International Conference on Case Histories in Geotechnical Engineering
(1984) - First International Conference on Case Histories in Geotechnical Engineering

08 May 1984, 10:15 am - 5:00 pm

Open Pit Mine Rock Dump Geotechnical Evaluation

Z. M. Zovodni
Kennecott, Engineering Construction, Salt Lake City, Utah

J. D. Tygesen
Kennecott, Utah Copper Division, Bingham Canyon, Utah

S. C. Pereus
SOHIO, Alternate Energy Development, Cleveland, Ohio

Open Pit Mine Rock Dump Geotechnical Evaluation

Z.M. Zovodni

Principal Geotechnical Engineer, Kennecott, Engineering Construction, Salt Lake City, Utah, U.S.A.

J.D. Tygesen

Engineer, Kennecott, Utah Copper Division, Bingham Canyon, Utah, U.S.A.

S.C. Pereus

Economic Analyst, SOHIO, Alternate Energy Development, Cleveland, Ohio, U.S.A.

SYNOPSIS: Open pit mining generally involves moving large quantities of waste rock to disposal areas which are usually located near the mine. This waste rock must be disposed of in a safe, economical, and environmentally acceptable manner. The stability of the waste dump depends to a great extent on the physical properties of the underlying foundation. Information must be obtained to define and assess the strength, consolidation, distribution, topographic and hydrogeologic properties for the foundation materials. Methods for obtaining estimates of the material properties include: laboratory and field testing, back analysis, and indirect estimates from other material properties.

Mining operations in mountainous terrain generally necessitate development of waste rock dumps on areas of moderate to steeply sloping terrain. The design and monitoring of these waste embankments are an integral part of the mine planning function, and present a challenge to the geotechnical engineer. Close coordination with mining operations is also required to ensure proper dump construction.

Described is a case history of a large scale rail dump settlement episode which extended over an area of approximately 20 acres. Boundary and crest tension cracks closely followed original drainage topography leading to the belief that displacements were foundation soil (clay) related. Active and passive blocks were distinctly exhibited. Concentrated dumping with attendant foundation pore pressure buildup were principal causes for the settlement.

INTRODUCTION

Major failures in rock dumps do not occur unless they are either very high (in excess of 500 ft), have poor foundation conditions, or become liquefied ("blowout" failure). For a dump to fail, the shearing resistance of the foundation soil must be significantly less than that of rock dump or the slope of the ground must be considerable (greater than 25°). The current discussion will examine the foundation failure mode of high (greater than 200 ft), single lift mine waste dumps deposited by end dumping from trucks or rail cars onto a residual soil foundation.

WASTE DUMP FOUNDATION INVESTIGATIONS

All potential waste dump sites require a geotechnical foundation investigation. The basic objective of such studies is to identify surface and subsurface soil, rock, and hydrologic properties. Laboratory testing is required to determine the classification, strength, moisture, hydraulic conductivity, attenuation and consolidation characteristics of the foundation material. Zavodni et al (1981) have described these procedures in some detail.

A geologic/topographic base map of the dump foundation area needs to be developed. Physical parameters of major bedrock structural discontinuities (e.g., faults, bedding planes, foliation, joints, contacts) need to be identified for subsequent input into the stability analysis. The topographic map is required to assess the stability, failure runout distance, volume, surface configuration and boulder spray of the dump. Foundation soils should be mapped to assess the presence of weak anomalous zones employing the Unified Soil Classification System.

Subsurface waste dump foundation investigations should also be performed. These could include: (a) backhoe trenching, seismic refraction surveys and shallow boreholes; and (b) hydrogeologic investigations using borehole hydraulic conductivity tests and infiltration tests.

Foundation pore water pressures are believed to play a significant role in dump stability. Pore pressure is known to dissipate very slowly within clay soils. The effective strength of a CL-ML (Unified Soil Classification System) foundation soil is decreased by an increase in pore pressure. Factors influencing pore pressure include:

a) Consolidation characteristics of the soil.
b) Length of the drainage path (thickness of the clay).
c) Hydraulic conductivity of the soil.
d) Time allowed for dissipation.
e) Degree of saturation.
f) Stress history

Results of a foundation soil pore pressure monitoring experiment at the toe of an active waste dump indicate that such a monitoring system can be useful in assessing dump instability as related to foundation failure, Zavodni et al (1981). Ideally, the dumping rate should be adjusted to allow for maximum foundation pore pressure dissipation.

Laboratory tests that should be performed include:

a) Grain size analyses and Atterberg limits.
b) Direct shear for soil and critical rock discontinuities (undrained conditions).
c) Triaxial for soil (consolidated and unconsolidated undrained conditions).
d) Hydraulic conductivity on undisturbed foundation samples.

e) Density and moisture content.
f) Consolidation.
g) Attenuation.
h) Water quality.

The shear strength of the foundation soils is one of the major parameters influencing dump stability. The foundation strength will vary indirectly with clay content and directly with sand/gravel content. Thickness of foundation soils is also an important parameter influencing dump stability. Rock outcrops without unfavorably oriented discontinuities have shown to be a stabilizing agent.

FOUNDATION TOPOGRAPHIC EFFECT

Active dumps resting on steeply inclined foundations (greater than 25°) have been noted by the authors to undergo more frequent surface readjustments and tend to be more prone to blasting induced failures than comparable dumps resting on shallow foundation inclinations.

Lateral and toe foundation topographic constraints improve dump stability; a valley fill will tend to be more stable (i.e., more lateral confinement) than a side hill fill. A toe buttress created by either a previous dump failure or a natural topographic ridge has been demonstrated to significantly improve dump stability.

The foundation inclination also has an important impact on boulder spray distance ahead of a high end dumped waste pile. Aerial photography and ground studies of dump surfaces exceeding 300 ft in height have allowed empirical boulder spray design distances to be formulated. These range from as little as 50-100 ft for a foundation angle of +5° to +25° and extend from 400 ft up to 1,000 ft ahead of a dump toe for a foundation inclination between -11° to -25° (see Table 1).
The nature of the foundation ground cover and waste material type (i.e., size and strength) also influence boulder spray distance.

TABLE I - Boulder Spray For Dumps Exceeding 300 Ft Height

Foundation Angle	Boulder Spray Distance (Max)
+ 5° to +25°	50 - 100 ft
+ 5° to - 5°	100 - 300 ft
- 5° to -11°	300 - 400 ft
-11° to -25°	400 - 1,000 ft

SLIDE RUNOUT DISTANCE

Dump slide debris has been noted to travel further than would be predicted by the static frictional resistance. The term "sturzstrom" has been applied to a slide where mobile frictional resistance is considerably less than the static friction. Campbell and Shaw (1978), have documented a direct empirical relationship between the height of a waste dump and failure runout distance. A similar relationship has been noted by Zavodni et al (1981) after back analyzing several large scale (greater than 10 million tons) dump foundation failures (see Figure 1). It has also been observed that foundation slope (fdt) will alter this empirical relationship (shallower foundations resulting in shorter runout distances) as will the foundation soil conditions and size of slide. Rapid loading of saturated soil ahead of a failing dump may cause the slide material to travel further than under dry conditions.

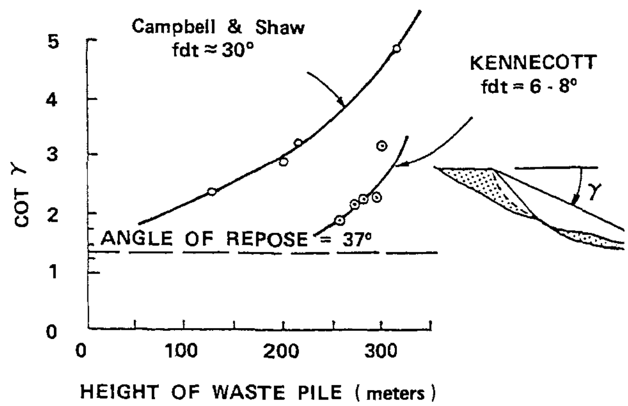

Figure 1 Relationship Between Height of Waste Rock Dump and Runout Angle

DUMP SETTLEMENT

All high end dumped rock waste dumps incur settlement. Crest settlement caused by compaction of material and surface sloughing from oversteepening (due to accumulation of fines near the crest) will occur immediately after waste deposition. Normal crest settlement rates without attendant instability have been measured between 0.4-1.3 ft/day along active 800-1,000 ft high truck dumps at Kennecott. After leaving the truck dump idle for two weeks, 15 ft wide, 4-8 ft deep slump zones have occurred near the berm of the dump. These slump zones typically move along established internal shear zones that parallel the dump face, causing a stair-stepping effect from the berm to the dump back. Settlement rates have not been documented for the train dumps, where low dump heights (less than 250 ft) and evenly distributed dumping application have resulted in smaller crest settlements than for the high truck dumps.

Settlement rate is a function of dump height, loading rate, location from the crest and type of material. The bulk of the movement occurs within the first 2-3 months after material placement. The total cumulative settlement can be as high as 20% of the overall dump height based on Kennecott dump stratigraphic data.

Long term settlement records have not been maintained on mine waste dumps. Prior to the mid to late 1960's, little concern was devoted to proper dump design and construction, let alone monitoring. It was not until failures at Aberfan, Wales (1966) and Buffalo Creek, West Virginia (1966 & 1972) (Singh, 1976) that the hazards of these waste structures were realized. Even today, published data on the vertical movement involved in mine waste embankments are limited. Therefore, long term records of settlement are only available from civil earth dam projects. Settlement in these structures, measured over a 30-year period, shows that 50% of the overall settlement occurs in the first 2-8 years (Sherard et al 1967).

CASE HISTORY

A large scale rail dump settlement/slide event developed between May 1981 and August 1982. The slide extended over a 20 acre surface area and involved some 4.3 million tons of mine waste rock. The height of the waste dump was approximately 250 ft and the main crest tension crack was located some 450 ft from the dump crest in August 1982. Approximately 2 million tons of waste rock were deposited in the active settlement area between May 1981 and August 1982. Figure 2 shows the rail dump settlement/slide area.

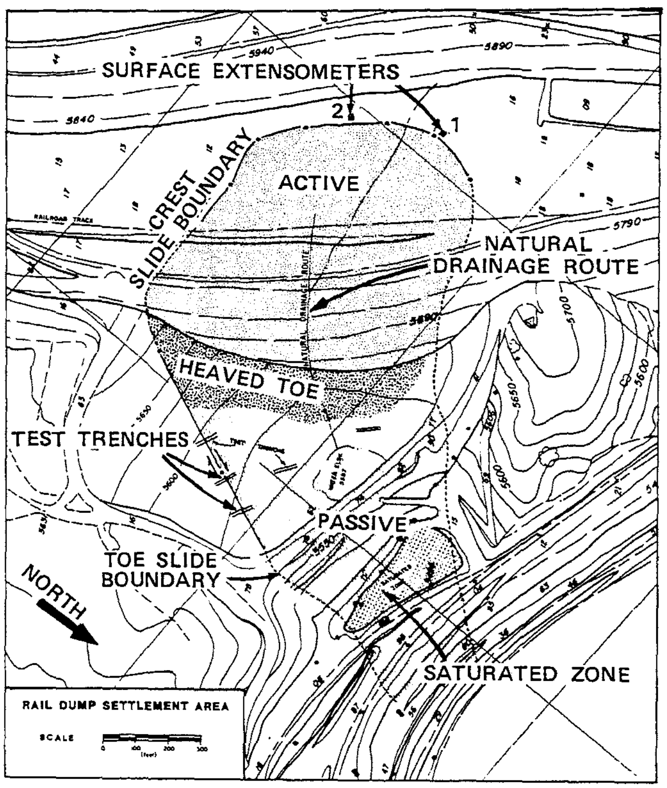

Figure 2 - Rail Dump Settlement Area

Site Characteristics

The rail dump was deposited on a residual soil foundation partly within a narrow drainage basin having an average slope of 13°. The foundation is composed of low plasticity silt and clay soils (CL-ML) ranging from a thin veneer on the valley abutments to about 20 ft in the valley center. The soil is largely saturated and rests on latite breccia bedrock. In situ borehole conductivity tests revealed hydraulic conductivity values ranging from 1 to 7×10^{-5} cm/sec for the fine-grained soils and from 2×10^{-6} to 9×10^{-8} cm/sec for the volcanic bedrock.

Triaxial consolidated undrained tests with and without pore pressure measurements were performed on typical silty clay foundation material. Employing the linear Mohr-Coulomb failure envelope, cohesions (c) ranged from 0-2,600 psf and angles of internal friction (Ø) varied from 5-23°.

Settlement/Slide History

The rail dump settlement/slide area was first detected in May 1981 when crest tension cracks were noted during a routine visual inspection. (Other early warning monitoring techniques currently employed on the dumps include: (a) continuous measurement of crest settlement by a laser beacon and toe displacement by inclinometers; and (b) periodic level surveying of the crest). The failure was considerably smaller than shown in Figure 2 as only 250 ft separated the dump crest from the rear tension crack where a differential displacement of 0.5 ft had developed. No evidence of seepage or movement was noted at the toe and no ground cracking was observed ahead of the dump toe. A foundation creep failure was suspected, triggered by concentrated dumping with attendant high foundation pore pressure buildup.

A wire line extensometer was immediately installed across the rear crescent shaped tension crack. The largest crack displacement was noted directly above the former stream channel. The tension crack paralleled the original drainage basin topography, leading to the conclusion that displacements were foundation soil related.

Slide displacement was monitored continuously employing modified Stevens Type F water-level recorders adapted as extensometers. (A second extensometer was installed in July 1982). The extensometers were oriented to best record net vector displacement. These instruments are rugged, provide reliable data and do not require electrical power. They are widely used by Kennecott in pit and waste dump monitoring. The extensometers are attached to trip switches that activate a warning device upon significant displacement. Figure 3 illustrates the rail dump displacement record during July and August 1982. The peak velocity noted during the entire rail dump slide episode was 2.5 inches/day in mid-July 1982.

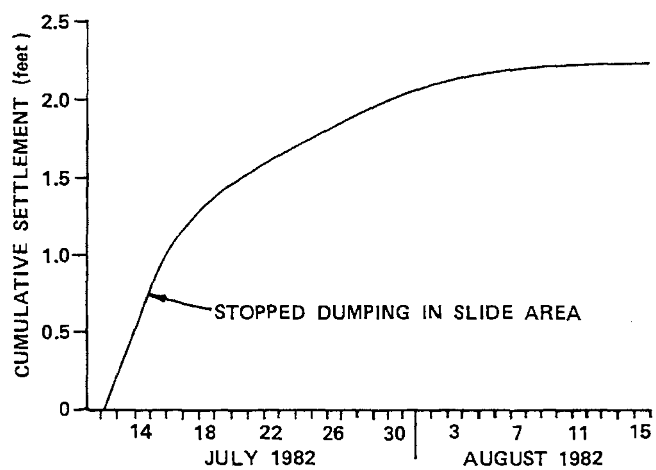

Figure 3 - Cumulative Settlement From Surface Extensometer #2

The settlement area was back analyzed in May 1981 using the Spencer limiting equilibrium analysis. A 0.93 safety factor was computed. Active dumping and monitoring continued through mid-July 1982 at this economically favorable "short dump" site. Between May 1981 and July

1982 a total net displacement of 25.6 ft was recorded at the crest tension crack. The main crest tension crack location remained constant during the entire 15 months of slide activity. Periodic track alignments and dump backfilling were required during the gradual creep failure.

Dump toe heave, along with active seepage was first noticed in November 1981. This was followed in April 1982 by the development of extensive soil cracks ahead of the dump toe and an increased dump movement rate. The toe cracks extended up to 500 ft ahead of the toe heave zone over an area that had become saturated. Such an extensive passive block slide had not been previously observed at this property. It was monitored daily along the lateral crack boundaries where a peak displacement of 1.25 inches/day was recorded in mid-July 1982. A survey network was also established at several points within the moving passive block.

By July 1982 the dump toe had heaved six to twelve feet vertically and the passive block slide cracks had advanced some 800 ft ahead of the toe heave zone. A maximum cumulative transverse movement of approximately 45 ft had been recorded along the southern toe heave slide boundary. The development of the far reaching toe cracks had not been anticipated and it became apparent that remedial measures had to be taken to prevent encroachment on important mining structures down slope.

Stability Analyses

Stability analyses were performed to determine the safest, most economical method of dump stabilization. The settlement area was back analyzed using the Spencer limiting equilibrium analysis. Two models were utilized: (a) Case 1 represented a waste dump mass sliding along a weak, continuous, saturated clay soil foundation (approximately 15 ft thick) paralleling bedrock; (b) Case 2 assumed the slide to consist of an active and passive block. Stabilization measures evaluated included: toe surcharge, trenching to bedrock, and dump slope angle reduction. Results of the analyses and assumptions are presented in Table II. Figures 4 and 5 show typical summarized models.

TABLE II - Rail Dump Stability Analyses

Case	Safety Factor	Comments
I*	0.58	Failure plane exits at toe bulge. (Existing condition 7/82)
IA*	0.59	90' high surcharge without trench.
IB*	0.56	75' wide excavation at toe of dump.
IC*	1.04	150' wide excavation at toe of dump with a 90' surcharge.
ID*	1.24	70' wide excavation just past the toe bulge, 90' high surcharge employing waste dump material thereby reducing dump slope angle from 37° angle of repose to 20°.
IIA**	1.01	Slide consists of two blocks. Ø obtained by back calculating @ FS = 1.0. (Existing condition 7/82)

TABLE II (continued)

Case	Safety Factor	Comments
IIB**	0.93	Block 1; 75' wide excavation just pas toe bulge.
IIC**	0.96	Block 1 and portion of Block 2; 75' wide excavation 400 ft past toe bulge
IID**	1.52	Block 2; passive section of slide ma: only.

* Analysis employs laboratory determined soil foun tion shear strength Ø = 7°; c = 180 psf

** Analysis employs back calculated shear strength Ø = 10°; c = 180 psf

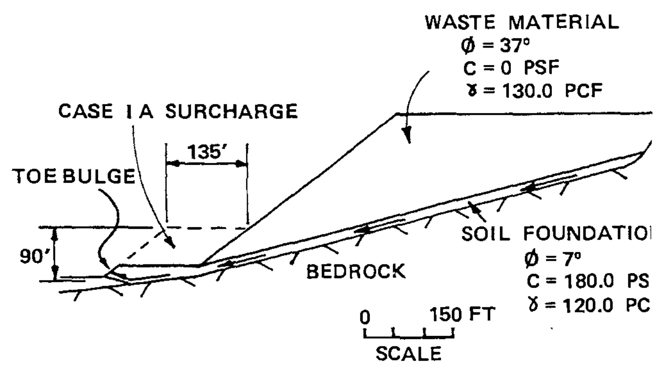

Figure 4 - Rail Dump Stability Models for Analyses of Cases I and IA

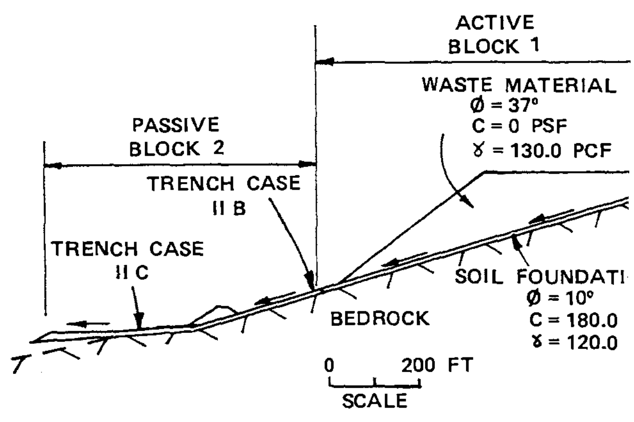

Figure 5 - Rail Dump Stability Model for Analysis of Case IIA

Case 1 (Fig. 4) represents the existing (July 1982) condition where the waste dump mass is sliding along the weak, continuous, saturated clay soil foundation (approximately 15 ft thick) paralleling bedrock. The computed safety factor of 0.58 was considerably lower than the 0.93 calculated 14 months earlier using the same strength parameters when the slide was of much smaller proportions.

Cases 1A-1D assessed the impact of various dump stabilization measures including: toe surcharge, a trench to bedrock extending some 700 ft laterally to intercept the entire clay failure zone, and dump slope angle reduction. Case 1A (Fig. 4) demonstrated that a toe rock surcharge placed directly over the clay foundation bulge would have practically no impact on increasing stability. Case 1B demonstrated that a 75 ft wide trench excavated to bedrock directly at the toe bulge would slightly decrease the safety factor. Case 1C demonstrated that with continued dumping, a 150 ft wide trench to bedrock with a 90 ft surcharge would be required to attain limiting equilibrium; the trench and surcharge would have to extend across the entire dump toe region. Case 1D demonstrated that a S.F. of 1.24 could be attained by reducing the overall dump slope angle to 20° and constructing a toe surcharge.

Case II (Fig 5) assumed that the slide consisted of two blocks (1 = active block; 2 = passive block), and that the known moving mass had a safety factor approaching 1.0. The purpose of this analysis was to assess the influence of alternate trench excavations on the stability of the moving mass and to back calculate foundation strength parameters. Case IIA demonstrated the existing (July 1982) condition; it resulted in back calculated foundation strength parameters that closely matched the laboratory determined values (Ø = 10° vs. 7°).

Failure interceptor trenches 75 ft wide extending to bedrock across the entire moving mass were analyzed at two locations. The Case IIB trench was just past the toe bulge and the Case IIC trench was 400 ft past the toe bulge (see Fig. 5). Safety factors of 0.93 and 0.96 demonstrated that the overall stability of the slide would be reduced if a trench was excavated in the lower block. However, the impact of the trench on stability would be minimized by placing the trench as far from the dump toe bulge as possible. Both trenches would intercept the failure surface and thereby stabilize the down slope area by separating the active and passive blocks. The Case IIC trench would extend some 400 ft laterally as opposed to the 700 ft long Case IIB trench.

Case IID evaluated the stability of passive Block 2 alone assuming that an open trench would be placed below the dump toe bulge. The safety factor for this case was 1.52, indicating an acceptable degree of stability for the lower passive block if separated from the active block.

Three test trenches were cut to bedrock in July 1982 along the eastern toe boundary (see Fig 2). The trenches were successful in showing that the sliding mass could be separated into an active and passive section. However, the trenches did not intersect the entire moving mass, causing the existing stresses to be diverted developing new failure boundaries. For a relief trench to be successful, it had to intersect the entire moving mass.

Stabilization Measures

Geotechnical and operational options were presented to mine management. Geotechnical recommendations were to discontinue dumping in the settlement/slide area and reduce water infiltration into the passive block. If these steps failed to stabilize the passive block, then a trench to bedrock should be constructed some 400 ft past the toe bulge, (Case IIC) thereby stabilizing the critical down slope block. Once the settlement/slide area had stabilized, limited dumping should only be resumed if short haul economics were more favorable than dump foundation preparation. This preparation would include a 75 ft wide toe interceptor trench to bedrock filled with rock dump material to a height of 90 ft reducing the overall dump face angle to approximately 20° (Case ID).

Management decided to immediately discontinue dumping in the slide area (July 15, 1982) and to reduce water infiltration into the passive block by installing a sump with a gravity flow pipeline network directly below the toe bulge. These actions successfully arrested the rail dump settlement (see Fig. 3). Dumping will not be renewed at the site and the toe drainage system will be maintained.

CONCLUSIONS

The rail dump settlement/slide episode provided valuable geotechnical data for a large mass at semi-equilibrium under constantly varying loading conditions. Active and passive blocks were clearly exhibited; the passive block extended over an unusually large distance ahead of the active dump failure toe. It was demonstrated that the settlement/slide movement rate was directly linked to the dumping rate. Numerical stabilization analyses revealed that a careful study is required to prevent toe surcharge from possibly increasing the driving force and excavation from accelerating the moving block. The early warning dump monitoring program proved to be effective; continuous crest monitoring allowed dumping under controlled conditions.

ACKNOWLEDGEMENTS

Sincere appreciation is extended to Kennecott for permission to publish this paper. The contributions made by various Kennecott engineers are greatly appreciated.

REFERENCES

Campbell, D.B. and Shaw, W.H. (1978), "Performance of a Waste Rock Dump on Moderate to Steeply Sloping Foundations," Proc. of the First International Symposium on Stability in Coal Mining, Vancouver, B.C., Canada, Chapter 27.

Sherard, J.L., Woodward, R.J., Gizienski, S.F., and Clevenger, W.A., (1967), "Earth and Earth-Rock Dams Engineering Problems of Design and Construction", John Wiley and Sons, Inc., New York.

Singh, M.M., (1976), "Mine Waste Embankments Need Same Engineering Attention as Earth Fill Dams," Coal Age, May.

Zavodni, Z.M., Trexler, B.D. Jr., and Pilz, J. (1981), "Waste Dump Foundation Investigations and Treatment", Design of Non-Impounding Mine Waste Dumps, AIME Fall Meeting Conference, Denver, Colorado, U.S.A.