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# SEISMIC STABILITY ASSESSMENT OF THE MOA NICKEL TAILINGS FACILITY

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## SUMMARY

A detailed seismic stability assessment is presented for a tailings disposal facility at a mine operated by Moa Nickel S.A. The tailings are relatively soft and fine grained, and are impounded behind laterite fill embankments which have been constructed by a staged upstream method. The site is located in a region of moderate seismicity, but with the potential for large magnitude (8+) earthquakes. Ground conditions at the site vary significantly in terms of soil type, strength and depth to bedrock. Historically, some difficulties related to foundation conditions and embankment stability have been experienced at the tailings facility, but in recent years the mine has implemented measures to improve the performance of the facility progressively, including a detailed review of seismic stability.

The soils which underlie the site include a complicated network of soft organic and marine silts and clays and variable alluvial soils, including loose sands and sandy silts. A detailed geotechnical site investigation program has been carried out to define the characteristics and limits of these soils and of the tailings deposit. Field testwork included Standard Penetration Tests (SPT), Seismic Cone Penetration Tests (SCPT) and Shear Vane tests. Laboratory cyclic simple shear testing was performed on selected soil samples to investigate the cyclic resistance and dynamic strength and stiffness characteristics of the materials. Dynamic site response analyses and liquefaction assessments of the tailings and foundation soils have been completed using soil parameters derived from the testing. The potential for liquefaction or development of large strains due to seismic loading have been evaluated for each of the soil units. This was achieved using the results of the SCPT and a relationship between stress normalised cone tip resistance, friction ratio and cyclic resistance ratio. This technique allows the resistance to cyclic loading of all soil types ranging from clean sands to silts and clays to be examined. The post-cyclic strengths of the soils have been determined from the cyclic simple shear laboratory testing program.

Detailed seismic stability and deformation analyses for the tailings facility were completed at critical sections using both conventional limit equilibrium techniques and dynamic finite difference computer modelling. Design earthquakes representing an Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE) were considered. A computer model was developed to incorporate the reduction in strength and stiffness of the soils during seismic loading. Soils that are predicted to liquefy during the design earthquake were assigned a post-liquefaction residual strength based on the results of the cyclic simple shear laboratory tests. Appropriate remedial measures for improving the seismic resistance of the existing facility and the design for on-going staged expansions of the facility were defined. These include buttressing of certain perimeter embankments to prevent seismically induced large deformations that could lead to discharge of tailings or overtopping of the dam.

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## INTRODUCTION

The Pedro Sotto Alba Mine is located near the city of Moa in the province of Holguín in the southeastern region of Cuba, as shown on Figure 1. Limonite ore is mined and subjected to a pressure acid leach process to produce nickel and cobalt concentrate. Leach and neutralization residue from the metallurgical processing, referred to as acid leach tailings, are deposited in the active tailings facility. The relatively soft and fine grained tailings are impounded behind laterite fill embankments developed in stages by the upstream construction method.

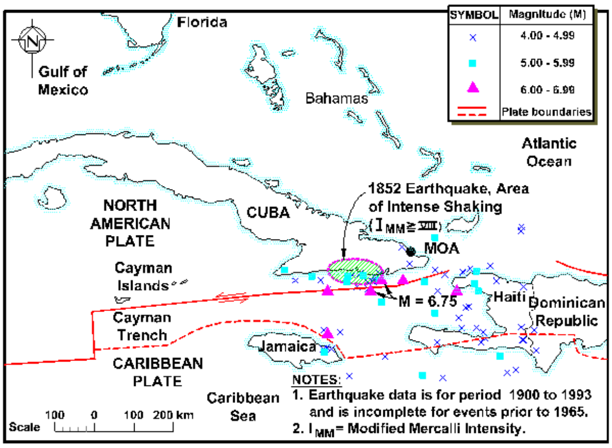


**Figure 1: Project location and regional seismicity**

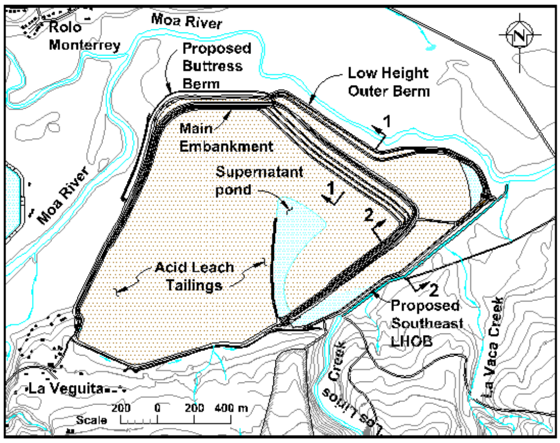


**Figure 2: Proposed site arrangement**

Ground conditions at the site vary significantly in terms of soil type, strength and depth to bedrock. Difficulties related to foundation conditions and embankment stability have occurred in the past at the tailings facility. However, in recent years the mine has implemented measures to improve the performance of the facility progressively. This has included a detailed seismic stability assessment, involving dynamic site response analyses, soil liquefaction potential and embankment deformation analyses. Both laboratory and field data were used to characterise the site conditions and provide material parameters for the analyses.

Rehabilitation of the facility was proposed in 5 phases over the period 1997 to 2001 with detailed design of each phase carried out prior to its construction. Each phase of construction generally includes buttressing of certain perimeter embankments to improve both the static and seismic stability of the structure, followed by further raising of the main embankment to provide the required tailings storage. Phase 1 included construction of a Low Height Outer Berm (LHOB) around the eastern side of the facility to contain an area inundated with tailings from past spills. Figure 2 shows the proposed arrangement at the end of Phase 5. Details of the rehabilitation plan for the tailings facility have been presented by Chalkley et al [1999].

## SEISMICITY

The site is located in a region of moderate seismicity with the majority of seismic events in the region associated with an east-west trending boundary between the Caribbean and North American Plates that extends from northern Honduras along the Cayman Trench between Jamaica and Southern Cuba. Historical seismic events in and around southeastern Cuba are included on Figure 1. Based on a seismic risk analysis completed for the site, a conservative maximum magnitude of 8.5 was adopted for the Maximum Credible Earthquake (MCE) with an estimated minimum source to site distance of about 70 km. For design purposes two earthquakes were considered, an Operating Basis Earthquake (OBE) of magnitude 8.0 which corresponds to the 1 in 475 year event, and a Maximum Design Earthquake (MDE) which is equivalent to the MCE. The tailings facility would be expected to function in a normal manner following occurrence of the OBE, while some minor damage would be acceptable from occurrence of the MDE, as long as the integrity of the facility is maintained and release of impounded tailings is prevented. The peak bedrock acceleration was estimated to be 0.12g for the OBE and 0.28g for the MDE.

## SITE CONDITIONS

The tailings facility is located on a low lying flood plain south of a meander in the Moa River and abuts against the coastal mountains to the south. The principal geological units in the vicinity of the facility consist of a complex sequence of Quaternary age sediments. These sediments include variable alluvial deposits ranging from fine sand and clayey silt typical of flood plain sediments, to silty sand and gravel with cobbles originating from past channels of local rivers and streams. In addition to the alluvial deposits, littoral (coastal) deposits of very soft to stiff organic clays and silts are present, as are very soft to firm marine clays and silts typical of deposition in marine environments. Generally, the sediments are stiffer and more competent to the west and southeast where alluvium is more predominant, and softer and weaker to the northeast of the site where organic and marine clays exist. Weathered ultramafic serpentine rock from the Upper Cretaceous age underlies the Quaternary deposits. Colluvial and residual laterized soil deposits, formed from the weathering and leaching of the parent serpentine bedrock, underlie the south and southeast perimeters of the site.

Field investigations carried out at the site included geotechnical drilling with Standard Penetration Testing (SPT), Seismic Cone Penetration Tests (SCPT) and Shear Vane testing. Data collected from these investigations were used to confirm the range of foundation conditions at the site, characterise the various soil types and provide data for estimating soil strength and stiffness parameters for use in dynamic response and deformation analyses. The stratigraphy through the northeastern and southeastern areas of the facility are shown by cross sections 1 and 2, respectively, on Figure 3. The locations of these cross sections are included on Figure 2.

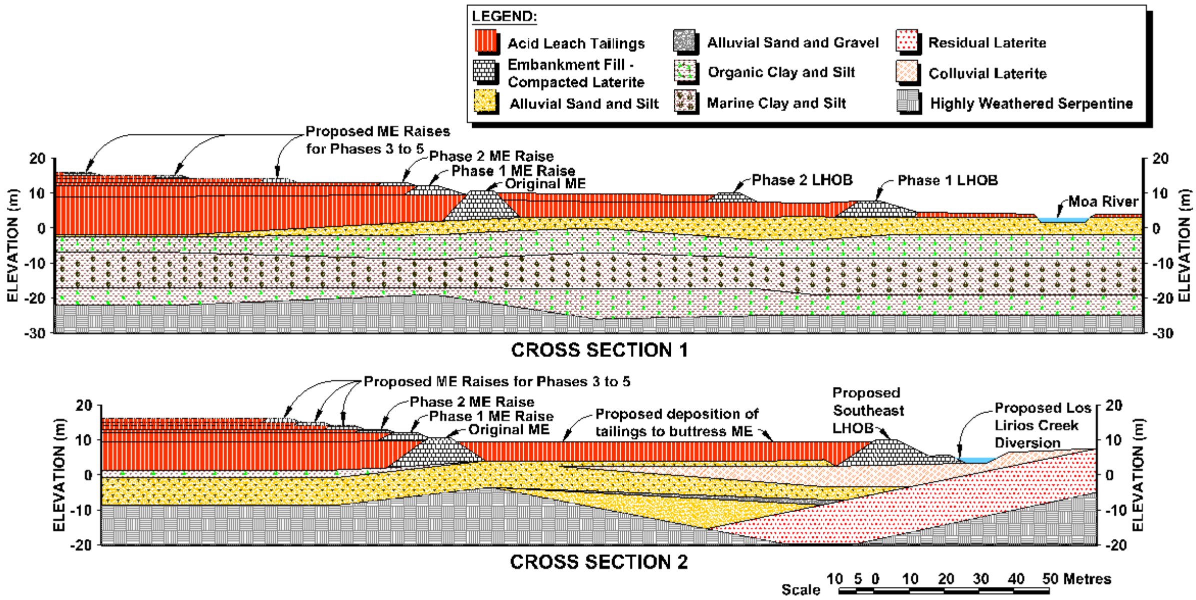


**Figure 3: Cross sections through the tailings facility**

**(including remedial measures to the end of Phase 5 construction)**

## LABORATORY TESTING

The objective of the laboratory testing program was to obtain detailed information on the liquefaction potential of the tailings and various foundation materials, and also their strength and stiffness parameters after cyclic loading. A series of cyclic simple shear (CSS) tests was conducted on undisturbed samples of tailings and foundation soils, including alluvial soils, organic silts and clays and marine silts and clays. For each material, the tests were carried out for a range of cyclic shear stress levels. CSS testing is believed to most closely simulate seismic loading conditions at the tailings facility.

One of the tests on the alluvium was carried out at a relatively low cyclic stress ratio of about 0.15, and terminated after 35 cycles. This is approximately the number of significant cycles that would be imposed by the magnitude 8.5 MDE event. This enabled the post-cyclic characteristics of the alluvium to be determined where liquefaction is not initiated during cyclic loading.

Figure 4 shows the CSS results for a typical sample of fine alluvium. The relationship between cyclic stress ratio and number of cycles to liquefaction and also the resulting normalised cyclic resistance ratio ($CRR_1$) are presented on Figure 4(a). The laboratory $CRR_1$ is defined as the cyclic resistance ratio for liquefaction at 15 cycles of uniform loading (representing an earthquake magnitude of 7.5) at a vertical effective stress of 100 kPa [Olsen, 1997]. The estimated value of $CRR_1$ for this sample was 0.205. Particle size analyses on the fine alluvium indicated that the fines content ranged from about 30 to 60 percent.

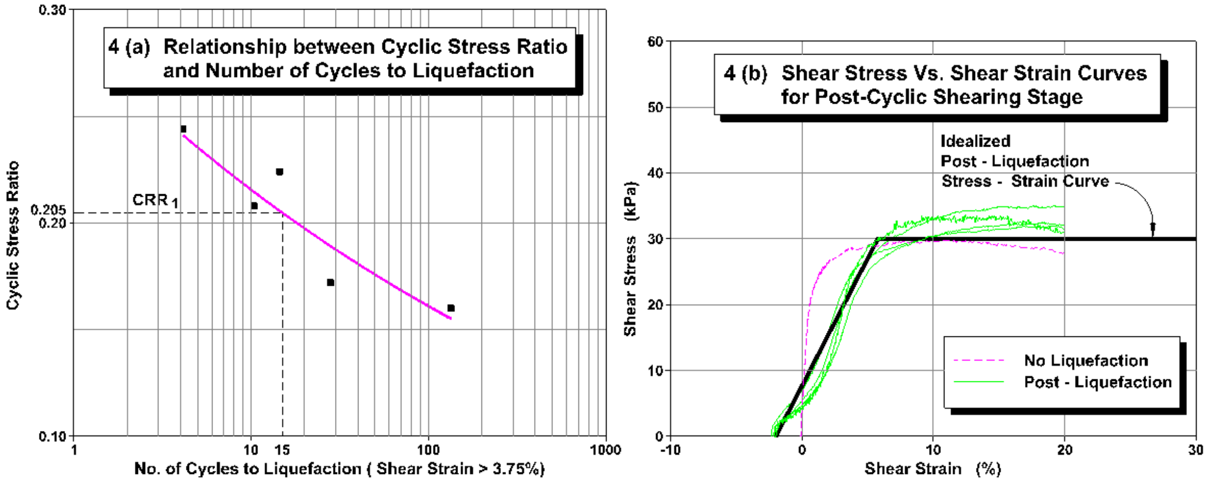


**Figure 4: Cyclic simple shear tests on fine alluvium (effective confining pressure = 100 kPa)**

The shear stress-strain curves from the post-cyclic shearing stage of the CSS tests are shown on Figure 4(b). The residual strength of the alluvium ranges from about 30 to 35 kPa. Included on this figure is the post-cyclic stress-strain curve for the alluvium sample that was not liquefied. As expected, the stiffness of this sample, although likely to be significantly reduced from its initial stiffness, is much higher than the post-cyclic stiffness of the other alluvium samples that liquefied during cyclic loading.

For the tailings, $CRR_1$ values of 0.198 and 0.172 were estimated for two different samples. Actual $CRR_1$ values for the tailings are likely to vary across the facility and with depth, depending on the age and degree of consolidation. The post-liquefaction (residual) shear strength of the tailings is approximately 40 kPa.

For the organic and marine silt and clay samples, cyclic loading was terminated after a maximum of 35 cycles. One static simple shear (SSS) test was also carried out on both clays to determine their strength characteristics prior to earthquake loading. Comparison of the results from the static and cyclic tests provides an indication of the reduction in stiffness and strength, if any, experienced by these clays due to cyclic loading. The CSS testing indicated that cyclic loading does not significantly reduce the strength of the organic and marine silts and clays. However, for tests where a cyclic stress ratio approaching 0.3 was applied, the post-cyclic stiffness was significantly reduced. This reduction in stiffness represents the softening of the silts and clays due to the development of large strains during cyclic loading. The undrained strength of clays is typically expressed as the undrained strength divided by vertical effective stress, signified as the $c_u/s_v'$ ratio. For both the organic and marine silts and clays the $c_u/s_v'$ is typically about 0.3.

## SITE RESPONSE ANALYSES

The liquefaction potential of the various soils was carried out by performing dynamic site response analyses to determine the cyclic stress ratios induced by the design earthquakes and by comparing them to the cyclic resistance ratios of the soils. Cyclic stress ratios were computed at selected vertical profiles across the entire tailings facility using the program SHAKE. Appropriate earthquake acceleration time history records were used, together with selected parameters that define the stratigraphy and the dynamic characteristics of the soil profile.

Maximum shear modulus (stiffness) values for the tailings and foundation materials were estimated using the shear wave velocity data collected during the SCPT site investigation program. At those locations where it was not possible to advance the SCPT through the full depth of alluvial materials to the underlying weathered serpentine, available SPT data from adjacent or nearby drill holes was used to estimate shear modulus values. Calculated values of maximum shear modulus ranged from very low values of less than 7,000 kPa in the soft organic and marine clays (shear wave measurements as low as 60 m/sec were recorded during the site investigation program), to more than 300,000 kPa in the very dense sands and gravels located on the northwest side of the facility. The underlying weathered serpentine was assigned a shear modulus of approximately 250,000 kPa, based on shear wave velocity measurements and SPT blow count data recorded at several drill hole locations across the site.

The plasticity of soils has a large influence on their response to seismic loading. Appropriate shear modulus reduction versus shear strain and damping ratio versus shear strain relationships were selected for each of the materials depending on measured plasticity indices obtained from laboratory testwork on representative samples. Relationships for soils with a Plasticity Index (PI) of 5 to 10 were used for the low plasticity tailings. For the foundation materials the selected relationships were dependent on the average PI for each location analysed. The plasticity of the alluvial soils varies across the site and also with depth, therefore, relationships ranging from cohesionless sands to medium plasticity soils (PI ranging from 20 to 40) were adopted. For the organic and marine silts and clays relationships for medium to high plasticity clays (PI between 20 to 80) were adopted. The highly weathered serpentine was assigned shear modulus and damping ratio relationships for an average PI in the range of 20 to 40.

Due to the uncertainties inherent in defining the characteristics of large magnitude earthquakes, a suite of five earthquake time-history records was generated for both the OBE and MDE events. These were developed to represent the typical range in ground motion characteristics (frequency content, amplitude and duration) for large magnitude events at intermediate to large epicentral distances. Each of the earthquake time-history records were scaled to the appropriate peak acceleration for the OBE and MDE events.

The analyses showed that the ground response varies greatly across the site, depending on the nature and depth of the foundation soils. The two particularly notable findings of the analyses were as follows: the alluvial soils along the southeastern edge of the facility are capable of transmitting very high ground motions; and, the soft organic and marine clays beneath the northeastern part of the facility attenuate the earthquake ground motion and are unable to transmit large accelerations to the overlying alluvium and tailings.

## LIQUEFACTION ASSESSMENT

The resistance of soils to cyclic loading and potential liquefaction is represented by the cyclic resistance ratio (CRR). Earthquake loading is defined as the average induced cyclic shear stress ratio (CSR). The factor of safety against liquefaction (or large strains for clays) is determined by dividing CRR by CSR.

In general, granular soils comprising saturated sands and silts are considered to be potentially liquefiable, whereas clayey soils with high fines contents are not liquefiable. Although not liquefiable, soft clays can accumulate large strains during an earthquake if the average earthquake induced shear stresses approach the static strength. Therefore, for clay soils the CRR represents an “equivalent liquefaction resistance.” For these analyses, soils with a calculated factor of safety of less than 1.1 were considered to liquefy, or exhibit large deformations in the case of clay soils.

In order to determine which soils are potentially liquefiable, the CRR values were calculated over the same vertical profiles as the CSR values estimated from the SHAKE analyses. Profiles of CRR were estimated using the SCPT data recorded during the site investigation programs. The SCPT independently measures tip stress (cone resistance) and sleeve friction resistance which, in combination, can be used to estimate the normalised cyclic resistance ratio ($CRR_1$). This technique requires stress normalisation of the cone resistance to a standard vertical effective stress of one atmosphere (approximately 100 kPa), and calculation of the friction ratio (sleeve friction divided by cone resistance). The $CRR_1$ determined from field data is the cyclic resistance ratio of the soil for a magnitude 7.5 earthquake and a vertical effective stress of 100 kPa. The relationship between normalised cone resistance, friction ratio and $CRR_1$ presented by Olsen [1997] was used.

A typical $CRR_1$ profile through the fine alluvial soils is shown on Figure 5. Also included on Figure 5 is the approximate sampling depth and estimated $CRR_1$ of the alluvium sample retrieved adjacent to this SCPT location. The average $CRR_1$ value of 0.205 estimated from laboratory CSS tests on this sample is in good agreement with the $CRR_1$ values from the SCPT data at the same depth. Despite the large variation in soil conditions, this agreement between laboratory and SCPT derived data was also found for other soil samples.

Typical values of $CRR_1$ for the organic and marine silts and clays are included on Figure 6. The average values of $CRR_1$ are 0.33 for the organic silts and clays and 0.27 for the marine silts and clays. The results of the CSS tests on these silt and clays showed that they accumulated large strains when subjected to cyclic stress ratios close to these average $CRR_1$ values. This is to be expected as the average $CRR_1$ values are close to the estimated undrained strength ratios ($c_u/s_v$') of the silts and clays.

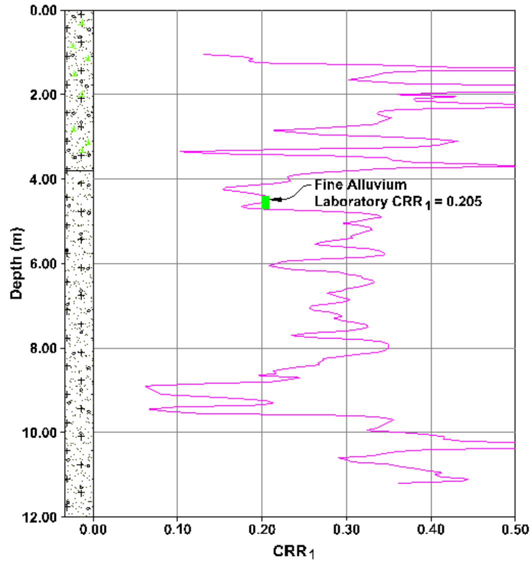


Figure 5: SCPT derived $CRR_1$ profile through alluvial sands and silts

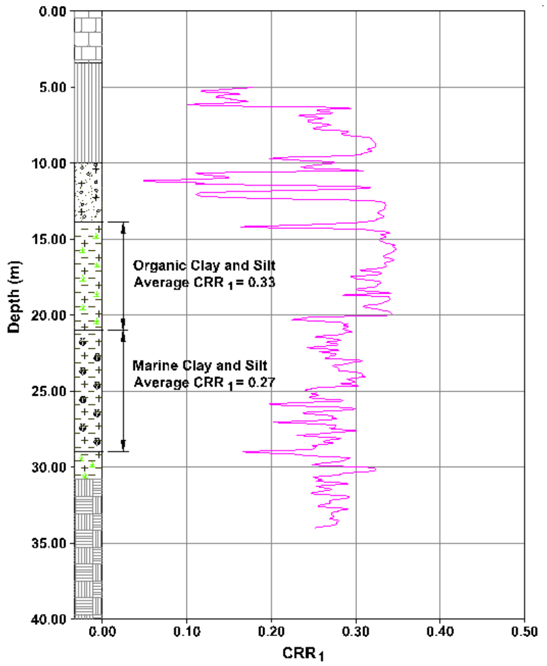


Figure 6: SCPT derived $CRR_1$ profile through organic and marine clays and silts

Estimated values of $CRR_1$ were corrected for both vertical effective stress and design earthquake magnitude. Magnitude scaling factors of 0.89 for the magnitude 8.0 OBE and 0.80 for the magnitude 8.5 MDE were adopted. Appropriate relationships between effective confining pressure and correction factor ($K_s$) were used depending on the soil type [NCEER, 1997].

The site response and liquefaction analyses demonstrated that the variety of soil types, ranging from soft silts and clays to very dense sand and gravel alluvium, behave very differently during earthquake shaking. The soft silts and clays that underlie much of the northeastern side of the facility (Section 1) have a low strength but attenuate strong ground shaking. Although these silts and clays are not liquefiable, they will strain soften during earthquake shaking with the potential to experience large deformations. The alluvial sands and silts located along the southeastern side of the facility (Section 2) are capable of generating very large ground motions, resulting in complete liquefaction of these soils and overlying tailings.

Liquefaction is predicted to occur in the tailings and alluvial foundation soils along the north and southeast sides of the facility in the event of an OBE. Liquefaction is also predicted to occur in these materials on the east and west ends of the LHOB for the MDE. None of the tailings or foundation soils are predicted to liquefy under the northeast side of the facility (Section 1), due to the attenuation of earthquake ground motions in the underlying soft organic and marine clay foundation soils. However, deformation of these soft soils is expected.

## EMBANKMENT STABILITY

Embankment stability was initially analysed using limit equilibrium methods to determine the minimum factor of safety for critical sections identified by the liquefaction assessment. A number of cross sections around the perimeter of the facility were examined. The area along the northeast side, near cross section 1, was found to be the most critical for static stability due to the occurrence of a deep soft organic and marine clay foundation. Prior to rehabilitation measures, the stability in this area was found to be marginal. This condition was improved by buttressing the Main Embankment (ME) along the northeast side with a Low Height Outer Berm (LHOB).

Post-liquefaction stability analyses were performed to determine if any of the cross sections had the potential for a critical failure in the event of liquefaction of the tailings and alluvial foundation soils. This type of failure would result in very large embankment deformations. Appropriate residual strengths based on the laboratory CSS testing were used to represent liquefied materials. For factors of safety less than about 1.1, the potential for a critical failure during, or shortly after, earthquake loading was assumed. The analyses indicated that the existing southeastern corner of the facility (Section 2) is susceptible to a critical failure if liquefaction of the tailings and foundation soils is initiated by earthquake shaking.

Based on the findings of the liquefaction assessment and post-liquefaction stability analyses, the seismic deformation of those cross sections with the potential for critical failure were examined using a total stress dynamic method and the finite difference stress-strain program FLAC [Cundell, 1995]. Of particular interest were sections through the northeastern side of the facility which is underlain by the very soft silts and clays (Section 1), and along the southeastern edge (Section 2) where the potential for a critical failure was identified.

The pre and post liquefaction strength and stiffness parameters were selected using the field and laboratory test data. For the pre-liquefaction phase of earthquake loading, appropriate soil stiffness values were determined using the maximum shear moduli and appropriate shear modulus reduction factors selected from the results of the SHAKE analyses. For the liquefiable soil layers, the SHAKE analyses were also used to estimate the time during the earthquake when the cyclic stresses would initiate liquefaction. Complete liquefaction of the entire soil layer was assumed at the same time. This method of triggering liquefaction was considered appropriate given the large cyclic stresses that were predicted to be generated very quickly during application of the design earthquakes. For the liquefiable tailings and alluvial soils, the strength and stiffness parameters were reset to post-liquefaction values and the dynamic analysis continued. Idealised post-liquefaction stress-strain relationships that define the residual strength and stiffness properties of the liquefied material were used. Included on Figure 4(b) is a typical idealised stress-strain relationship for the liquefiable alluvium. For the soft organic and marine silts and clays a reduced stiffness, based on the laboratory CSS tests, was assigned to model softening due to the large strains imposed by earthquake loading.

For future seismic deformation analyses proposed for the tailings facility, a more sophisticated method of liquefaction triggering is to be adopted. This method tracks the cumulative shear stresses generated within each soil "element" of the FLAC model during earthquake loading. Liquefaction is triggered within an element when sufficient cycles of shear stress have accumulated. This method allows liquefaction to occur in the most susceptible areas first, followed by progressive liquefaction of other areas as earthquake loading continues. Details of this method are presented by Beaty and Byrne [1999].

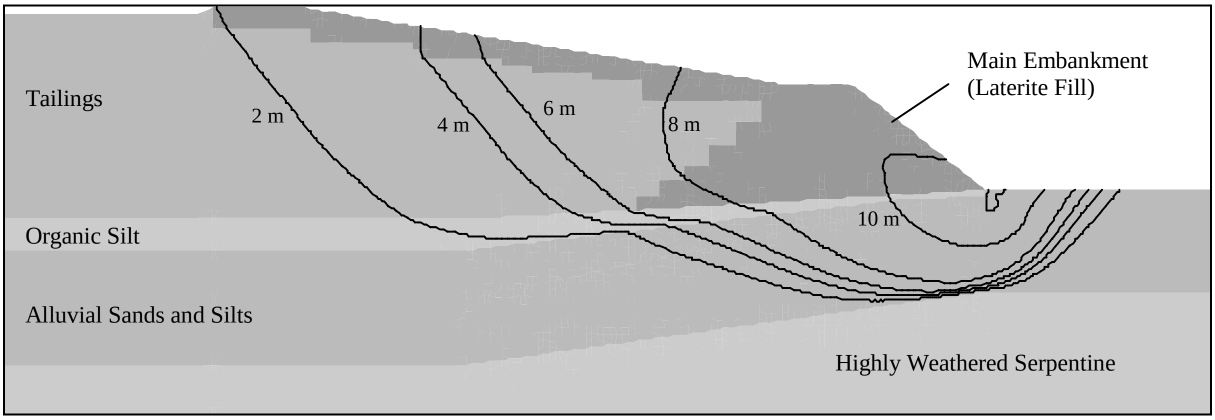


**Figure 7: Contours of maximum displacement from seismic deformation (FLAC) analyses - Section 2 (Phase 5 configuration without any tailings buttressing)**

The seismic deformation analyses indicated that for the southeast side at Section 2, large deformations of several metres will occur if there is no buttressing, possibly leading to release of solids and supernatant water under both the OBE and MDE events. Figure 7 shows contours of the predicted deformations for Section 2 without any buttressing (note that the deformed shape of the embankment has not been shown for clarity).

For the LHOB and ME along the northeast side of the facility, predicted deformations under the MDE event were significant but tolerable. The maximum deformations were estimated to be approximately two metres horizontally and one metre vertically, and are predominantly related to large strains within the soft marine clay layer. For the OBE event estimated maximum displacements were about 0.6 metres. These displacements can be accommodated by the operating freeboard requirements of the ME and LHOB.

## REMEDIAL MEASURES

Based on the findings of the seismic stability assessment appropriate remedial measures are being implemented. To improve stability and remove the potential for large seismically induced deformations along the southeastern edge of the facility, buttressing of the Main Embankment is required. This will be achieved by extending the Low Height Outer Berm along the southeastern edge on more competent foundation soils (colluvial laterite), to provide containment so that tailings can be deposited against the ME to provide the required buttressing (see Figure 3, Section 2). Additional seismic deformation analyses were carried out to determine the extent of buttressing required to reduce the seismically induced deformations to acceptable values. The analyses indicated that embankment deformations will be significantly reduced to less than 0.5 metres by these remedial measures and can be satisfactorily accommodated by the operating freeboard requirements.

Tailings were used for buttressing of the embankments at the facility because they exhibit a relatively high residual strength. In addition, the use of tailings offers certain advantages including cost-effective provision of additional tailings storage and satisfying critical schedule requirements for construction and operations.

Further work is still required to complete the seismic stability assessment for the tailings facility. Areas requiring further work include the north and northwest sides of the facility. It is expected that placement of a buttress berm along the toe of the existing embankment in these areas will satisfy seismic stability concerns.

## ACKNOWLEDGEMENTS

We wish to take this opportunity to thank the directors of Moa Nickel and Knight Piésold for the opportunity to present this paper.

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