Design of a non-liquefiable sand shear key to improve the stability of an overburden waste dump constructed over a tailings storage facility

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ABSTRACT: The Fort Hills Dump (FHD) Stage 3 is an in-pit overburden waste dump under construction at the Syncrude Canada Ltd. (Syncrude) Aurora North Mine. The dump is located on top of a tailings sand and fines deposit, 70 m in thickness, with a footprint of 2.5 km^2. The design height of the dump is 50 metres. To provide a suitable foundation for dump construction, a track-packed sand cap of 10 m thickness was constructed on top of the tailings deposit.

A key concern with the FHD was the performance of the dump as the underlying tailings consolidated. Cracks could develop in the base of the dump, which could be infilled with water from the surrounding topography and the consolidating tailings beneath. This could result in a soft zone at the base of the dump which might threaten overall stability. Hence, a shear key was incorporated in the design to provide additional shear resistance and self-healing against cracking, and to limit the extent of dump base softening towards the dump toe. The shear key was constructed of mechanically placed and compacted tailings sand and was located where the dump was potentially most vulnerable to cracking and softening.

Another concern was that material placed within the shear key could become saturated as the dump settled. As such, the shear key needed to remain dilative over the expected stress ranges. To assess the compaction requirements (i.e. minimum initial dry density) for the shear key to remain dense, laboratory testing was completed to estimate the critical state line (CSL) for the tailings sand and to predict its performance under load.

1 INTRODUCTION

The Aurora Fort Hills Dump (FHD) at the Syncrude Aurora North Mine serves as a primary waste storage area for overburden materials removed during mining. The dump was constructed in multiple stages, with Stage 3 located on top of the Aurora East Pit Northwest (AEPN-W) in-pit tailings storage facility (TSF). While overburden waste dumps are common throughout the oil sands, and several TSFs have been capped, this would be the most significant waste dump to be constructed on an infilled tailings pond. Construction of the dump on top of a TSF posed several challenges for the design team including:

- Total and differential settlement of the dump and underlying tailings
- Slope stability of overburden and potentially liquefiable tailings
- Softening of clays along the base of the dump due to water ingress
- Proximity to active in-pit deposition areas
- Space constraints around the dump footprint.

To mitigate these challenges, several major design defenses were developed, including a track-packed sand cap over the TSF, active drainage during construction of the dump, a higher specification exterior shell zone, a step over into an adjacent area, incorporation of an inverted shear key into the design of the FHD and construction controls and specifications for the initial placement of the dump. The focus of this paper is the design of the shear key and the development of placement and compaction criteria for the material that was selected for construction.

2 BACKGROUND

2.1 *Location*

The FHD is located at the Syncrude Aurora North Mine site, approximately 80 km north of Fort McMurray, Alberta (Figure 1). Stage 1 was completed in 2010 on original ground, north of the in-pit tailings storage facility AEPN-W (existing tailings facilities shown on Figure 2). Stage 2 was also constructed on original ground, west of AEPN-W. Stage 3 is currently being constructed on top of the infilled AEPN-W and will tie into Stage 2 to the west and Stage 1 to the north. AEPN-W was bounded by the old pit high wall to the north and west and hydraulic fill dykes to the east (Dyke 1 North) and south (Dyke 1 West) that separated AEPN-W from the active TSFs AEPN-E and AEPS to the east and south respectively. Stage 3 steps over Dyke 1N in the northeast corner of the footprint onto the upper portion of the beach of the adjacent AEPN-E in-pit tailings storage facility.

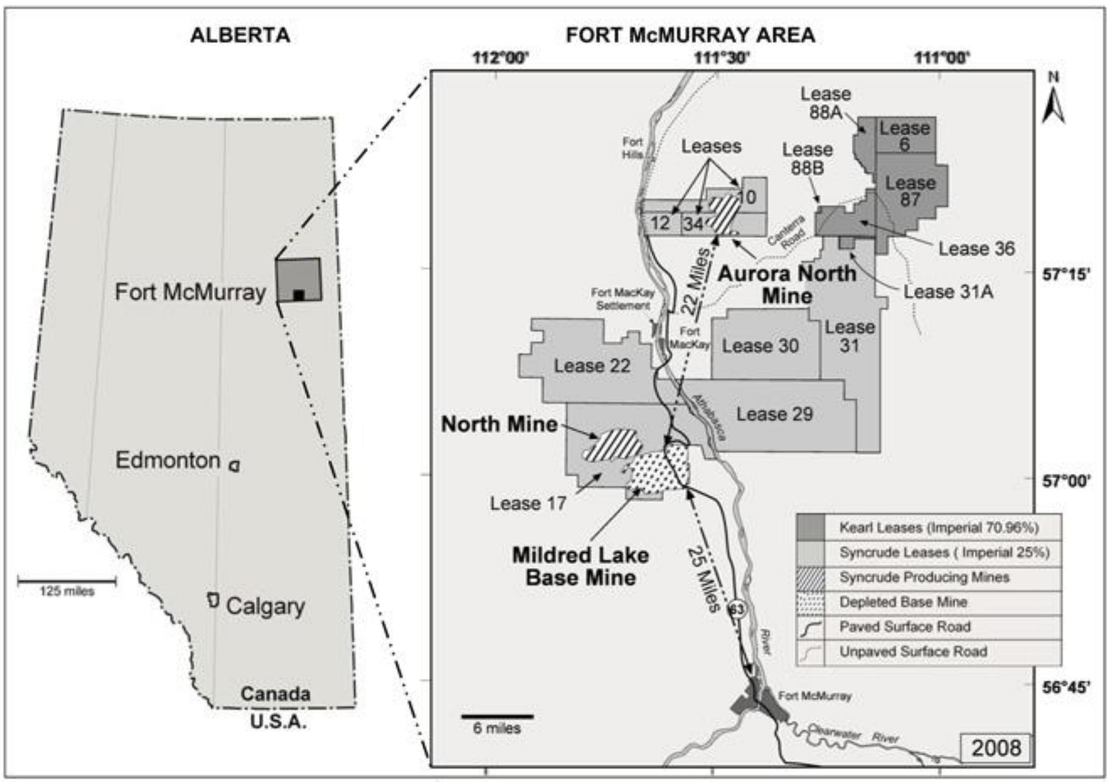

Figure 1. Location of the Syncrude Aurora North Mine Site (United States Securities and Exchange Commission, 2008)

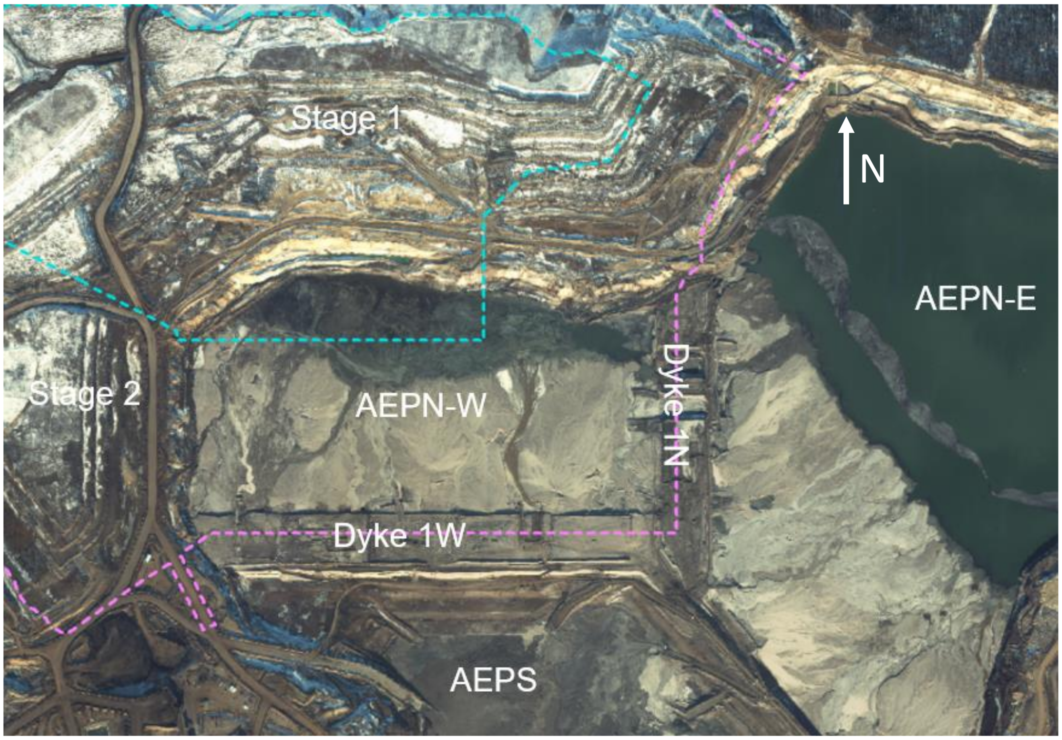

Figure 2. Layout of in-pit storage facilities within the original Aurora East Pit

2.2 *Foundation materials*

Following completion of mining activities, deposition within AEPN-W started in 2012 and followed standard beach construction methods within the oil sands, with tailings sand primarily discharged and deposited from the south dyke. The tailings deposit is a combination of beach above water (BAW) and beach below water (BBW) tailings. The upper beach portions of the BAW nearest to the discharge were track-packed, and the pond was generally located towards the north of AEPN-W, against the old pit high wall. The facility contained zones of coarse and fine tailings, fluid, and bitumen (sometimes in layers). At the end of active deposition, the tailings deposit within AEPN-W was up to 70 m deep.

As part of the dump design, a 10 m thick tracked-packed sand cap was hydraulically placed over the tailings. Placement of the sand cap proceeded simultaneously from the south to north and from east to west, then finally from the northwest to the southeast as soft material and supernatant fluid in the north part of the tailings deposit was channeled through trenches and culverts located through Dyke 1N, flowing into AEPN-E.

Several cone penetration test (CPT) investigation campaigns were conducted across AEPN-W to characterize the contents of the facility. The investigations found that there was significant heterogeneity and layering of fine and coarse material within the deposit, and that a significant portion of the foundation was potentially liquefiable. A general profile of the contents of AEPN-W is shown in Figure 3.

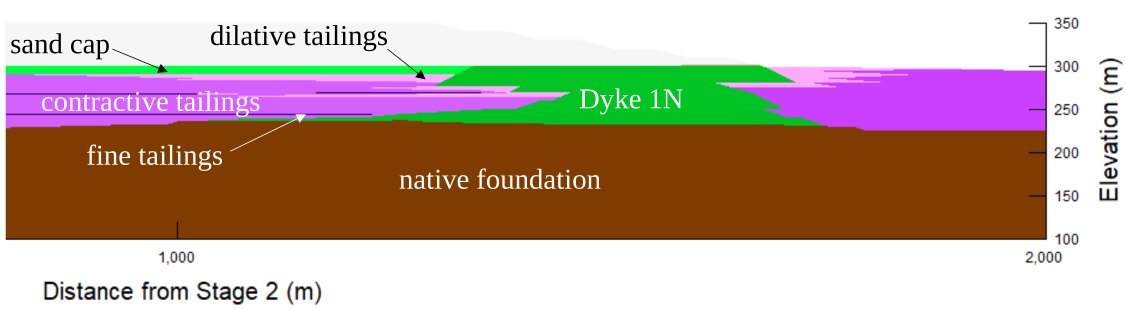

Figure 3. General profile of AEPN-W

2.3 *Waste dump characteristics*

The FHD is an overburden waste dump comprised of two main placement zones, an interior zone and an exterior shell. Typical overburden is made up of a mixture of medium plastic clays and clay tills, and medium to high plastic clay shales from the Clearwater formation. Materials were placed in 2.5 m to 5 m thick lifts and compacted by haul truck traffic. Stage 3 of the FHD was proposed to be 50 m high, with a footprint of 2.5 km east to west and 1 km north to south. The downstream slope was 4 horizontal to 1 vertical (4H:1V), with benches every 10 m in height, resulting in a compound slope of 8H:1V. Included within the profile was a shear key, which is the focus of this paper. A general cross-section of Stage 3 is shown in Figure 4.

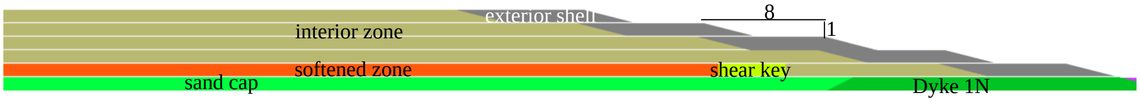

Figure 4. General cross-section of the waste dump (west to east)

3 ANALYSIS AND ASSESSMENT OF OVERBURDEN WASTE DUMP

3.1 *Overview*

As part of the design of Stage 3 of the FHD, assessments were conducted for stability, liquefaction susceptibility, constructability, cracking/softening of construction materials, seepage, fluid management, capping, waste placement methodology, monitoring, and total storage. These assessments identified several issues that needed to be addressed through the design. The following section discusses those issues that are relevant to the development of the design of the shear key.

3.2 *Key issues*

The shear key was meant to address the potential for instability to develop through the base of the waste dump, due to softening of the dump materials. This could happen as the dump settled differentially during construction, resulting in cracks forming along the base and ingress of water from seepage along sand layers in the old high wall, consolidation of the tailings below and proximity to the water table.

Several CPT campaigns were conducted to assess the foundation materials. Consistent with typical beach deposition, the foundation conditions were more favourable towards the south near the discharge locations and less favourable towards the north and east, where the pond had historically been located. CPT soundings revealed that a significant thickness of the foundation was highly compressible, with up to 70 m of normally consolidated clays, silts and sands present in the northern portion of the foundation. The locations with the most compressible foundation material also corresponded to the highest part of the structure. A settlement assessment showed that significant amounts of settlement could be expected to occur over the life of the structure and that differential settlements were expected along Dyke 1N, in areas where the pond had been located, adjacent to the dyke during operation.

A series of 1D settlement assessments for the proposed dump were made and contours of the potential settlement were developed. From the settlement contours, an assessment of the differential settlement and the potential strains along the base of the dump was made, which indicated that cracks were likely to develop along the base of the dump along transitions between the tailings beach (areas of high settlement) and the pit wall and dykes (areas with low settlement). The potential magnitude of settlement was sufficiently large for the base of the dump to be submerged below the phreatic surface.

During the operation of AEPN-W, fluid levels were controlled by a system of pumps and drains within the containment dykes. The system drew down the phreatic surface within the tailings by several metres. This system is intended to remain operational during construction of the dump, but an assessment was made to understand the effects if the system was decommissioned, which, when coupled with local seepage from original ground to the north, would cause fluid levels within the structure to rise significantly.

With the potential for high plastic clays shales, placed relatively dry, to be present at the base of the dump, along with the potential for cracking and ingress of fluid due to seepage and consolidation of the underlying foundation, a key issue for the dump was the potential for the material along the base to soften, resulting in a low strength zone developing along the base of the dump. Stability analyses identified that both large deep-seated failures extending back into the dump and deep-seated failures along the side slopes were possible depending upon the extent of the softened zone.

4 DESIGN OF THE SHEAR KEY

To address the issues described in the previous section, the design of the dump was modified to include a 10 m high, 50 m wide sand shear key, which would serve to increase resistance along the base of the dump to slope failure and to curtail the lateral progression of the softened zone. The key parts of the shear key design, namely, the location, compaction specification and construction methodology, are described in more detail below.

4.1 *Location*

For the shear key to remain effective throughout the life of the dump it had to be optimally located to improve stability for all configurations of the dump. When locating the shear key the following factors were considered:

- Locations of maximum strain in the base of the dump
- Geometry of potential slip surfaces
- Potential locations for fluid ingress
- Construction access and constructability.

The amount of shear strain in the dump was assessed by considering the contour intervals generated from the settlement analyses as well as by using a simplified 2D stress deformation model. The highest tensile strains were present where contour bands were the closest together. These tended to be located near the transition between uncompacted and compacted material such as between uncompacted and tracked packed beaches along the dykes.

The location of the shear key also considered the geometry of the slip surfaces through the dump. If the shear key was too close to the toe, there would be the potential for failure to occur upstream of the shear key through softened material at the base of the dump. Conversely, if the shear key was located too far upstream, failure surfaces could develop which passed through liquefiable tailings or through a softened zone downstream of the shear key (Figure 5).

The shear key also needed to be in an area where it could be constructed with typical mine equipment and with preference to areas closer to material borrow sources. Based on the assessment, the shear key was located 250 m from the dump toe.

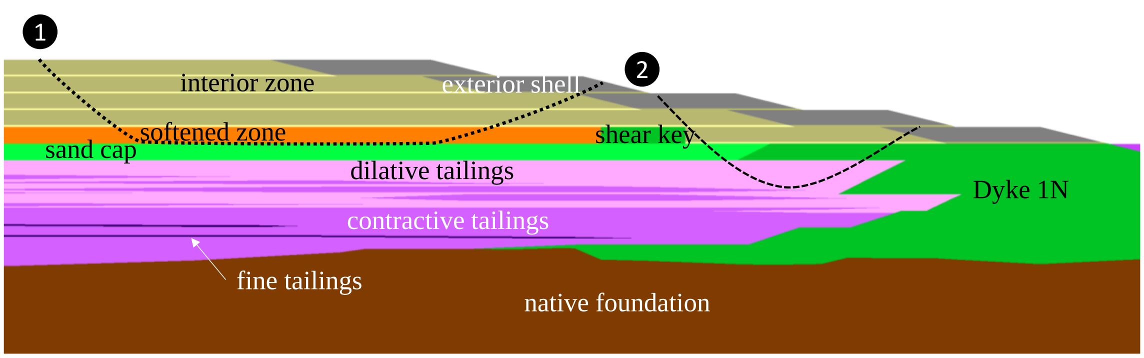

Figure 5. Potential slip surfaces for the dump with a softened base (1) upstream and (2) downstream of the shear key

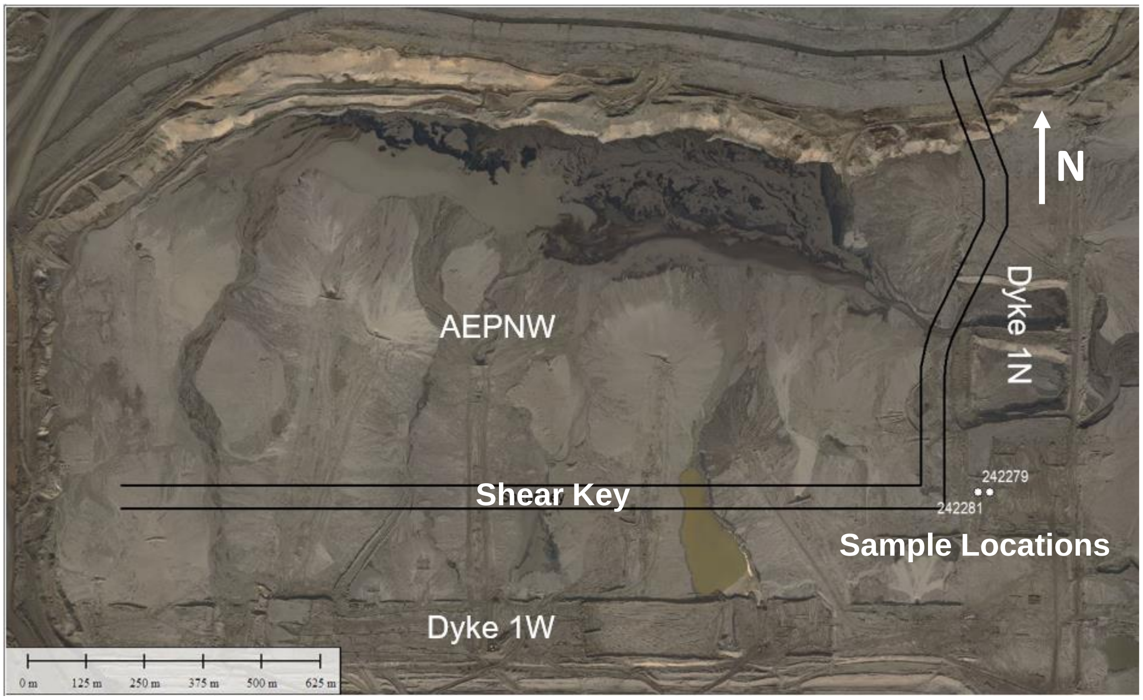

Figure 6. Plan view of shear key location

4.2 *Parameter selection and laboratory testing*

Given the potential magnitude of predicted settlement of the base of the overburden dump, as well as the abundance of water within the foundation and adjacent terrain, it was anticipated that part or all of the shear key would become saturated during the life of the structure. It was important to the stability of the dump that the shear key responded in a dilative manner when subjected to shear, i.e. that it was not at risk of liquefaction for the various loading configurations of the structure.

To address the performance of the shear key, a laboratory program was developed for characterization and selection of the potential borrow sources. The material assessment for the shear key consisted of:

- Consolidated drained (CD) triaxial tests
- Consolidated undrained (CU) triaxial tests
- Oedometer tests
- Direct shear tests
- Particle size distribution (PSD) using sieve and hydrometer
- Minimum and maximum void ratio
- Proctor density
- Critical state line (CSL) assessment.

Several PSDs were determined, to assess the variability of the material. The two samples shown in Figure 7 represented the bounding gradations for the material and were used in most of the assessments. The materials tested were sourced from similar areas and from ore that was processed and deposited similarly. Before conducting the laboratory study, a comprehensive literature review was performed on CSLs for tailings. Two public references were found for Syncrude Base Mine tailings, provided in Sladen and Handford (1987) and Sobkowicz and Handford (1990). The available gradation is also provided in Figure 7. Testing from Sobkowicz and Handford (1990) did not provide a full PSD. However, the samples were described as poorly graded and the D50 was noted as 0.13 mm to 0.17 mm with a geotechnical fines content of 4% to 11%. It was noted that the published PSDs were finer than the materials tested as part of this program.

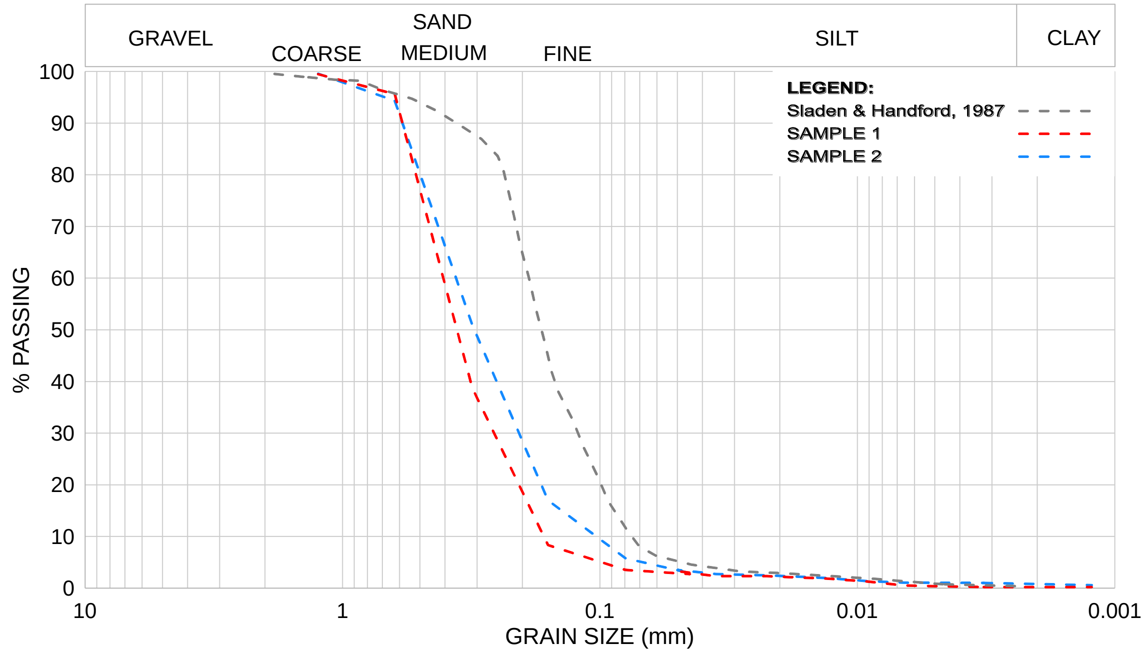

Figure 7. Particle size distribution of the tested samples and from literature

To assess the CSL, a series of CU and CD tests were conducted over the range of fines contents expected for the sands used to construct the shear key. The triaxial test methodology used in the testing was adopted from the recommendations provided by Jefferies and Been (2016), ASTM D4767 and ASTM D7181. Samples were mixed to homogenize the material and were air-dried to 5% moisture content. Air drying was used instead of oven drying due to the presence of bitumen. Reconstituted triaxial test specimens were prepared using the moist tamping method, where the samples were built in seven equal lifts of the same density (Ladd, 1978). Samples were prepared such that they would remain loose of critical state after saturation and consolidation. Samples were sheared in the triaxial apparatus at a constant rate until 25% axial strain (Figure 8). The resulting CSLs are compared to the CSLs from the literature review in Figure 9.

When comparing the CSLs to those available in the literature, it is noted that while the slope of the CSL is similar, there is a substantial difference in the intercept. The similarity in the slope of the CSL is likely due to compressibility and crushability of the sand grains, which is expected since while they are from different sites, they are from the same formation. At the same time, the variation in the intercept is likely due to the differences in PSD, grain shape and mineralogy as discussed in Jefferies and Been (2016). A summary of the results is provided in Table 1.

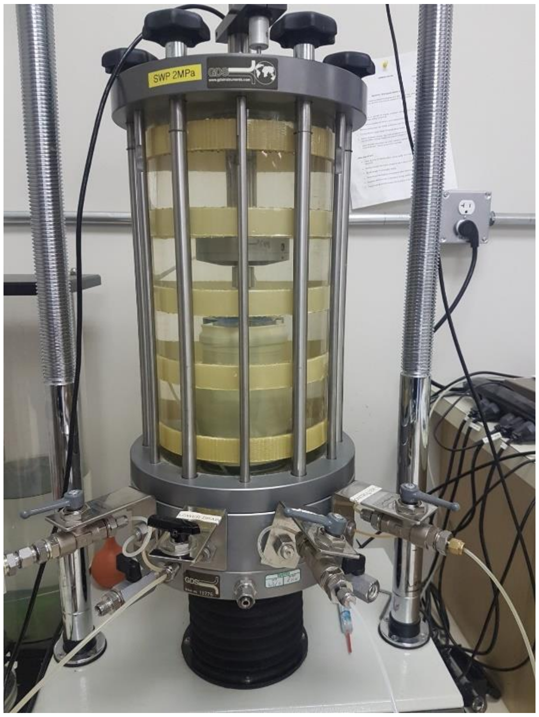

Figure 8. Triaxial apparatus

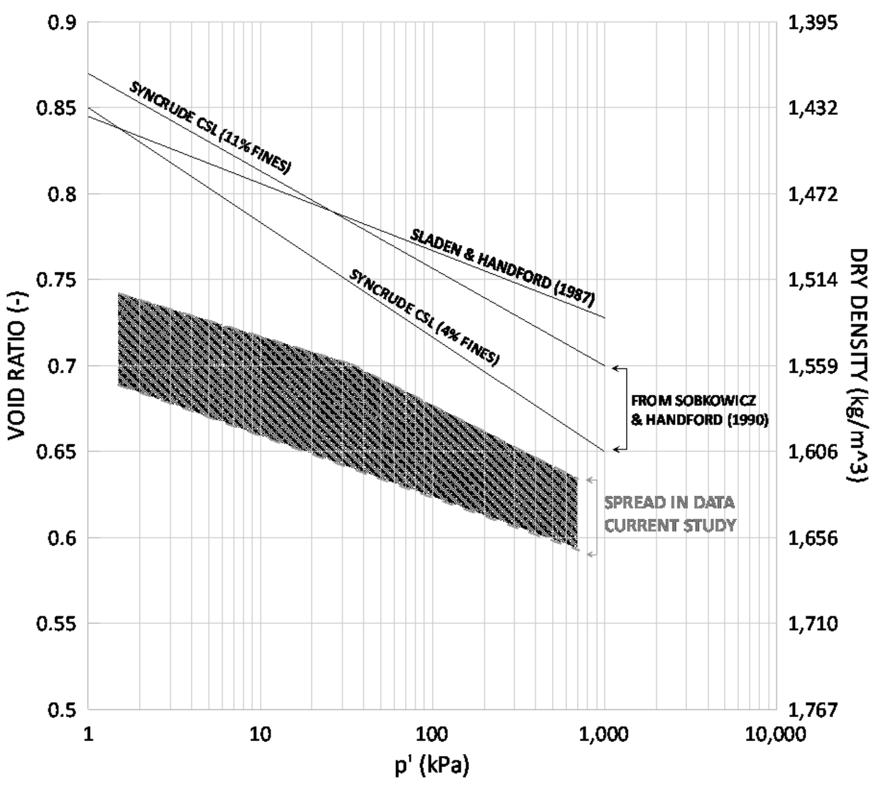

Figure 9. Critical state line assessment

Table 1. Design parameters for the shear key

Parameter	Value
Friction angle	32
Critical state intercept, Γ	0.71 – 0.73
Critical state slope, λ_{10}	0.035 – 0.04
Consolidation index, Cc	0.09
Rebound index, Cr	0.009
Proctor density (kg/m^3)	1730 - 1750
Minimum density (kg/m^3)	1440
Maximum density (kg/m^3)	1875

4.3 *Development of compaction specification*

Given the potential for the shear key to saturate during operation, it needed to be placed dense enough to remain dilative for the expected operational stress ranges. To determine the necessary compaction specification in order to achieve these goals several factors needed to be considered:

- Densities associated with dilative behaviour
- Stress range expected within the shear key
- Expected change in void ratio during operation
- Relationship between relative compaction and relative density
- Ability to achieve the necessary compaction.

Typical construction activities for sand usually specify compaction in the range between 95% and 100% Standard Proctor density. However, some activities require a higher compaction specification to meet the intended purpose. Increasing the placement density also comes with an increased cost, i.e. it could require thinner lifts or different equipment than what is available, so it was important for such a large volume of construction to assess how dense was dense enough?

A series of CSLs were developed for the potential borrow materials to assess over what densities the material would dilate. The CSL defines the border between contractive and dilative behaviour at various confining stresses, where materials loose of the CSL will contract and strain soften in undrained shear and can experience significant strength loss through the process of liquefaction. The issue for the shear key is that material would initially be placed dense, equivalent to the energy imparted by the compaction equipment, and as dump material was built on top of the shear key, the material would continue to densify, however, the overall state of the material, i.e. the distance to the CSL, would decrease (Figure 10). Therefore, the relative compaction (RC) of the material at placement had to be sufficient to maintain a dense state throughout the life of the dump.

To assess the necessary compaction, simulations of several dump cross-sections were analysed using FLAC, a finite difference software used to model stress deformation behaviour, to examine the expected stress change in the shear key during operation.

The stress path that the compacted sand would follow during loading was determined by oedometer tests on samples compacted to different levels. Using the rebound index from oedometer testing with the expected stress change from the modeling, the change in void ratio of the shear key could be estimated during the life of the structure (Figure 10). The compaction specification was selected such that the material would remain dense of the CSL by a state parameter offset of at least -0.05 at a constant mean effective stress, where state is the distance to the CSL in terms of void ratio. An offset of -0.05 was chosen because testing has shown that materials can still experience strain-softening for states between -0.05 and 0 (Jefferies and Been, 2016). An allowance was also made for the accuracy of determining the degree of compaction of the shear key sand during construction.

Based on the analysis and laboratory testing a compaction of 98% Standard Proctor Maximum Dry Density (SPMDD) was selected for the shear key.

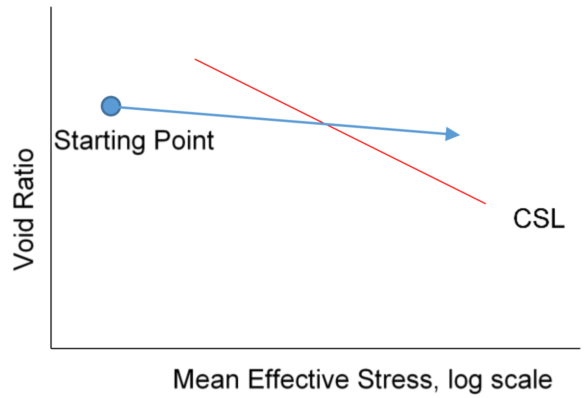

Figure 10. Stress path relative to critical state during dump construction

4.4 *Quality assurance and control methodology*

In standard civil construction, compaction specifications are established and materials are placed in controlled lifts, typically 200 mm to 300 mm thick, using a vibratory roller for compaction (of sand). Compliance with the compaction specification can then be assessed through the use of a nuclear densometer, which has an effective measurement depth of about 300 mm. Construction of the shear key initially planned to use much larger equipment than the standard vibratory rollers used on most construction sites. A test strip was carried out to examine the effectiveness of using mine equipment for compaction. It was found that the equipment was not practical to obtain the necessary compaction level in the shear key and so conventional compaction equipment and lift thicknesses were used instead. Quality control (QC) of the shear key was carried out by establishing a method specification based on the equipment type, number of passes and material placement during the test strip. Nuclear densometer tests along with an assessment of the material gradation and Proctor density were conducted for quality assurance (QA).

5 OBSERVATIONS, LEARNINGS, PERSPECTIVES

It is perhaps useful to reflect on the process of dump assessment and shear key design activities as described in this paper, in order to itemize a few learnings and perspectives which may inform the reader and others contemplating similar designs:

- Recognize the potential for liquefaction of any granular portion or zone of a structure, irrespective of the placement method, and specify a compaction requirement that addresses all stresses and failure modes.
- Consider a range of loading and pore pressures to which the structure will be subjected.
- Bear in mind that a shear key is but one tool in a useful toolbox of design remedies and may be best applied in combination with other design components such as drainage, compaction and buttressing.
- The target compaction density is influenced by many factors, including available material and equipment, risk and cost. In the end, the final compaction specification which was selected was perhaps slightly lower than originally anticipated.
- When selecting compaction requirements to ensure granular materials are placed in a dilative state (for the range of anticipated effective stresses), take account of the accuracy of both the laboratory density/CSL testing and the field compaction testing.
- Team based, collaborative solutions may require more intentional communication, but ultimately serve to deliver a robust, balanced design which is able to cater to a wide range of geotechnical performance demands.

6 CONCLUSIONS

To support the stability of a large waste dump constructed on top of an infilled tailings pond, a 10 m thick, 50 m wide shear key was designed. A typical compaction specification for standard civil construction of structures or embankments is in the range of 95% to 100%, depending upon the application. These specifications are based upon years of construction experience. Given the size and construction methodology of the shear key, the variance between 95% and 100% compaction presented significant cost implications. Based on a series of advanced tests a compaction specification of 98% SPMDD was selected to guard against the potential for liquefaction of the shear key after dump construction was completed. Field trials allowed development of a method specification for QC.

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