upper portion of sand in the cell tends to ge more compaction than the lower portion, sandwich effect results, with alternating laye of denser and looser sand.

In the last few years, the cell constructio procedures have been slightly modified with th bulldozers operating over the sand as it placed. This has resulted in the sand receivi a greater compactive effort and although th sandwich effect still occurs, the averag densities are measurably higher.

All of the 1972 cone penetration data a summarized on Figure 17. The left hand side the figure summarizes the cone bearing val (q_c) for all four test holes. The right ha side of the figure presents a typical plot cone bearing and friction sleeve values for single test hole. The cone bearing values the upper 40 ft of the summary plot are qui variable ranging from loose to dense. Below t 40 ft depths, the cone bearing values a generally low, indicating a loose to medi dense sand. This change in the density profi reflects the change from on-dam cycloning early hydraulic placement procedures whi occurred approximately 2 years after the sta of operations.

The 1980 cone penetration data are summarized Figure 18. The left hand side of the figu summarizes the cone bearing values for a 5 test holes. The right hand side of the figu presents a typical plot of cone bearing a friction sleeve values for a single test hole.

A comparison of the cone penetration data fr the 1972 and 1980 cone penetration tests clear indicates the different densities achieved the three construction procedures used. Brief summarizing this comparison shows:

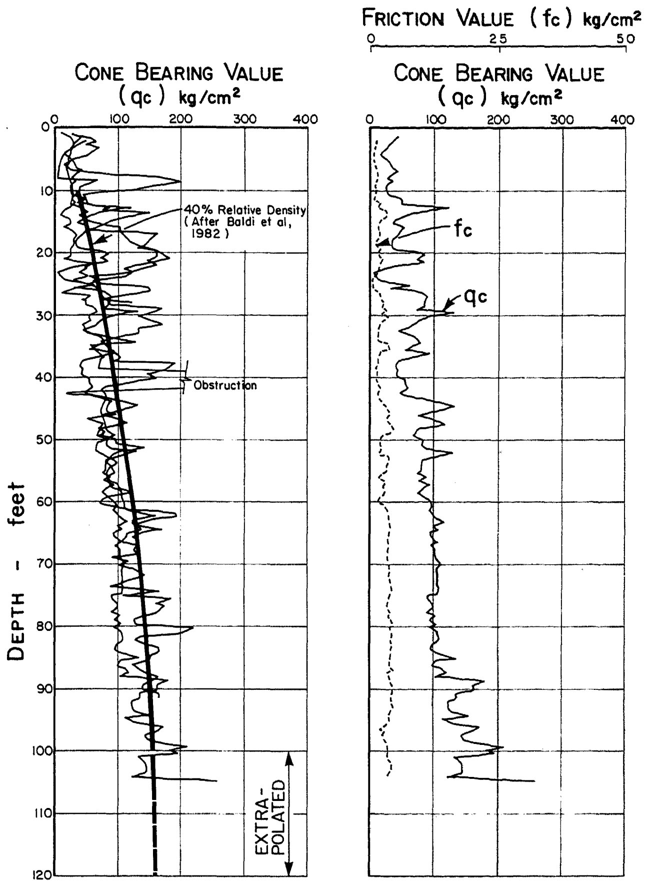

Figure 17
Summary of 1972 Cone Penetration Data

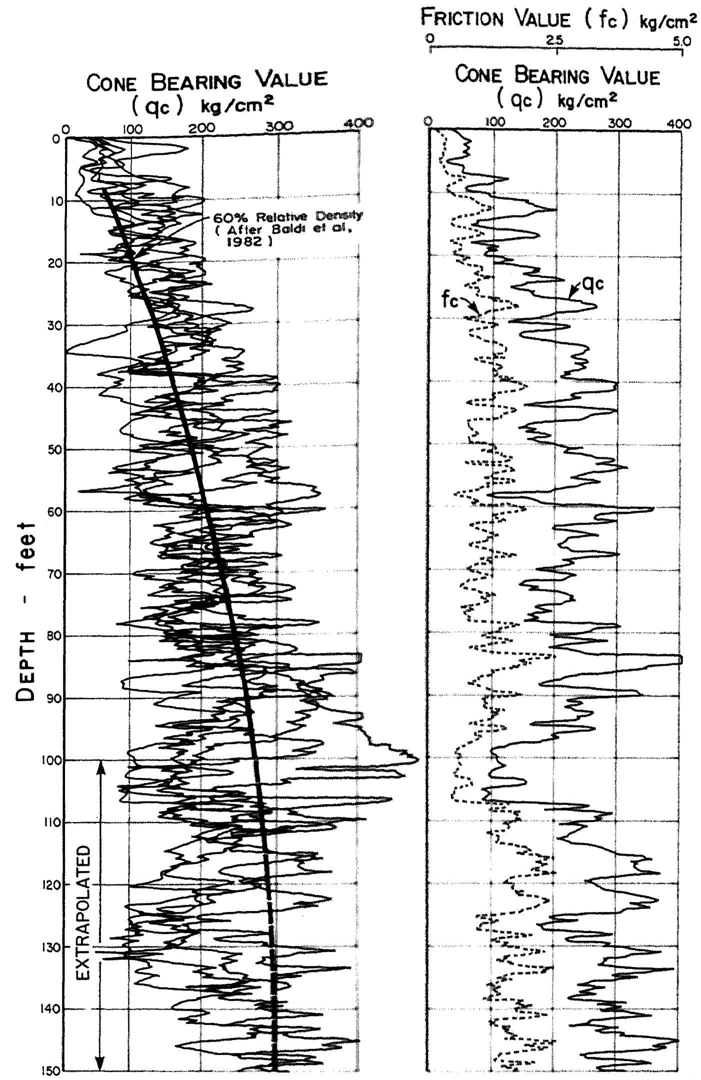

Figure 18
Summary of 1980 Cone Penetration Data

(1) The sand below a depth of approximately 40 ft in Figure 17, which has had no bulldozer traffic, has lower and generally more uniform densities than the sand above the 40 ft depth, which has had limited bulldozer traffic.

(2) The sand above a depth of approximately 40ft in Figure 17, which has had limited bulldozer traffic is generally less dense than the upper sands in Figure 18, which have had more bulldozer traffic.

(3) By overlaying Figure 17 on Figure 18, it may be seen that the average density for the sand placed without benefit of bulldozer traffic (Figure 17) falls just below the lower density boundary of the sand placed with bulldozer traffic (Figure 18). This clearly indicates the benefits derived from operating bulldozers in the cells as the sand is placed.

Cone penetration tests have been used for many years to evaluate the density and bearing capacity of sands. Recent work by Baldi et al (1982) presents a series of curves, relating relative density to cone bearing values (q_c), to equivalent sand depths of approximately 100 ft. These curves are presented on Figure 19.

For purposes of comparing Baldi's curves with the cone bearing values obtained in this investigation, the author has taken the liberty of extrapolating these curves to a total equivalent sand depth of 150 ft. Overlaying these curves on the 1980 cone bearing data of Figure 18 indicates that the in situ relative density of the sand ranges from about 40% to 70% with an average value of about 60%. In making this assessment the obviously very low and very high cone bearing values were ignored. For ease of reference, the 60% relative density curve also has been plotted on Figure 18.

A similar comparison of Baldi's curves and the 1972 data indicates an average relative density in the upper 40 ft of about 50% and for remainder of the depth, an average relative density of somewhat less than 40%. To illustrate this item, the 40% relative density curve has been plotted on Figure 17.

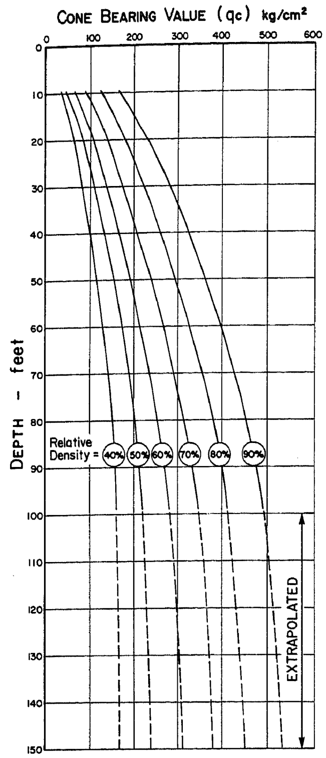

Figure 19
Relative Density Relationship for
Uncemented and Unaged Quartz Sands
(after Baldi et al 1982)

As previously discussed, construction procedures have varied since the tailings dam was begun. The sands placed most recently, generally have the highest average densities, while the sands placed initially have the lowest. The relative densities indicated by the Baldi curves appear to support this item. It is interesting to note that the insitu densities inferred from the indirect static cone and standard penetration testing programs indicate reasonable agreement with those determined directly from undisturbed soil samples.

Seismic Wave Velocities - Downhole measurements of compressional and shear wave velocities were carried out in 5 deep drill holes. The downhole velocity measurements were made by recording at various depths vertically travelling seismic waves (shear and compression) from sources on the ground surface. The work was carried out by Geo-Recon (1980) using generally accepted methods and field procedures and is considered to be a reasonably accurate presentation of site conditions. The shear wave velocities obtained from these tests were used to determine the maximum shear modulus, corresponding to low shear strains, which is required for the dynamic analyses. Consistent and similar test results were obtained in each of the 5 deep drill holes. Figure 20 presents a summary of the data from two of the test holes.

Piezometer Installations - Standpipe piezometers have been used to measure piezometric levels within the sand tailings dam since the beginning of construction. Considerable problems have been encountered in maintaining thes piezometers as construction operations ofte cause breakage. During the 1980 investigations a new piezometer system was installed usin pneumatic piezometers that are remotely read an more easily maintained throughout th construction operations. To date (1983), th pneumatic piezometers have performed well Also, during the 1980 investigations, piezometric profiles were obtained both in the dam an in the upstream impervious tailings beach usin the Fugro Piezometric Cone.

Figure 21 presents typical longitudinal an transverse sections through the sand dam showing a recent set (1983) of piezometri levels. (The location of these sections i shown on Figure 12). These levels are generall quite low and rise locally when sand and wate are being placed in a construction cell. Th height of the water mounding that occurs under sand cell is a function of the size of the cell its location, and the length of time durin which sand and water are added to any give location. Local rises in piezometric levels o 20 to 50 ft have occurred in the past.

The piezometric water levels near the left abutment of the sand dam (Section B in Figure 21 were higher than normal during the past year a evidenced by the September 1983 readings. Thre factors are considered to be contributing t this condition. First, is the recent construction of a sand cell in the area. Second, is th natural runoff from the left abutment of th valley. Runoff tends to follow old channel (shallow depressions in the topography) tha cross under the dam on the left abutment downstream of the dam centreline. Seepage flow from this source are believed to hav contributed water to the area ever since th sand dam became large enough to encompass thes natural depressions. Third, is the possible partial plugging of the underlying finger drai by piping of sand into the drain, as evidence by the sinkhole that developed over this drain To date, the rise in the piezometric levels i this area presents no problem as piezometri levels are still less than the values assume for the stability analyses for the end o construction case. Moreover, they slowl decrease when construction in the area ceases Close observations are being maintained in thi area to check on both piezometric levels and an possible additional sinkhole developments Further site investigations are planned for nex spring (1984). Should piezometric level continue to rise in the area, it may becom necessary to install additional drainage work and/or increase the size of the ultimate to berm.

Section A on Figure 21 presents data from two double piezometer installations located in th tailings beach at distances of 200 ft and 400 f upstream of the centreline of the sand tailing dam. These piezometers were installed t determine the phreatic surface under the beac and measure any existing gradients. Additiona piezometric data for the beach area wer obtained during the cone penetration testin program when several Fugro piezometric con holes were also made in this area.

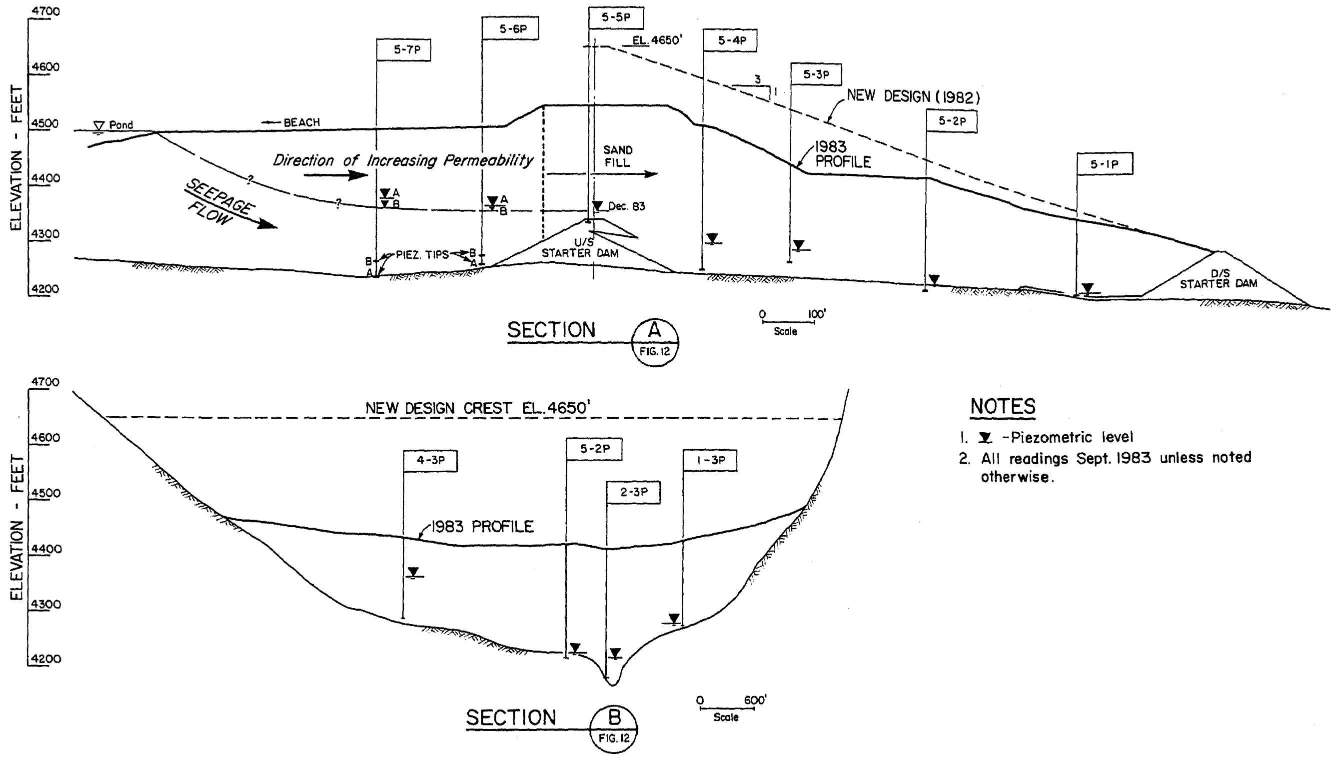

Figure 21
Piezometric Sections Through Sand Dam

Seepage patterns on the beach, in front of the sand dam, are quite complex, as several factors affect the seepage flows. A major contributing seepage flow is from the pond. As both the foundation and the starter dam provide impervious lower boundaries, all flow must pass over the starter dam and into the sand tailings dam and its drainage system. As shown on Section A of Figure 21, the phreatic surface between the freewater in the pond and the sand dam is located well below the surface of the beach. As indicated by piezometers 5-7P and 5-6P, a decreasing piezometric gradient exists both upwards and towards the sand dam. A portion of these gradients is likely attributable to dissipation of consolidation pore pressures in the beach deposits and the remainder attributable to the head of the water in the pond above the crest of the impervious starter dam.

Superimposed on the above pattern of seepage from the pond water, is the effect of downward seepage from the water and the fine tailings being spigotted onto the surface of the beach from the sand dam. Part of this surface water runs off the surface of the beach and into the pond while the rest percolates downwards towards the underlying phreatic surface. As the beach contains lenses of impervious slimes and layers of ice, randomly distributed both vertically and horizontally, the downward percolation of beach water is frequently interrupted. This results in the development of isolated perched water tables between the beach surface and the phreatic surface. The existence of both the impervious zones and the perched water tables has been confirmed by the Fugro piezometer cone tests that were made in this area.

Currently, (1983) the total seepage generated by all contributing factors, (pond water, beach water, and consolidation of tailings) is relatively small and is estimated to be in the order of 500 gpm. On the other hand, the construction water that is used to transport the cycloned sand and which percolates through the sand dam and is collected by the underdrains is estimated to average approximately 1500-2000 gpm. In addition to the above volumes of construction water and seepage, some runoff from both abutments is also intercepted by the sand dam and its underdrainage system. In total, an estimated 2500 to 3000 gpm is presently collected in the downstream seepage recovery pond. When construction of the dam is completed and the sand drains, the volume of seepage water from the pond that must be considered is estimated to be less than 500 gpm.

Several piezometers and 4 relief wells were installed at the beginning of construction in pervious zones within the glacial till foundation soils, immediately downstream of the rockfill toe dam. This instrumentation was intended to measure any build-up of excess pore pressures that might develop as the tailings pond was raised. Figure 12 presents the location of this instrumentation. Figure 22 presents sections illustrating the effects of the rising tailings pond on the foundation pore pressures. Table I presents a summary of readings obtained during the period September 1970 to February 1983.

A review of these data indicates that there has been a slight increase in foundation pore pressures as the dam was raised. These increases range from a minimum of near zero on the right

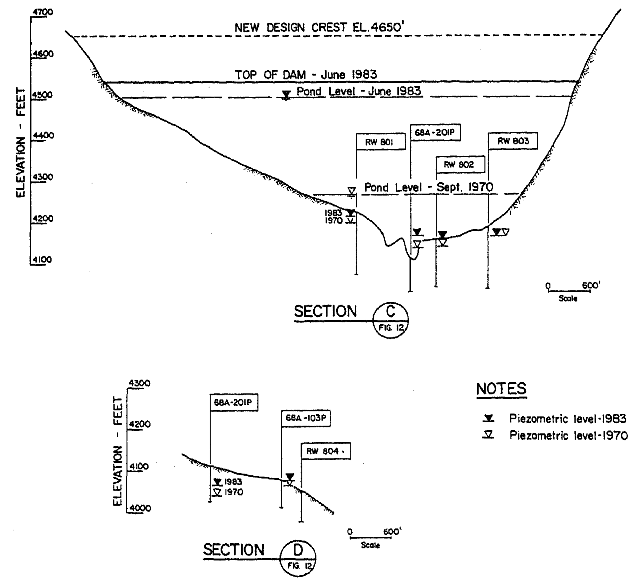

Figure 22
Piezometric Sections - Foundations D/S of Sand Dam

abutment (R.W. 803) to a maximum of approximately 30 ft at mid-valley (68A-201P). During this same period, the water levels in the tailings pond rose approximately 235 ft.

One of the relief wells (R.W. 804) has flowed slightly since its installation. The other three have risen very slowly since the start of construction with relief well 802 beginning to seep a very small volume of water in 1982. The rise in head in the three originally non-flowing relief wells since their installation has ranged approximately between 0 and 20 ft. These values agree closely with measured piezometer readings in the same area.

These relatively small piezometric increases that have developed within the foundation soils as the tailings pond was raised are considered acceptable and no cause for concern. Moreover, they confirm that the pervious lenses within the glacial till deposit are isolated from the tailings pond and therefore do not directly transmit high pond pressures to the foundation soils underlying the tailings dam.

Laboratory Investigations

Index Properties - Index property tests on the sand fill included: grain size analyses, specific gravities, maximum and minimum densities, scanning electron micrographs, and dry unit weight determinations on undisturbed samples obtained from the field investigations.

Grain size distribution curves for all samples tested to date are summarized by the narrow band presented on Figure 23. The majority of the cycloned tailings sand falls along the coarse side of this band having 5 to 10% of material

TABLE I

FOUNDATION PIEZOMETRIC PRESSURES DOWNSTREAM OF SAND DAM

	PIEZOMETRIC ELEVATIONS (ft)				
INSTRUMENTATION	SEPT 1970	OCT 1980	AUG 1981	SEPT 1982	FEB 1983
Piezometer #68A-103P	4067	4086	4079	4081	4079
Piezometer #68A-201P	4140	4157	4159	4173	4169
Relief Well #801	4202	4218	4220	4222	4222
Relief Well #802	4140	4152	4158	start flowing at 4159 flow less than .01 gpm	
Relief Well #803	4169	4171	4170	4170	4170
Relief Well #804	flowed from outset at less than 0.1 gpm				
Reference Figure	for location of piezometers and relief wells				

passing a number 200 sieve. As indicated by Figure 23, the sand fill is a clean, uniform, free-draining material.

The specific gravity of the sandfill averages 2.70. The average maximum density is 106.6 pcf and the average minimum density is 83.5 pcf. These average values were obtained from ten different series of tests on representative sand samples.

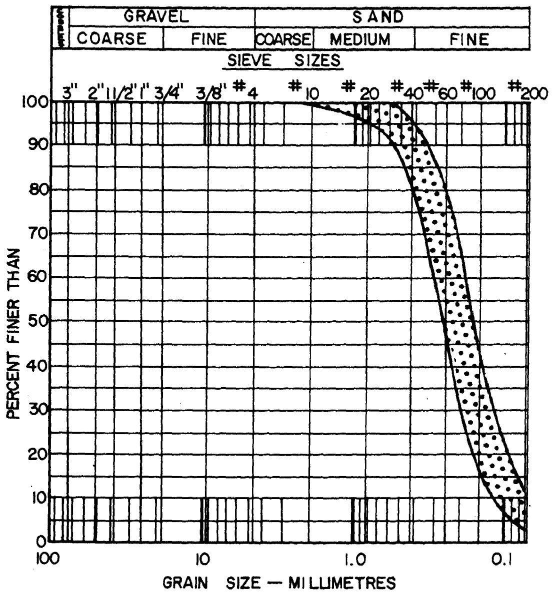

Figure 23

Range of Gradation for Cycloned Sand

Scanning electron micrographs of individual sand grains taken before and after static triaxial testing indicate that no obvious changes in the shape or surface characteristics of the sand grains occurred during testing.

Dry unit weights of the sandfill, determined from undisturbed tube samples are presented on Figure 13. This figure summarizes all insitu dry unit weights obtained from the 5 deep drill holes made in the sandfill dam during the 1980 investigations. The insitu density sampling program was concentrated on obtaining undisturbed samples from within the saturated zone at the bottom of the sand fill dam. In the unsaturated zone samples covering the entire range of densities were obtained. The very low density values shown are probably local deposits of windblown sand or material placed in a loose bulked state in dykes as discussed in the following section.

A series of dry unit weight determinations also was made on sand samples obtained at selected locations on or near the surface of the sand fill. The purpose of these tests was to determine the range of densities over which the sand was initially placed in the dam. The results of these tests are presented in Table II following.

TABLE II

AVERAGE RELATIVE DENSITY VALUES FOR SURFACE SANDS

Type of Deposit	Wind Blown Sand	Loosely Bulked Bulldozed Dykes	Sand Cell No Bulldozer	Sand Cell Bulldozer
Relative Density	0	0	40%	55%

The windblown sand and loosely bulldozed dyke deposits are randomly scattered throughout the sand fill. They generally parallel the centreline of the dam in the form of long narrow strips. They are not considered to significantly affect the overall performance of the dam, however, they do show up in all of the insitu tests as local zones of looser than normal sand.

The bulk of the sand in the dam is placed at initial densities ranging between 35% and 60% with an average value in the order of 50%. As the initial placement density has a very significant effect on the ultimate density of any given depth, a fairly wide range of insitu dry densities exist at the Brenda site. This is indicated by the fairly wide range of insitu density values plotted in Figure 13. For purposes of the steady-state seismic analyses, which are briefly discussed in a following section of the paper, the consolidation Curve A, on Figure 13, which is considered to represent the lower boundary of the insitu density range, was conservatively used to determine the variation in void ratio with depth of the sand fill.

Consolidation Characteristics - Consolidation characteristics of the sandfill were examined using both one-dimensional and triaxial consolidation tests on undisturbed specimens. Both tests showed that primary consolidation of the test specimens was completed well within one minute after load application. In addition, compression curves from both the one-dimensional and the triaxial consolidation tests exhibited the same shape in the plot of void ratio versus effective vertical (major principal) stress.

Consolidation test data in the form of "dry unit weight versus depth" plots are presented in Figure 24. Curves A and B are from consolidation tests carried out on reconstituted specimens of the tailings sand. Curve C is based on a consolidation test made on an undisturbed sample taken at a depth of 172 ft. Curve B, the reconstituted sample, and Curve C, the undisturbed sample both had approximately the same density at the start of the consolidation test. The apparent higher compressibility at lower vertical effective stresses of the undisturbed sample is believed to be attributable to two causes. First, possible sample disturbance prior to testing and second, imperfect fitting of the specimen in the consolidation ring. At higher vertical effective stresses, the two curves are parallel suggesting that either type of test could be used to estimate density changes in the sand with depth. Eliminating the sample and consolidation ring fitting disturbance would move consolidation Curve C further to the left and probably into closer agreement with Curve B.

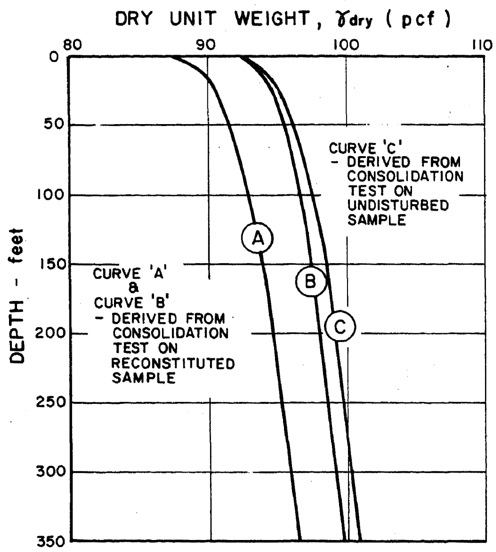

Figure 24
Consolidation Test - Computed Density vs. Depth Curves

Consolidation test Curves A and B for the two reconstituted samples are superimposed on Figure 13, the sample tube density versus depth plot. The agreement between the plotted actual insitu densities and the values that would be predicted using the consolidation curves is remarkably good. This suggests that simple, one-dimensional, consolidation tests on reconstituted sand samples could be used to predict, with acceptable accuracy, density changes with depth, for uniform sand fills, such as the Brenda tailings dam.

Static Shear Characteristics - Static triaxial, consolidated-drained tests were performed on undisturbed samples of the sandfill to determine its stress-strain characteristics and strengths under static shear loadings. On the basis of these tests, the average effective friction angle for the sand throughout the range of confining pressures used in the stability analyses was assumed to be 35° and the effective cohesion, zero.

Steady-State Undrained Strength - Static triaxial, consolidated-undrained tests were used to determine the steady-state line for the tailings sand fill. Figure 25 presents a summary of these data on a steady-state plot. The scatter that produces a band on the steady-state plot for the undisturbed samples is caused primarily by minor variations in physical properties from sample to sample. Carefully controlled tests on undisturbed and remolded specimens of the same material showed that, in every case, the undisturbed specimens demonstrated slightly higher strengths. The higher strengths are attributed to the structure of the undisturbed samples, which may include slight cementing. However, it should also be noted that the "steady-state band" is satisfactorily defined in Figure 25 by the several tests that were made on reconstituted sand samples.

Dynamic Shear Characteristics - Resonant column tests and cyclic triaxial hysteresis tests were performed on specimens of the sandfill to obtain data on the variations of shear modulus and damping ratio with shear strain. A summary of the shear modulus data obtained from resonant column tests, cyclic triaxial hysteresis tests, and insitu geophysical field tests is presented on Figure 26. Also plotted on this figure are the suggested curves of Seed and Idriss (1970). At very low shear strains, the laboratory data plots well below the Seed and Idriss curves. However, within the range of shear strain obtained from the finite element analysis, the laboratory data agrees fairly closely with the upper range of these curves.

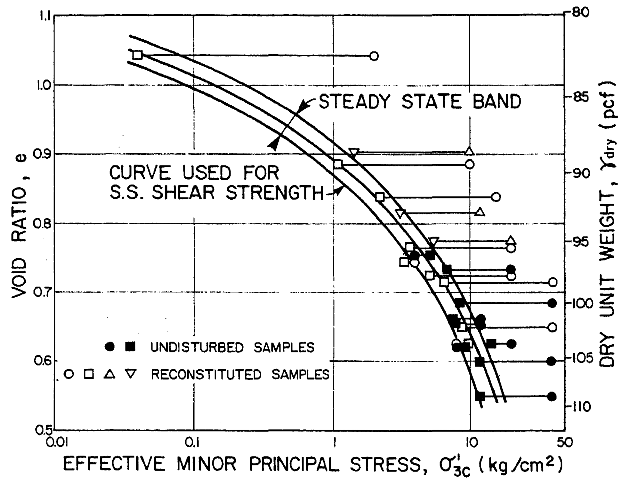

Figure 25
Steady-State Plot for Cycloned-Sand

A summary of the damping ratio results obtained from cyclic triaxial hysteresis tests is presented on Figure 27. The cyclic triaxial hysteresis test values plot along the lower boundary of the Seed and Idriss curves which are also shown on this figure.

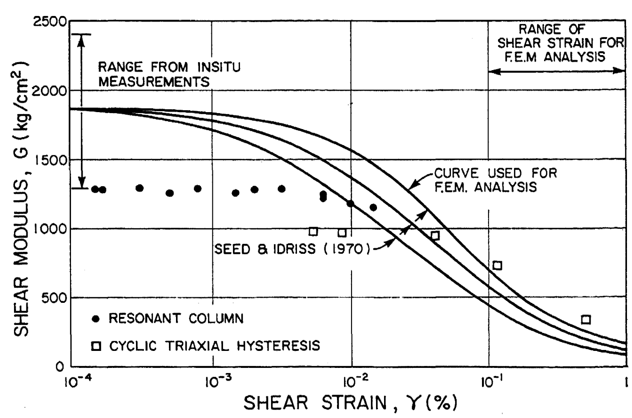

Figure 26
Summary of Shear Modulus Data

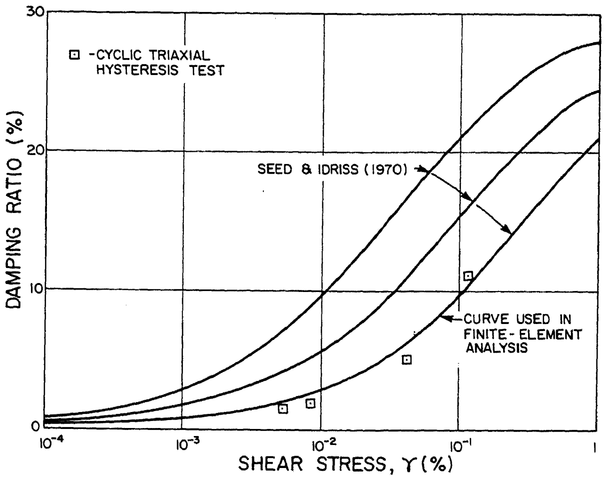

Figure 27
Summary of Damping Ratio Data

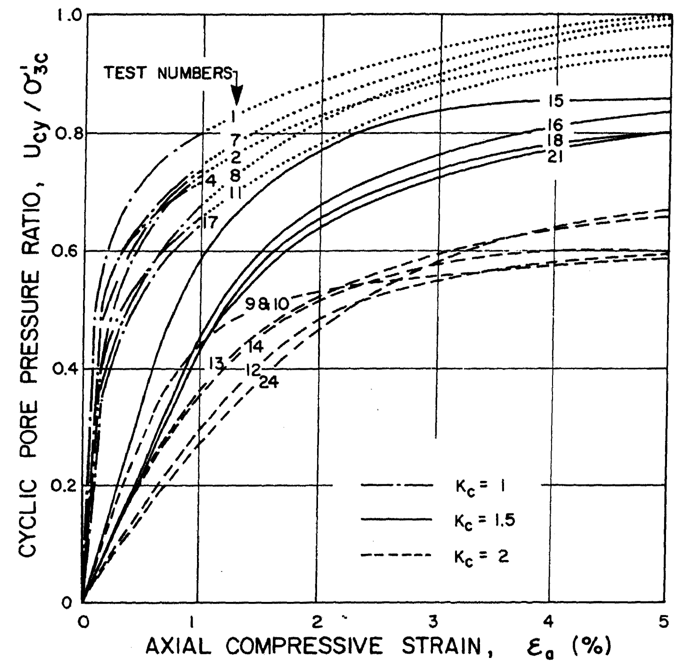

Figure 29
Summary Cyclic Pore Pressures vs. Compressive Strain

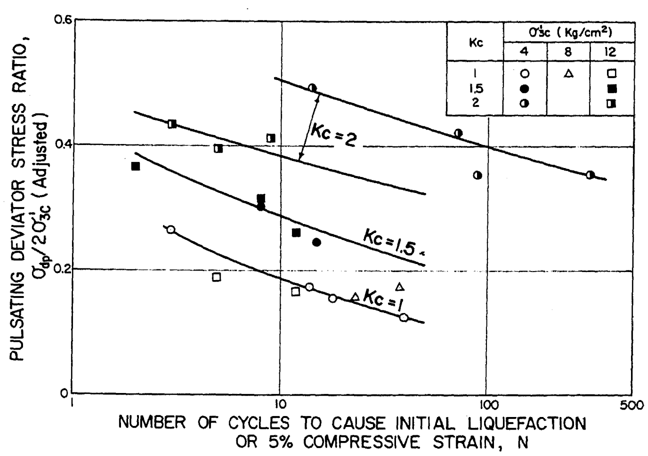

Figure 28
Summary of Cyclic Triaxial Test Results

Comprehensive, cyclic triaxial consolidated-undrained tests were also conducted on specimens of the sandfill to establish the increase of compressive strain as well as the build-up of pore pressure in the test specimens, in response to cyclic shear loadings. A summary of the cyclic triaxial test results is presented on Figure 28 (pulsating deviator stress ratio versus number of cycles to cause 5% compressive strain) and Figure 29 (Cyclic pore pressure versus axial strain).

SEISMIC ANALYSES

General

The seismic stability of a tailings dam depends on factors such as: the geometry of the dam section, the phreatic surface within the dam, the shear strength parameters for the material making up the dam section and its foundations, and the external forces caused by earthquakes. The stability analysis for static conditions is relatively simple to perform, and is normally based on limit-equilibrium methods of one kind or another. However, the inclusion of dynamic forces in the stability analysis adds appreciable complications, and, depending on the extent to which the soils would suffer significant shear strength loss under the earthquake loads, the applicable methods of analysis can differ. For soils that undergo little or no loss of strength under seismic loading, Seed (1979) in his Rankine Lecture indicated that a deformation type of analysis would normally suffice. For soils subject to significant loss of strength, both the selection of methods of analyses and the interpretation of the results require considerable engineering judgement. For tailings dams constructed of hydraulically-placed, sandfill materials of medium density, such as the Brenda Cycloned-Sand Dam, the potential for substantial pore pressure build-up and consequent loss of resistance to deformation in the saturated portion of the sandfill is of major concern. Current "state-of-the-art" methods of dynamic analyses for such structures involve the use of finite element procedures.

Finite Element Dynamic Analysis

The application of the finite element method to the analysis of a tailings dam requires special attention to the fact that, unlike a water retention dam, the tailings dam also retains, in its pond, loose, partially to normally consolidated, tailings materials. Upon the arrival of the initial earthquake shocks, these stored tailings are likely to liquefy, and this transformation of the tailings from solid to liquid state imposes a sudden additional thrust onto the tailings dam. The new force which is applied to the saturated dam materials under essentially undrained conditions, significantly alters the state of stresses in the dam at the onset of the earthquake.

Finite element dynamic analysis procedures for analyzing earth and rockfill dams and tailings dams have been described by Seed (1979) and Finn (1982) respectively. These solutions, which represent our best current "state-of-the-art" engineering knowledge on this subject are complex; involve complicated computer programs; require the input of a broad spectrum of engineering skills and experience; and are time consuming and expensive. A detailed discussion of such analyses is far beyond the scope of this paper and the brief outline presented following is only intended to highlight the steps taken for the analysis of the Brenda sand dam.

(1) Determine the static and dynamic properties of the soils comprising the dam, the upstream tailings beach, and the foundation.

(2) Subject representative samples of the embankment materials to laboratory tests simulating the combined effects of the initial static stresses and the superimposed cyclic stresses and determine their effects in terms of the generation of excess pore-water pressures and the development of strains.

(3) Determine the static, pre-earthquake stresses within the dam.

(4) Determine the time history of base excitation to which the dam and its foundation may be subjected by the design earthquake.

(5) Determine the dynamic response for the dam and its foundation to the seismic loading from Step 4 above and the resulting cyclic stresses and strains. (This step might need adjustments to compensate for the additional shear forces caused by potential liquefaction of the tailings pond).

(6) Evaluate the overall cyclic strain potential and performance of the dam.

(7) Carry out a post-earthquake, limit equilibrium analysis using: the effective friction angle for the sand; the effective normal stress on a potential failure surface which has been computed by subtracting from the total stress on the failure surface the sum of the following pore pressures: (1) the original static pore pressures established by the steady seepage condition; (2) the pore pressure rise caused by the pond liquefaction; (3) the pore pressure rise caused by earthquake shaking. In making this post-earthquake analysis <u>it is also assumed that the lower limiting value of the shear strength along the failure surface is defined by the steady-state undrained strength regardless of the pore pressure values calculated from the dynamic finite element analyses.</u> For the post-earthquake stability analysis, the tailings in the pond are assumed to remain in the liquefied state, thus applying the full hydrostatic pressure of tailings against the dam.

<u>Strain Potentials</u> - The general practice in evaluating finite element analyses for conventional water retention dams is to examine the strain potentials of saturated elements. Normally, strain potentials within the limit of 5% are considered acceptable. On that basis, the strain potentials estimated for the Brenda sand tailings dam are generally favourable. Figure 30 presents a comparison of induced cyclic shear stresses with those causing 0.5% and 5% strains along the base of the dam at station 49+50, the maximum section through the dam. Of all the sections examined, only station 28+00 displayed a small zone near the downstream toe where the 5% criterion was exceeded. However, at this limited section, the confinement provided by the unsaturated zone above and the compacted berm downstream, is expected to prevent excessive movements in the area. Consequently, the Brenda sand dam was considered to satisfy the normal strain potential criterion.

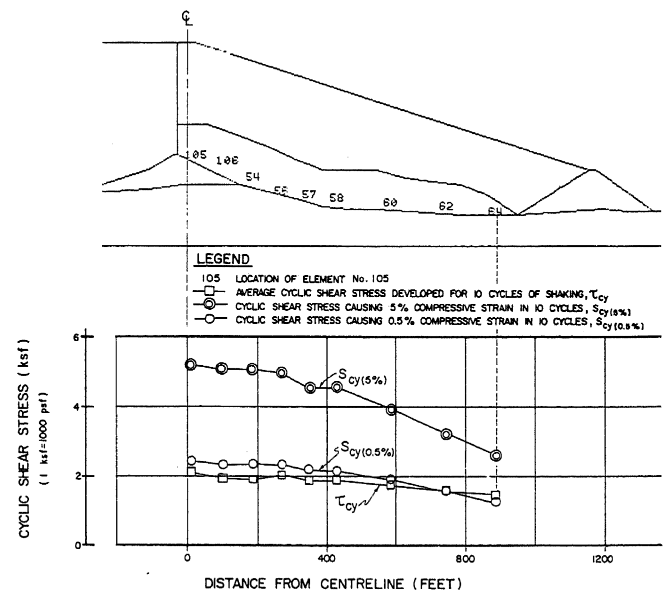

Figure 30
Comparison of Induced Cyclic Shear
0.5% and 5% Strain

<u>Post-Earthquake Stability</u> - The post-earthquake stability of the Brenda sand dam (Step 7 above) was determined using a computer program for a limit-equilibrium stability analysis, based on Janbu's method of slices. In the analysis, the excess pore pressures caused by the pond liquefaction and by the earthquake shaking, which are calculated from the finite element analysis, were added to the initial static pore pressures. Two conditions, long-term and end-of-construction, were analyzed.

For the long-term condition, the sand fill is assumed to be partially drained with the bottom 20 ft of sand fill fully saturated over all but the central gully section, there the assumed depth of saturation is increased to 50 ft. These saturated zones are assumed to taper off at the downstream toe of the dam. The tailings pond is assumed to be completely filled. Figure 31 illustrates these assumptions and presents the results of the post-earthquake stability analysis, for the long-term condition, at the maximum dam section, station 49+50.

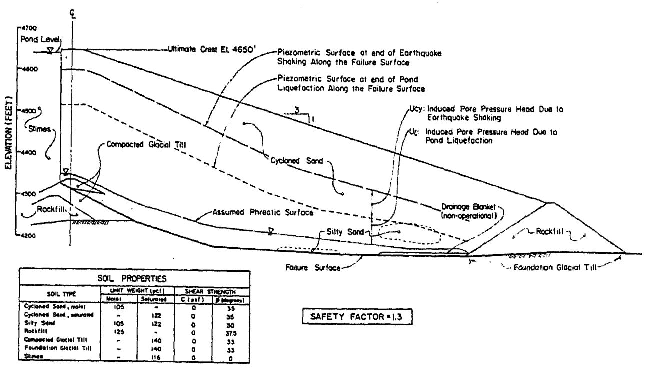

Figure 31
Post-Earthquake Stability Analyses
at Maximum Dam Section (49+50)

For the end of construction condition, the saturated zone at the base of the dam is assumed to be everywhere 60 ft thicker than assumed for the long-term condition due to the effect of the large volumes of water used during construction. Construction of the sand dam is assumed to be completed before the ultimate tailings pond level is reached. The analyses assume that the pond level is 50 ft below the crest of the completed dam. Figure 32 illustrates these assumptions and presents the results for the end of construction condition, at the maximum section, station 49+50. The beneficial effects of having the pond 50 ft lower are offset by the 60 ft higher phreatic line in the sand dam. The net result is no change from the factor of safety computed for the long-term condition.

Table III summarizes the safety factors obtained for the post-earthquake stability analyses for the three dam sections examined in detail for both the end-of-construction and long-term conditions. (Crest elevation 4650 ft.)

TABLE III

SUMMARY OF FACTORS OF SAFETY
POST-EARTHQUAKE STABILITY ANALYSES

Section Station	End of Construction Condition	Long Term Condition
28+00 (with Sand Berm)	1.1	1.1
45+00	1.2	1.2
49+50	1.3	1.3

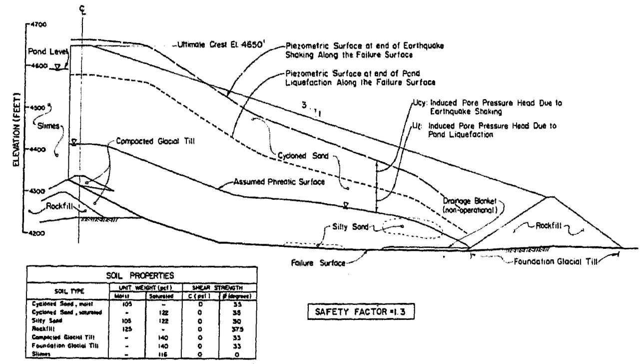

Figure 32
End of Construction Earthquake
Stability Analyses at Maximum Dam
Sections (49+50)

The safety factors determined for the post-earthquake condition are considered acceptable for this structure, particularly in view of the relatively conservative assumptions that have been made throughout the analyses. The more important of these assumptions are:

(1) The conservative nature of the design earthquake selected for the site. (Richter magnitude 6.5 with epicentral distance of 11 miles).

(2) The value of the design peak horizontal acceleration of 0.38 g, as determined using the attenuation relation proposed by Hasegawa et al (1981). (The Milne (1977) and Schnabel and Seed (1972) relations would result in a value of approximately 0.29 g).

(3) The tailings pond becomes fully liquefied at the onset of the design earthquake. This is a severe assumption that does not consider such things as: the dynamic properties of the tailings, the magnitude of the earthquake, and the interaction between tailings in the pond and the stronger fill materials in the dam.

(4) The conservative selection of soil parameters from the field and laboratory investigations.

(5) The conservative projections that were used to select both the end-of-construction and long-term piezometric levels within the sand dam.

(6) The neglect of any beneficial effects of aging and potential bonding of the cycloned sand.

Steady-State Strength Analysis

In addition to carrying out the finite element dynamic analysis described in the previous section, steady-state analyses were also made. These analyses were used to determine the "bottom-line" strength of the sand tailings dam. The concepts of the steady-state strength type of analysis have been described by Castro (1969). In effect, the steady-state undrained strength is the lowest possible strength which a given sand, in a contractive state, at a given density and under a given confining pressure could reach if it were subjected to an earthquake of sufficient magnitude and duration to force the sand to lose enough strength that it reaches the steady-state condition. In this type of analysis the steady-state undrained strength along the potential failure surface, as determined from laboratory tests, is used in a conventional, static, limit-equilibrium, stability analysis. The analysis must include the increase in the horizontal thrust due to the liquefied tailings in the pond. The inertial forces caused by the earthquake are not included as they are of very short duration and the dam is assumed capable of safely absorbing the horizontal movements that occur during the brief period that these inertial forces act on the dam.

The steady-state method is a quick and relatively inexpensive form of analysis and represents the worst possible conditions that could develop for a sand dam of low to medium density during an earthquake. At sites involving loose, saturated materials where the seismic risk is high, the method is considered to provide a reasonable assessment of the tailings dam's stability under earthquake loading. However, for medium-dense sands at sites of low to moderate seismicity the extent of seismic loading is usually insufficient to force the materials completely into a steady-state condition. For these conditions the steady-state analysis is considered to provide a conservative assessment of the lower bound of the dam's seismic stability. Sand tailings dams constructed using downstream methods of construction and hydraulic fill placement methods usually fall into this medium density category. If such a tailings dam successfully passes the steady-state analysis, more sophisticated analyses obviously are not required. On the other hand, if the steady-state analysis indicates that a problem in stability might exist, then the next stage in the evaluation would be to run either a simplified dynamic analysis or a finite element dynamic analysis.

DISCUSSION

This cycloned-sand, tailings dam, case history was prepared with three major objectives in mind. Briefly, these were:

(1) To introduce into the engineering literature the large amount of field and laboratory data that has been accumulated to date for the cycloned-sand tailings dam;

(2) To describe and illustrate both the construction procedures used and some of the construction problems encountered in building this sand dam; and

(3) To outline the results obtained from a "state-of-the-art" finite element dynamic analysis of the stability of the cycloned sand dam under earthquake loading.

The prime function of the field and laboratory studies was to provide the static and dynamic parameters required to carry out a "state-of-the-art" finite element dynamic analysis of th cycloned-sand dam. The paper limits itself t assessing the field and laboratory data to th extent necessary to determine these parameters In assessing these data, several significan items became apparent, including the following:

(1) Reconstituted test specimens of the cyclon ed-sand may be used to carry out conven tional, one-dimensional consolidation test that provide results which very closely cor respond to those obtained using undisturbed cycloned-sand samples. Moreover, thes results may be used to predict the increas in density with depth, of the cycloned-sand The results of such predictions agre closely with those densities measured fro "undisturbed" soil samples.

(2) Reconstituted test specimens may also b used to define the "steady-state band" fo the cycloned-sand.

(3) Fixed-piston, double-rodded, soil samplers using thin-walled sample tubes and ver careful sampling techniques may be used t recover relatively undisturbed, saturate soil samples from depths of up to 250 f and at distances of up to 60 ft below th water table.

(4) The standard penetration and static con data obtained to maximum depths of 250 f and 230 ft respectively, appear to b reasonably consistent and useful as a guid for assessing the insitu density of th cycloned-sand. In this regard, the stati cone data is particularly interesting as i provides a continuous record of cone resis tance and the energy applied at the tip i accurately known. However, further analy ses of the SPT, static cone and measure insitu densities is beyond the scope of th present paper.

The first two objectives of the paper are uncom plicated, as they only involve the presentatio of factual data and the description of event that occurred. The third objective involves complex subject that deserves some furthe discussion:

As indicated in the paper, conventional practic in assessing the results of finite elemen dynamic analysis on a conventional water storag dam is to assume that strain potentials of 5% c less indicate a satisfactory structure. Fo medium dense, cycloned-sand tailings dams, suc as Brenda, this criterion may not be adequate The cyclic triaxial tests on the Brenda cyclone sands indicated that very high pore pressure will develop at very low strains (Figure 29) Consequently, although the strain potential ma be well under 5%, the induced pore pressures i the tailings dam may be high and the dam ma have an unacceptably low, post-earthquake facto of safety.

Another factor that should be kept in mind whe carrying out a finite element dynamic analysi on a sand tailings dam, is that the most criti cal section in the dam is not necessarily th maximum section. For high sand dams founded c stiff foundations, the most critical section likely corresponds to some intermediate height of dam. The reason being that the high dam, having a long fundamental period has a low level of response to the high frequencies of the earthquake, whereas the lower sections, having a shorter period, may have a much higher level of response. This item, which is discussed by Finn (1982), proved to be a significant factor at the Brenda sand dam, where an intermediate-height, abutment section was found to be the most critical part of the dam.

In making the "finite element" post-earthquake, stability analyses described in the paper, the steady-state strength has been assumed in all instances to define the lower boundary or residual strength of the sand. Whenever the pore pressures, as determined from the finite element analysis became so high that the computed effective stresses became less than the steady-state residual values, they were disregarded and the steady-state values were used in the analysis. This approach is consistent with the concept of the steady-state undrained strength. It is interesting to note that if the design earthquake were large enough, the above condition would exist over the entire failure surface and the post-earthquake "finite element" analysis would become equal to a post-earthquake steady-state analysis.

The total movements of the cycloned-sand tailings dam that may occur during a major earthquake are expected to be appreciable. However, as a sand tailings dam is usually a massive structure, has mainly tailings rather than water against its upstream face, and provides a generous freeboard between the free water surface in the pond and the crest of the dam, it can absorb safely large settlements and downstream movements during an earthquake. Consequently, provided the sand tailings dam has an adequate post-earthquake factor of safety, it should be able to safely withstand the loadings of the design earthquake and maintain its integrity. This is considered to be the case for the Brenda Mines, cycloned-sand, tailings dam.

The current approach used by the writers' firm for evaluating the stability of a sand tailings dam when subjected to earthquake forces, involves the use of a three-stage process. Details of the method are described by Lo et al (1982). Briefly, the method starts with the simplest analysis and proceeds to the more complex and costly analyses only if required. In ascending order of cost and complexity, the three stages of dynamic analyses used are:

(1) Steady-State Strength Analysis, Castro (1969)

(2) Simplified Dynamic Analysis, Klohn et al (1978), Finn et al (1978), Grigg et al (1979)

(3) Finite Element Dynamic Analysis, Seed (1979), Finn (1982)

If the tailings dam successfully passes the steady-state analysis, the dam is obviously safe and more sophisticated analyses are not required. On the other hand, if the steady-state analysis indicates that a stability problem might exist, the next stage of analysis, a simplified dynamic analysis, would be carried out.

The SEISLOP (Grigg et al, 1979) simplified dynamic method of analysis is simpler and less costly to run than are the programs for the finite element analysis, while at the same time acknowledging the energy input levels from the earthquake. Moreover, it attempts to approximate pore pressure conditions that would fall in the gap between those corresponding to the fully drained static state and those associated with the fully developed, undrained steady-state.

Application of the finite element dynamic analysis is reserved for those major tailings dams where failure would pose a serious threat to life and property downstream and where the stage 1 (steady-state) and/or stage 2 (simplified dynamic) analyses have indicated that a more detailed review of the stability problem is required. The Brenda Mines, cycloned-sand, tailings dam falls into this category.

A recent paper by Klohn et al (1983) compares the computed, post-earthquake, factors of safety obtained for three separate sections through a sand tailings dam, using each of the three suggested stages of analysis. As might be expected, this comparison demonstrates that: the steady-state analysis provides the lowest factors of safety (most conservative); the finite element dynamic analysis provides the highest factors of safety; and the simplified dynamic analysis (SEISLOP) provides factors of safety that fall between these two extremes. These results are very encouraging and suggest that a two-stage analysis using only the steady-state and SEISLOP procedures might be adequate for all but the most unusual and/or critical sand tailings dams.

CONCLUSIONS

In conclusion, the author considers that:

(1) The extensive field and laboratory investigations carried out at this site have provided the basic data and soil parameters required to make "state-of-the-art" seismic analyses of the cycloned-sand, tailings dam.

(2) On the basis of these seismic analyses, the Brenda Mines Cycloned-Sand Tailings Dam, when completed to its ultimate height of 530 ft, can safely withstand the maximum credible earthquake for this site (Richter Magnitude 6.5).

ACKNOWLEDGEMENTS

The author wishes to express his sincere thanks to Messrs. C.H. Maartman and R.C.Y. Lo, who reviewed the original manuscript and provided several useful criticisms and suggestions. The contribution of Mr. F. Chow, who prepared the several figures that accompany the paper is also appreciated. Finally, the author is grateful to the management and operating staff of Brenda Mines for their cooperation and support throughout the investigations and analyses and for their permission to publish the data contained in this paper.

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