A Unique Experience with Liquefaction Assessment of Impounded Brown Coal Ash

Radin Espandar [1], Mark Locke [2] and James Faithful [3]

1. *Principal Dams Engineer, GHD*
2. *Principal Dams Engineer, GHD*
3. *Technical Services Manager – Mine, ENGIE*

Brown coal ash has the potential to be a hazard to the environment and local communities if its storage is not well managed. The risk of releasing contained ash from an ash tailings dam due to earthquake induced liquefaction is a concern for mining lease holders, mining regulators and the community. Ash tailings dams are typically raised by excavating and compacting reclaimed ash to form new embankments over slurry deposited ash, relying on drying consolidation and minor cementation for stability. Understanding the post-earthquake behaviour of the brown coal ash is necessary to assess the overall stability of an ash tailings dam during and after seismic loading events. A particular concern is the seismic motion may break cementation bonds within the ash resulting in a large reduction in shear strength (i.e. sensitive soil behaviour) and potential instability. There is limited information available for black coal ash however, brown coal ash has different properties to black coal ash and no known work has been carried out to date in this area.

The dynamic and post-earthquake behaviour, including liquefaction susceptibility, of the brown coal ash was studied, specifically for Hazelwood Ash Pond No. 4 Raise (HAP4A) in Latrobe Valley, Victoria. In this study, different well-known methods for liquefaction susceptibility, including the methods based on the index parameters, the cone penetration test (CPT) and the cyclic triaxial testing, were used and the results were compared.

It was found that the impounded brown coal ash is susceptible to liquefaction and /or cyclic softening. Triggering of the liquefaction or softening was assessed based on the results of cyclic triaxial test. In this methodology, the relationship among axial strain (ε_a), Cyclic Stress Ratio (CSR) and number of uniform cycles (N_{equ}) was determined based on the triaxial test results. Then, a site-specific CSR was determined using the ground response analysis. The CSR and number of uniform cycles (N_{equ}) for each ash layer was calculated and added to the ε_a-CSR- N_{equ} graph to determine the expected axial strain during an MCE event. It was found that the calculated axial strain for the ash embankment and ash deposits during site specific Maximum Credible Earthquake (MCE) are less than the axial strain of the ash material required for triggering of liquefaction and the brown coal ash in HAP4A does not liquefy and/or soften the material during an MCE event. Also it was found that the insitu tests which break the cementation between particles (such as CPT) does not provide accurate results on triggering or sensitivity.

Keywords: *Brown Coal Ash, Liquefaction, Tailings, Hazelwood, Cyclic Triaxial Test.*

Introduction

Australia is the fourth-largest coal producing country in the world. Unlike many other states, the major coalfields of Victoria contain brown coal, a fuel generally unsuitable for combustion without specialized technology. At present, most electricity in Victoria is generated by burning brown coal in thermal power stations in the Latrobe Valley. A by-product of that process, ash, is typically transported as a slurry to an ash tailings dam for permanent disposal.

Ash tailings dams can pose a significant threat to the community and the environment, due to their size, design and the nature of the materials they store. Ash tailings dams are typically raised by excavating and placing reclaimed ash material to form new embankments over slurry deposited ash, similar to an upstream raise of a tailings dam. Ash embankments are required to be stable for any likely loads during operation as well as post closure, including earthquake. Liquefaction is a major cause of damage during earthquakes. Classical liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore-water pressure and reduced effective stress (Marcuson 1978). Typically, the engineering assessment of liquefaction is undertaken in two steps: 1) Identification of susceptibility to liquefaction, and 2) Assessment of triggering (initiation) of liquefaction during an earthquake. There are several methods that have been developed for screening of soils susceptible to liquefaction. However, the mechanical properties of brown coal ash differ from those of soil and therefore some of the screening methods applicable for soils may not provide reliable results when used to assess brown coal ash.

Brown coal ash

Particle size analyses indicates that Latrobe Valley ash is quite variable, a typical size distribution of Hazelwood brown coal ash is shown in Figure 1, which shows Hazelwood ash tends to be a silt material (MH). The size distribution appears similar to that of black coal ash. The particle density of the fly ash was measured as 2520 kg/m^3. Brown coal ash has a very high moisture content (generally higher than 100%) and high porosity (more than 50%).

The comparison of the shapes of the black coal and brown coal fly ash particles, using Scanning Electron Microscope (SEM) techniques, shows that while particles of the black coal fly ash are generally spherical in nature, the shape of the individual particle of the brown coal ash varies widely from spherical particles to rectangular shapes (Figure 2).

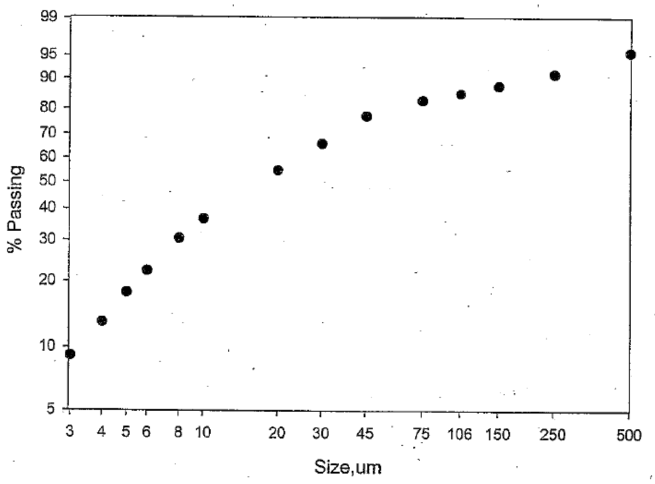
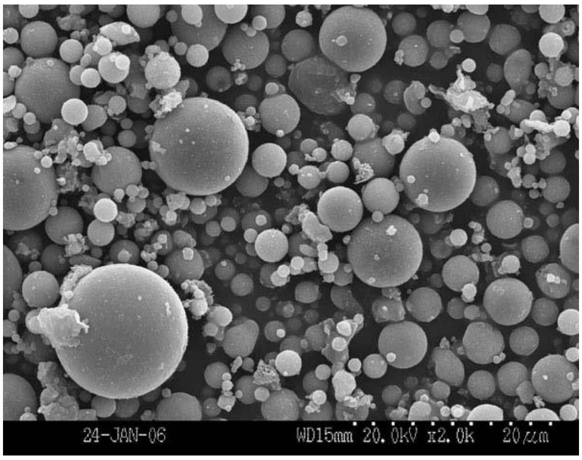
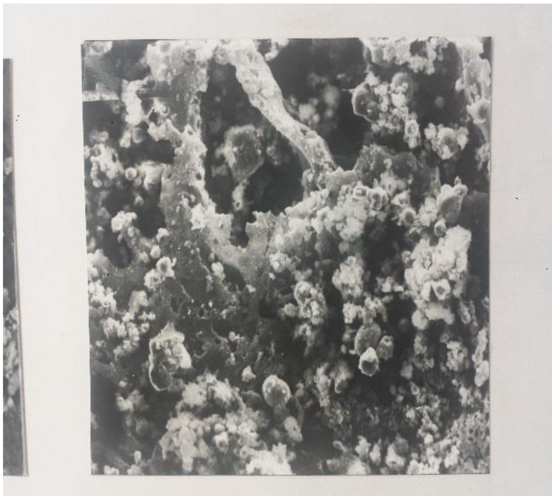


Figure 1. Size distribution of fly ash from Hazelwood power station (Sigma 2003)





a. Black coal ash X600 b. Brown coal Ash X2000

Figure 2. Microphotographs of a. Typical Black coal ash and b. Yallourn Brown coal Ash

Geotechnical investigation,

A comprehensive geotechnical investigation has been carried out in Hazelwood Ash Pond No. 4 Raise (HAP4A) between 2012 and 2015, including No. 17 CPT probing, No. 6 deep boreholes, No. 88 shallow hand augers (to 3.0 m depth) with vane shear tests at 0.5 m intervals in the selected boreholes, one dilatometer probe (DMT) to determine in-situ deformation modulus and shear wave velocity, and No. 23 undisturbed samples for cyclic triaxial test by pushing thin walled tubes into the ash. In addition, a considerable amount of lab tests has been undertaken. Details of the investigations are not provided but specific aspects are referred to within the paper.

Susceptibility to liquefaction

Most published literature on liquefaction assessment is based on natural materials (sand and silt) rather than ash. The very limited literature on liquefaction assessment of ash suggests that the different behaviour of deposited ash may mean that normal methods are not appropriate.

ANCOLD (2016) suggests several approaches to assess the susceptibility of soils to liquefaction including: 1) Geology and age of deposit and 2) Soil gradation, plasticity and moisture content including interpretation based on Cone Penetration Test (CPT) data. ANCOLD recommends using at least two and preferably more of the methods stated. These approaches were employed for initial screening of liquefaction susceptibility for the impounded brown coal ash in Hazelwood Ash Pond No. 4 Raise (HAP4A).

Screening methods

In accordance with ANCOLD (1998, 2012, 2016) mine tailings with non-plastic or very low plasticity fills are most susceptible to liquefaction. In addition, the tailings are a recent man-made deposit, which more likely to be susceptible to liquefaction. The impounded brown coal ash in HAP4A is mostly categorised as high plasticity silt (MH), and hence, is not traditionally considered liquefiable. The main concern is that ash is pozzolanic and develops cementation (diagenesis) over time. If this occurs relatively early after deposition then the ash can form a stiff matrix skeleton that appears strong and results in low compressibility but locks in a high porosity skeleton. The high porosity skeleton can collapse on strong loadings with sufficient magnitude to break the cementation, such as strong earthquakes. This would result in increased pore pressure, which may lead to liquefaction, considering very high moisture content of the impounded ash.

Liquefaction susceptibility based on soil gradation, index parameters and moisture content

There is a range of published literature discussing liquefaction susceptibility of fine grained soils with no clear consensus. The most widely used methods are modified Chinese criteria, (Wang 1979) (Seed and Idriss 1982), Seed et al (2003), Boulanger and Idriss (2006) and Bray and Sancio (2006). ANCOLD (2016) suggests that the modified Chinese criteria is not reliable and should not be used, the other methods were considered for the brown coal ash.

Seed et al. (2003) proposed new criterial for the assessment of liquefaction susceptibility of fine grained soils, and this is now recommended by ANCOLD (2016). As shown in Figure 3, soils with PI≤12, LL ≤37 and moisture content to liquid limit ratios (mc/LL) >0.8 fall into Zone A and are considered to be potentially liquefiable. Soils with 12<PI≤20, 37 < LL≤ 47 and mc/LL >0.85 fall into zone B, are classified to be moderately susceptible to liquefaction and need further testing. Soils lying out of these boundaries are not considered to be susceptible to classical liquefaction although they should be checked for potential sensitivity. Seed et al. (2003) criteria applies to soils with fines content (FC) ≥20 for PI>12 and FC≥35 for PI<12. Existing results for HAP4A are plotted relative to these limits in Figure 3. Based on this methodology, the ash is deemed as non-liquefiable.

ANCOLD (2016) proposed Boulanger and Idriss (2006) as another method to identify liquefiable soils. The method use PI < 7 to identify soils exhibiting “sand-like” behaviour that are susceptible to liquefaction and PI ≥7 to identify soils exhibiting “clay-like” behaviour that are judged to not be susceptible to liquefaction. The method suggests that there is a significant difference in the cyclic response of fine-grained soils due to minor changes in PI for low plasticity soils. Clay-like soils may soften due to the loss of effective stress resulting from the build-up of positive excess pore pressures, but the term liquefaction is reserved for sand-like soils. The high PI of these ash samples suggests that “clay-like” behaviour can be expected. For soils with clay-like behaviour, laboratory testing is recommended.

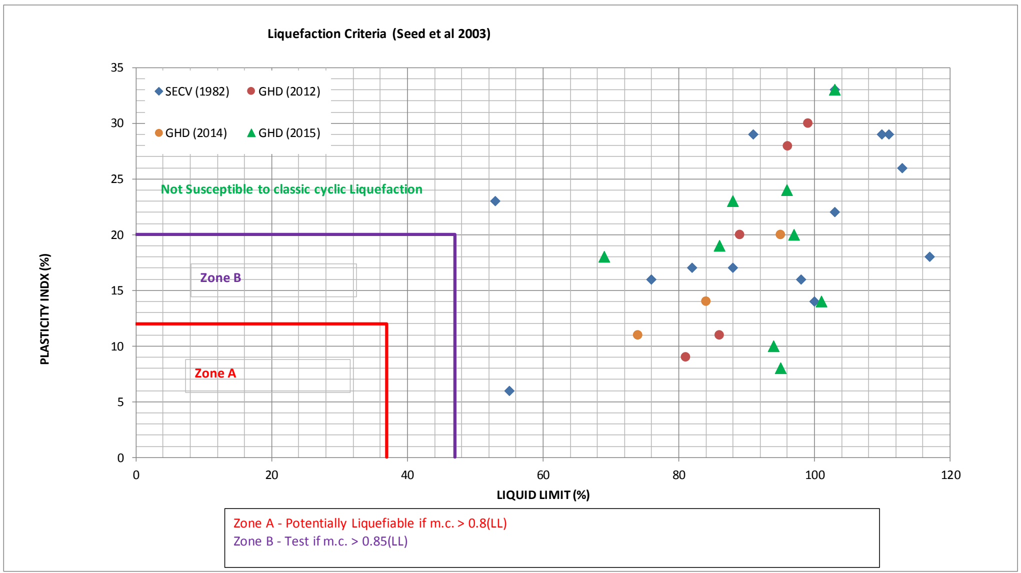


Figure 3. Results of liquefaction susceptibility based on Seed et al (2003)

The method proposed by Bray and Sancio (2006) was also considered for this study. Based primarily on cyclic testing of “undisturbed” silts and clays specimens of the 1999 Adapazar earthquake in Turkey, Bray and Sancio (2006) found that soils with Plasticity Index (PI) < 12 and moisture content to liquid limit ratios (mc/LL) > 0.85 were susceptible to liquefaction, as evidenced by a dramatic loss of strength resulting from increased pore water pressure and reduced effective stress. According to Bray and Sancio (2006) soils in the range 12 < Plasticity Index (PI) < 18 and moisture content to liquid limit ratios (mc/LL) > 0.80 are moderately susceptible to liquefaction and they suggest further laboratory testing to confirm; whereas soils having PI>18 are considered to be non-liquefiable under low effective stress levels. Note that there may be cases where sensitive soils with PI > 18 undergo severe strength loss because of earthquake induced straining, so the proposed criteria should be applied with engineering judgement. Existing results for HAP4A are plotted relative to these limits in Figure 4. Based on this methodology, parts of the ash deposits are potentially liquefiable. The method has not been proposed in ANCOLD (2016).

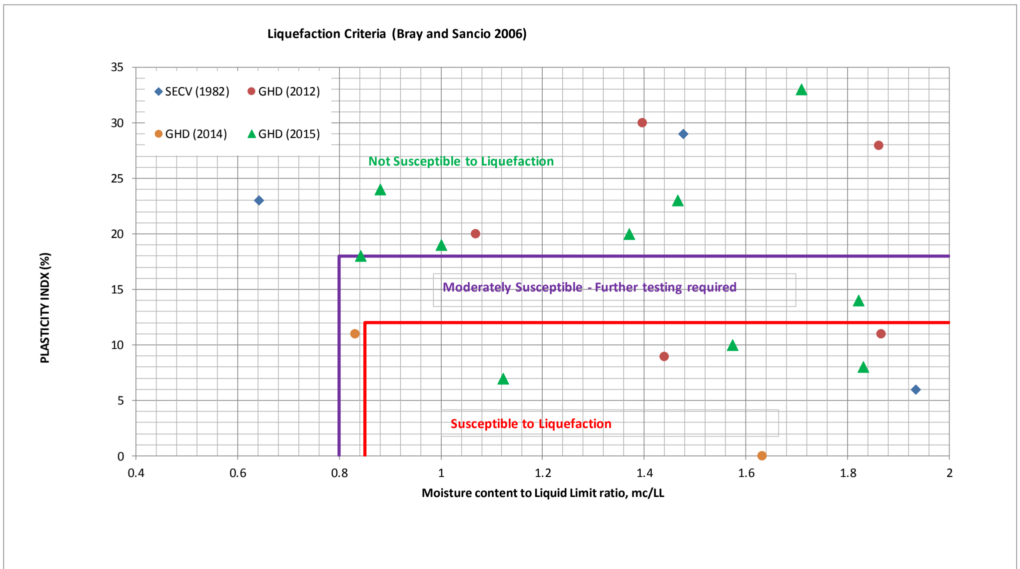


Figure 4. Results of liquefaction susceptibility based on Bray and Sancio (2006)

Liquefaction susceptibility based on Cone Penetration Test (CPT)

ANCOLD (2016) proposed CPT based methods to assess the susceptibility to liquefaction, especially considering Robertson and Wride (1998) Soil Behaviour Type (SBT) index I_c and Robertson (2010). Figure 5 was prepared for one of the CPT probes through the crest of an ash embankment and into the underlying slurry deposited ash to show the susceptibility to liquefaction based on Robertson and Wride (1998) and Robertson (2010). The red points indicate the data points which are susceptible to cyclic liquefaction (Zones A1 and A2), while the looser soils in Zone A2 are more susceptible to strength loss and flow liquefaction. The green points (Zones B and C) correspond to fine-grained, predominantly claylike soils for which it is more appropriate to use procedures similar to, or modified from, those used to evaluate the undrained shear strength (such as field vane shear tests, CPT and shear strength tests on high quality thin-walled tube samples). The soils in Zones B and C are susceptible to cyclic softening, but the softer soils in Zone C can be more sensitive and susceptible to potential strength loss and possible flow liquefaction. Robertson (2010) recommends that for moderate to high risk projects, undisturbed sampling and testing be carried out for soils in Zones B and C to determine the soil response. .A significant number of points are red (located in Zone A1 and Zone A2) showing the ash in HAP4A is susceptible to liquefaction. The other CPT results, carried out in HAP4A (as a sample for the brown coal ash), show almost the same results, i.e. this method predicts the brown coal ash is susceptible to liquefaction.

Typically, the CPT cannot be expected to provide accurate prediction of soil type based on physical characteristics but provide a guide to the mechanical characteristics of the soil, or so-called SBT. The CPT method suggests that susceptibility to liquefaction depends on the SBT index (I_c). Comparing the interpretations of the CPT probes with the adjacent borehole logs in HAP4A shows that the SBT of liquefiable soils were mostly predicted as sandy material while the boreholes generally encountered high plastic silt (MH). While variability of the brown ash is expected both vertically and laterally, there was a clear trend of the CPT predicting sandy soil behaviour while fine grained materials were observed in adjacent boreholes.

We believe that that cementation between ash particles may affect CPT based interpretation of soil behaviour. When CPT cone drives into the ash material, a high tip resistance is required to break the cementation but low frictional resistance applies to the CPT sleeve after the cementation is broken. This mechanical behaviour is typically interpreted as sandy material in the CPT probing, i.e. silty sand to sandy silt. These observations suggest that typical CPT liquefaction assessment for soils is not appropriate for the ash and other methods should be considered.

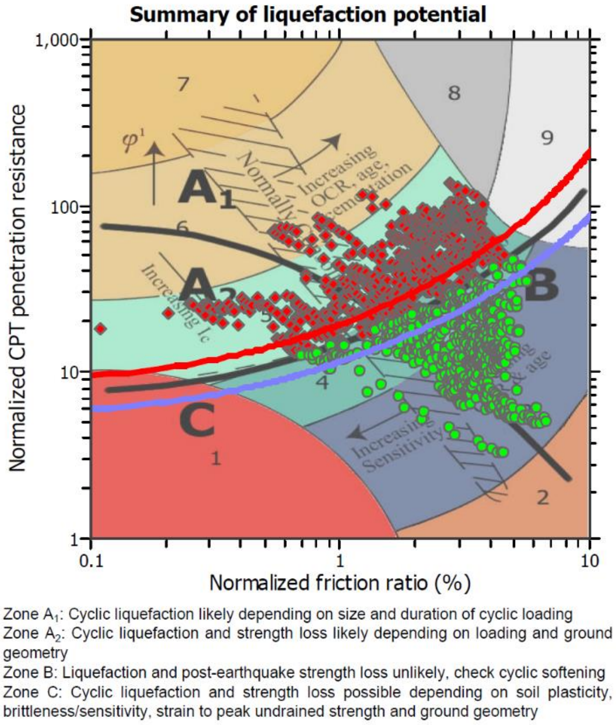


Figure 5. Results of liquefaction susceptibility for a sample CPT probe at crest of ash embankment in HAP4A

Conclusion on susceptibility of brown coal ash to liquefaction

As recommended by ANCOLD (2016) the study considered several methods for assessing susceptibility to liquefaction and identified some inconsistencies. In general, the ash is high plasticity silt, which is not typically considered susceptible to liquefaction. The Seed et al (2003) method categorised the impounded brown coal ash as non-liquefiable. In contrary, the Bray and Sancio (2006) method identified that the material is susceptible to liquefaction. The CPT results indicate the material is susceptible to liquefaction, and also identified soils susceptible to cyclic softening if they are sensitive. However, comparing the CPT index parameter with the nearby borehole show that the CPT interpretation of the material is not accurate.

Based on this uncertainty, the brown coal ash may be susceptible to liquefaction and/or strain softening and further assessment was required. The assessment of triggering of liquefaction/softening during the design earthquake is the next step of the liquefaction assessment.

Assessment of triggering potential

Although a large amount of literature exists on cyclic resistance of sands and fine-grained soils, little has been done to determine the liquefaction potential of black coal ash and almost none on the brown coal ash. There are two general types of approaches available for assessment of triggering potential: use of laboratory testing of “undisturbed” samples, and use of empirical relationship based on correlation of observed field behaviour with various in-situ “index” tests. The use of laboratory testing is complicated by difficulties associated with sample disturbance during both sampling and reconsolidation. Definition of the design earthquake is the first step in the quantitative assessment of triggering of liquefaction.

Design Earthquake

Selection of design earthquake

ANCOLD (2012) recommends Maximum Credible Earthquake (MCE) should be used for post closure studies but taking into account expected long-term properties of the tailings. The consequence category of the pond was determined as “Significant”. Therefore, earthquake events of 1: 10,000 AEP were determined as the MCE event, which is one order of magnitude higher than recommended MDE earthquakes by ANCOLD (2012).

Seismic hazard assessment

A site-specific seismic hazard assessment was undertaken (SRC 2014). A magnitude-distance deaggregation plot was produced for the site for a range of frequencies and a return period of 10,000 years. A frequency range of 33.3 to 100 Hz was applied. The results show that the dominant source of ground motion, which would be most destructive to a structure with a short period at Hazelwood is likely to be from magnitude ML 6.5 to ML 7.5 event at a distance of approximately between 16 to 20 km from the site. The hazard from the prominent fault is from Yarram Fault with an approximate length of 52 km and an assumed slip rate of 40 m/Myr with a closest distance of approximately 16 km from the site itself.

The study provided response spectra and time histories for three different earthquakes at the basement rock based on an average shear wave velocity (V_{S30}) of 750 m/s, representing the bedrock at considerable depth. Available data from the 1980-05-25 Mammoth Lakes ML 6.3 earthquake, the 1983-07-22 Coalinga D ML 5.7 earthquake, and the 1989-10-18 Loma Prieta ML 6.9 earthquake were scaled to produce the time histories.

Ground response analysis

A ground response analysis was undertaken using the time histories at bedrock level to determine the seismic response within the soil profile to the top level of ash in HAP4A. The latest version of computer code "SHAKE2000", which is the most widely used program to date for computing the seismic response of horizontally layered soil deposits, was employed for this analysis. Dynamic response analysis using SHAKE is not recommended for non-level ground such as embankments, however, the HAP4A raise is a low height embankment on a horizontal ash surface and the one-dimensional model was considered acceptable in this case. Also, in this case there are approximately 600m of sedimentary sequences overlying 'bedrock' and several studies in the Latrobe Valley have identified significant attenuation of earthquake motion through these sedimentary materials. It was considered more appropriate to carry out this site specific ground response analysis to determine the earthquake response for the ash storage, rather than rely on the typical V_{s30} characterisation used in PSHA which only relates to the shear wave velocity of the top 30m of foundation.

Plots of the Peak Ground Acceleration (PGA) against depth are presented in Figure 6. The graphs present the amplification and attenuation of the PGA through different layers of soil profile. The maximum PGA (among six time histories) was attenuated from 0.95g at the bedrock level to 0.35g in HAP4A at the ash surface. The magnitude of the MCE was calculated as 7.2, which is inside the range proposed by the de-aggregation plot.

The Ground Response Analysis predicts the dynamic shear stresses in various layers of the ash during the earthquake. This data was used for estimating the Cyclic Stress Ratio (CSR), which is the ratio of uniform cyclic shear stress ($\tau_{cyc} = 0.65\ \tau_{max}$) to consolidation stress (σ'_{vc}), i.e. CSR = $\tau_{cyc}/\ \sigma'_{vc}$. The CSR was then compared with the cyclic resistance ratio (CRR) of the soil determined by laboratory testing as described below.

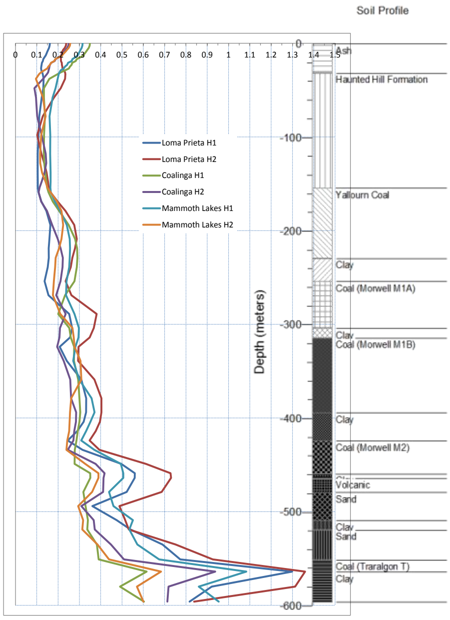


Figure 6 Peak Ground Acceleration (PGA) through soil profile of HAP4A

Liquefaction triggering based on CPT

The potential for liquefaction triggering using CPT results based on the earthquake movement predicted by the ground response analysis described above, i.e. 0.35g and M7.2 is presented in Figure 7. Significant zones of ash show factor of safety against liquefaction less than 1.0, which suggests that the material would liquefy or strain soften during the assumed earthquake. However, given the uncertainties noted earlier in the CPT interpretation, the results of CPT liquefaction assessment should be confirmed by laboratory tests, such as soil grading.

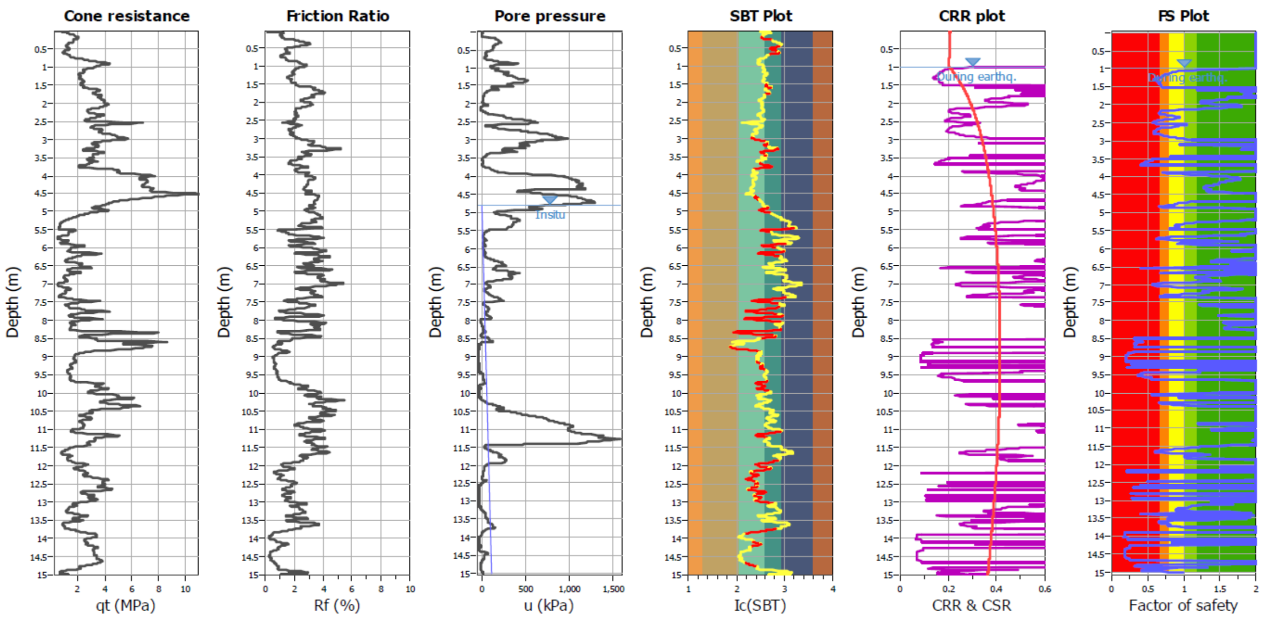


Figure 7. Factor of safety against liquefaction for a CPT probe on the crest of ash embankment in HAP4A. (0-4m: Ash Embankment, 4-15 Ash deposits in HAP4)

Liquefaction triggering based on cyclic triaxial test

ANCOLD (2012) has not proposed cyclic tests as one of the methods for liquefaction assessment for soils or ash. ANCOLD (2016) proposed the cyclic simple shear and cyclic triaxial tests to provide input to the geological model. An experimental program along with an analytical study was conducted to assess the triggering of liquefaction of impounded brown coal ash and evaluate the post-earthquake shear strength.

The cyclic strength of the impounded ash material was measured using standard cyclic triaxial test according to ASTM D5311-11 (2011), and the post-cyclic shear strengths were evaluated using consolidated-undrained triaxial shear test.

The undrained cyclic resistance ratio of soil (CRR) against the triggering of liquefaction was estimated in this study by measuring the number of uniform cycles to reach a limit shear strain for a range of applied shear stresses. In this approach, single amplitude shear strain criteria of 2.5%, 3.75% (Moriwaki et al 1982), 5% (Ishihara et al 1980) and 7.5% (Poulos et al 1985) are often used. In the current study, a shear strain of 2.5% (or axial strain of about 1.75%) was used to define triggering of liquefaction (or significant deformation of a non-liquefiable fine-grained material). It is noted that this is lower than adopted by many researchers, it was thought that this lower limit would represent a degree of conservatism, i.e. predicting liquefaction at a lower applied stress, and was appropriate given the variability in the ash samples.

Sample preparation

The University of Wollongong (UOW) were engaged to carry out the cyclic undrained triaxial tests. The sampling tubes were 450 mm in length and 50 mm in diameter. Further dimensioning of samples was necessary, in order to prepare test specimens of 100 mm in length and 50 mm in diameter. The specimens were first statically extruded from the tubes and were subsequently split across the length using a soil lathe to meet the required dimensions. Extrusion from sampling tube was carried out at extremely low rate of movement (0.05 mm/minute) to minimise disturbance. Owing to the specimens being composed of intercalating layers of fine-grained material and finer ash, the recovery percentage of viable test specimens for testing was about two specimens per tube. Distinct dissimilarly between some specimens was noted. Figure 8 presents a photo of five specimens taken from ash deposits beneath the ash embankment prior to the test. The laboratory noted that the samples appeared in good condition although no specific testing for degree of disturbance was undertaken.

Test set up

A series of cyclic undrained triaxial tests were performed on undisturbed specimens consolidated isotropically (K_0 = 1.0) to the estimated in-situ vertical stresses. This was an attempt to test the samples in their insitu state using the ‘Recompression’ technique. Samples were subjected to six different cyclic stress ratios (CSR = $\sigma_{(cy)}$ / 2 σ'_{vc} where, σ'_{vc} = vertical effective stress and $\sigma_{(cy)}$ =single amplitude cyclic axial stress, ASTM D5311-11, 2011) of 0.2, 0.4, 0.5, 0.6, 0.75, and 1.0, respectively.

Double amplitude cyclic stresses were applied to each specimen at a frequency of 1 Hz (i.e. possible earthquake conditions in Australia), and the accumulated axial strains and the excess pore water pressure reached at the end of number of load cycles (N) = 10, 30, and in a few cases N = 100, were recorded. This was followed by a strain-controlled monotonic loading phase under undrained conditions to measure the post-cyclic shear strength. The monotonic loading was carried out at constant rate of displacement (0.05 mm/minute).

Figure 8. Five specimens taken before commencing testing retrieved from 6 m to 6.45 m depth tubes.

Results

Specimens tested at higher CSR of 0.75 and 1.0 reached near-failure and complete failure conditions respectively, during the application of cyclic loading. In general, specimen tested at CSR ≤ 0.75 exhibited axial

strain less than 1.75 %, thus indicating that they are not liquefiable, using the criteria specified for liquefaction. These results proved beyond doubt that CSR = 1.0 were critical in view of potential to liquefaction, even at a lower number of loading cycles (e.g. $N \leq 10$).
The values of excess pore water pressure at the 1.75% axial strain, i.e. liquefaction triggering axial strain, and the maximum recorded at the end of the cyclic testing, were presented in Table 1.

Table 1. Excess Pore Pressure at onset of liquefaction

Test Description	Number of Cycles to reach to 1.75% axial strain	Excess pore pressure ratio at 1.75% axial strain during cyclic test	Maximum measured excess pore pressure ratio during cyclic test
CSR=0.75, N=10	5.2	0.64	0.80 at 10 cycles
CSR=0.75, N=30	3.3	0.60	0.83 at 4.5 cycles
CSR=0.40, N=100	97	0.84	0.86 at 100 cycles

The main outcome of the cyclic triaxial test is the graph presented in Figure 9. To develop this figure, the number of cycles required to reach a range of axial strains for tests at different CSRs was determined. For example a test at CSR = 0.4 measured strains of 0.25%, 0.5% at N=1 and 8 respectively. All test data was plotted as the open symbols in Figure 9, and lines of best fit for data at the selected axial strains were plotted. The scatter in this data indicates the degree of variability in these samples. Note that the solid symbols in Figure 9 relate to predicted CSR from the ground response analysis and are described below.

As mentioned earlier $\varepsilon_a = 1.75$ % was assessed as the axial strain required to trigger the liquefaction. Figure 9 suggests this occurs at 12 cycles for CSR = 0.6 or 100 cycles for CSR = 0.4. Idriss (1999) suggests $N_{equ} = 11$ ('N_{equ}' is the equivalent number of uniform stress cycles for each layer of the model at 65% of the peak cyclic stress ratio) for the M7.2 design earthquake for sands. This number for clayey material is higher, as was estimated to be about 30 for clays for an earthquake magnitude of 7.5. The appropriate N_{equ} for intermediate soils has not been defined. This study adopted the SHAKE results to determine 'N_{equ}' using the conversion factors proposed by Seed et al. (1975). This number varies between 1.7 and 10.5 for different earthquakes.

Methodology to apply the test results to the field impounded ash

The liquefaction assessment was carried out according to the methodology proposed for black coal ash (Zand et al 2007) with a few modifications as follows:

1. The shear stress results are obtained from SHAKE ground response analysis from each layer of ash in the model.
2. The peak shear stress (τ_{max}) for each layer is determined.
3. 'N_{equ}', the equivalent number of uniform stress cycles, is determined based on Seed et al. (1975) by converting an irregular shear stress history to a uniform cyclic shear stress with amplitude of 0.65 τ_{max} using the results of SHAKE analysis.
4. The effective stress for each layer is calculated (σ'_{vc}).
5. The CSR is calculated through normalising the uniform cyclic shear stress ($\tau_{cyc} = 0.65\ \tau_{max}$) by the initial effective vertical stress, i.e. CSR = $\tau_{cyc}/\ \sigma'_{vc}$. This is taken as the 'field' CSR.
6. The 'field' CSR was then divided by a factor of 0.9 to estimate the 'equivalent laboratory' CSR. This is an empirical correction factor included to correct for multidirectional shaking conditions in a real earthquake compared with unidirectional laboratory results (Seed et al 1975). Note that a factor of 0.96 is commonly used for clays but the lower value was adopted for this study.
7. The data for each ash layer is plotted in the CSR vs N_{equ} graph obtained from cyclic triaxial test. The amount of axial strain is determined.
8. If the axial strain is less than the triggering strain for liquefaction (determined through cyclic triaxial test), i.e. $\varepsilon_a = 1.75\%$, then the layer is assessed as non-liquefiable. Otherwise, the layer is liquefiable for the assumed earthquake.

Assessment of the liquefaction triggering of impounded ash in HAP4A based on cyclic triaxial test results

In addition to the laboratory data, Figure 9 also presents the results of the assessment of liquefaction triggering based on the ground response analysis. The impounded ash was divided in 15 layers in the SHAKE model to provide a more accurate assessment. Two horizontal components of three nominated earthquakes were applied in the analyses and the condition, which provides higher cyclic stress (CSR) for each layer, was plotted along with the equivalent number of uniform cycles N determined from the SHAKE analysis. Figure 9 then allows estimation of the expected axial strain for each layer and comparison with the strain based liquefaction trigger of 1.75%.

The key observation of Figure 9 is that all predicted stress and earthquake cycle conditions (solid points) are below the 1.75% axial strain adopted to represent triggering liquefaction. Therefore, the brown coal ash is considered as non-liquefiable during MCE events. However, some of the ash layers in the HAP4A are predicted to experience major axial strain (up to 1.7%) during the MCE. This means that the ash will not fully liquefied but possibly lose part of its strength during MCE. This understanding is important for the post closure stability of the ash embankments.

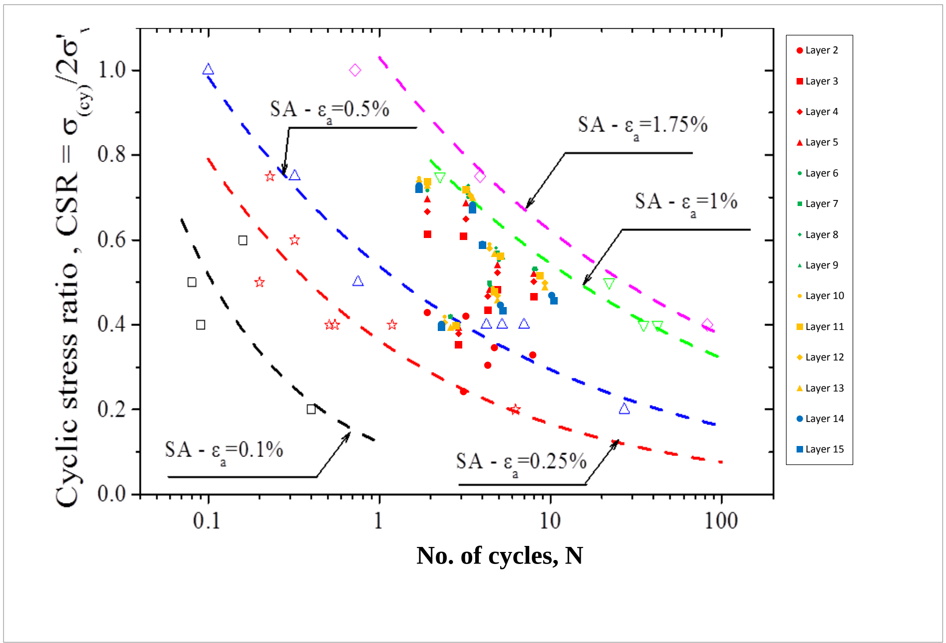


Figure 9 ***Results of triggering of liquefaction of impounded brown coal ash in HAP4A for MCE events (GHD 2015)***

Conclusions and Recommendations

Conclusions

A liquefaction assessment on the impounded brown coal ash was undertaken for Hazelwood Ash Pond No. 4 Raise (HAP4A). The ash material would be considered susceptible to liquefaction using older assessment methods. However, the impounded ash is a high porosity material, with cementation between the particles and demonstrating high sensitivity. There was a concern that breaking the cementation between particles during a strong loading may increase the pore pressure, which consequently may lead to liquefaction or cyclic softening. The susceptibility to liquefaction or cyclic softening was assessed based on geology of deposits, soil gradation, index parameters and moisture content. The following conclusions have been derived:

- Generally, mine tailings are a recent man-made deposit with non-plastic or very low plasticity fills are susceptible to liquefaction. The impounded brown coal ash in HAP4A is mostly categorised as high plasticity silt, hence less likely to be susceptible to liquefaction, but still might be susceptible to cyclic softening.
- Different methods are available to assess the susceptibility to liquefaction of the fine-grained materials. Two methods were used in the current work, i.e. Seed et al (2003), and Bray and Sancio (2006). The

former method predicts that the ash is not susceptible to liquefaction, whereas the latter method predicts liquefaction is possible. All methods suggest the results should be checked by laboratory testing.
- The soil behaviour type estimated from CPT probing suggests that the impounded brown coal ash is susceptible to liquefaction or cyclic softening.

Although there were inconsistencies in the results of assessment using different methods, it was assumed that the brown coal ash is susceptible to liquefaction and/or strain softening and therefore the assessment of triggering liquefaction was undertaken.

There are two general approaches available for assessment of triggering, use of in-situ index testing (such as CPT), and use of laboratory testing. The following results have been obtained from the triggering assessment:
- CPT based assessment suggests that the impounded ash material is liquefiable and/or strain softened. However, comparing the CPT results with the adjacent borehole logs shows that the prediction of the soil behaviour type in the CPT often did not match the observed material. This is due to cementation of the ash, which increases the tip resistance with a reduction in friction ratio. Thus, the CPT results for the impounded ash may be unreliable.
- A program of cyclic triaxial testing was undertaken at a range of stress conditions to examine the dynamic behaviour of the ash. Sufficient testing was undertaken to produce curves of predicted axial strain for varying cyclic stress ratio (CSR) and number of cycles (N). For example, the results suggest that the impounded ash would reach the axial strain limit selected to define the onset of liquefaction at 10 cycles for CSR = 0.6 or 100 cycles for CSR = 0.4
- The results of ground response analysis show that the earthquake at the site would be attenuated from the deep bedrock through the sedimentary sequences to the surface. The amount of attenuation was found to be different for different earthquake time histories.
- The ground response analysis was used to estimate the CSR experienced in different layers of the impounded ash during the assumed MCE events. This applied CSR and number of earthquake cycles from the ground response analysis were compared with the combined laboratory test results to estimate axial strains. These were all sufficiently below the adopted strain limit for the onset of liquefaction (considering the equivalent number of uniform cycles) and therefore, it was concluded that liquefaction would not trigger during MCE events.

The result of the current study is the impounded brown coal ash is liquefiable/considerable strain softened at high stresses or number of dynamic cycles, but liquefaction/ full loss of strength would not be activated during the maximum credible earthquake at HAP4A.

Recommendations

Based on the results of the study the following items are recommended:
- ANCOLD (2016) proposed Bray and Sancio (2006) for estimation of the effects of cyclic softening of non-liquefied clays and plastic silts, but not as a method to assess susceptibility to liquefaction. It is recommended that ANCOLD (2016) consider including this method in the final guideline.
- CPT investigations for ash or similar material with cementation between particles should be confirmed with adjacent sampling (eg. boreholes). If the CPT index parameter does not match with the observed material then liquefaction assessment should not be undertaken solely using CPT data.
- It is recommended that cyclic testing be used as one of the methods for assessment of liquefaction triggering for these unusual materials.

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