TECHNICAL MEMORANDUM

Golder Associates

Golder Associates Ltd.
#500 - 4260 Still Creek Drive
Burnaby, B.C., Canada V5C 6C6
Telephone: 604-298-6623
Fax Access: 604-298-5253

TO: Cam Scott – SRK Consulting Inc.
DATE: March 24, 2004
FROM: John Cunning, Mike Jefferies
JOB NO: 03-1413-080
EMAIL: JCUNNING@GOLDER.COM
RE: **ROSE CREEK TAILINGS IMPOUNMENT**
INTERMEDIATE DAM STABILITY ASSESSMENT

The following presents our draft Intermediate Dam Stability Assessment for review. This technical memorandum forms Addendum #1 to our February 9, 2004 Draft Rose Creek Tailings Impoundment Site Characterization and Seismic Stability Assessment Report. Following review this is planned to be incorporated as Chapter 9 of the final report.

9.0 SLOPE STABILITY ANALYSES

9.1 Overview

The stability and displacements of the Intermediate Dam during the design earthquake scenario were investigated. The methodology comprised psuedostatic limit equilibrium stability analyses to define a yield ground acceleration, and the subsequent use of the Newmark method to estimate ground displacement. Some key features of this approach are presented as follows.

The psuedostatic method neglects the distribution and time history of inertial forces in the dam during an earthquake. Instead, the effect of ground motion is idealized as a uniform horizontal acceleration acting to destabilize the slope. The output of the analysis is the value of this acceleration that just brings the slope to incipient movement, and this is refereed to as the yield acceleration. The method only considers the non-liquefaction situations. A crucial assumption of the method is that the soil does not weaken during any earthquake induced displacements. Reduced strength values may be appropriate even when considering compact soils to allow for dilation during shear causing a reduction in soil density and correspondingly strength.

The Newmark (1965) method developed from the recognition that almost all earthfill structures will be overloaded by strong ground motion, but that such overloading does not necessarily mean the structure fails. Rather displacements will occur but these can be quite small and stop when the ground motion stops. The displacements in the case of well constructed earth dams are often in the order of less than 1 m and may not affect the serviceability of the sturcture. The Newmark method computes the horizontal displacements by assuming the earthfill above the slip surface identified in the pseudostatic analysis deforms as a rigid block on that surface. The Newmark method can be applied either using simple bounding curves based on dominant characteristics of the design ground motion or with more detailed results using actual earthquake records. The latter approach was used here, with the records being supplied to us by Dr. G. Atkinson (Atkinson 2003).

9.2 Idealization of Dam

A single cross-section of the Intermediate Dam was selected for analysis as presented in Figure 2.8. Referring to the mean sea level elevation datum, the current crest of the dam is at elevation 1049.4 m, the upstream pond assumed to be at the level of the spillway invert of elevation 1047.5 m and the downstream shell of the dam is submerged by the cross valley pond assumed to be at elevation 1031.2 m.

Pseudostatic analyses for earthquake load conditions were carried out for the long-term condition using effective stress strength parameters for drained conditions.

A discussion of the foundation conditions for this dam was presented in Section 2.3.3. The Intermediate Dam foundation ranges from a compact fine grained material in the proximity of the original Rose Creek channel to relatively coarse sand and gravel till and some alluvium near the north abutment and some frozen till with some colluvium near the south abutment. The dam shell was constructed from compacted sand and gravel and the core from glacial till core both obtained from a local borrows. Drainage and filter sand and gravel zones were placed on both sides of the core and under the downstream rockfill shell.

Limited laboratory strength testing data was reviewed for foundation soils, borrow materials used for shell construction and tailings. This data was used to estimate ranges of expected material properties for use in the analyses which is summarized in Table 9-1.

TABLE 9-1: SUMMARY OF MATERIAL PROPERTIES USED IN SLOPE STABILITY ANALYSES

Material	Saturated Unit Weight (kN/m^3)	Effective Stress Parameters		Basis for Property Selection
		Cohesion, c' (kPa)	Angle of Internal Friction (°)	
Dam Core – glacial till	21	0 35	36 32.5	Consolidated drained triaxial testing
Shell – sand and gravel	12	0	32 to 36	Consolidated drained triaxial testing and previous experience with material with similar grain size distribution
Foundation Sand and Gravel	21	0	32 to 36	Previous experience with material with similar grain size distribution
Tailings	24	0	30	Consolidated drained triaxial testing

Phreatic conditions through the dam cross section have been estimated based on the geometry of the inclined till core and assuming that the drainage layer at the base of the downstream shell remains effective. The assumption of drainage layer efficiency requires confirmation in the field, and consideration to the installation of piezometers for this purpose should be given for the site works being planned for 2004.

Figure 9.1 illustrates an example of the idealized dam section used in the analyses. Other idealizations used the same geometry and only varied material properties in accordance with Table 9.1.

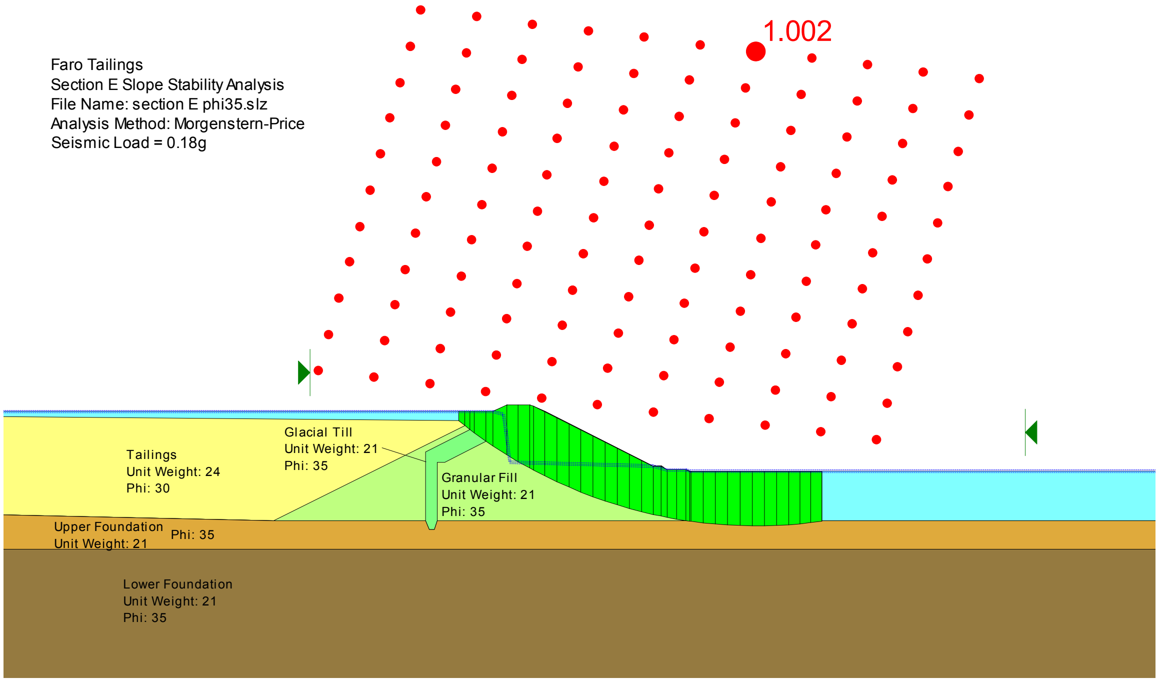


Figure 9.1 – Idealized Dam Cross Section Considered in Analyses

9.3 Estimated Yield Acceleration

Limit equilibrium analysis was carried out with the Slope/W software using a Morgenstern & Price non-circular slip surface method. Slope/W does not output the yield acceleration directly. It is therefore necessary to run a series of analyses with varying horizontal accelerations and plot the computed factor of safety (FOS) against the input acceleration. The yield acceleration is that corresponding to a factor of safety of unity.

The range of possible failure surfaces was limited to those that passing through the dam crest immediate upstream tailings, such that failure on this surface would result in release of tailings impounded by the dam.

Figure 9.2 presents the psuedostatic FOS based against the seismic accelerations calculated for a range of the shell and foundation materials friction angles that correspond to the uncertainty identified in Table 9.1.

The static FOS corresponds to zero horizontal acceleration and ranges from about 1.3 to 1.6, depending upon what the actual friction angles of the shell and foundation are.

As horizontal acceleration is increased, the factor of safety reduces. A FOS = 1 gives the yield acceleration (a_y) which lies in the range $0.11\text{g} < a_y < 0.18\text{g}$, see Figure 9.2.

A FOS less than unity during an earthquake does not mean "failure", merely that the dam will deform. The extent of this deformation is computed in the next section.

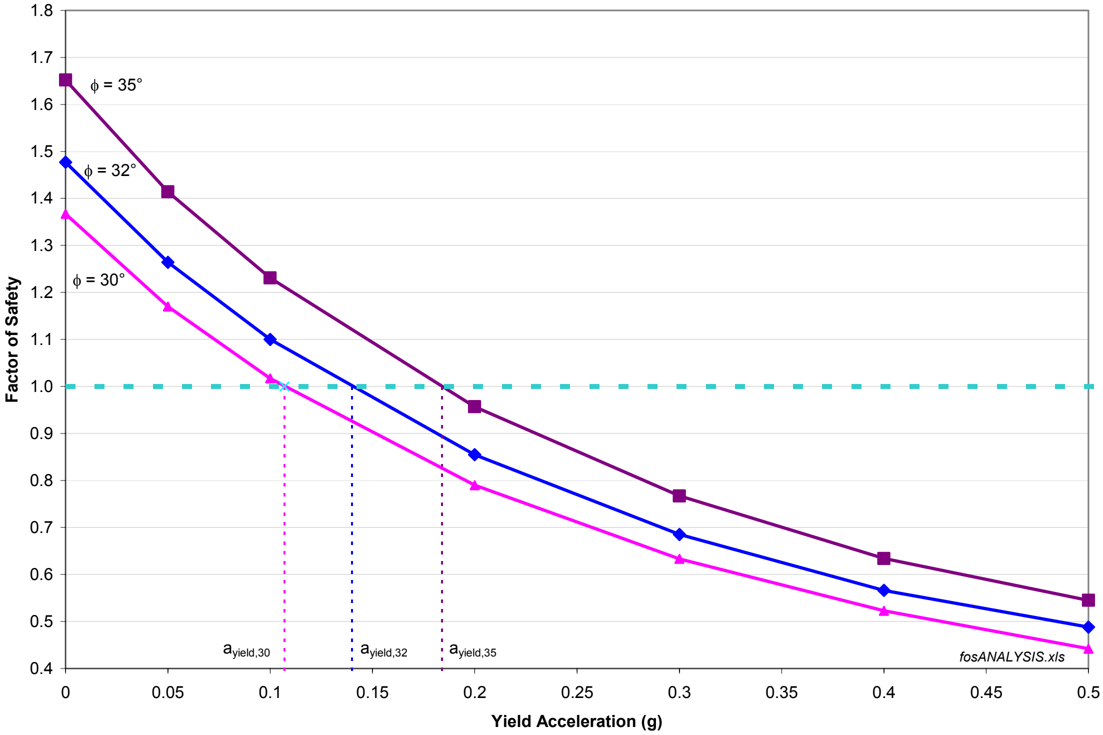


Figure 9.2: Determination of yield acceleration from slope stability analysis

9.4 Deformation Analysis

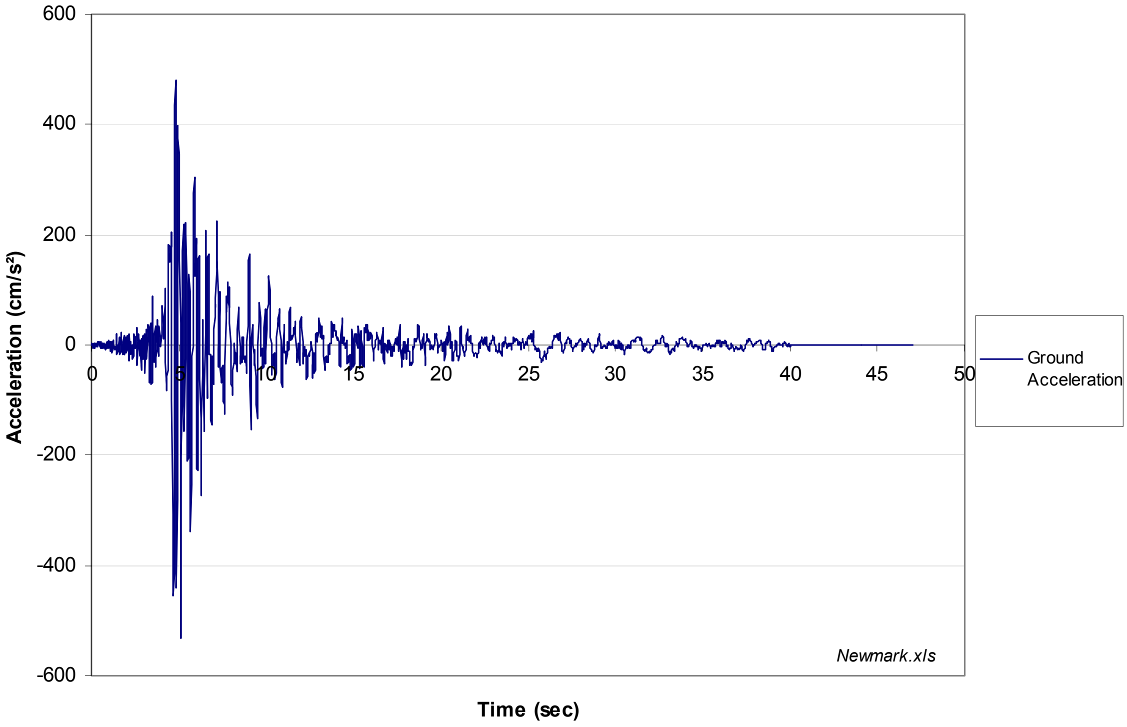
As presented in Section 6.1, the site specific seismic hazard assessment indicates a peak ground acceleration (PGA) in the range from a medium of 0.3 g to a mean of 0.5 g for an 0.0001 p.a. spectrum. Included in the seismic hazard assessment were the acceleration time history records for three earthquake ground motions which would produce a similar PGA for this site (Atkinson 2003). For the present purposes these three strong ground motion records in two directions each, as selected by others, were used for estimating the dam movement. These records are summarized on Table 9.2 (only horizontal records were used).

Table 9.2: Summary of Ground Motion Records

Earthquake	Component	a_{max} (g)	v_{max} (cm/s)	Newmark Method Maximum Calculated Displacement (cm)	
				$a_{yield} = 0.11g$	$a_{yield} = 0.18g$
Loma Prieta Earthquake - Gilroy Sewage Plant	90° 0°	0.37 0.54	43.8 35.8	38.5 42.5	34.5 36.4
Loma Prieta Earthquake - Lick Lab	90° 0°	0.41 0.44	21.9 22.0	30.6 36.5	27.2 31.5
Northridge Earthquake - Pacoima Dam	265° 175°	0.43 0.41	31.4 45.0	23.4 23.3	20.4 19.4

The recorded ground motions were used at face value and without computing the effect that dam response might have on them. Although shear modulus and damping of the dam control whether ground motions are amplified or attenuated by the dam, there is no data for these properties in the dam. The calculation is approximate and, as will be illustrated, relatively insensitive to the actual a_{max}.

The Newmark method was implemented in a visual basic program that was verified against the simplified limiting values presented by Newmark (1965). This program output the computed ground and dam displacement, with the overall deformation being taken as the vector sum of the two (the calculations are related to an inertial frame of reference). Figure 9.3 presents and example of the computed output and which shows how displacement accumulates.



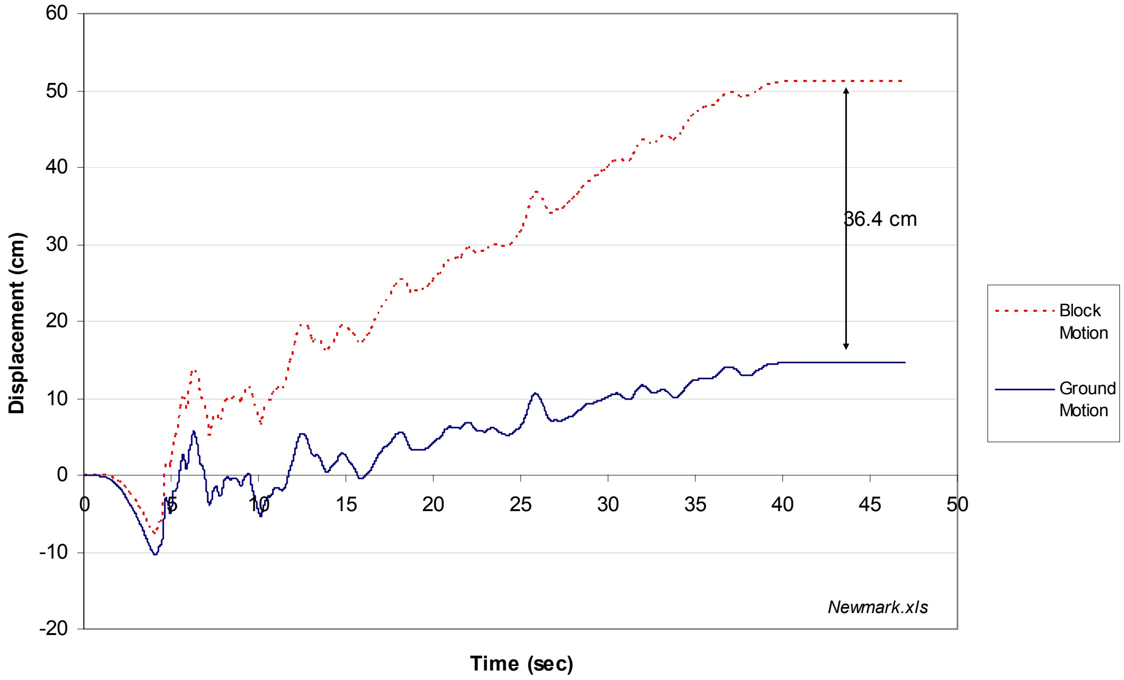


Figure 9.3: Example of computed dam displacement response in Loma Prieta earthquake (Gilroy Sewage Plant, 0°) for $a_{yield} = 0.18g$

Because computed motions are vector, the orientation of the dam to the ground motions matters. This was allowed for by computing with each record twice, once assuming that positive acceleration was orientated downslope and once assuming that negative acceleration was downslope. Whichever assumption gave the greatest displacement was taken as the assumption to adopt for that record. This condition was adopted for all six records provided.

Figure 9.4 shows the results from two simulations of the same record, one with a_y=0.11g and the other with a_y=0.18g. The results of which indicate that the computed yield acceleration range results in about a 5 cm range of computed displacements.

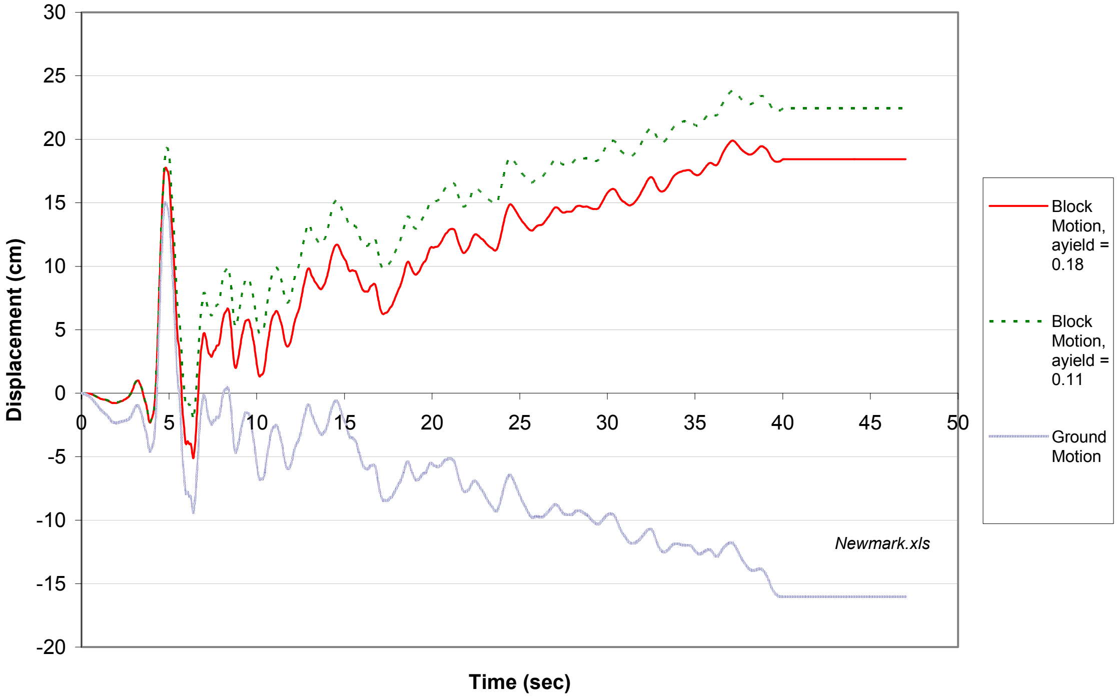


Figure 9.4: Effect of varying yield acceleration on dam displacement

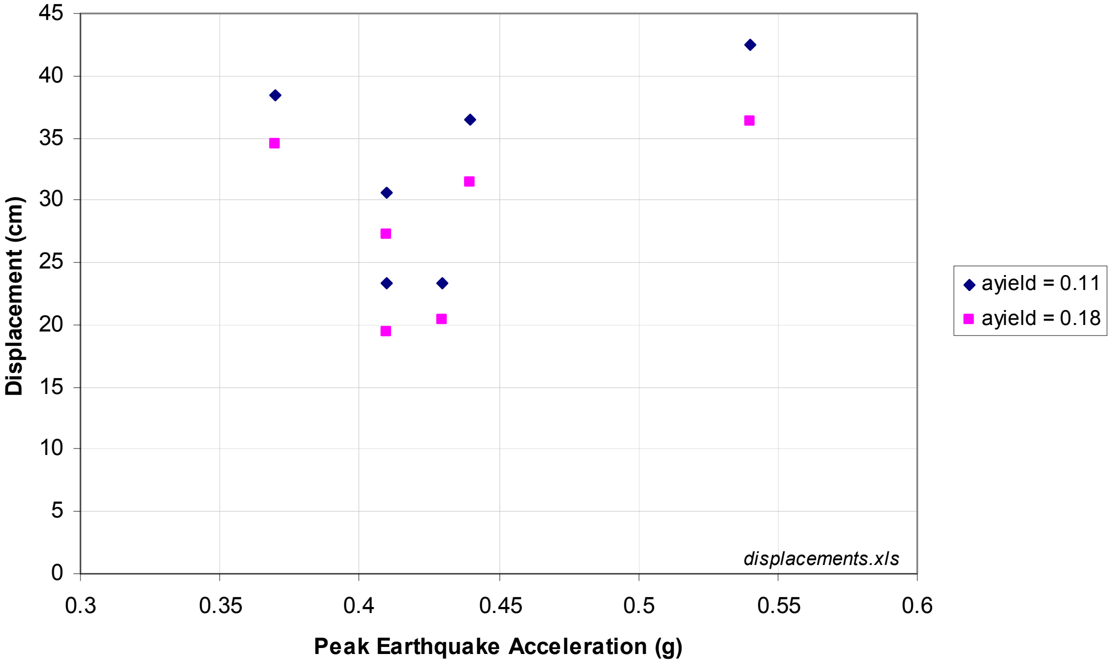


Figure 9.5: Computed maximum displacements of block for various earthquakes

The overall estimates of maximum horizontal displacement are presented on Figure 9.5. For the range of earthquake acceleration time histories considered that the dam would experience between 0.2 and 0.5 m horizontal deformation and it would be estimated that vertical settlement would be about half the horizontal based on the downstream slope angle of the dam.

/