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Seismic Stability of the Mt. Frederick Sidecast

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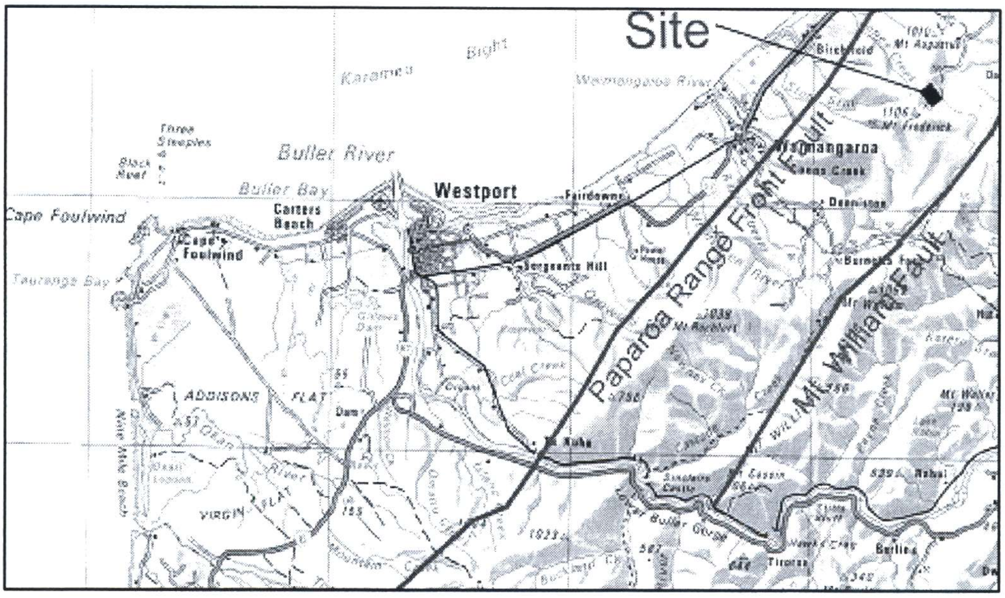
Summary: As a part of Solid Energy's progressive rehabilitation of the Stockton Mine, it was determined that significant rehabilitation of the existing Mt Frederick Sidecast would be required to ensure the long-term stability of the sidecast. The goal of this rehabilitation is to reshape the existing stockpile into a more stable slope configuration, and the construction of a comprehensive stormwater collection and discharge system. The reconfigured slopes will facilitate the final revegetation of the exposed surfaces and the management of surface and groundwater discharges from the area. This paper briefly describes the specific details of the project and the significance of seismic conditions on the long-term stability of the reconfigured stockpile.

INTRODUCTION

The Mt. Frederick Sidecast (sidecast) is a large waste rock stockpile containing overburden materials stripped from the surface during the mining process. As shown on Figure 1, the site is located on the west coast of New Zealand's South Island within the Buller Coalfield about 25km north-east of the town of Westport at the southern end of the Stockton Opencast Mine. The mine is situated on an elevated plateau at an altitude of between 600-1000m above sea level, and the sidecast is located at the higher altitudes between 880-1010m above sea level. Annual rainfall at the site is on the order of 6-8m per year.



SITE DESCRIPTION AND PROJECT HISTORY

The sidecast contains approximately 3 million cubic meters (m^3) of materials that were end-tipped into an area on the northeast face of Mt. Frederick and the upper Herbert Stream catchment. Below the sidecast, a sediment retention dam was constructed to control and divert stormwater and sediment runoff from entering Herbert Stream. A recent aerial photo of the sidecast is shown in Figure 2.

Figure 2. Aerial view of the existing Mt Frederick Sidecast

The first dumping of materials into the sidecast started in July 1992 and continued on and off until May 1998. The overburden waste rock consist of sandstones / siltstones of the Brunner Coal Measure geologic group. A large portion of these materials could be described as rockfill, but after excavation and placement a significant portion of the materials have broken down to silt, sand, and gravel size particles. Segregation has occurred within these materials during end-tipping from the top of the slope, and subsequent surface erosion since original deposition has created a large debris fan at the toe of the sidecast. An intermingling of coarser fan-like deposits within finer-grained sediments (sands & silts) has occurred within the sediment basin at the toe of the slope. A series of well-developed surface erosion channels and shallow surficial slumps dominate the current surface of the sidecast. Ongoing slope failures recently led to a large debris flow that blocked the existing stormwater channel adjacent to the crest of the dam. This led to the construction of a rock bund near the toe of the slope to divert future runoff and debris flows laterally across the sediment basin to the north. The sidecast is considered to be marginally stable in its present condition, and it is expected that surface erosion and shallow surface slumping will continue.

The Herbert Stream sediment dam was built to divert stormwater runoff from the active mine above the sidecast and the Herbert Stream catchment into the site-wide stormwater and sediment control system in the Plover Stream catchment. The dam was constructed in four stages, and the present crest of the dam is located 100-150m downstream from the toe of the sidecast. The dam is currently 30m high and 220m long and contains roughly 250,000m^3 of fill material. The dam was constructed from waste rock fill sourced from four different areas within the mining block, end-dumped in near horizontal lifts and compacted by equipment travel. It is believed that a zone of large cobbles and boulders was placed at the foundation contact in the old stream channel to act as an internal drain for the rockfill embankment. A low permeability zone was placed on the upstream face of the upper 10m of the dam. Due to the nature of the materials used and methods of construction, the existing dam is considered to be a relatively pervious structure.

A mixture of sediments and talus fan deposits from the sidecast fill the available storage space behind the sediment dam. Judging by the short time between embankment raises, sediment accumulation behind the dam appears to have occurred rapidly. It is difficult to ascertain how much mixing of sidecast waste rock and impounded sediments may have occurred and how far this mixed zone may extend into the sediments. However, it is believed that significant sediment accumulation may have occurred below RL880 prior to the deposition of large amounts of talus fan materials on top of the impounded sediments.

GEOTECHNICAL INVESTIGATIONS

Field investigations and lab testing were undertaken to determine the soil strength parameters for the impounded sediments and waste rock, assess the variability of these properties at depth, locate the piezometric surface within the dam, and confirm the presence of a colluvial layer at the bedrock contact (*MWH 2001*). Two boreholes were drilled through the dam and standard penetration tests (SPT) were undertaken to establish rockfill strength properties. Cuttings were logged and a colluvium layer was recorded in both boreholes, and slotted PVC standpipes were installed in each borehole to monitor water levels and identify the piezometric surface. Two cone penetration tests (CPT) were undertaken in the saturated loose sediments that have accumulated behind the dam. A total of 33 Scala penetrometer tests were undertaken, with 31 tests performed in the impounded sediments and 2 tests performed on the crest of the dam. A total of 60 neutron density measurements were undertaken, with 52 tests located across the impounded sediments and 8 tests located on the crest of the dam. Five shallow hand-dug inspection pits were excavated along the north side of the sediment basin and around the perimeter of the dam to investigate the extent of the colluvium layer in this area, and hand shear vane tests were used to characterise the strength of the colluvium.

Several weak zones were detected within the sediment dam in both boreholes during the drilling and SPT testing. It is believed these zones are likely attributed to intercepting layers of poor compaction along the old downstream batter slopes from earlier stages of embankment construction. Water level readings from the piezometers indicate that the majority of the rockfill is dry. The observed seepage patterns suggest that internal seepage likely drains rapidly through the rockfill embankment to the basal contact and exits along the original stream channel.

CPT results and pore pressure dissipation tests indicate that the impounded sediments are dominated with very loose sand materials with thin layers of finer-grained silt or clay material. Tip resistances were generally less than 0.8 MPa, and the CPT traces suggest that there is up to 20% of finer-grained material within the impounded sediment column. The impounded sediments appear to consist of thin interfingered layers comprised of roughly 40-45% sand, 45-50% silty to clayey sand, and 10% clay (in thin isolated stringers). It doesn't appear that the interfingered stringers are laterally continuous, and the extent of these thin bands is believed to vary significantly across the sediment basin.

Three distinct sediment environments were noted on the surface of the impounded sediments. These consist of: a well-developed zone of talus fan sediments located at the base of the sidecast extends out across the majority of the impounded sediments; a thin fan of braided stream sediments (30-40m wide) along the northern side of the sediment basin; and a zone of streambed sediments (30-40m wide) associated with the existing stormwater control channel running parallel to the sediment dam crest. Generally, the sandstone rock fragments increase in size and abundance from east to west and finer-grained more unstable materials were found to increase in thickness from west to east and from south to north (adjacent to the sediment dam and north side of the sediment basin).

Observations from the shallow inspection pits around the perimeter of the sediment dam confirmed that the colluvium layer appears to exist consistently across the site. This layer appears to consist of clay rich sands with gravel, overlying peat and soil (which was interpreted to be the original vegetation layer) that in turn overlies sandstone. Shear vane results in the fine-grained peat portion of the material ranged from 10-14 kPa indicating that this layer is very soft. However, it is believed this material will have been consolidated under significant vertical stress beneath the sidecast, the dam, and impounded sediments; and will therefore have increased in strength. As the colluvium layer is of limited thickness and is surrounded by free draining material, it is anticipated that it will consolidate and gain strength rapidly, if further load is applied by the reconfiguration of the existing sidecast.

A total of 14 near-surface samples were tested to determine the soil strength properties for the impounded sediments and waste rock materials. This included 6 gradation tests, 4 compaction tests, two shear strength tests (insitu and remoulded), and one permeability test. Assumed soil strength parameters used in the stability analyses were selected based on these results. Residual liquefied undrained strength of the sediment was assessed using the recommendations of *Seed and Harder,* which relate the strength to SPT N value and fines content. The final soil properties adopted for the analysis are summarised in the Table 1.

Table 1 - Soil properties adopted for stability analysis

Material	**Density**	**Static Strength Parameters**		**Seismic Strength Parameters**	
		Shear strength	**Cohesion**	**Shear strength**	**Cohesion**
	γ (kN/m^3)	**Φ (degrees)**	**c (kPa)**	**Φ (degrees)**	**c (kPa)**
Colluvium	19	30	2	30	2
Waste and dam rock	22.2	37	0	37	0
Sediment (dry)	18	29	0	27	0
Sediment (saturated)	18	29	0	0	1

REVIEW OF SEISMIC HAZARDS

The sidecast is located in an area of high seismicity with a number of known active faults. The epicentres of two major historic earthquakes were located near the site (15km from the 1968 Inangahua earthquake M7.1 and 30km from the 1929 Murchison earthquake M7.8). Engineering Geology Ltd performed a seismic hazard evaluation (*March 2001*) for the site that included a review of the available literature on the seismicity of the site area, and provided the estimated ground motions to be used in the seismic stability evaluations. Since there are no direct measurements available for the site, ground motions were characterised in terms of a probabilistic estimate of the Peak Ground Acceleration (PGA) based on the historical record for the 50-yr, 150-yr, 450-yr, and 1000-yr return period level of shaking for the site. Summarised in Table 2 are the known active faults located within 65km of the site, estimates of the Maximum Credible Earthquake (MCE), maximum PGA, and the recurrence interval for these active faults.

Table 2 - Known Active Faults in the Vicinity of the Site

Seismic Source	Distance from site (km)	Estimated MCE (Magnitude)	Estimated Maximum PGA at Site (g)	Recurrence Intervals (years)
Paparoa Range Front Fault	4	7.1	0.97	5,000
Mt. William Fault	4	6.5	0.76	Not Known
Inangahua Fault	15	7.4	0.53	4,400
Lyell Fault	22	6.7	0.24	Not Known
White Creek Fault	30	7.6	0.36	34,000
Cape Foulwind Fault	50	6.5	0.08	Not Known
Alpine Fault	65	8.0	0.29	300

Due to their proximity to the site, the Paparoa Range and Mt Williams Faults have the greatest influence on seismic hazards to the site. The probabilistic PGA for the 150-yr return period was estimated to be 0.32g and the probabilistic PGA for MCE for the site was estimated to be 0.97g. However, where the design criteria for an embankment structure requires minimal damage, it is appropriate to use a horizontal force coefficient less than the PGA. For most seismic events, the PGA is associated with one pulse that is considerably greater than the average acceleration and it acts over a very short duration (fraction of a second). Consequently, a value of two-thirds PGA is more representative of the stresses that will be applied by earthquake loading. Therefore, the seismic analysis has been performed with a coefficient of 2/3 PGA for the 150-yr return period ground motions (i.e., 0.21g) and the MCE ground motions (i.e., 0.65g).

SELECTION OF ACCEPTABLE FOS FOR DESIGN

Industry practice for the design of embankment dams (*NZSOLD, ANCOLD, and USBR*) is to design for two levels of earthquake loading (Maximum Design Earthquake or MDE, and Operating Basis Earthquake or OBE). The MDE is the maximum level of ground motion for which a dam should be designed or analysed (usually either the MCE or a 1 in 10,000 annual exceedance probability event), and following the MDE some damage is allowable but it must not lead to catastrophic failure. The OBE is usually selected on a probabilistic basis typically representing the ground motions with an annual exceedance probability of 1 in 150, and following the OBE there should be either no damage or minor repairable damage. In terms of expected loss of life and economic loss, the Herbert Stream sediment dam is considered to be a very low to low potential impact dam (*NZSOLD*). Therefore, the ground motion for the 150yr return period event was selected for the target level for this analysis. Typically acceptable values for recommended factors of safety (FOS) are provided in Table 3.

Table 3 - Typically Accepted Factor of Safety Values for Stability Analyses

For static stability cases	FOS > 1.5
For seismic stability for OBE event (with little or no significant damage) using pseudo-static analysis	FOS > 1.2 to 1.3
For MCE event (accepted that some damage will occur) or assess deformation	FOS > 1.0

It should be recognised that these criteria are largely based on lessons learned from the historic performance of water retention dams during earthquakes, and vary depending upon the extent to which the material properties and the location of phreatic surfaces are known. In this case, these values were considered conservative given the nature of the structure and the potential consequences of failure.

In high seismic areas, it is likely that FOS <1.0 under the MCE may occur using a pseudo-static approach. In these cases it is common that some assessment of the expected deformations is made to estimate the magnitude of the damage. Since site-specific investigations have confirmed the material properties and the location of the phreatic surface used for analyses, it was considered that less conservative FOS values are warranted. During the course of this analysis, Solid Energy agreed to accept limited damage to the reconfigured sidecast when subjected to ground shaking at or exceeding the OBE event. Consequently, the FOS values selected for this analysis are provided in Table 4.

Table 4 - Factor of Safety Values Selected for Stability Analyses

Minimum acceptable FOS for static stability	FOS = 1.4
Minimum acceptable FOS for the OBE level shaking using pseudo-static analysis (minor damage to waste stockpile, but no significant damage to sediment dam)	FOS = 1.1
Rather than adhere to a minimum acceptable FOS for the MCE level shaking, it was decided to perform a deformation analysis to confirm the magnitude of potential displacements	

SLOPE STABILITY ASSESSMENT

Initially an analysis of shallow failures within the rockfill was undertaken. Measurements of the existing sidecast slope angles indicated that the natural angle of repose of the material was between 35° and 40°, and a design angle of internal friction of 37° was adopted. A simple sliding block analysis of shallow failures on the dry rockfill slope was used to determine that a slope of approximately 3H:1V was necessary to meet the OBE performance requirements. Using this slope angle, preliminary layouts of the reconfigured stockpile were undertaken.

The intent of preliminary layouts was to make the reconfiguration a balanced cut to fill operation and retain all of the overburden materials within the Herbert Stream catchment. However, it soon became apparent that this would not be possible without placing a significant quantity of materials on the potentially liquefiable sediments. Analysis of the liquefaction potential using methods by *Seed* and *Davis & Berrill* confirmed the susceptibility of the saturated sediments to liquefaction at the OBE level of shaking, therefore any material placed on the sediment was likely to be severely disturbed during seismic shaking. Other treatment options (such as densification, drainage, etc.) were investigated, and found to be prohibitively expensive for the scale of the site, the decision was made to place as little of the reconfigured stockpile on the impounded sediments as possible. Solid Energy agreed that a large volume of materials would have to be removed from the Mt Frederick site to achieve this goal. The proposed remedial earthworks will consist of flattening and recontouring the sidecast to create a stepped landform by a series of 10m wide benches spaced on 15m vertical intervals with 2.7H:1V batters slopes to produce an overall slope of 3.3H:1V as shown on figure 3.

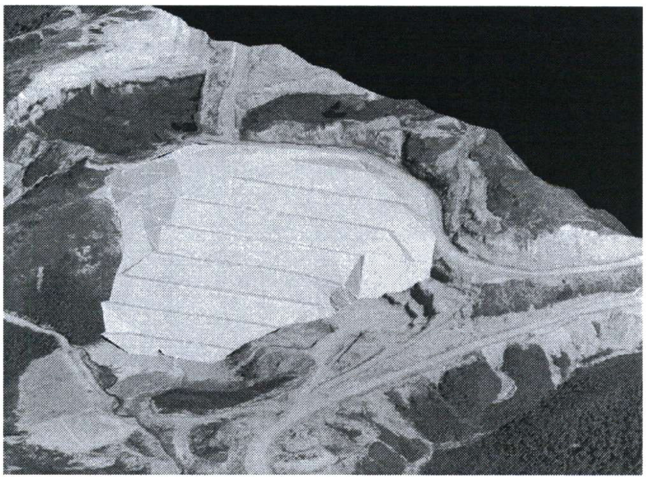


Figure 3. Computer generated view of proposed reconfigured sidecast

Once an acceptable balance of these requirements had been found, the stability analysis of three typical cross sections of the entire slope (including the impounded sediments and dam) were undertaken for both static and seismic load cases using a Generalised Limiting Equilibrium (GLE) approach using Slope/W software. Seismic accelerations were modelled using a constant pseudo-static force acting out from the slope. The effect of the earthquake loading was modelled by applying an additional horizontal inertia force to the centroid of the sliding mass being analysed. The critical parameters for the static and seismic stability were the present steep sidecast slope angles; the presence of saturated loose liquefiable sediments at the toe of the slope, the location of the piezometric surface; and the presence of a soft residual soil layer above the bedrock contact. Since the field investigations confirmed the majority of the rockfill in the sediment dam is dry, it was determined the stability of the dam is acceptable and the dam can largely be left in place in its present condition. Proposed earthworks to reconfigure the sidecast will incorporate the construction of a series of permanent stormwater channels to control surface runoff and divert stormwater flows off the reconfigured sidecast and around the impounded sediments to significantly reduce the recharge to the groundwater system within the sediments.

The aims of the final analysis was to: confirm the static stability of a "global" failure (i.e. a failure involving more than simply a shallow skin of material either on the dam or the stockpile); establish the relative size of failure that could be induced by the liquefaction of the impounded sediments; and confirm that the portion of the stockpile that was not affected by liquefaction also complies with the seismic design requirements. An example of the slope stability model used for this analysis is shown in Figure 4.

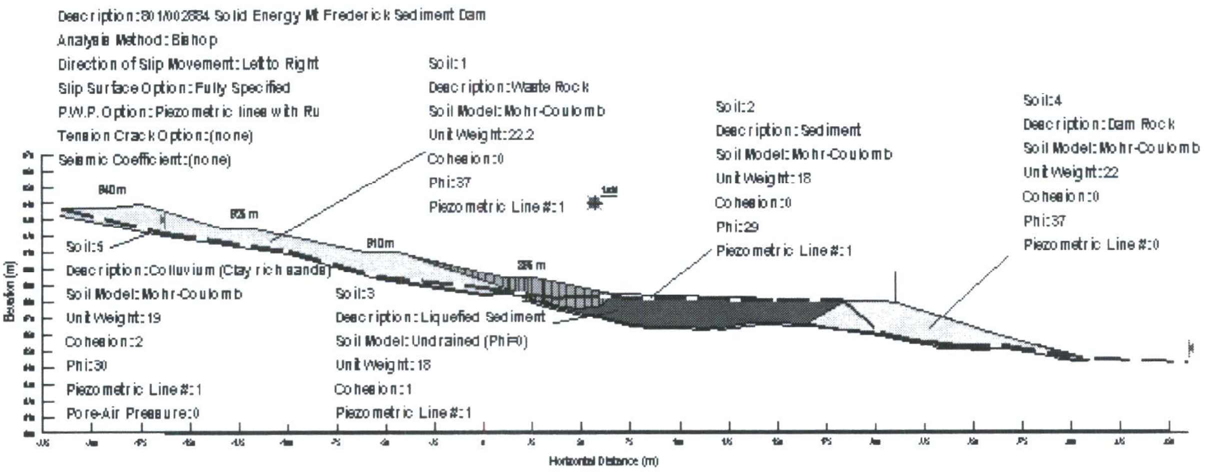


Figure 4 - Example of Stability Analysis output

This model represents the situation of the soil being in a liquefied state but without an external applied acceleration. Any failure surface with a FOS less than one under these conditions cannot resist the continuous static stresses and will fail. Surfaces that include less of the rockfill than shown in Figure 4 would also have a FOS less than one and would be expected to slump. It should also be noted that during the earthquake, after the sediments have liquefied, there would be instantaneous failures during the earthquake pulses along the failure surface shown that will result in significant deformations. It was therefore necessary to apply some judgement to the further extent of the slope that would undergo so much deformation as to have essentially failed. Results of the analysis are summarised in Table 5:

Table 5 – Summary of Stability Analysis Results

Cross Section	Static Stability		Seismic Stability @ OBE (excluding liquefaction effects)		Effect of Liquefaction
	Shallow FOS	Global FOS	Shallow FOS	Global FOS	
A - A	2.1	1.9	1.2	1.0	Slumping to first bench
C - C	2.1	2.0	1.2	1.1	Slumping to second bench
H - H	2.1	2.1	1.2	1.1	Slumping to second bench

The performance of the reconfigured sidecast and sediment dam statically and under the OBE was considered sufficiently close to the design requirements to be acceptable. Liquefaction of the impounded sediments would cause post-liquefaction settlements in the order of 1-2m (*Ambraseys 1988* and *Braziar 1992)*. Given that the slope is beginning to yield at the OBE level of shaking, it is anticipated that the MCE event will cause displacements in the order of 1-3m (*Makdisi and Seed*) along the lower portion of the sidecast. For slopes of this type, with low consequence of failure, this order of movement is considered acceptable. It was considered that at this level of shaking many natural slopes in the areas would undergo significant deformations and many natural slips would occur, therefore some disruption of this man-made slope seems reasonable.

CONCLUSIONS

In its present condition, the existing sidecast is considered to be vulnerable to instability. The proposed remedial works to flatten the sidecast slopes and improving the site-wide stormwater drainage should ensure long-term stability. Results of stability analyses undertaken indicate that the permanent slope configuration will be stable for the OBE level of ground shaking. Minor slumping at the toe of the slope is possible due to seismically-induced liquefaction of the impounded sediments under the OBE level of ground shaking, and more significant deformations of the slope will likely occur under the MCE level of shaking. These deformations and the related consequences are considered acceptable.

In relation to the general topic of earthquake engineering, this project highlights:

- The vulnerability of dumped rockfill embankments and slopes in high seismicity zones and the potential need for remedial work on such structures.
- The need for site specific earthquake hazard estimates to accurately define the risk.
- The use of residual liquefied shear strengths to estimate the effects of liquefaction on structures.
- The need to consider slope deformations at extreme levels of earthquake shaking.
- The need to use engineering judgement to decide acceptable structure performance at various levels of earthquake shaking and the need to communicate these to project owners and stakeholders.

FOOTNOTE

This paper was originally presented in the poster session and included in the proceedings of the 2003 Pacific Conference on Earthquake Engineering (Feb 2003). Since this paper was prepared, some slight modifications to the final remedial design were made that were not reflected here. The bulk earthworks are currently underway, and this work is expected to be largely complete by the time this conference convenes.

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