EEG-82

EVALUATION OF PROPOSED PANEL CLOSURE MODIFICATIONS AT WIPP

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December 2001

FOREWORD

The purpose of the New Mexico Environmental Evaluation Group (EEG) is to conduct an independent technical evaluation of the Waste Isolation Pilot Plant (WIPP) Project to ensure the protection of the public health and safety and the environment of New Mexico. The WIPP Project, located in southeastern New Mexico, became operational in March 1999 for the disposal of transuranic (TRU) radioactive wastes generated by the national defense programs. The EEG was established in 1978 with funds provided by the U. S. Department of Energy (DOE) to the State of New Mexico. Public Law 100-456, the National Defense Authorization Act, Fiscal Year 1989, Section 1433, assigned EEG to the New Mexico Institute of Mining and Technology and continued the original contract DE-AC04-79AL10752 through DOE contract DE-ACO4-89AL58309. The National Defense Authorization Act for Fiscal Year 1994, Public Law 103-160, and the National Defense Authorization Act for Fiscal Year 2000, Public Law 106-65, continued the authorization.

EEG performs independent technical analyses of the suitability of the proposed site; the design of the repository, its operation, and its long-term integrity; suitability and safety of the transportation systems; suitability of the Waste Acceptance Criteria and the compliance of the generator sites with them; and related subjects. These analyses include assessments of reports issued by the DOE and its contractors, other federal agencies and organizations, as they relate to the potential health, safety and environmental impacts associated with WIPP. Another important function of EEG is the independent on- and off-site environmental monitoring of radioactivity in air, water, and soil.

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Director

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ACKNOWLEDGMENTS

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ACRONYMS

AAR   Alkali aggregate reactivity

DOE   U.S. Department of Energy

EEG   Environmental Evaluation Group

EPA   U.S. Environmental Protection Agency

TRU   Transuranic

WIPP   Waste Isolation Pilot Plant

EXECUTIVE SUMMARY

A key component in the design of the WIPP repository is the installation of concrete structures as panel seals in the intake and exhaust drifts after a panel has been filled with waste containers. As noted in the EPA final rule, the panel seal closure system is intended to block brine flow between the waste panels at the WIPP. On April 17, 2001, the DOE proposed seven modifications to the EPA concerning the design of the panel closure system.

EPA approval of these modifications is necessary since the details of the panel design are specified in EPA's final rule as a condition for WIPP certification. However, the EPA has not determined whether a rulemaking would be required for these proposed design modifications. On September 4, 2001, the DOE withdrew the request, noting that it would be resubmitted on a future date.

The Environmental Evaluation Group (EEG) contracted with two engineers, Dr. John Abel and Dr. Rusty Morgan, to evaluate the proposed modifications. The EEG has accepted the conclusions and recommendations from these two experts: 1) replacement of Salado Mass Concrete with a generic salt-based concrete; 2) replacement of the explosion wall with a construction wall; 3) replacement of freshwater grouting with salt-based grouting; 4) option to allow surface or underground mixing; and 5) option to allow up to one year for completion of closure. The proposed modification to allow local carbonate river rock as aggregate is acceptable pending demonstration that no problems will exist in the resulting concrete. The proposed modification to give the contractor discretion in removal of steel forms is not supported. Instead, several recommendations are made to specifically reduce the number of forms left, thereby reducing potential migration pathways.

1.0 INTRODUCTION

The Waste Isolation Pilot Plant (WIPP) Project, located in Southeastern New Mexico has been constructed by the U.S. Department of Energy (DOE) to provide permanent disposal of long-lived transuranic (TRU) waste from the U.S. defense activities and programs. The facility must comply with 40 CFR 191, Subpart A during the period when radioactive waste are being emplaced (operating period) and with 40 CFR 191, Subpart B and 40 CFR 194 for long-term disposal. The U.S. Environmental Protection Agency (EPA) concluded that WIPP met the requirements of 40 CFR 191 and 194 and made a Certification Decision in May 1998 (EPA 1998). The repository began receiving radioactive TRU wastes in March 1999.

The underground WIPP facility design includes eight panels for disposing of transuranic waste (see Figure 1). At the present time waste is being emplaced in Panel 1 and excavation of Panel 2 has been completed. Each panel includes seven waste disposal rooms as well as a ventilation intake drift and a ventilation exhaust drift.

A key component in the design of the WIPP repository is the installation of concrete structures as panel seals in the intake and exhaust drifts after a panel has been filled with waste containers. The panel seals are required to rectify the damage done to the natural formation by excavation and are, at best, an imperfect attempt to recapture the characteristics of the original rock (Silva and Chaturvedi 1995). As noted in the EPA final rule, the panel seal closure system is intended to block brine flow between the waste panels at the WIPP. The DOE application (DOE 1996a) identified four design options. As a specific condition of compliance, the EPA mandated the use of Option D. But the agency also determined that the use of a Salado Mass Concrete – using brine rather than fresh water – would produce concrete seal permeabilities in the repository more consistent with the values used in the DOE performance assessment (EPA 1998, 27355).

In an April 17, 2001 letter from Dr. Inés Triay to Mr. Frank Marcinowski of EPA (Appendix A), the Carlsbad Field Office (CBFO) of the DOE proposed several panel closure design
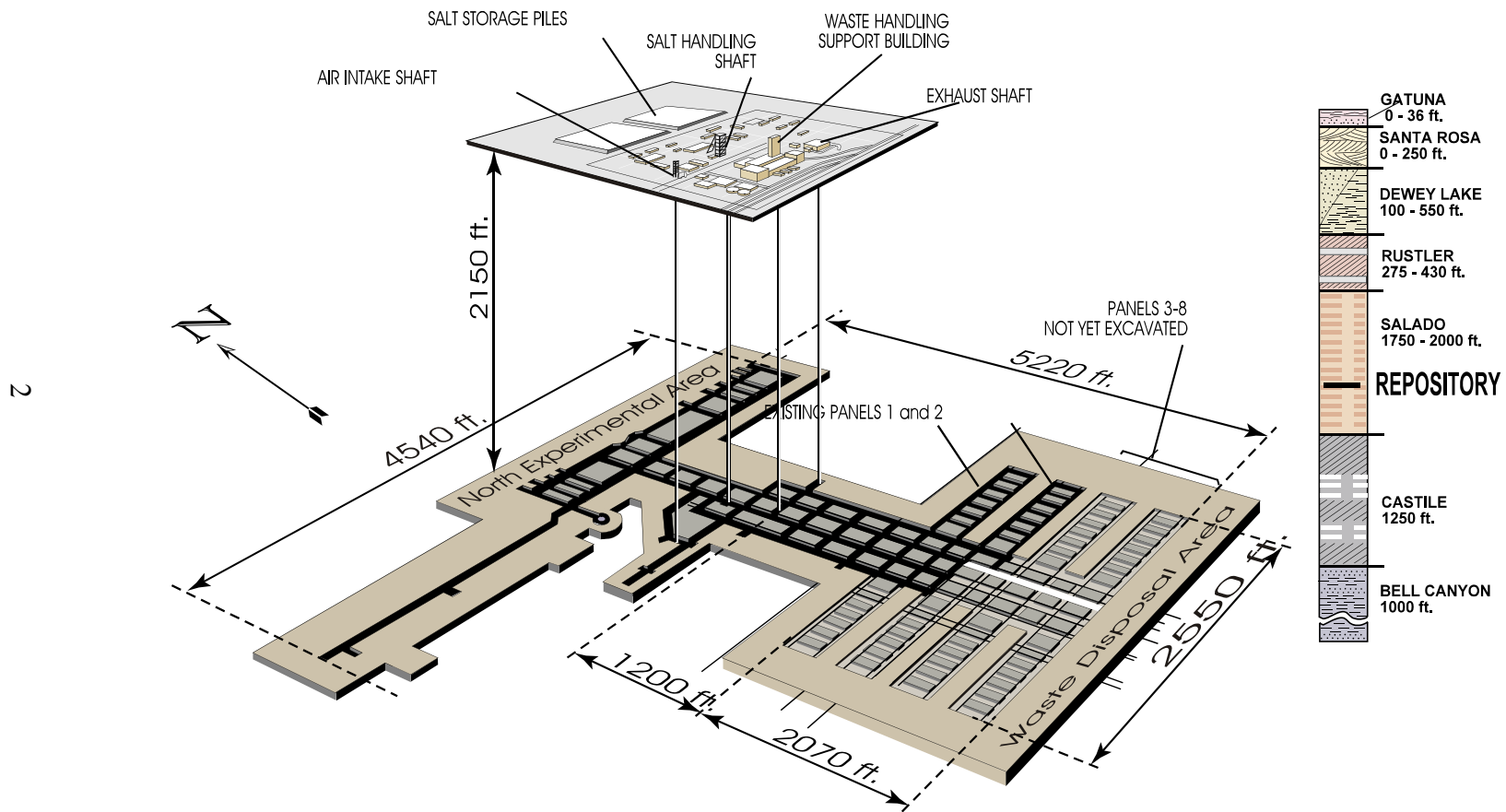
Figure 1. The WIPP facility and stratigraphic sequence. Panel 1 is currently in use. The mining of Panel 2 was completed on October 13, 2000. SOURCE: DOE, 2000.

modifications. EPA approval of the changes proposed by DOE is required since the details of the panel designs are specified in EPA's final rule as a condition for WIPP certification (EPA 1998, 27355). The EPA final rule allows for a modification to the design of the facility. Significant modification requires a rulemaking in accordance with the WIPP compliance criteria (40 CFR §§ 194.65-66). [1]

The Environmental Evaluation Group (EEG), in its role of providing technical evaluations on the design, construction, and operation of the WIPP Project, contracted with two engineers that are expert in relevant aspects of panel seal design and construction to evaluate these proposed enhancements. Dr. John F. Abel, Jr., a Mining Engineer from Golden, Colorado, evaluated the proposed enhancements concentrating on bulkhead and masonry wall stability. Dr. D. R. Morgan, a Materials Engineer from Vancouver, B.C., Canada, evaluated three of the proposed enhancements: (a) changes in proposed aggregate; (b) change to a salt-based grout; and (c) change in mass concrete requirements. The reports of Dr. Abel and Dr. Morgan are included as Appendices B and C to this report. The EEG has accepted the conclusion and recommendations contained in these two reports as summarized below.

The proposed enhancements were subsequently withdrawn from consideration by DOE in Dr. Triay's letter to Mr. Marcinowski, dated September 4, 2001 (Appendix D). This letter indicated that the topic was expected to be revisited at some time in the future. Toward this end, and because of the time spent in evaluation of the proposed modifications, EEG decided to proceed with this report on the proposed modfications.

2.0 PROPOSED PANEL CLOSURE DESIGN MODIFICATIONS

The proposed enhancements are discussed in the order used in Dr. Triay's letter of April 17, 2001. More details can be obtained from the appended reports.

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[1] The EPA has not yet published an opinion as to whether or not the changes proposed by DOE constitute a modification.

2.1   Replace Salado Mass Concrete with a Generic Salt-based Concrete

This proposed enhancement is acceptable and probably preferable since it gives the Contractor more flexibility and responsibility in meeting performance-based objectives. However, in order to ensure adequate performance, the project specification should be written in rigorous performance-based specification language. In addition, more detail should be provided in the specification regarding permissible constituent materials for the mass concrete components such as salt and shrinkage compensating materials.

It is appropriate that the Contractor be supplied with pertinent information regarding specifications for Salado Mass Concrete. This information can provide the Contractor with a starting point for generic salt-based mixture proportioning. However, the responsibility for concrete performance would reside with the Contractor.

2.2   Replace the Explosion Wall with a Construction Wall

This proposed enhancement is acceptable. The analysis in Dr. Abel's report indicates that the 12-foot thick explosion-isolation masonry wall is not needed. The panel closure bulkhead will adequately protect against the design basis 480 psi methane explosion which cannot occur prior to (at least) 15 years after panel closure. The strength of the 4-foot thick construction-isolation masonry walls is sufficient to protect against the design pressure generated by a roof fall within the panel.

2.3   Replace Freshwater Grouting with Salt-based Grouting

EEG agrees with the proposal to replace the freshwater grout with a salt-based grout since it will counteract the tendency for dissolution (and hence void formation) of fresh water based grouts. This is apparently only a point for clarification. The design report detailing the original panel closure options (DOE 1996), specifies that if the Salado Mass Concrete is used instead of a fresh

water/plain cement concrete, the contractor shall use a salt saturated grout. This would be the case for any salt-based concrete.

2.4   Option to Allow Local Carbonate River Rock Aggregate in Lieu of Crushed Quartz

It may be possible to demonstrate that this option is acceptable. However, Dr. Morgan raised three concerns. One concern is that the coefficient of thermal expansion of the aggregate influences the coefficient of expansion of the concrete containing such aggregate. Dr. Morgan states, "Serious differences in the coefficients of thermal expansion have been reported to occur with aggregates with very low expansion, such as certain granites, limestones, and marbles." Therefore, it will be necessary to demonstrate that this is not a problem with the proposed local carbonate river rock.

A second concern is that naturally rounded gravels used in concrete production are better if they have a certain "crush-count". Dr. Morgan states, "There are certain advantages to having partially fractured faces in a sufficient percentage of the aggregate particles, including enhanced compressive, flexural and tensile strength development in the concrete made with such particles, compared to concrete made with natural rounded particles only." Consideration should be given to using aggregate with a partial crush-count if this option is chosen.

A final concern is that some carbonate aggregate is chemically reactive, resulting in deleterious expansion of the concrete. Therefore, an evaluation of the alkali aggregate reactivity (AAR) susceptibility should be conducted.

2.5   Option to Allow Surface or Underground Mixing

This proposed enhancement is acceptable. Dr. Abel concluded that either surface or underground mixing was adequate, provided the critical time between mixing and placement in the form is met. It may be easier to meet the time limitation by underground mixing. In fact, as with Proposed Enhancement I (replacement of Salado Mass Concrete with a generic salt-based

concrete), it gives the contractor more flexibility and responsibility in meeting performance based objectives.

2.6   Option to Allow Steel Forms to be Left in Place or Removed

This option is more complicated than the title implies because it also would allow the contractor the flexibility to modify the design of the bulkhead. Abel's analysis of the current design and his recommendations should be seriously considered. The current four cell design with the steel forms remaining is inferior to a monolithic single cell because there are many more potential leakage flow paths through the bulkhead. However, the size of these bulkheads exceeds that of known continuous pours. Abel recommends the following approach for dealing with this dilemma:

It is recommended that the panel bulkhead specifications:
1)   provide an incentive for the contractor to minimize the number of cells (preferably to one).
2)   require that each cell be filled as a continuous monolithic concrete pour,
3)   require the contractor support the fluid concrete in all cells with external structures,
4)   require the contractor to remove the support structures and forms between internal cells,
5)   provide for a rough form surface between internal cell walls (possibly with a layer of burlap),
6)   assure that some grout points are located at the roof concrete/rock salt contact and
7)   prevent the use of all internal form spacer supports.

2.7   Option to Allow up to One-year for Completion of Closure in Lieu of 180 Days

This option is acceptable. Significant gas generation concentrations take much longer than one year to occur and it is preferable to do the construction properly without the pressure of an artificial deadline.

3.0 CONCLUSIONS

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Of the seven proposed modifications, the EEG readily accepts five: 1) replacement of Salado Mass Concrete with a generic salt-based concrete, 2) replacement of the explosion wall with a construction wall, 3) replacement of freshwater grouting with salt-based grouting, 4) option to allow surface or underground mixing, and 5) option to allow up to one-year for completion of closure. The proposed modification to allow local carbonate river rock is acceptable pending demonstration that no problems will exist in the resulting concrete. The proposed modification to give the contractor discretion in removal of steel forms is not supported by the EEG. Instead, the EEG has proposed a number of recommendations to specifically reduce the number of forms which are left, thereby reducing potential migration pathways.

REFERENCES

40 CFR 191 Subpart A. 2001. Environmental standards for management and storage. Title 40, Protection of environment; Chapter I, Environmental Protection Agency; Code of Federal Regulations. Washington (DC): National Archives and Records Administration.

40 CFR 191 Subpart B. 2001. Environmental standards for disposal. Title 40, Protection of environment; Chapter I, Environmental Protection Agency; Code of Federal Regulations. Washington (DC): National Archives and Records Administration.

40 CFR 194. 2001. Criteria for the certification and recertification of the Waste Isolation Pilot Plant's compliance regulations. Title 40, Protection of environment; Chapter I, Environmental Protection Agency; Code of Federal Regulations. Washington (DC): National Archives and Records Administration.

Silva, Matthew; Chaturvedi, Lokesh. 1995. Need for engineered barriers at the Waste Isolation Pilot Plant. In Transactions of the American Nuclear Society, October 29- November 2, 1995. La Grange Park (IL): American Nuclear Society, TANSAO 73:123-124.

[DOE] US Department of Energy. 1996. Detailed design report for an operational phase panel-closure system. Carlsbad: DOE. DOE/WIPP96-2150.

[DOE] US Department of Energy. 1996a. Title 40 CFR 191 compliance certification application for the Waste Isolation Pilot Plant, October 1996. Carlsbad (NM): Carlsbad Area Office. DOE/CAO-1996-2184. 21 volumes.

[EPA] US Environmental Protection Agency. 1998 May 18. Criteria for the certification and recertification of the Waste Isolation Pilot Plant's compliance with 40 CFR 191 disposal regulations: certification decision; final rule. Federal Register 63(5):27354-27406.

APPENDICES

APPENDIX A

April 17, 2001 letter, Triay to Marcinowski
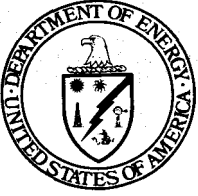
Department of Energy
Carlsbad Area Office
P. O. Box 3090
Carlsbad, New Mexico 88221

April 17, 2001

Mr. Frank Marcinowski
Office of Radiation and Indoor Air
U.S. Environmental Protection Agency
401 M. Street, S. W.
Washington, DC 20460

Dear Mr. Marcinowski:

This purpose of this letter is to inform the Environmental Protection Agency (EPA), per the requirements of the Title 40 CFR Part 194 Final Rule, Supplementary Information, Section VIII.A.1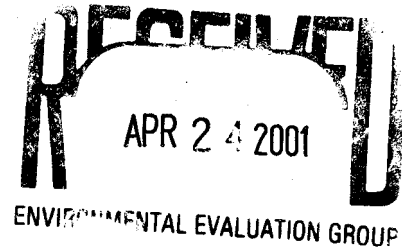.(b), regarding minor enhancements proposed to the panel closure construction specifications for the Waste Isolation Pilot Plant (WIPP) repository. As you are aware, the purpose of the panel closures is for the hazardous waste disposal unit closure and to control potential volatile organic compound (VOC) releases during waste management operations.

Secondarily, in terms of long-term performance, the Compliance Certification Application and the EPA final rule note that the closure system will also influence fluid connections between waste panels. The present panel closure design, as required in the EPA final rule, is extremely conservative (restricting VOC releases to levels that are more than two orders of magnitude less than the applicable standard). We previously briefed your agency regarding this subject on April 13, 2000 in Washington, DC and on December 12, 2000 in Carlsbad, NM.

The enhancements were identified during our continuing evaluation of engineering issues associated with operating the WIPP. The identified improvements include the following:

- Replace Salado Mass Concrete with a generic salt-based concrete
- Replace the Explosion Wall with a Construction Wall
- Replace freshwater grouting with salt-based grouting
- Option to allow local carbonate river rock aggregate in lieu of crushed quartz
- Option to allow surface- or underground-mixing
- Option to allow steel forms to be left in place or removed
- Option to allow up to one-year for completion of closure in lieu of 180-days

CBFO:ORC:DDM:SJJ:01-0753:UFC:5486
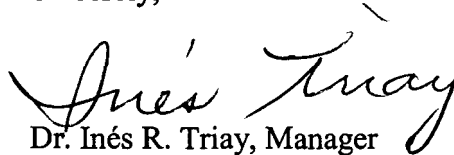
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The enclosed package contains a detailed description of the enhancements and our analysis of their effects. Included also are edited versions of the technical specifications for the panel closures which incorporate the enhancements. Our analysis demonstrates that these enhancements are clearly minor, will not compromise the performance of the closures, and do not impact long-term compliance. However, we believe that implementation of these enhancements would allow construction flexibility which will increase worker safety and greatly improve the constructibility of the panel closures and better ensure the closures perform as required.

If you or your staff have any questions regarding this matter, please contact Daryl Mercer at (505) 234-7452.

Sincerely,

Dr. Inés R. Triay, Manager
Carlsbad Field Office

Enclosure: Panel Closure Enhancements

cc w/ enclosure:
D. Huizenga, DOE EM
M. Kruger, EPA-ORIA
S. Ghose, EPA-ORIA
C. Byrum, EPA-ORIA
N. Stone, EPA, Region VI
M. Silva, EEG
B. Lilly, DOE/CBFO
J. Plum, DOE/CBFO
D. Mercer, DOE/CBFO
G. Maples, WTS
F. Hansen, SNL

cc w/o enclosure:
S. Hunt, DOE/CBFO
H. Johnson, DOE/CBFO

CBFO:ORC:DDM:SJJ:01-0753:UFC:5486

APPENDIX B

Review of Panel Closure Bulkhead Enhancements, Waste Isolation Pilot Plant
John F. Abel

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REVIEW OF PANEL CLOSURE BULKHEAD ENHANCEMENTS

WASTE ISOLATION PILOT PLANT (WIPP)

Report to

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by

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July 18, 2001

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EXECUTIVE SUMMARY

As indicated on Table 1, the strength of the 4,000 psi plain concrete panel closure bulkheads (Figure 1) is sufficient to resist the 480 psi design pressure from the postulated methane explosion, during and after the planned 35-year life of the facility. The strength of the 4-ft thick construction-isolation masonry walls is sufficient to isolate the panel closure bulkhead construction areas from the design pressure generated by a panel room roof fall. The 12-ft thick explosion-isolation masonry wall, (2,500 psi) solid concrete blocks (3,500 psi) mortared with cement (2,500 psi) and hitched 6-in into the adjacent rock salt, designed for the Intake Drift and the Exhaust Drift is insufficient to resist the 480 psi methane explosion pressure (Figure 2). There is no apparent reason for an explosion-isolation wall during the six-month or one year construction of the panel closure bulkheads. Methane concentration will not increase until panel ventilation is stopped, which should only start when construction starts on the isolation walls (Figure 3).

The "Panel Closure Enhancements" refer to the main panel closure bulkheads as "The concrete monolith ---". Water retaining bulkheads (Figure 4) are normally designed and constructed as monolithic, i.e. continuous, pours without cold joints or interior form walls. The planned four cell panel closure bulkhead construction will not provide a uniform massive bulkhead but will contain three interior steel form walls. Even if a way is found to remove the three interior steel form walls, after the concrete has set in each cell, a smooth surfaced cold joint will still be present between cells. In order of declining potential effectiveness, the following panel closure bulkhead construction modifications should decrease the bulkhead leakage potential,

1) constructing truly monolithic panel closure bulkheads or
2) externally supporting the bulkhead cell forms, i.e. eliminating the form spacers, or
3) reducing the number of cells.

The size of these bulkheads, approximately 990 cu yds for the 36-ft thick Intake Drift and approximately 565 cu yd for the 26-ft thick Exhaust Drift, exceeds known continuous monolithic bulkhead pours. The Jackpot Mine west decline tapered bulkhead, approximately 20.4-ft wide by 15.5-ft high by 25-ft thick, involved only 311 cu yds of monolithic concrete pour. At West Driefontein Mine four approximately 350 cu yd monolithic sand/cement plugs were constructed in twenty days, during a mine flooding emergency.

Penetrations through a water retaining bulkhead are normally minimized, whenever possible, because they represent potential leakage flow paths through bulkheads. Essential penetrations, such as a bypass pipe, include waterstops to assure no leakage along the penetrations. The forms are externally braced. The planned 4 cell panel closure bulkhead (Figure 1) includes the use of longitudinal 1-in diameter form spacers, which also support the form wall until the concrete sets. Figure 5 indicates 55 form spacer penetrations across Cell #2 and Cell #3. Figure 6 indicates 22 form spacer penetrations across Cell #1 and Cell #4. Multiple potential flow paths are apparent across the individual cells and the steel plate and angle forms provide multiple potential connections between form spacers in adjacent cells.

The forms for water impoundment bulkheads are typically constructed of timber posts hitched into the roof and floor and braced externally by pipe columns to floor, wall and roof anchors for temporary support of the fresh concrete during construction. This would eliminate the form spacer bars. However, the bulkhead volumes will probably require at least one internal cold joint. An airtight panel closure seal may be difficult to achieve, by any method, but more so with multiple bulkhead penetrations.

The panel closure bulkheads would be classed as tapered two-way pressure bulkheads, i.e. to resist equal pressure from either side. Tapered water retaining bulkheads are typically installed to increase the shear resistance of a low-strength rock from a hydrostatic pressure acting in only one direction. If the panel closure bulkheads are airtight, closure within the panel rooms could potentially increase to 100 psi on the waste side of the bulkheads 35 years after construction (Figure 7). The two-way taper of the panel closure bulkheads was apparently designed to permit the safe excavation of the disturbed rock zones (DRZ) above and below the intake and exhaust drifts.

The strength of the concrete is critical to the strength of the panel closure bulkheads. The aggregate strength is a minor concrete strength factor which can be compensated for, if necessary, by increasing the cement in the mix. In addition, carbonate aggregate concrete probably has a lower coefficient of thermal expansion than quartz aggregate. Figure 8 indicates that concrete with quartz aggregate has a higher coefficient of expansion (approximately 6.6 millionths per °F) than either limestone aggregate concrete (approximately 3.8 millionths per °F) or dolomite aggregate concrete (approximately 5.3 millionths per °F).

Table 1. Bulkhead stability for Intake Drift (20-ft by 13-ft) and Exhaust Drift (14-ft by 12-ft)

Bulkhead or Barrier Location	Bulkhead or Barrier Type	Bulkhead Thickness (ft)	Pressure Gradient FS	Concrete/ Masonry Shear FS	Rock Salt Shear FS	Worst-Case Bending Stress FS	Best-Case Bending Stress FS
Intake Drift	Plain Concrete Bulkhead	36 31.3[1]	2.40	2.40	5.50	3.61	8.55
Exhaust Drift	Plain Concrete Bulkhead	26 21.3[1]	1.54	2.92	1.75	3.85	5.24
Intake Drift	Masonry Explosion-Isolation	12	1.03	0.63	6.37[2]	0.32	0.75
Exhaust Drift	Masonry Explosion-Isolation	12	1.03	0.77	7.68[2]	0.65	0.88
Intake Drift	Masonry Construction-Isolation[3]	4	>2300	>1450	14.6[2]	44.7	>190

Notes: [1] - Sloping face of waste disposal (pressure) side of plain concrete barrier does not provide rock salt shear resistance from the force developed by 480 psi factored design explosion pressure applied to barrier face. The rock salt shear path was therefore reduced. The bond strength between the concrete and the rock salt was conservatively assumed to be negligible. [2] - Hitched 6 inches into roof, ribs and floor rock salt. [3] - Pressure from roof fall in central part of nearest panel room.

Figure 1.   Exhaust drift longitudinal cross section through panel
            closure bulkhead
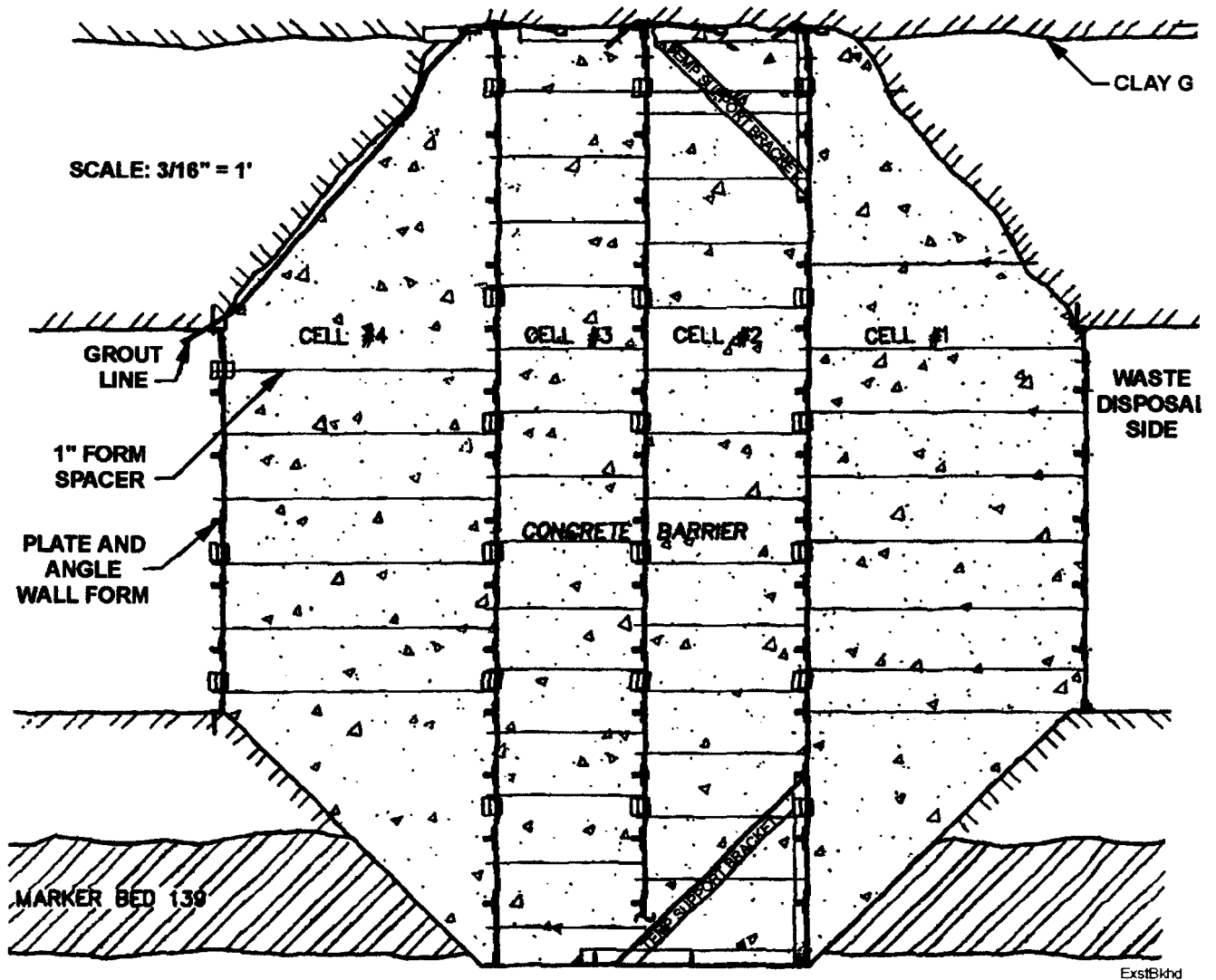
Figure 2.   Explosion-isolation masonry wall bulkhead
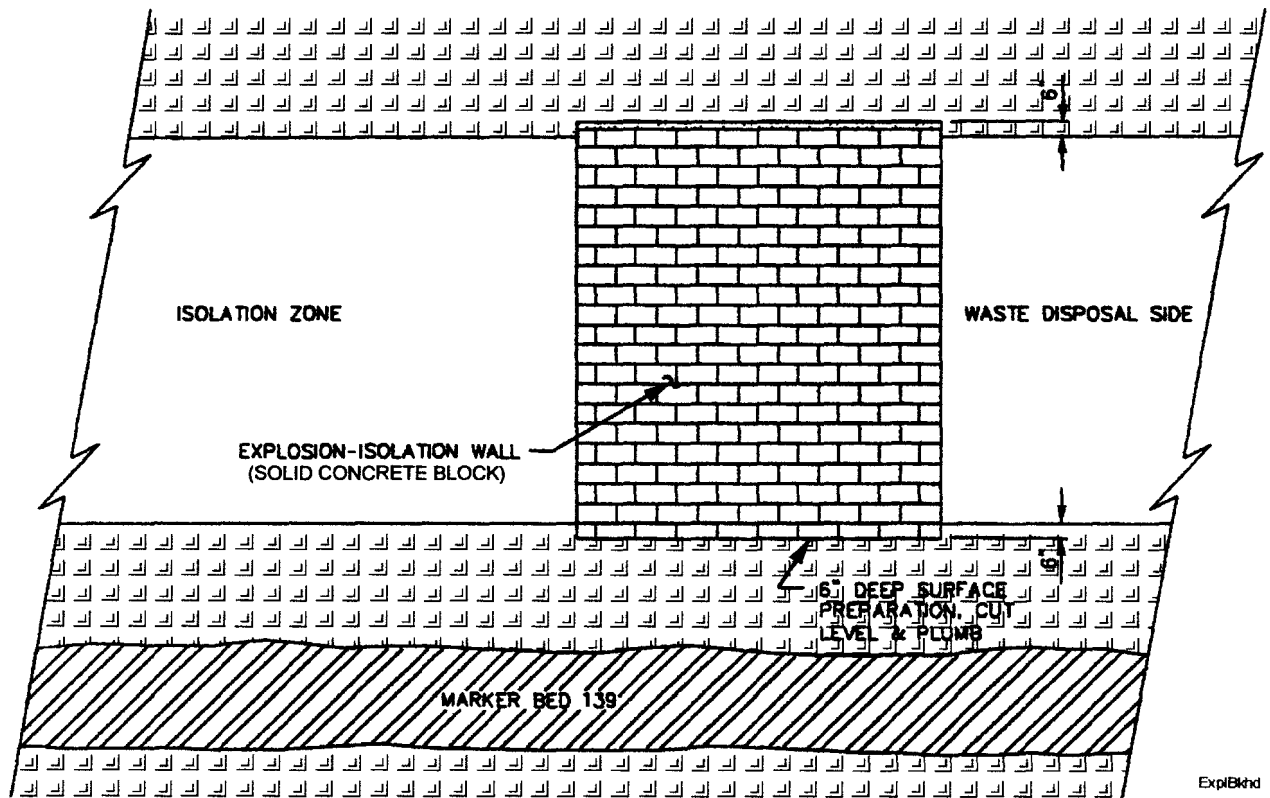
Figure 3.   Methane concentration in waste panel over time

— **With Closure**   **0.1 Mole per drum per year**
-- **Without Closure**
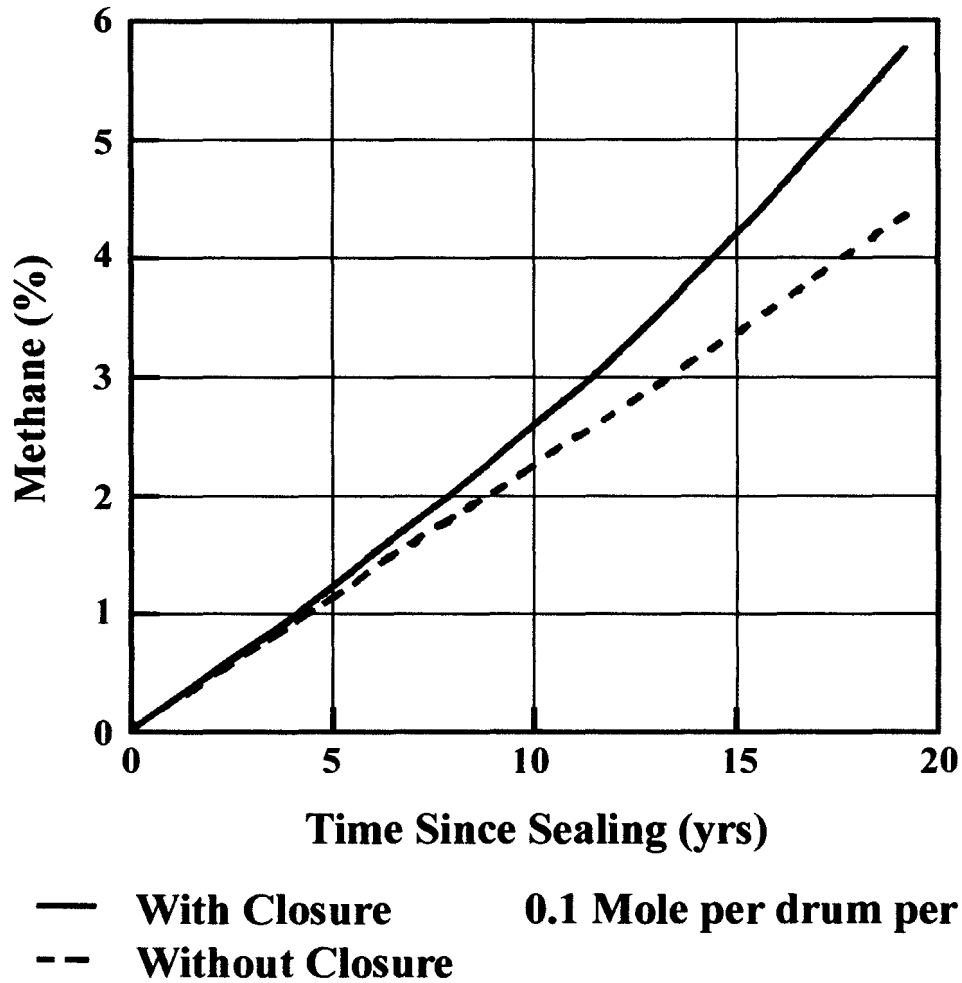
Methane concentration in waste panel over time
(modified from Dykes, et. al., 1997)

MthnTime

Figure 4.   Water impoundment bulkhead types
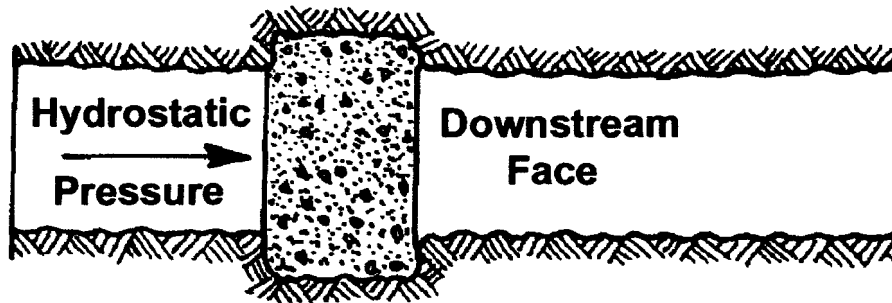
Slab keyed into walls
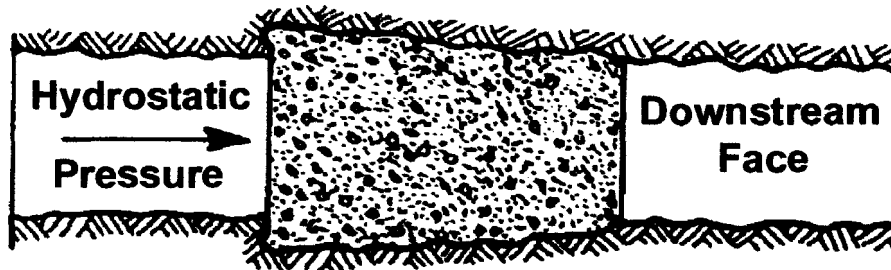
Taper plug
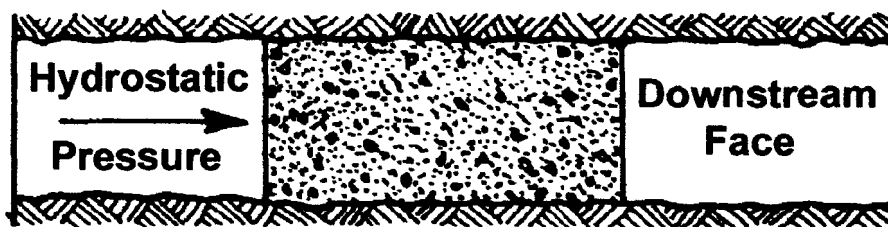
Parallel plug

(Adapted from Garrett & Campbell Pitt, 1958) Blkhtyps

Figure 5.   Intake drift lateral cross section through center
of closure bulkhead
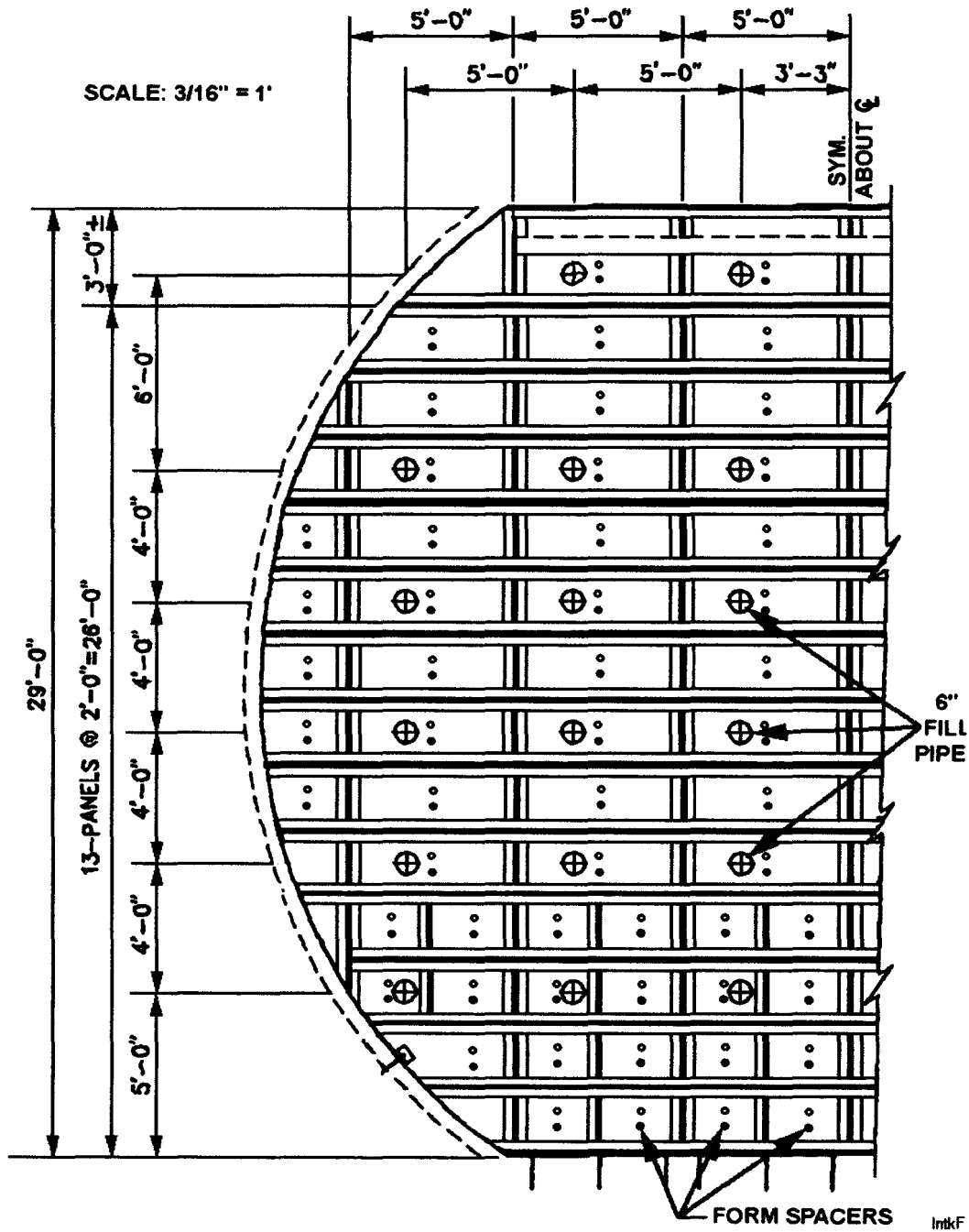
Figure 6.   Intake drift lateral cross section inside panel
closure bulkhead face
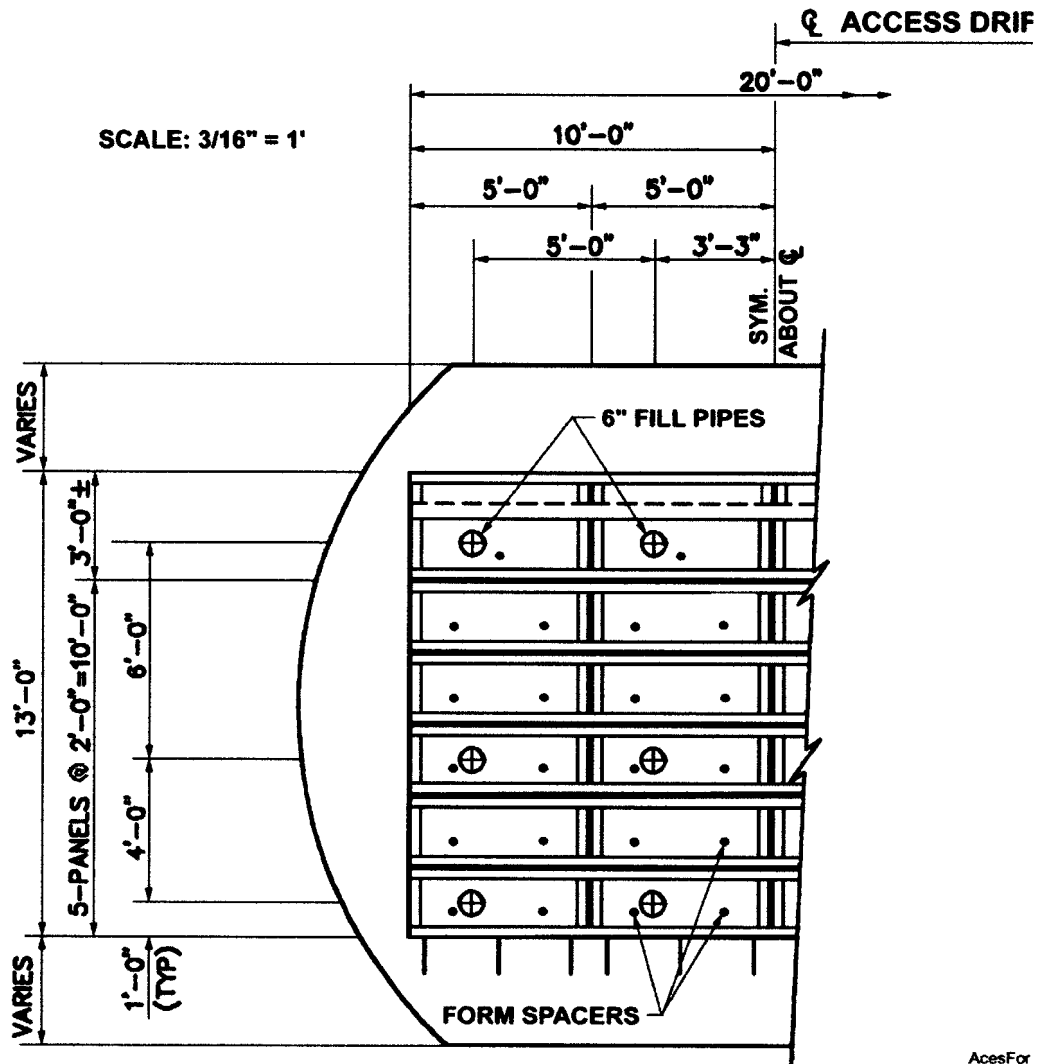
Figure 7.   Pressure buildup from panel closure after panel
             closure bulkhead construction
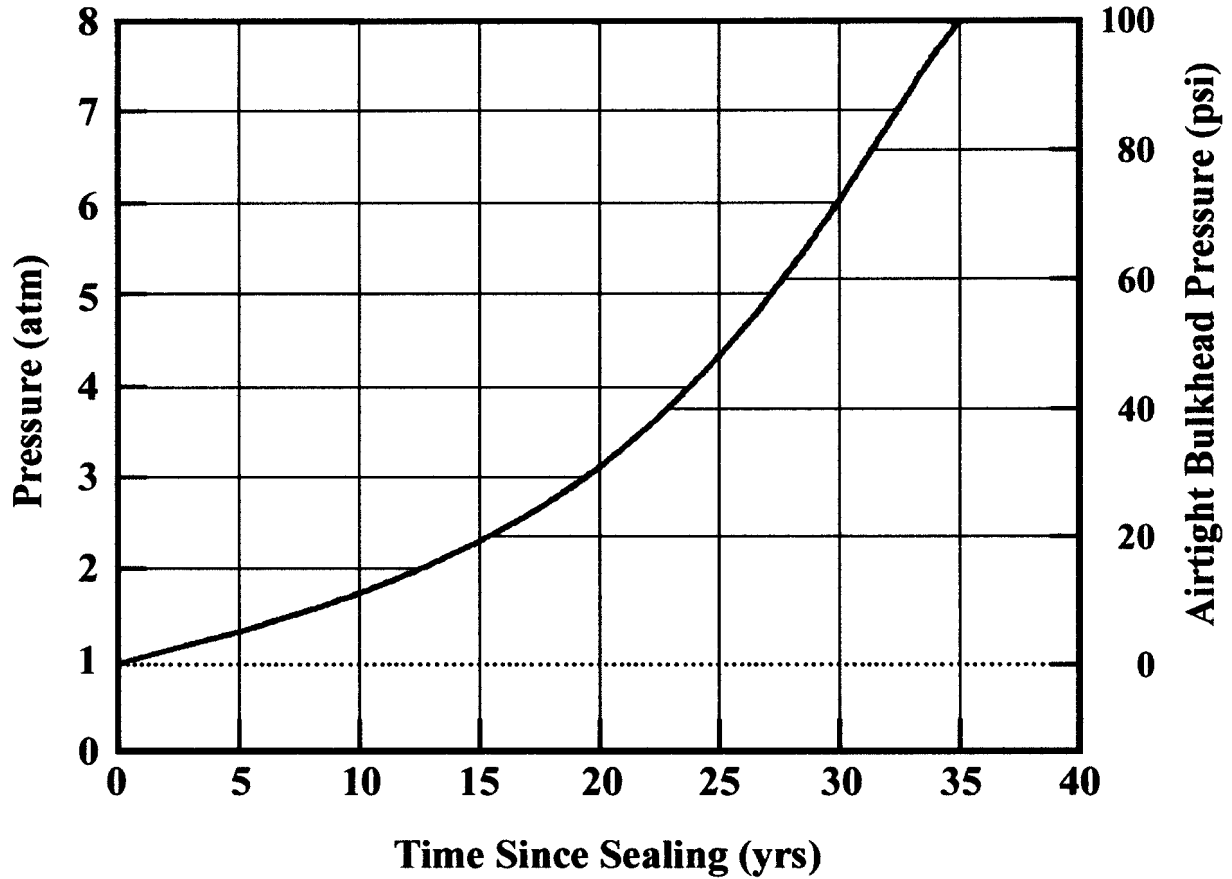
Pressure buildup in a panel after closure
(modified from Dykes, et. al., 1997)

PanlPres

Figure 8.   Thermal coefficients of expansion of neat cements,
            mortars and concretes
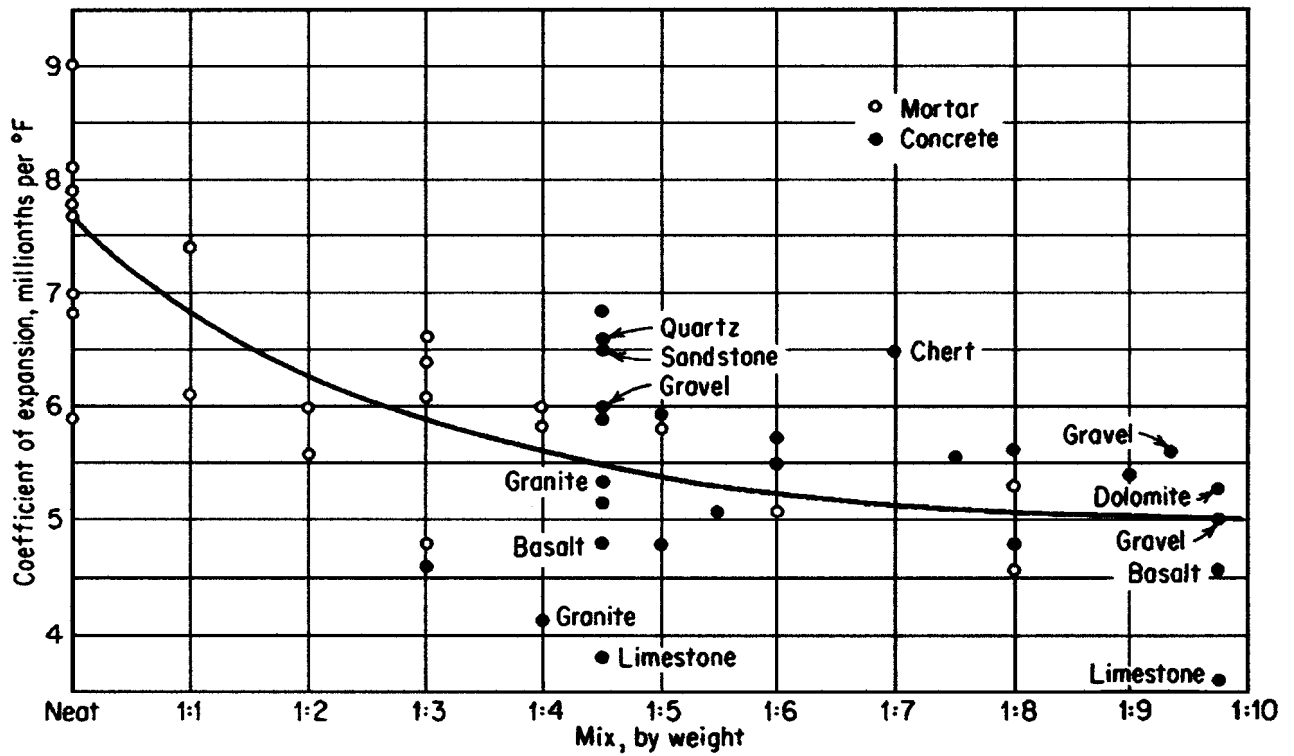
Fig. 12.8.   Thermal coefficients of expansion of neat cements, mortars, and concretes.
(*From U.S. Bureau of Reclamation* [106].)

TempExpn

INTRODUCTION

This report was prepared as directed by Matthew K. Silva of the Environmental Evaluation Group and Mark Levin of Mining and Environmental Services LLC for the purpose of reviewing the proposed Panel Closure Enhancements to the panel closure system area bulkheads and isolation walls, shown on Figure 9. The original optional designs for the panel closure systems are presented on Figure 10. The proposed enhancements to the Panel Closure System are described in the Compliance Certification Application (CCA). This review is specifically directed at evaluating the ability of the enhanced panel closure bulkheads and walls to provide an equivalent, or improved, panel closure. The specific enhancements proposed and reviewed are:

1) replacing the 12-ft thick explosion-isolation wall with the 4-ft construction-isolation wall,

2) allowing the use of a generic salt-based concrete in lieu of Salado-based concrete,

3) allowing the use of local carbonate river rock aggregate in lieu of crushed quartz aggregate,

4) allowing either surface or underground mixing of the concrete,

5) allowing flexibility in the bulkhead forming practices,

6) replacing freshwater grout with salt-based grout and

7) extending the construction period from 180 days to up to one year.

Item 1) relates to the necessity for and ability of the 12-ft thick explosion-isolation masonry wall to protect the workers constructing the panel bulkheads from the design methane explosion pressure. Items 2), 3), 4), 6) and 7) relate primarily to the resulting strength of the concrete in the panel closure bulkheads, over the minimum operational period of 35 years. Item 5) relates to the ability of the resulting flexibly formed panel closure bulkheads to resist both short term methane explosion pressure and long term panel closure-induced gas pressure and to prevent long term brine flow from the panels.

Figure 9. Location of panel closure system areas

Location of panel closure system areas (from DOE/WIPP-96-2150)
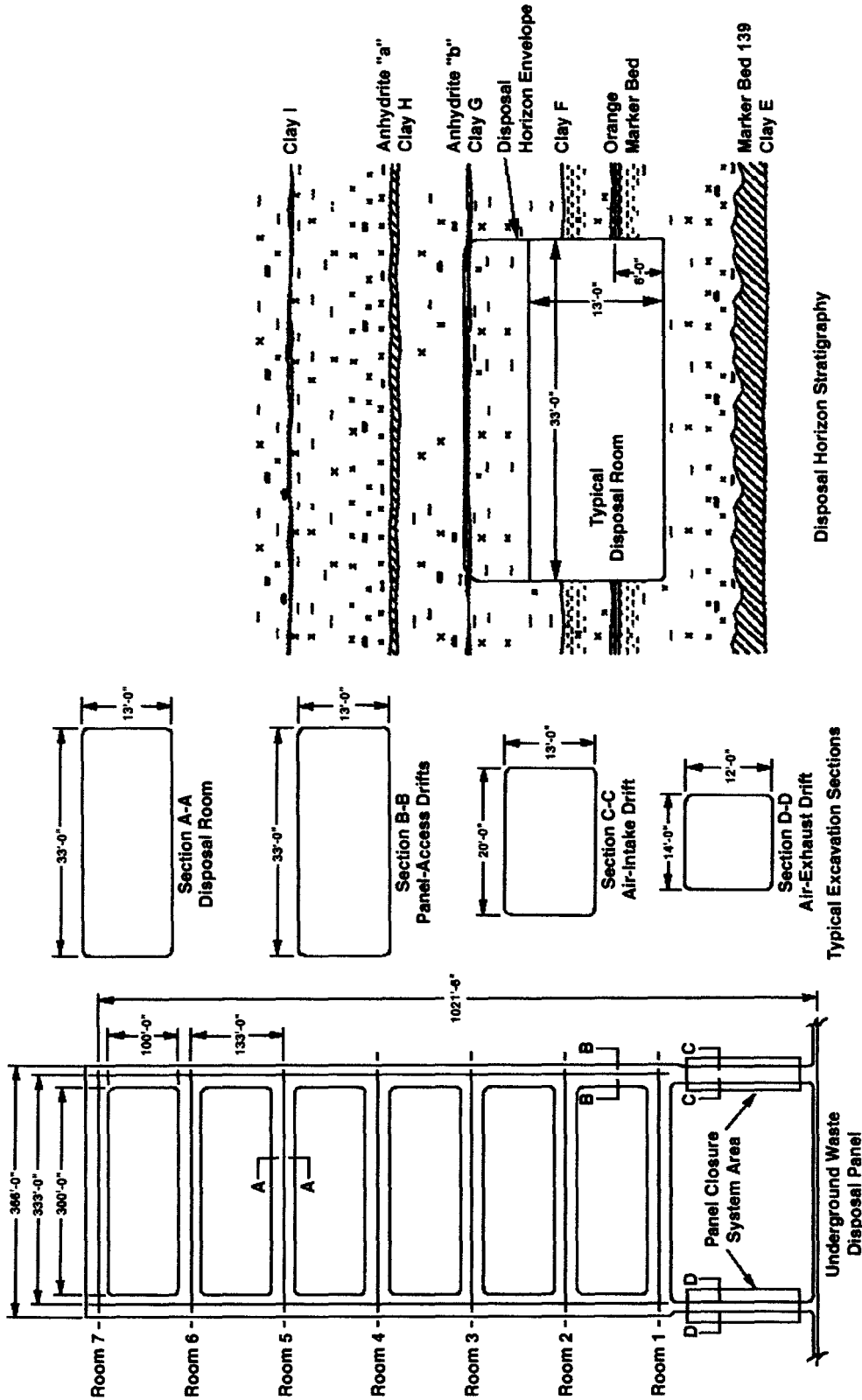
Note: Figure is not to scale. All dimensions shown are nominal.

Figure 10. Optional designs for panel closure systems

**A. CONCRETE BARRIER WITHOUT DRZ REMOVED
AND CONSTRUCTION-ISOLATION WALL**

**B. CONCRETE BARRIER WITHOUT DRZ REMOVED
AND EXPLOSION-ISOLATION WALL**

**C. CONCRETE BARRIER WITH DRZ REMOVED
AND CONSTRUCTION-ISOLATION WALL**

**D. CONCRETE BARRIER WITH DRZ REMOVED
AND EXPLOSION-ISOLATION WALL**
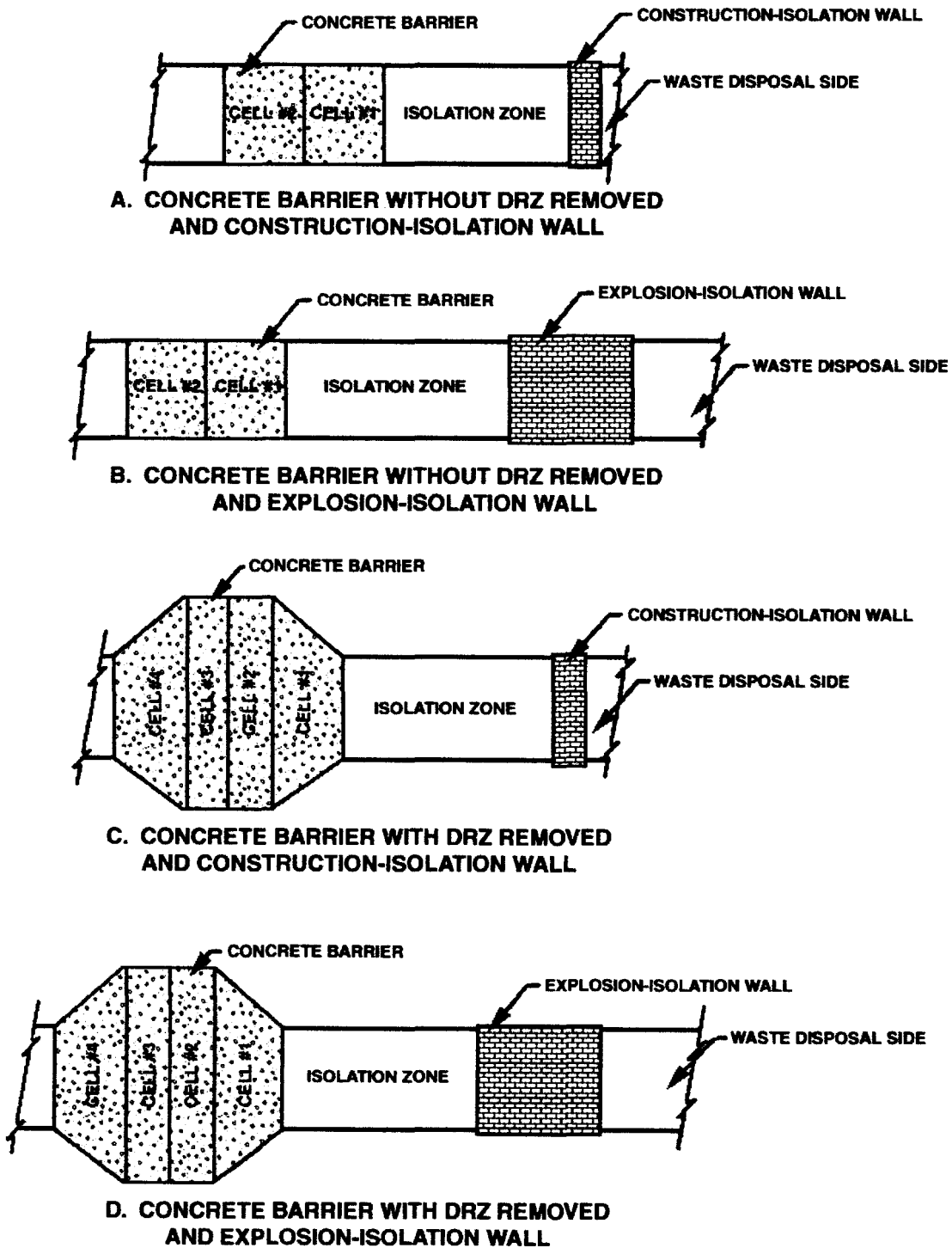
Optional designs for panel closure systems
(modified from DOE/WIPP-96-2150)   WPBlkhd1

CONCRETE BULKHEAD AND MASONRY WALL STABILITY

The 4,000 psi concrete panel closure bulkheads must be capable of resisting the transient 480 psi design overpressure from a methane explosion within closed panels during the planned 35 year life of the facility. The panel closure bulkheads must also be capable of containing the continuous and rising air pressure resulting from the creep closure of the of the panel rooms for the planned 35 year life of the facility. Figure 7 indicates the air pressure inside a sealed panel could increase to approximately 100 psi over 35 years. The panel closure bulkhead designed for the 14-ft wide by 12-ft high Exhaust Drift (Figure 1) is 26-ft thick, 29-ft high and 33.5-ft wide. The panel closure bulkhead designed for the 20-ft wide by 13-ft high Intake Drift is 36-ft thick, 29-ft high and 35.5-ft wide.

In addition, it was postulated that "selection of a thick enough" optional explosion-isolation wall could isolate the main panel closure bulkheads from the design methane explosion pressure. A 12-ft thick, 3500 psi solid masonry concrete block wall mortared with 2500 psi mortar was selected for the explosion-isolation wall.

A 4-ft thick construction-isolation masonry wall is specified "to provide isolation during construction of the main concrete barrier." The panel closure bulkhead construction area will be isolated from the overpressure developed by a possible roof fall in the nearest waste filled room during the 6-month to one-year bulkhead construction period.

PANEL CLOSURE BULKHEADS

The purpose of the panel closure bulkheads, located in the Intake and Exhaust Drifts where indicated on Figure 9, is to seal the waste emplaced in the seven panel rooms from the remaining active, operational, part of the WIPP. Figure 11 shows a typical active panel with waste emplaced in two rooms and indicates the continuing ventilation of the waste filled rooms and panel access drifts. The tapered concrete bulkheads were also designed to block potential leakage paths along Clay G in the near roof and in the anhydrite Marker Bed 139 beneath the floor. Bed separation at Clay G and low angle shear fractures in the roof salt starting at the ribside and angling up to Clay G that preceded the two roof falls in the Site and Preliminary Design Validation (SPDV) area demonstrate the progressive damage in the immediate rock salt roof. Similarly, the fracturing and heave of room floors has been traced to upward buckling of the anhydrite Marker Bed 139. The need to remove the damaged rock zones (DRZ) and block the potential leakage paths in the roof and floor is

the reason that the two-way tapered plain concrete bulkheads, shown on Figure 1, were designed.

The essentially rigid concrete panel closure bulkheads will draw load as the salt creeps toward the adjacent open drifts and rooms. The vertical and lateral horizontal stress acting on the roughly spherical panel closure bulkheads could eventually approach twice the overburden pressure, approximately 4,300 psi (Goodier, 1933; Edwards, 1955). The high stress concentration against the panel closure bulkheads will resist fracture propagation in the rock salt over, under and around the bulkheads, as indicated on Figure 12. The applied pressure would have to overcome (exceed) the rock salt/concrete contact pressure to propagate a fracture around the panel closure bulkheads.

The ground pressure applied to all sides of the panel closure bulkheads during the 17 years needed to reach an explosive 5% methane concentration (Figure 3) will grip the top, bottom and sides of the bulkheads. The force of this grip will resist flexure of the bulkhead concrete, in effect forming a fixed-end circular deep plate. The fixity of the panel bulkheads will reduce the bending moment developed when the bulkhead is loaded by the design explosion pressure applied from the waste emplacement side.

The ability of the planned plain concrete panel closure bulkheads was conservatively checked using the water impoundment bulkhead design method (Abel, 1998). The panel closure bulkheads are fully capable of resisting the 480 psi design methane explosion pressure. The 480 psi design methane explosion pressure is equal to the hydrostatic head of 1,108 feet of water. The panel closure bulkheads are effectively plain concrete because no tensile reinforcement is provided, or necessary, to resist the deep-beam bending stresses. The bulkhead lengths are sufficient to reduce the flexural bending stresses to less than the American Concrete Institute (ACI) allowable 206 psi tensile strength of 4,000 psi compressive strength concrete. Appendix A presents the calculations for the Intake Drift bulkhead and Appendix B for the Exhaust Drift bulkhead. Table 1 presents the various factors of safety for the panel closure bulkheads. The lowest factor of safety for the 36-ft thick Intake Drift bulkhead is 2.40 against both pressure gradient and concrete shear and for the 26-ft thick Exhaust Drift bulkhead is 1.54 against pressure gradient and 2.92 against concrete shear. The water impoundment bulkhead method only considers the grout pressure along the rock/concrete contact and does not provide any credit for the pressure from creep closure pressure of the rock salt.

The panel closure bulkhead design (Figure 1 and Figure 10) is not a monolith because it is planned to be built using four

cells. The steel forms will restrain the concrete pumped into each cell until the initial set, which will take approximately 4 hours (Figure 13). Most water retaining bulkheads (Figure 4) are designed and constructed as monolithic, i.e. continuous, pours without cold joints or interior form walls. Figure 14 provides a longitudinal cross section of the reinforced concrete American Tunnel Bulkhead #1, designed to resist a 1550-ft head (670 psi) and supporting 1010-ft (438 psi). Note the waterstops on the bypass pipe and the grout holes drilled through the concrete to low-pressure grout the roof and rib contact between the 11,000 psi compressive strength latite porphyry and the 3,000 psi concrete.

Some multi-cell bulkheads have been constructed when construction problems or emergencies required. At the Summitville Mine a second 20-ft long cell was added when water loss through the 129 psi compressive strength altered latite porphyry and over the initial 6.5-ft long Chandler Adit bulkhead became excessive. At the West Driefontein Mine, in response to a 67,000 gpm water inrush, a three cell bulkhead was progressively constructed on the 12-Level, the initial "temporary" sand and cement bag diversion plug, followed by a 15-ft long thickening bulkhead against the "temporary" plug and the final 60-ft long sand-concrete extension bulkhead. At the Rocanville Mine, in response to a 6,250 gpm brine inrush into the potash mine, an 87-ft thick five cell bulkhead was constructed, an initial 8-ft thick bulkhead, a 16-ft thick second cell, a 25-ft thick third cell and a final 37-ft thick cell (Figure 15). In every case, the cell forms were externally supported and the internal bulkhead forms were stripped before constructing the subsequent cell. The planned four cell panel closure bulkhead construction would not provide a monolithic, i.e. uniform and continuous, bulkhead but could contain as many as three interior steel form walls supported during filling by the form spacers.

Penetrations through a water retaining bulkhead are normally minimized, whenever possible, because they represent potential leakage paths through bulkheads. Essential penetrations, such as bypass and sampling and pressure monitoring pipes, include waterstops to assure no leakage along the penetrations. The planned 4 cell panel closure bulkhead (Figure 1) includes the use of longitudinal 1-in diameter form spacers, which also support the facility side plate and angle form wall until the concrete sets. Figure 5 indicates 55 form spacer penetrations across Cell #2 and Cell #3. Figure 6 indicates 22 form spacer penetrations across Cell #1 and Cell #4. Multiple potential flow paths are apparent across the individual cells. If not removed after the cell is filled, the steel plate and angle forms provide multiple potential leakage paths between form spacers in adjacent cells. The bond strength between the steel plates and the concrete is

necessarily low. Even if a way is found to remove the three
interior steel form walls, after the concrete has set in each
cell, a smooth surfaced cold joint will still be present between
cells.

In order of declining potential effectiveness, the following
panel closure bulkhead construction modifications should decrease
the bulkhead leakage potential,

1) constructing truly monolithic single-cell panel
    closure bulkheads or
2) externally supporting the bulkhead cell forms, i.e.
    eliminating the form spacers, or
3) reducing the number of cells.

The panel closure bulkheads are tapered two-way pressure
bulkheads, i.e. designed to resist equal pressure from either
side. The panel closure bulkheads are apparently designed to be
airtight for the 35-year life of the facility. Closure of the
panel rooms and panel access drifts is predicted to result in 100
psi of effective pressure on the waste side of the bulkheads over
a 35 years period after completion of the panel closure bulkheads
(Figure 7). The two-way taper of the panel closure bulkheads was
apparently designed to permit the safe excavation of the
disturbed rock zones (DRZ) above and below the intake and exhaust
drifts. The ample panel closure bulkhead factors of safety
against the 480 psi design methane explosion pressure should be
more than quadrupled for the closure pressure.

Tapered bulkhead water containment bulkheads are normally
constructed when the rock shear strength is lower than the
concrete design shear strength. Figure 4 shows a typical
tapered water containment bulkhead, designed to resist
hydrostatic pressure from one side. Successful tapered
bulkheads have been constructed at the Summitville Mine, the
Jackpot Mine declines and at IMC's K2 Mine. Tapering a bulkhead
increases the length of the worst-case potential shear surface
in the lower strength rock adjacent to the rock/concrete
interface and tightens the concrete bulkhead against the rock
when hydrostatic pressure is applied. In most water impoundment
cases, when the rock strength exceeds the concrete strength,
parallel plugs have proven to be effective. Loofbourow in the
Society of Mining Engineers (SME) Mining Engineering Handbook
(1973, Sec 26.7.4) states "no indication of structural failure
resulting from thrust was noted" in the case of ten bulkheads
subjected by hydraulic pressures in excess of 1000 psi and which
relied solely on normal rock surface irregularities, referred to
as a "parallel plug" on Figure 4. Garrett and Campbell-Pitt
(1961) reported the successful results from 26 mine bulkheads,

twelve with parallel plugs, that relied solely on the
irregularity of the tunnel walls, and 14 with taper plugs.

Parallel panel closure bulkheads would probably be
successful in resisting the potential applied pressures, Options
A and B on Figure 10. However, the progressive deterioration of
the adjacent rock salt and anhydrite and clay layer bed
separations suggests that the two-way tapered bulkhead will
assure the long term functioning of airtight panel closure
bulkheads.

EXPLOSION-ISOLATION MASONRY WALLS

The 12-ft thick explosion-isolation masonry walls for the
Intake Drift and the Exhaust Drift (Figure 2) are planned to be
built with solid concrete blocks (3,500 psi) mortared with
cement and hitched 6-in into the roof, ribs and floor. The
explosion-isolation masonry wall is apparently designed to
resist the 480 psi methane explosion pressure during the six
months to one year of panel bulkhead construction.

The ability of the planned explosion-isolation masonry
walls to resist the 480 psi methane explosion pressure was
conservatively checked using the water impoundment bulkhead
design method (Abel, 1998). Table 1 presents the results of
this analysis. The calculations for the Intake Drift are
presented in Appendix C and for the Exhaust Drift in Appendix D.
The results of this analysis predicts that the planned 12-ft
thick explosion-isolation masonry walls would not be capable of
resisting the 480 psi design methane explosion pressure. The
predicted failure of the explosion-isolation wall is indicated
on Figure 12. In the Intake Drift, the thickness of an
explosion-isolation masonry bulkhead would have to be
approximately 20-ft to contain the 480 psi methane explosion
pressure and in the Exhaust Drift approximately 15-ft.

Figure 3 presents the predicted 17+ year methane
concentration time interval between cessation of panel
ventilation and the methane concentration reaching the minimum
5% lower explosive limit. The predicted methane concentration
should not exceed approximately 0.25% during a one-year panel
closure bulkhead construction period. Therefore, a methane
explosion should not be possible, and an explosion-isolation
masonry wall not necessary.

CONSTRUCTION-ISOLATION MASONRY WALLS

A 4-ft thick construction-isolation masonry wall has been designed to isolate the panel closure bulkhead construction from the emplaced waste in a completed panel and from the transient overpressure from a 15,000 ton roof fall in central 140-ft of Room 1 in the completed panel, closest to the Intake and Exhaust Drifts. The design roof fall is equal in weight and length to the roof fall in Site and Preliminary Design Validation Room 1 (SPDV1). The SPDV1 fall distance was the 13-ft room height, whereas the fall distance with emplaced waste drums is limited to 3.5-ft. Figure 16 presents the predicted 0.4 PSF (0.003 psi) maximum overpressure and the design 10.1 PSF (0.070 psi) air overpressure from the roof fall.

The ability of the planned construction-isolation masonry walls to resist the 0.070 psi design roof fall pressure was conservatively checked using the water impoundment bulkhead design method (Abel, 1998). Table 1 presents the results of this analysis. The calculations for the 4-ft thick Intake Drift construction-isolation masonry wall are presented in Appendix E.

The 4-ft thick construction-isolation masonry wall planned for the 20-ft wide by 13-ft high Intake Drift is fully capable of resisting the roof fall design overpressure. A similar 4-ft thick construction-isolation masonry wall planned for the 14-ft wide by 12-ft high Exhaust Drift will have even higher factors of safety against the roof fall design overpressure.

Figure 11.Typical active panel configuration with ventilation splits
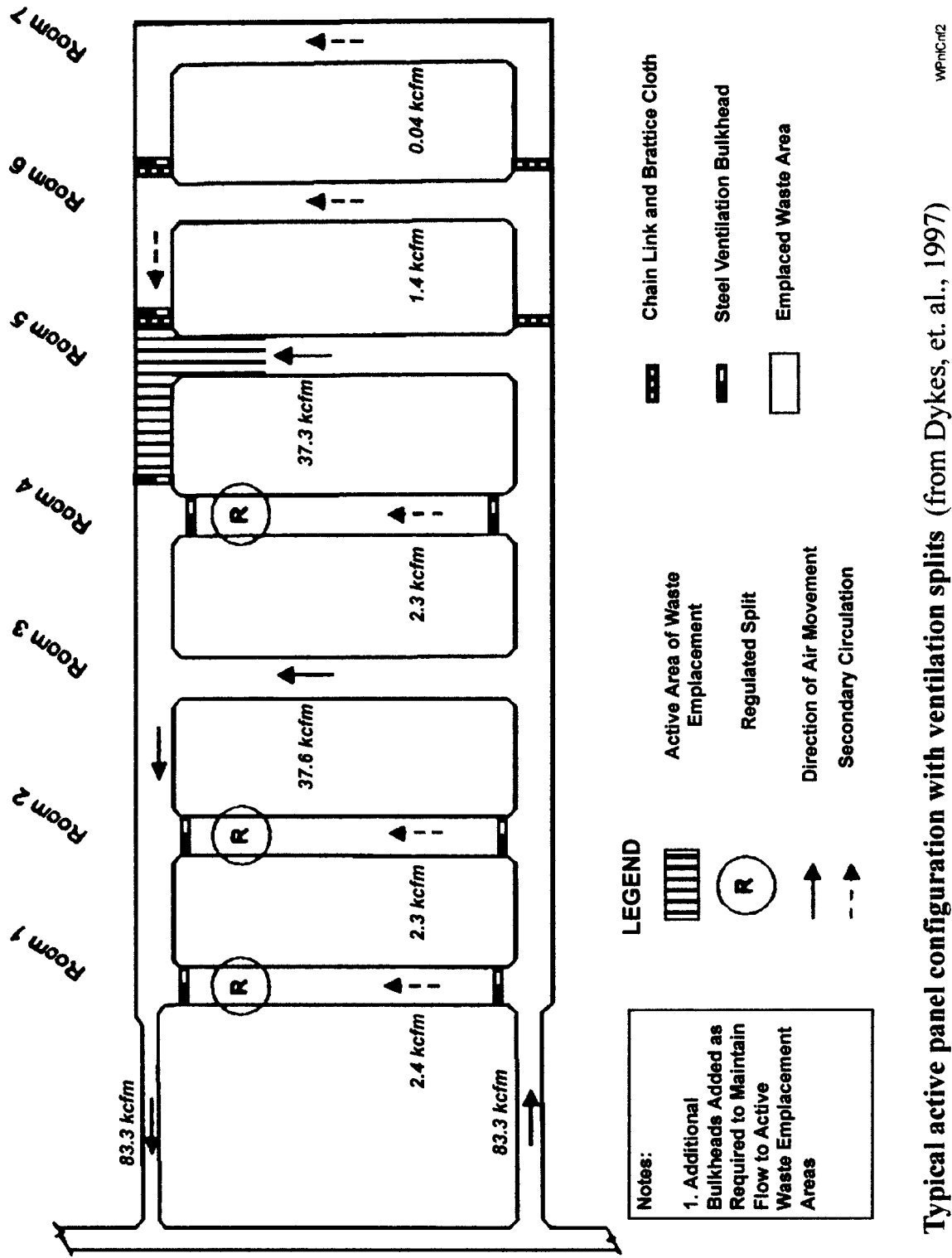
Typical active panel configuration with ventilation splits (from Dykes, et. al., 1997)

Figure 12. Fracture propagation modes of bulkhead failure
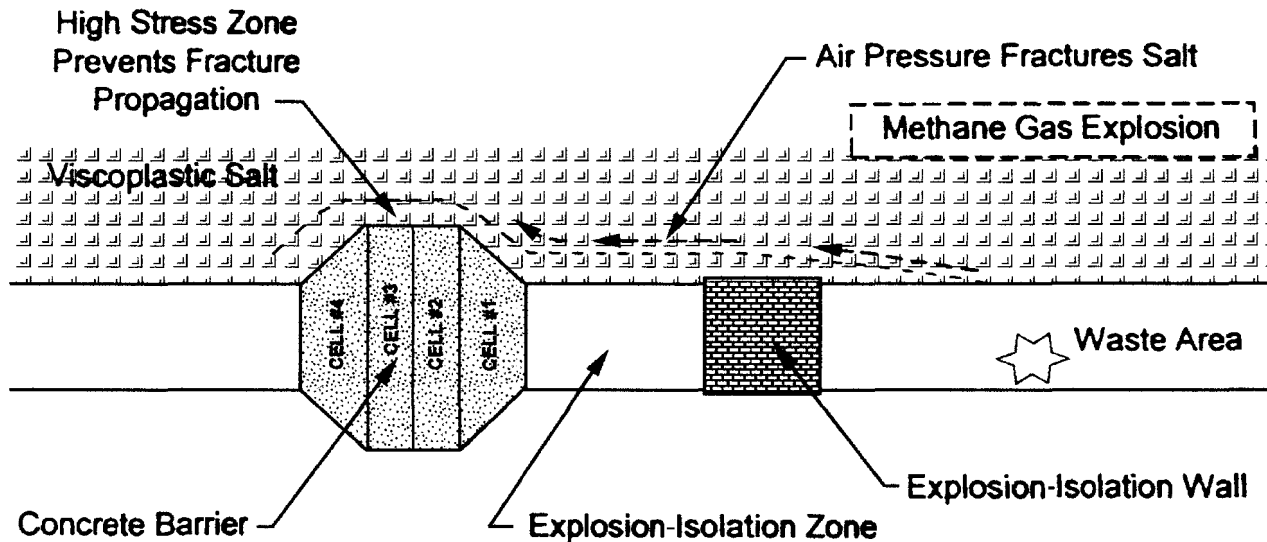
Fracture propagation mode of failure with stable explosion-isolation wall
(modified from Dykes, et. al., 1997)
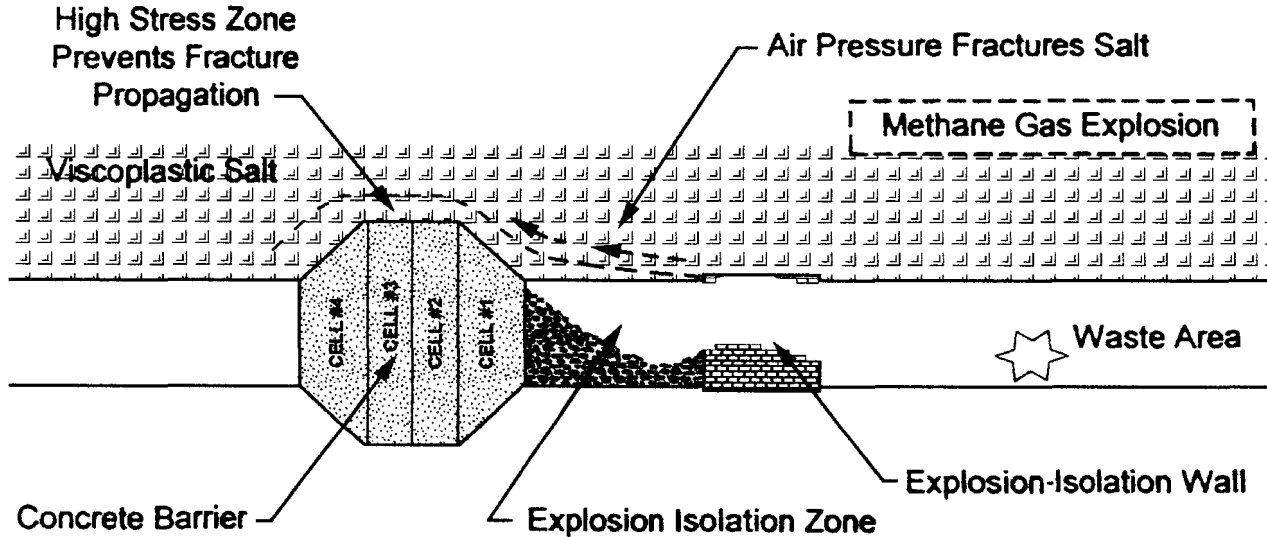
Fracture propagation mode of failure for explosion-isolation wall failure
(modified from Dykes, et. al., 1997)

ExploBk1

Figure 13. Example of effect of temperature on setting time of concrete
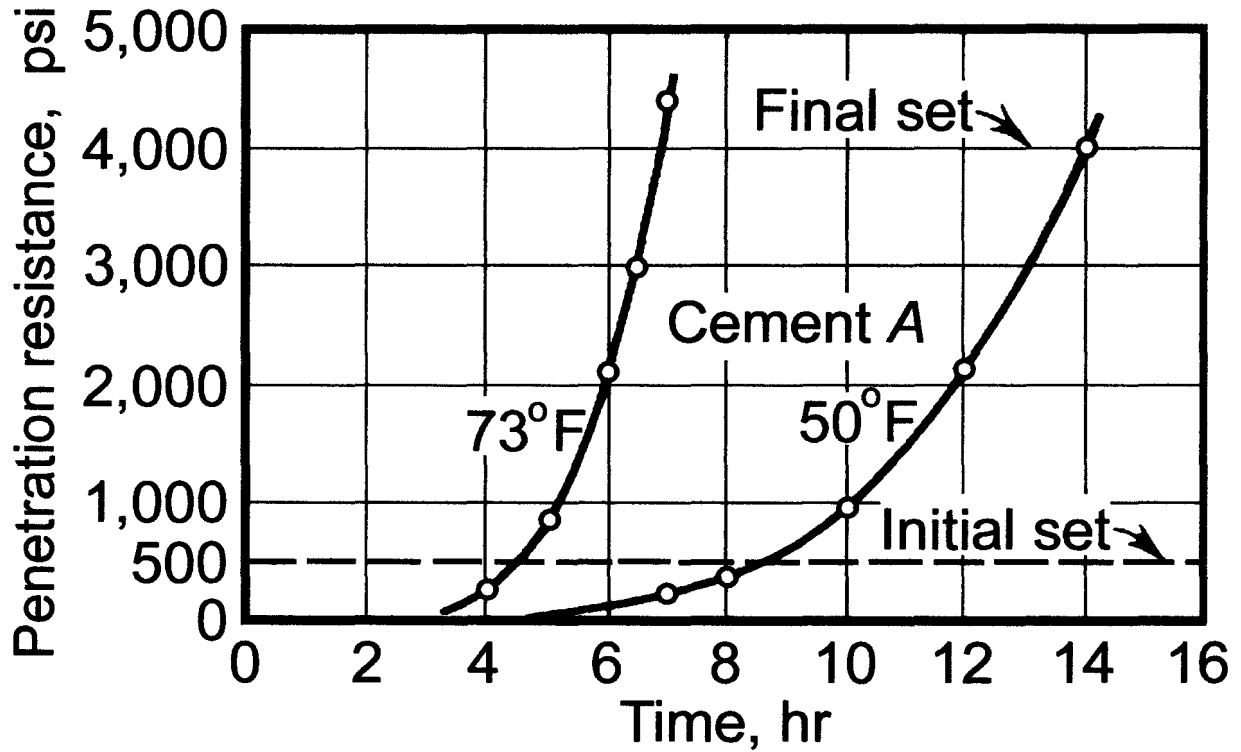
Example of effect of temperature on setting time of concrete
(Troxell, et. al., 1968)

SetTime2

Figure 14. Cross section through monolithic bulkhead (Abel, 1998)

Sunnyside Gold Corp.   **American Tunnel Bulkhead #1**

Longitudinal Cross Section
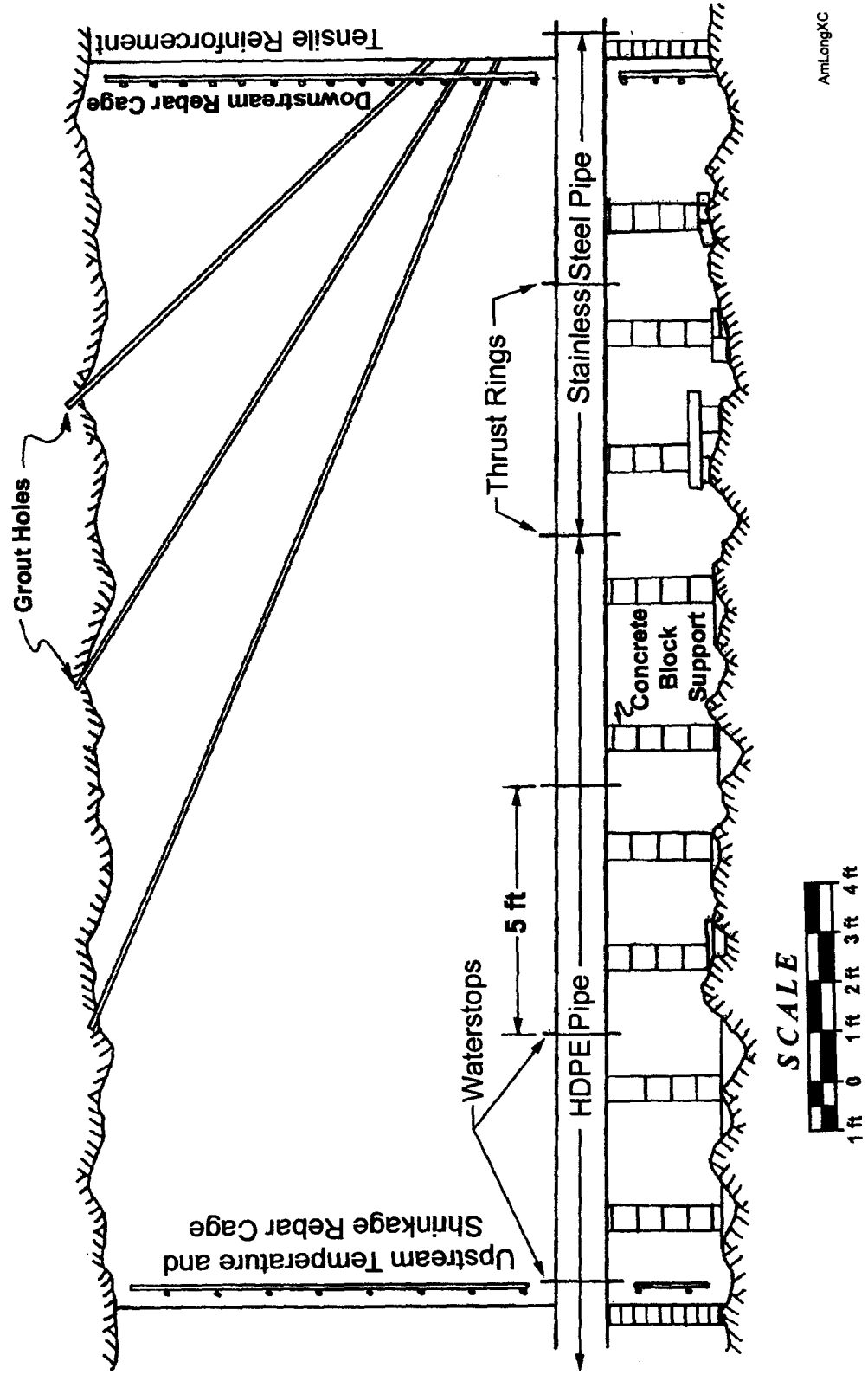
Figure 15. Rocanville inflow tunnel bulkheads and plugs
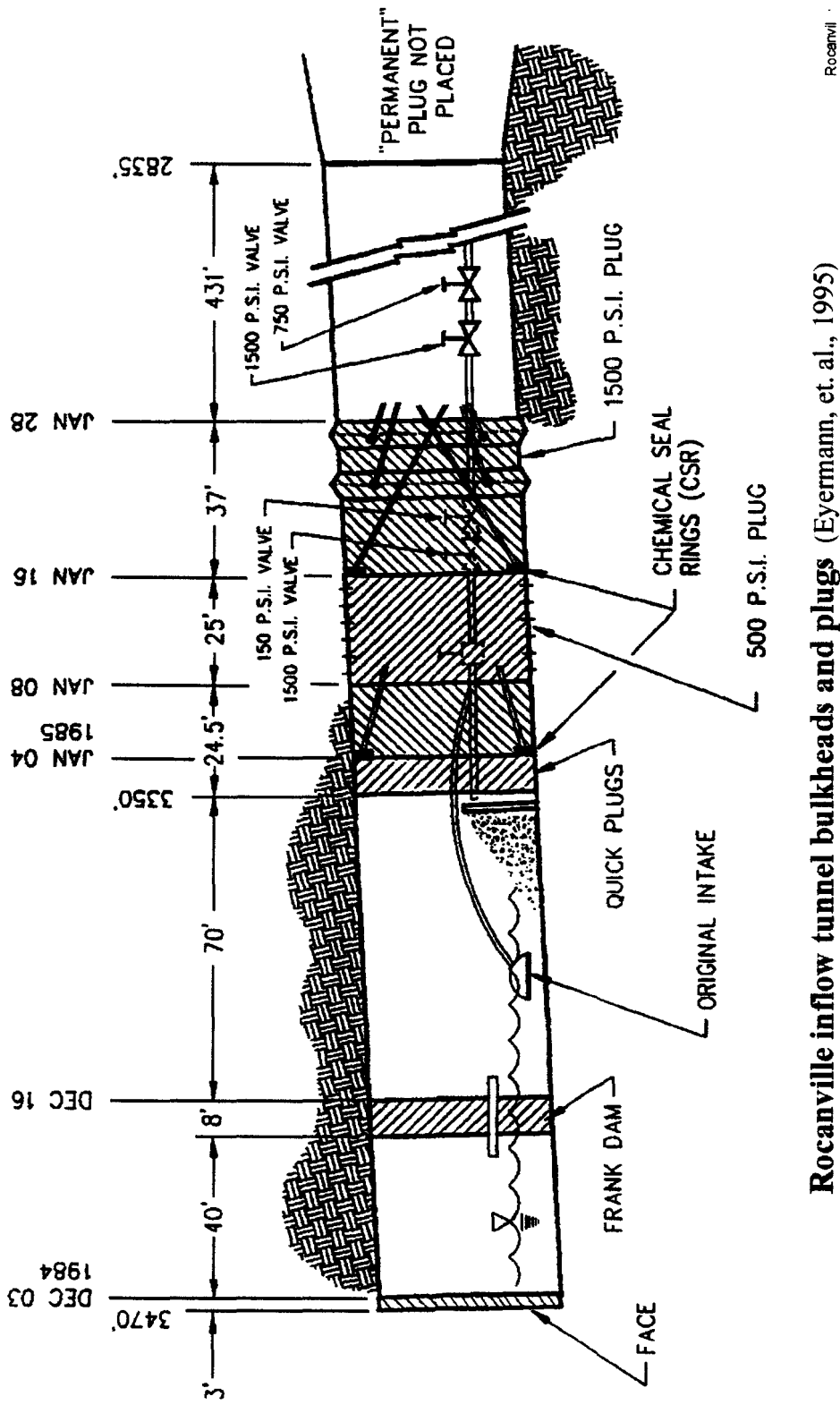
Rocanville inflow tunnel bulkheads and plugs (Eyermann, et. al., 1995)

Figure 16. Pressure transient from a roof fall

Elapsed Time (seconds)
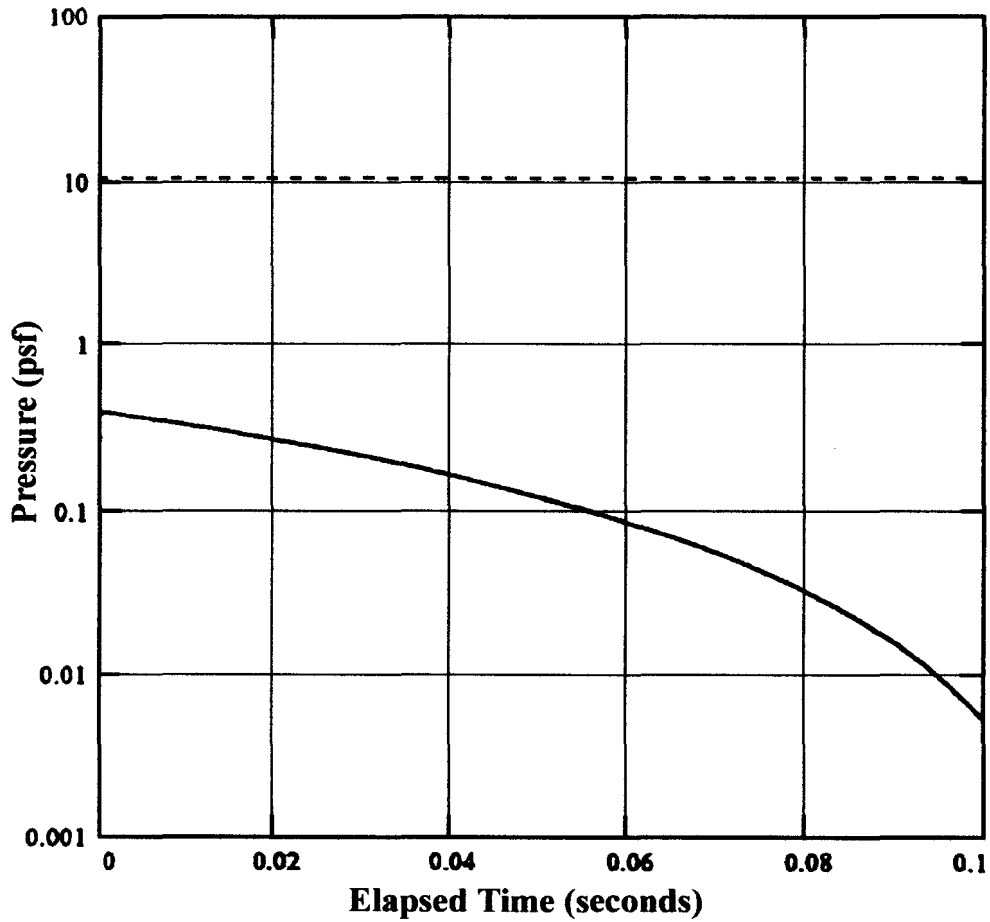
— Predicted Pressure Loading
- - Design Pressure Loading

Pressure transient for a roof fall
(modified from Dykes, et. al., 1997)   RoofFall

CONSTRUCTION ENHANCEMENTS

The remaining panel closure enhancements relate primarily to the resulting strength of the concrete in the panel closure bulkheads, over the minimum operational period of 35 years. The strength of the concrete is critical to the strength of the panel closure bulkheads.

The proposal to allow the mass concrete to utilize any rock salt in the mix depends on the ability to consistently meet the strength specification. The unconfined compressive strength of the 16 Permian evaporite beds tabulated in Appendix F ranges from 2300 psi to 5880 psi and the cohesion from 540 to 1580 psi. The unconfined compressive strength of the evaporite beds tabulated in Appendix G, including the Permian beds in Appendix F, ranges from 2260 psi to 7510 psi and the cohesion from 540 to 1790 psi. The random selection of the salt from any tested salt bed should be capable of providing equivalent strength properties. In addition, the strength variation across a single evaporite bed in the Salado formation is significant. The unconfined compressive strength of the Mississippi Potash Company's Cycle 7 bed (> 85% salt) ranged from 1610 psi to 4950 psi, as shown on Figure 17. The important factor would appear that the salt component of the mix be consistent, i. e. well mixed from one source.

Aggregate strength is a minor concrete strength factor which can be compensated for, if necessary, by increasing the cement in the mix and verified by testing. In addition, carbonate aggregate concrete probably has a lower coefficient of thermal expansion that quartz aggregate. Figure 8 indicates that concrete with quartz aggregate has a higher coefficient of expansion (approximately 6.6 millionths per °F) than either limestone aggregate concrete (approximately 3.8 millionths per °F) or dolomite aggregate concrete (approximately 5.3 millionths per °F). It should be possible to use either the Pennsylvanian Atoka limestone or the Permian, Guadalupian, Bell Canyon limestone river rock for concrete aggregate. Rounded river rock should facilitate slick line pumping and form filling.

Surface or underground mixing of the concrete is of no significance. Sunnyside Cold Corp. has constructed five water impoundment bulkheads using underground mixing and pumping stations, four using supersacks, surface mixing and pumping and one with surface mixing and transporting by rail in Moran cars. The critical factor is limiting the time between mixing and placement in the form. Troxell, et. al. (1968) state:

Figure 17. Strength of Mississippi Potash Co.'s Cycle 7 bed
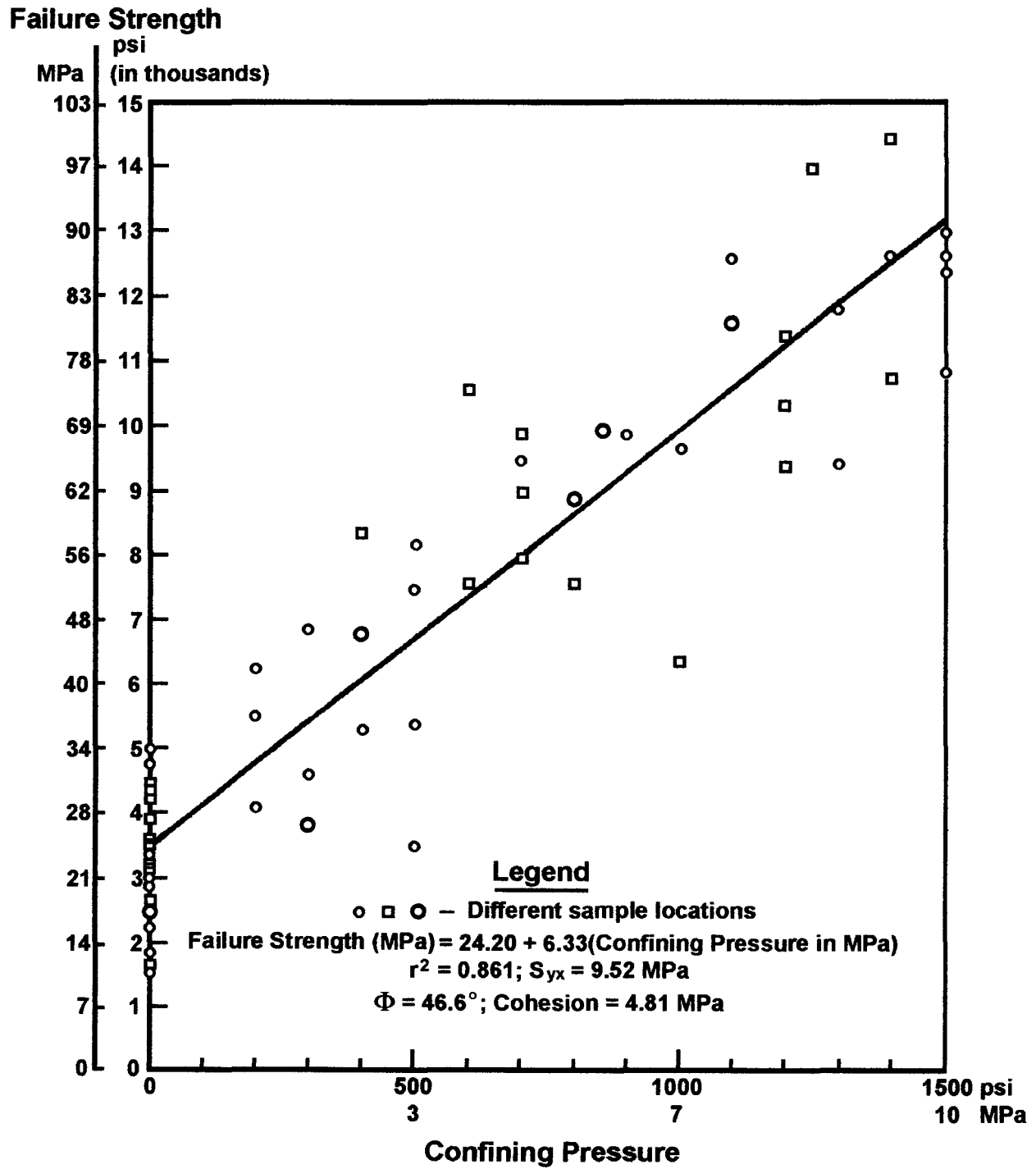
Strength of Mississippi Potash Co.'s Cycle 7 bed   (Abel and Djahanguiri, 1984)   SaltNuc5

Current specifications for ready-mixed concrete require that the concrete be discharged from the truck within 1-1/2 hr or before the drum has had 300 revolutions (whichever comes first) after the water is added to the batch, or the cement to the moist aggregate. Under specially favorable conditions, periods up to 2 and 3 hr may be allowed. Conversely, under unfavorable conditions where air temperatures are unusually high, or the ingredients of the concrete are such that an unusually quick time of set or loss of plasticity may occur, it may be necessary to substitute a shorter period.

If the water is added to the bulkhead concrete on the surface, it will be difficult to meet the time limitation. Transporting supersacks underground and adding the water and mixing close to the bulkhead sites would appear to be the most reasonable method.

Replacing freshwater grout with salt-based grout should be done in order to minimize salt dissolution in the adjacent rock salt and in the salt-based concrete. Salt dissolution could weaken the contact zone and potentially provide a leakage path for brine, methane explosion pressure and closure compressed gas pressure behind the panel closure bulkheads.

It should be expected that the best contractor bid will result from the least restrictive specification. There is no apparent reason to require panel closure construction within six months rather than one year.

CONCLUSIONS AND RECOMMENDATIONS

The most important of the specific enhancements proposed, allowing flexibility in the bulkhead forming practices is troubling because the planned bulkhead design contains multiple potential leakage paths within cells, between cells and through the panel closure bulkheads. It is recommended that the panel bulkhead specifications:

1) provide an incentive for the contractor to minimize the number of cells (preferably to one),
2) require that each cell be filled as a continuous monolithic concrete pour,
3) require the contractor support the fluid concrete in all cells with external structures,
4) require the contractor to remove the support structures and forms between internal cells,
5) provide for a rough form surface between internal cell walls, (possibly with a layer of burlap),
6) assure that some grout points are located at the roof concrete/rock salt contact and
7) prevent the use of all internal form spacer supports.

If built as recommended, the panel closure bulkheads should be capable of providing more than the required 35 years of protection from brine migration, 480 psi of methane explosion pressure and 100 psi of panel closure pressure.

The planned 12-ft thick explosion-isolation masonry walls should not be built. The methane concentration will take more than 17 years to rise to the 5% lower explosive limit after stopping panel ventilation. If built, the planned 12-ft thick explosion-isolation masonry walls would probably be incapable of resisting the design 480 psi methane explosion pressure.

The 4-ft thick construction-isolation wall is recommended to isolate the panel closure construction areas. The 4-ft thick construction-isolation walls are more than adequate to support the potential roof fall overpressure.

Generic salt-based concrete should be equally as effective as Salado-based concrete.

It should be possible to use local limestone or dolomite river rock for concrete aggregate. Limestone aggregate concrete should have a lower coefficient of thermal expansion than crushed quartz aggregate concrete. River rock should pump more readily than crusher product.

The contractor should be allowed to use either surface or underground mixing of the concrete. However, underground mixing of supersacks is recommended to assure sufficient time for placing the concrete in the forms.

Salt-based grout is recommended to eliminate the possibility of weakening the salt-based concrete along the rock salt/concrete contact by dissolution. Grouting across the roof contact is essential to assure that voids are filled.

There does not appear to be a time imperative requiring a 180-day construction period rather than a one year period.

REFERENCES

Abel, Jr., J.F. & F. Djahanguiri, 1984, Application of Performance Data from Evaporite Mines to Salt Nuclear Waste Repository Design: Int'l. Jour. of Mining Engrg., v 2, p 323-340

Abel, Jr., J.F., 1998, Bulkhead design for acid mine drainage: in Proc Western U.S. Mining-Impacted Watersheds, Joint Conf on Remediation and Ecological Risk Assessment Technologies, Denver, CO, 36 p

American Concrete Institute, 1995, Building code requirements for structural concrete (ACI 318-95) and commentary - ACI 318R-95: 369 p

American Institute of Steel Construction, 1989, Manual of steel construction, allowable stress design: 9th ed, AISC, Inc., 1121 p

Brown, A., Consultants, Inc., 1992b, Adit plugging case histories: Report 1288B/921105, Nov 5, 17 p

Chekan, G.J., 1985, Design of bulkheads for controlling water in underground mines: USBM Info Circ 9020, 36 p

Department of Energy, 2001, Panel closure enhancements: Carlsbad Office, 24 p

Djahanguiri, F. & J.F. Abel, Jr., 1997, Design and construction of a bulkhead for a simulated underground leaching stope: Mining Engrg., v. 49, no. 1, Jan, p. 57-64

Dykes, A.A., J.B. Case & R.K. Deremer, 1997, Analysis of roof falls and methane gas explosions in closed rooms and panels: Report PLG-1167, Westinghouse Electric Corp, 124 p

Edwards, R.H., 1951, Stress concentrations around spheroidal inclusions and cavities: Jour Applied Mechanics, Am Soc Mech Engineers Trans, v 73, p 19-30

Einarson, D.S. & J.F. Abel, Jr., 1990, Tunnel Bulkheads for Acid Mine Drainage: Proc Symp on Unique Underground Structures, v 2, p. 71-1 to 71-20

Eyermann, T.J., L.L. Van Sambeek & F.D. Hansen, 1995, Case studies of sealing methods and materials used in salt and potash mining industries: Sandia Report SAND95-1120, 73 p

REFERENCES (Continued)

Fenning, R.W., 1926, Gaseous combustion at medium pressures: Phil Trans Royal Soc of London, Ser A, v 225, p 331-356

Garrett, W.S. & L.T. Campbell Pitt, 1958, Tests on an experimental underground bulkhead for high pressures: Jour S. African Inst Mining and Metallurgy Congress, Oct, p 123-143

Garrett, W.S. & L.T. Campbell Pitt, 1961, Design and construction of underground bulkheads and water barriers: in Proc 7th Commonwealth Mining and Metallurgy Congress, v 3, p 1283-1301

Goodier, J.N., 1933, Concentration of stress around spherical and cylindrical inclusions and flaws: Jour Applied Mechanics, Am Soc Mech Engineers Trans, v 55, p 39-44

Lancaster, F.H., 1964, Research into underground plugs Transvall and Orange Free State Chamber of Mines: Research Report 27/64, Aug, 130 p (Govt Mining Engr of S. Africa)

Loofbourow, R.L., 1973, Groundwater and groundwater control: in SME Mining Engineering Handbook, Section 26.7.4, Underground bulkheads and plugs, p 2646-2648

Louw, A., 1970, Ordeal by water: Mining Congress Jour, Part 1, Mar, p 43-56, Part 2, Apr, p 99-103

Nagy, J., E.C. Seiler, J.W. Conn & H.C. Verakis, 1971, Explosion development in closed vessels: USBM Rpt of Investigations 7507, 49 p

Troxell, G.E., H.E. Davis & J.W. Kelly, 1968, Composition and properties of concrete: 2nd ed, McGraw-Hill, 529 p

U.S. Department of Energy, WIPP Project Office, 1996, Detailed design report for an operational phase panel-closure system: DOE/WIPP-96-2150, 208 p

U.S. Bureau of Reclamation, 1963, Concrete Manual: Denver, CO, 642 p

World Mining, 1969, How West Driefontein gold mine fought and won the flood battle: US Edition, Mar, v 5, no 3, p 18-23 & 36

APPENDIX A. INTAKE DRIFT BULKHEAD DESIGN CALCULATIONS

Notation:

C = compressive bending force (lb)

D = dead load $\left(\frac{lb}{ft}\right)$

FS = factor of safety

$\sqrt{f'_c}$ = square root of f'_c

f'_s = concrete shear strength $\left(2\sqrt{f'_c} = 126 \text{ psi}\right)$

H = depth below surface (2150 ft)

I = moment of inertia (in⁴)

ℓ = Intake Drift width (20 ft)

M_n = nominal beam moment (ft·lb)

S = section modulus (in³)

T_e = effective bulkhead thickness (28 ft)

V_c = concrete shear strength (lb)

V_u = factored shear force (lb)

W = bulkhead load (lb)

ρ_a = allowable pressure head (psi)

ρ_g = pressure gradient $\left(\frac{psi}{ft}\right)$

γ_w = water density (62.4PCF)

ϕ = plain concrete strength reduction factors
    0.65 plain concrete flexure, compression
    shear and bearing

c = centroidal distance (in)

F = fluid load $\left(\frac{lb}{ft}\right)$

f'_c = concrete comp strength (4,000 psi)

f_{cl} = concrete tensile strength $\left(5\phi\sqrt{f'_c} \text{ psi}\right)$

$\left[5(0.65)\sqrt{4000} = 206 \text{ psi}\right]$

h = Intake Drift height (13 ft)

L = live, dynamic, load $\left(\frac{lb}{ft}\right)$

M = bending moment (ft·lb)

M_u = factored beam moment (ft·lb)

T = overall bulkhead thickness (36 ft)

U = required strength $\left(\frac{lb}{ft}\right)$

V_n = nominal shear force (lb)

v_s = shear stress (psi)

ω = uniform bulkhead load $\left(\frac{lb}{ft}\right)$

ρ_d = dynamic pressure head (240 psi)

γ_c = concrete density (151PCF)

γ_s = salt density (140 PCF)

σ_s = flexure stress (psi)

Load factors (ACI 318-95, Sec 9.2.1)
    Static fluid load factor (F) = 1.4;
    Live (dynamic) load factor (L) = 1.7

Load factor (DOE, 1996, Appendix PCS: 2.2.3.1)
    Live (dynamic) load factor (L) = 2

Allowable pressure gradient:

Low pressure grouting of concrete-rock salt contact but not rock salt, gradient allowable = 41 psi/ft (Garrett & Campbell-Pitt, 1958, Chekan, 1985, p11), with factor of safety of 4

Intake Drift bulkhead, design dynamic pressure head = $L\rho_d$ = 2(240) = 480 psi

Required bulkhead thickness with low pressure grouting on concrete/rock salt bulkhead contact:

$$T = \frac{\rho}{41} = \frac{480}{41} = 11.7 \text{ ft}$$

Appendix A. Intake Drift bulkhead design calculations (Continued)

Pressure gradient along minimum effective bulkhead thickness T_e= 28 ft

$$\rho_g = \frac{\rho}{28} = \frac{480}{28} = 17.1 \text{ psi/ft}$$

Factor of Safety against leakage of explosion gasses along concrete/rock salt contact around 28-ft effective bulkhead thickness is:

$$\underline{FS} = \frac{41}{17.1} = \underline{2.40}$$

Allowable concrete shear on Intake Drift perimeter:

$$f_s' = 2\sqrt{f_c'} = 2\sqrt{4000} = 126 \text{ psi} \qquad \text{(ACI 318-95, Sec 11.3.1.1)}$$

$$T = \frac{\rho_a h \ell}{2(h+\ell)f_s'}$$

$$\rho_a = \frac{2T(h+\ell)f_s'}{h\ell} = \frac{2(36)(13+20)126}{13(20)} = \frac{299400}{260} = 1152 \text{ psi}$$

$$W = \rho_d h\ell = 480(13)20(144) = 17,970,000 \text{ lb}$$

$$v_s = \frac{W}{[2(h+\ell)]T(144)} = \frac{17970000}{[2(13+20)]36(144)} = \frac{17970000}{342100} = 52.5 \text{ psi}$$

Factor of Safety against concrete shear failure $= \frac{f_s'}{v_s} = \frac{126}{52.5} = 2.40$

Allowable rock salt shear force along concrete/rock salt contact, Intake Drift bulkhead:

Rock salt cohesion (C_{rs}) approximately 1070 psi (See appendix F.)

Length of minimum shear path in rock salt (L_{rs}) adjacent to concrete/rock salt contact:

$$L_{rs} = \sqrt{8^2 + 8^2} + 0.5 + 9.75 + 9.75 = 31.3 \text{ ft}^2/\text{ft of perimeter}$$

Minimum rock salt perimeter = 2(13+20) = 66 ft

Total effective bulkhead shear area = 2,066 ft^2

Maximum rock salt shear resistance = 2066(1070)144 = 318,300,000 lb

Appendix A. Intake Drift bulkhead design calculations (Continued)

Maximum shear force on panel side bulkhead face and outward inclined panel side concrete/rock salt contact potentially opened by explosion gas pressure:

Face area = 13(20) = 260 ft^2

Vertical component of sloping area = $2(13+20)8 + \frac{\pi 8^2}{4}$ = 528 + 50 = 578 ft^2

Design maximum thrust = (578+260)(480)144 = 57,920,000 lb

Factor of Safety against rock salt shear failure = $\frac{318,300,000}{57,920,000}$ = 5.50

Plain concrete deep beam bending stress design, Intake Drift (ACI 318-95, Sec 9.3.5, Sec 10.5; ACI318.1-89, Sec 6.2.1)

for 480 psi design dynamic pressure head:

$\omega = U = 2\rho_d(144) = 2(240)144 = 69,120 \left(\frac{lb}{ft^2}\right)$

Bulkhead deep beam griped at rock salt ribs by creep pressure (worst-case)

$M_n = \frac{\omega \ell^2}{24} = \frac{69120(20^2)}{24} = 1,152,000$ ft·lb

$M_u = \frac{M_n}{0.65} = \frac{1152000}{0.65} = 1,772,000$ ft·lb

$S = \frac{I}{c} = \frac{\frac{bT^3}{12}}{\frac{T}{2}} = \frac{\frac{1(T^3)(12^3)}{12}}{\frac{T(12)}{2}} = \frac{144T^2}{6}$

$f'_{cl} = 5\phi\sqrt{f'_c} = 5(0.65)\sqrt{4000} = 206$ psi

$f'_{cl} = 206 = \sigma = \frac{M_u c}{I} = \frac{M_u}{S} = \frac{1772000}{\frac{144T^2}{6}} = \frac{73830}{T^2}$

$T = \sqrt{\frac{73830}{206}} = \sqrt{358.4} = 18.9$ ft thick plain concrete bulkhead is required for worst-case rib to rib fixed bulkhead.

$\sigma_s = \frac{M_u}{S} = \frac{M_u}{\frac{144T^2}{6}} = \frac{1772000}{\frac{144(36^2)}{6}} = \frac{1772000}{31100} = 57.0$ psi

$FS = \frac{f'_{cl}}{\sigma_s} = \frac{206}{57.0} = 3.61$

Therefore, a 36-ft thick plain-concrete bulkhead worst-case griped at both ribs of the 20-ft wide Intake Drift, is acceptable as a panel closure bulkhead.

Appendix A. Intake Drift bulkhead design calculations (Continued)

Bulkhead deep beam griped at Intake Drift rock salt roof and floor (best-case)

$$M_n = \frac{\omega \ell^2}{24} = \frac{69120(13^2)}{24} = 486,700 \text{ ft·lb}$$

$$M_u = \frac{M_n}{0.65} = \frac{486700}{0.65} = 748,800 \text{ ft·lb}$$

$$S = \frac{I}{c} = \frac{\frac{bT^3}{12}}{\frac{T}{2}} = \frac{\frac{1(T^3)(12^3)}{12}}{\frac{T(12)}{2}} = \frac{144T^2}{6}$$

$$f'_{cl} = 5\phi\sqrt{f'_c} = 5(0.65)\sqrt{4000} = 206 \text{ psi}$$

$$f'_{cl} = 206 = \sigma = \frac{M_u c}{I} = \frac{M_u}{S} = \frac{748800}{\frac{144T^2}{6}} = \frac{31200}{T^2}$$

$$T = \sqrt{\frac{31200}{206}} = \sqrt{151.5} = 12.3\text{-ft thick plain concrete bulkhead is required for best-case}$$
roof to floor fixed bulkhead.

$$\sigma_s = \frac{M_u}{S} = \frac{M_u}{\frac{144T^2}{6}} = \frac{748800}{\frac{144(36^2)}{6}} = \frac{748800}{31100} = 24.1 \text{ psi}$$

$$FS = \frac{f'_{cl}}{\sigma_s} = \frac{206}{24.1} = 8.55$$

Therefore, a 36-ft thick plain-concrete bulkhead, best-case griped at roof and floor of the 13-ft high Intake Drift, is acceptable as a panel closure bulkhead.

APPENDIX B. EXHAUST DRIFT BULKHEAD DESIGN CALCULATIONS

Notation:

C = compressive bending force (lb)

D = dead load $\left(\frac{lb}{ft}\right)$

FS = factor of safety

$\sqrt{f'_c}$ = square root of f'_c

f'_s = concrete shear strength $\left(2\sqrt{f'_c} = 126 \text{ psi}\right)$

H = depth below surface (2150 ft)

I = moment of inertia (in^4)

ℓ = Exhaust Drift width (14 ft)

M_n = nominal beam moment (ft·lb)

S = section modulus (in^3)

T_e = effective bulkhead thickness (18 ft)

V_c = concrete shear strength (lb)

V_u = factored shear force (lb)

W = bulkhead load (lb)

ρ_a = allowable pressure head (psi)

ρ_g = pressure gradient $\left(\frac{psi}{ft}\right)$

γ_w = water density (62.4PCF)

ϕ = plain concrete strength reduction factors
　　0.65 plain concrete flexure, compression
　　shear and bearing

c = centroidal distance (in)

F = fluid load $\left(\frac{lb}{ft}\right)$

f'_c = concrete comp strength (4,000 psi)

f_{cl} = concrete tensile strength $\left(5\phi\sqrt{f'_c} \text{ psi}\right)$

　　$\left[5(0.65)\sqrt{4000} = 206 \text{ psi}\right]$

h = Exhaust Drift height (12 ft)

L = live, dynamic, load $\left(\frac{lb}{ft}\right)$

M = bending moment (ft·lb)

M_u = factored beam moment (ft·lb)

T = overall bulkhead thickness (26 ft)

U = required strength $\left(\frac{lb}{ft}\right)$

V_n = nominal shear force (lb)

v_s = shear stress (psi)

ω = uniform bulkhead load $\left(\frac{lb}{ft}\right)$

ρ_d = dynamic pressure head (240 psi)

γ_c = concrete density (151PCF)

γ_s = salt density (140 PCF)

σ_s = flexure stress (psi)

Load factors (ACI 318-95, Sec 9.2.1)
　　Static fluid load factor (F) = 1.4;
　　Live (dynamic) load factor (L) = 1.7

Load factor (DOE, 1996, Appendix PCS: 2.2.3.1)
　　Live (dynamic) load factor (L) = 2

Allowable pressure gradient:

　　Low pressure grouting of concrete-rock salt contact but not rock salt, gradient allowable = 41 psi/ft (Garrett & Campbell-Pitt, 1958, Chekan, 1985, p11), with factor of safety of 4

　　Exhaust Drift bulkhead, design dynamic pressure head = $L\rho_d$ = 2(240) = 480 psi

　　Required bulkhead length with low pressure grouting on concrete/rock salt bulkhead contact:

　　$T = \frac{\rho}{41} = \frac{480}{41} = 11.7 \text{ ft}$

Appendix B. Exhaust Drift bulkhead design calculations (Continued)

Pressure gradient along minimum effective bulkhead thickness T = 18 ft

$$p_g = \frac{p}{18} = \frac{480}{18} = 26.7 \text{ psi/ft}$$

Factor of Safety against leakage of explosion gasses along concrete/rock salt contact around 18-ft effective bulkhead thickness is:

$$\underline{FS} = \frac{41}{26.7} = \underline{1.54}$$

Allowable concrete shear on Exhaust Drift perimeter:

$$f'_s = 2\sqrt{f'_c} = 2\sqrt{4000} = 126 \text{ psi} \qquad \text{(ACI 318-95, Sec 11.3.1.1)}$$

$$T = \frac{p_a h\ell}{2(h+\ell)f'_s}$$

$$p_a = \frac{2T(h+\ell)f'_s}{h\ell} = \frac{2(36)(12+14)126}{12(14)} = \frac{235900}{168} = 1404 \text{ psi}$$

$$W = p_d h\ell = 480(12)14(144) = 11,610,000 \text{ lb}$$

$$V_s = \frac{W}{[2(h+\ell)]T(144)} = \frac{11610000}{[2(12+14)]36(144)} = \frac{11610000}{269600} = 43.1 \text{ psi}$$

Factor of Safety against concrete shear failure $= \frac{f'_s}{V_s} = \frac{126}{43.1} = 2.92$

Allowable rock salt shear force along concrete/rock salt contact, Exhaust Drift bulkhead:

Rock salt cohesion (C_{rs}) approximately 1070 psi (See appendix F.)

Length of minimum shear path in rock salt (L_{rs}) adjacent to concrete/rock salt contact:

$$L_{rs} = \sqrt{8^2 + 8^2} + 0.5 + 4.75 + 4.75 = 21.3 \text{ ft}^2/\text{ft of perimeter}$$

Minimum rock salt perimeter = 2(12+14) = 52 ft

Total effective bulkhead shear area = 1,108 ft^2

Maximum rock salt shear resistance = 1108(1070)144 = 170,700,000 lb

Appendix B. Exhaust Drift bulkhead design calculations (Continued)

Maximum shear force on panel side bulkhead face and outward inclined panel side concrete/rock salt contact potentially opened by explosion gas pressure:

Face area = 12(14) = 168 ft^2

Vertical component of sloping area = $2(12+14)8 + \frac{\pi 8^2}{4}$ = 416 + 50 = 466 ft^2

Design maximum thrust = (466+168)(1070)144 = 97,690,000 lb

Factor of Safety against rock salt shear failure = $\frac{170,700,000}{97,690,000}$ = 1.75

Plain concrete deep beam bending stress design, Exhaust Drift (ACI 318-95, Sec 9.3.5, Sec 10.5; ACI318.1-89, Sec 6.2.1)

for 480 psi design dynamic pressure head:

$\omega = U = 2\rho_d(144) = 2(240)144 = 69,120 \left(\frac{lb}{ft^2}\right)$

Bulkhead deep beam griped at rock salt ribs by creep pressure (worst-case)

$M_n = \frac{\omega \ell^2}{24} = \frac{69120(14^2)}{24} = 564,500$ ft·lb

$M_u = \frac{M_n}{0.65} = \frac{564500}{0.65} = 868,500$ ft·lb

$S = \frac{I}{c} = \frac{\frac{bT^3}{12}}{\frac{T}{2}} = \frac{\frac{1(T^3)(12^3)}{12}}{\frac{T(12)}{2}} = \frac{144T^2}{6}$

$f'_{cl} = 5\phi\sqrt{f'_c} = 5(0.65)\sqrt{4000} = 206$ psi

$f'_{cl} = 206 = \sigma = \frac{M_u c}{I} = \frac{M_u}{S} = \frac{868500}{\frac{144T^2}{6}} = \frac{36190}{T^2}$

$T = \sqrt{\frac{36190}{206}} = \sqrt{175.7} = 13.3$ ft thick plain concrete bulkhead is required for worst-case rib to rib fixed bulkhead.

$\sigma_s = \frac{M_u}{S} = \frac{M_u}{\frac{144T^2}{6}} = \frac{868500}{\frac{144(26^2)}{6}} = \frac{868500}{16220} = 53.5$ psi

$FS = \frac{f'_{cl}}{\sigma_s} = \frac{206}{53.5} = 3.85$

Therefore, a 26-ft thick plain-concrete bulkhead, worst-case griped at both ribsides of the 14-ft wide Exhaust Drift, is acceptable as a panel closure bulkhead.

Appendix B. Exhaust Drift bulkhead design calculations (Continued)

Bulkhead deep beam griped at Exhaust Drift rock salt roof and floor (best-case)

$$M_n = \frac{\omega \ell^2}{24} = \frac{69120(12^2)}{24} = 414,700 \text{ ft·lb}$$

$$M_u = \frac{M_n}{0.65} = \frac{414700}{0.65} = 638,000 \text{ ft·lb}$$

$$S = \frac{I}{c} = \frac{\frac{bT^3}{12}}{\frac{T}{2}} = \frac{\frac{1(T^3)(12^3)}{12}}{\frac{T(12)}{2}} = \frac{144T^2}{6}$$

$$f'_{cl} = 5\phi\sqrt{f'_c} = 5(0.65)\sqrt{4000} = 206 \text{ psi}$$

$$f'_{cl} = 190 = \sigma = \frac{M_u c}{I} = \frac{M_u}{S} = \frac{638000}{\frac{144T^2}{6}} = \frac{26580}{T^2}$$

$$T = \sqrt{\frac{26580}{206}} = \sqrt{129.0} = 11.4\text{-ft thick plain concrete bulkhead is required for best-case}$$
roof to floor fixed bulkhead.

$$\sigma_s = \frac{M_u}{S} = \frac{M_u}{\frac{144T^2}{6}} = \frac{638000}{\frac{144(26^2)}{6}} = \frac{638000}{16220} = 39.3 \text{ psi}$$

$$FS = \frac{f'_{cl}}{\sigma_s} = \frac{206}{39.3} = 5.24$$

Therefore, a 26-ft thick plain-concrete bulkhead, best-case griped at roof and floor of the 12-ft high Exhaust Drift, is acceptable as a panel closure bulkhead.

APPENDIX C. INTAKE DRIFT EXPLOSION-ISOLATION MASONRY WALL DESIGN
CALCULATIONS

Notation:

C = compressive bending force (lb)

D = dead load $\left(\frac{lb}{ft}\right)$

FS = factor of safety

$\sqrt{f'_c}$ = square root of f'_c

f'_s = masonry shear strength $\left(2\sqrt{f'_c} = 100 \text{ psi}\right)$

H = depth below surface (2150 ft)

I = moment of inertia (in^4)

ℓ = Intake Drift width (20 ft)

M_n = nominal beam moment (ft·lb)

S = section modulus (in^3)

U = required strength $\left(\frac{lb}{ft}\right)$

V_n = nominal shear force (lb)

v_s = shear stress (psi)

ω = uniform bulkhead load $\left(\frac{lb}{ft}\right)$

ρ_a = allowable pressure head (psi)

ρ_g = pressure gradient $\left(\frac{psi}{ft}\right)$

γ_w = water density (62.4PCF)

ϕ = plain masonry strength reduction factors
   0.65 plain concrete flexure, compression
      shear and bearing

c = centroidal distance (in)

F = fluid load $\left(\frac{lb}{ft}\right)$

f'_c = masonry comp strength (2,500 psi)

f_{cl} = masonry tensile strength $\left(5\phi\sqrt{f'_c} \text{ psi}\right)$

$\left[5(0.65)\sqrt{2500} = 162 \text{ psi}\right]$

h = Intake Drift height (13 ft)

L = live, dynamic, load $\left(\frac{lb}{ft}\right)$

M = bending moment (ft·lb)

M_u = factored beam moment (ft·lb)

T = overall bulkhead thickness (12 ft)

V_c = masonry shear strength (lb)

V_u = factored shear force (lb)

W = bulkhead load (lb)

ρ = design pressure head (480 psi)

ρ_d = dynamic pressure head (240 psi)

γ_c = masonry density (151PCF)

γ_s = salt density (140 PCF)

σ_s = flexure stress (psi)

Load factors (ACI 318-95, Sec 9.2.1)
     Static fluid load factor (F) = 1.4;
     Live (dynamic) load factor (L) = 1.7

Load factor (DOE, 1996, Appendix PCS: 2.2.3.1)
     Live (dynamic) load factor (L) = 2

Allowable pressure gradient:

    Low pressure grouting of masonry/rock salt contact but not rock salt, gradient allowable
    = 41 psi/ft (Garrett & Campbell-Pitt, 1958, Chekan, 1985, p11), with factor of safety of 4

    Intake Drift bulkhead, design dynamic pressure head = $L\rho_d = 2(240) = 480$ psi

Appendix C. Intake Drift explosion-isolation masonry wall design calculations (Continued)

Required bulkhead thickness with low pressure grouting on masonry/rock salt bulkhead contact:

$$T = \frac{\rho}{41} = \frac{480}{41} = 11.7 \text{ ft}$$

Pressure gradient along bulkhead thickness T = 12 ft

$$\rho_g = \frac{\rho}{28} = \frac{480}{12} = 40.0 \text{ psi/ft}$$

Factor of Safety against leakage of explosion gasses along masonry/rock salt contact around 12-ft effective bulkhead thickness is:

$$\underline{FS} = \frac{41}{40.0} = \underline{1.03}$$

Allowable masonry shear on Intake Drift perimeter:

$$f_s' = 2\sqrt{f_c'} = 2\sqrt{2500} = 100 \text{ psi} \qquad \text{(ACI 318-95, Sec 11.3.1.1)}$$

$$T = \frac{\rho_a h \ell}{2(h+\ell)f_s'}$$

$$\rho_a = \frac{2T(h+\ell)f_s'}{h\ell} = \frac{2(12)(13+20)100}{13(20)} = \frac{79200}{260} = 304.6 \text{ psi}$$

$$W = \rho_d h\ell = 480(13)20(144) = 17,970,000 \text{ lb}$$

$$v_s = \frac{W}{[2(h+\ell)]T(144)} = \frac{17970000}{[2(13+20)]12(144)} = \frac{17970000}{114000} = 157.6 \text{ psi}$$

Factor of Safety against masonry shear failure $= \frac{f_s'}{v_s} = \frac{100}{157.6} = \underline{\textbf{0.63}}$

Required masonry wall thickness to resist design methane explosion pressure

$$T = \frac{\rho h\ell}{2(h+\ell)f_s'} = \frac{480(13)20}{2[13+20]100} = \frac{124800}{6600} = 18.9 \text{ ft}$$

Allowable rock salt shear force along masonry/rock salt contact, Intake Drift explosion-isolation bulkhead:

Rock salt cohesion (C$_{rs}$)approximately 1070 psi (See appendix F.)

Length of minimum shear path in rock salt (L$_{rs}$) adjacent to masonry/rock salt contact:

L$_{rs}$ = 12 ft²/ft of perimeter

Appendix C. Intake Drift explosion-isolation masonry wall design calculations (Continued)

Minimum rock salt perimeter = 2(14+21) = 70 ft
based on perimeter hitched 6-in into roof, ribs and floor of Intake Drift

Total effective bulkhead shear area = 840 ft^2

Maximum rock salt shear resistance = 840(1070)144 = 129,400,000 lb

Maximum shear force on masonry/rock salt contact potentially opened by explosion gas pressure:

Face area = 14(21) = 294 ft^2

Design maximum thrust = (294)(480)144 = 20,320,000 lb

Factor of Safety against rock salt shear failure = $\frac{129,400,000}{20,320,000}$ = 6.37

Masonry explosion-isolation beam bending stress design, Intake Drift (ACI 318-95, Sec 9.3.5, Sec 10.5; ACI318.1-89, Sec 6.2.1) for 480 psi design dynamic pressure head:

$$\omega = U = 2\rho_d(144) = 2(240)144 = 69,120 \left(\tfrac{lb}{ft^2}\right)$$

Bulkhead deep beam griped at rock salt ribs by creep pressure (worst-case)

$$M_n = \frac{\omega \ell^2}{24} = \frac{69120(20^2)}{24} = 1,152,000 \text{ ft·lb}$$

$$M_u = \frac{M_n}{0.65} = \frac{1152000}{0.65} = 1,772,000 \text{ ft·lb}$$

$$S = \frac{I}{c} = \frac{\frac{bT^3}{12}}{\frac{T}{2}} = \frac{\frac{1(T^3)(12^3)}{12}}{\frac{T(12)}{2}} = \frac{144T^2}{6}$$

$$f'_{cl} = 5\phi\sqrt{f'_c} = 5(0.65)\sqrt{2500} = 162.5 \text{ psi}$$

$$f'_{cl} = 150 = \sigma = \frac{M_u c}{I} = \frac{M_u}{S} = \frac{1772000}{\frac{144T^2}{6}} = \frac{73830}{T^2}$$

$$T = \sqrt{\frac{73830}{162.5}} = \sqrt{454.3} = 21.3 \text{ ft thick masonry bulkhead is required for worst-case rib to}$$
rib fixed explosion-isolation bulkhead

$$\sigma_s = \frac{M_u}{S} = \frac{M_u}{\frac{144T^2}{6}} = \frac{1772000}{\frac{144(12^2)}{6}} = \frac{1772000}{3456} = 512.7 \text{ psi}$$

$$FS = \frac{f'_{cl}}{\sigma_s} = \frac{162.5}{512.7} = \mathbf{0.32}$$

Appendix C. Intake Drift explosion-isolation masonry wall design calculations (Continued)

Therefore, 12-ft thick cement-mortared masonry block bulkhead, worst-case griped at both ribsides of the 20-ft wide Intake Drift, is NOT acceptable as an explosion-isolation bulkhead.

Masonry bulkhead deep beam griped at Intake Drift rock salt roof and floor (best-case)

$$M_n = \frac{\omega \ell^2}{24} = \frac{69120(13^2)}{24} = 486{,}700 \text{ ft·lb}$$

$$M_u = \frac{M_n}{0.65} = \frac{486700}{0.65} = 748{,}800 \text{ ft·lb}$$

$$S = \frac{I}{c} = \frac{\frac{bT^3}{12}}{\frac{T}{2}} = \frac{\frac{1(T^3)(12^3)}{12}}{\frac{T(12)}{2}} = \frac{144T^2}{6}$$

$$f'_{cl} = 5\phi\sqrt{f'_c} = 5(0.65)\sqrt{2500} = 162.5 \text{ psi}$$

$$f'_{cl} = 162.5 = \sigma = \frac{M_u c}{I} = \frac{M_u}{S} = \frac{748800}{\frac{144T^2}{6}} = \frac{31200}{T^2}$$

$$T = \sqrt{\frac{31200}{162.5}} = \sqrt{192.0} = 13.8\text{-ft thick concrete block masonry bulkhead is required for}$$
best-case roof to floor fixed bulkhead.

$$\sigma_s = \frac{M_u}{S} = \frac{M_u}{\frac{144T^2}{6}} = \frac{748800}{\frac{144(12^2)}{6}} = \frac{748800}{3456} = 216.7 \text{ psi}$$

$$FS = \frac{f'_{cl}}{\sigma_s} = \frac{162.5}{216.7} = \underline{\mathbf{0.75}}$$

Therefore, 12-ft thick cement-mortared masonry block bulkhead, best-case griped at roof and floor of the 13-ft high Intake Drift, is NOT acceptable as an explosion-isolation bulkhead.

APPENDIX D.   EXHAUST DRIFT EXPLOSION-ISOLATION MASONRY WALL DESIGN
                    CALCULATIONS

Notation:

C = compressive bending force (lb)

D = dead load $\left(\frac{lb}{ft}\right)$

FS = factor of safety

$\sqrt{f'_c}$ = square root of f'_c

f'_s = masonry shear strength $\left(2\sqrt{f'_c} = 100 \text{ psi}\right)$

H = depth below surface (2150 ft)

I = moment of inertia (in⁴)

ℓ = Exhaust Drift width (14 ft)

M_n = nominal beam moment (ft·lb)

S = section modulus (in³)

U = required strength $\left(\frac{lb}{ft}\right)$

V_n = nominal shear force (lb)

v_s = shear stress (psi)

ω = uniform bulkhead load $\left(\frac{lb}{ft}\right)$

ρ_a = allowable pressure head (psi)

ρ_g = pressure gradient $\left(\frac{psi}{ft}\right)$

γ_w = water density (62.4PCF)

ϕ = plain masonry strength reduction factors
    0.65 plain concrete flexure, compression
        shear and bearing

c = centroidal distance (in)

F = fluid load $\left(\frac{lb}{ft}\right)$

f'_c = masonry comp strength (2,500 psi)

f_{cl} = masonry tensile strength $\left(5\phi\sqrt{f'_c} \text{ psi}\right)$

    $\left[5(0.65)\sqrt{2500} = 162 \text{ psi}\right]$

h = Exhaust Drift height (12 ft)

L = live, dynamic, load $\left(\frac{lb}{ft}\right)$

M = bending moment (ft·lb)

M_u = factored beam moment (ft·lb)

T = overall bulkhead thickness (12 ft)

V_c = masonry shear strength (lb)

V_u = factored shear force (lb)

W = bulkhead load (lb)

ρ = design pressure head (480 psi)

ρ_d = dynamic pressure head (240 psi)

γ_c = masonry density (151PCF)

γ_s = salt density (140 PCF)

σ_s = flexure stress (psi)

Load factors (ACI 318-95, Sec 9.2.1)
        Static fluid load factor (F) = 1.4;
        Live (dynamic) load factor (L) = 1.7

Load factor (DOE, 1996, Appendix PCS: 2.2.3.1)
        Live (dynamic) load factor (L) = 2

Allowable pressure gradient:

    Low pressure grouting of masonry/rock salt contact but not rock salt, gradient allowable
    = 41 psi/ft (Garrett & Campbell-Pitt, 1958, Chekan, 1985, p11), with factor of safety of 4

    Exhaust Drift bulkhead, design dynamic pressure head = $L\rho_d$ = 2(240) = 480 psi

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Appendix D. Exhaust Drift explosion-isolation wall design calculations (Continued)

Required bulkhead length with low pressure grouting on masonry/rock salt bulkhead contact:

$$T = \frac{p}{4I} = \frac{480}{4I} = 11.7 \text{ ft}$$

Pressure gradient along bulkhead thickness T = 12 ft

$$\rho_g = \frac{p}{12} = \frac{480}{12} = 40.0 \text{ psi/ft}$$

Factor of Safety against leakage of explosion gasses along masonry/rock salt contact around 12-ft effective bulkhead thickness is:

$$FS = \frac{41}{40.0} = 1.03$$

Allowable masonry shear on Exhaust Drift perimeter:

$$f_s' = 2\sqrt{f_c'} = 2\sqrt{2500} = 100 \text{ psi} \qquad (\text{ACI 318-95, Sec 11.3.1.1})$$

$$T = \frac{\rho_a h \ell}{2(h+\ell)f_s'}$$

$$\rho_a = \frac{2T(h+\ell)f_s'}{h\ell} = \frac{2(12)(12+14)100}{12(14)} = \frac{62400}{168} = 371.4 \text{ psi}$$

$$W = \rho_d h\ell = 480(12)14(144) = 11,610,000 \text{ lb}$$

$$V_s = \frac{W}{[2(h+\ell)]T(144)} = \frac{11610000}{2(12+14)12(144)} = \frac{11610000}{89860} = 129.2 \text{ psi}$$

Factor of Safety against masonry shear failure $= \frac{f_s'}{v_s} = \frac{100}{129.2} = \underline{\mathbf{0.77}}$

Required masonry wall thickness to resist design methane explosion pressure

$$T = \frac{\rho h\ell}{2(h+\ell)f_s'} = \frac{480(12)14}{2[13+20]100} = \frac{81640}{6600} = 12.2 \text{ ft}$$

Allowable rock salt shear force along masonry/rock salt contact, Exhaust Drift explosion-isolation bulkhead:

Rock salt cohesion (C_{rs})approximately 1070 psi (See appendix F.)

Length of minimum shear path in rock salt (L_{rs}) adjacent to masonry/rock salt contact:

$L_{rs} = 12 \text{ ft}^2/\text{ft of perimeter}$

Appendix D. Exhaust Drift explosion-isolation wall design calculations (Continued)

Minimum rock salt perimeter = 2(13+15) = 56 ft
  based on perimeter hitched 6-in into roof, ribs and floor of Exhaust Drift

Total effective bulkhead shear area = 672 ft^2

Maximum rock salt shear resistance = 672(1070)144 = 103,500,000 lb

Maximum shear force on panel side bulkhead face and outward inclined panel side masonry/rock salt contact potentially opened by explosion gas pressure:

Face area = 13(15) = 195 ft^2

Design maximum thrust = (195)(480)144 = 13,480,000 lb

Factor of Safety against rock salt shear failure $= \frac{103,500,000}{13,480,000} = 7.68$

Masonry explosion-isolation beam bending stress design, Exhaust Drift (ACI 318-95, Sec 9.3.5, Sec 10.5; ACI318.1-89, Sec 6.2.1) for 480 psi design dynamic pressure head:

$$\omega = U = 2\rho_d(144) = 2(240)144 = 69,120 \left(\tfrac{lb}{ft^2}\right)$$

Bulkhead deep beam griped at rock salt ribs by creep pressure (worst-case)

$$M_n = \frac{\omega \ell^2}{24} = \frac{69120(14^2)}{24} = 564,500 \text{ ft·lb}$$

$$M_u = \frac{M_n}{0.65} = \frac{564500}{0.65} = 868,500 \text{ ft·lb}$$

$$S = \frac{I}{c} = \frac{\frac{bT^3}{12}}{\frac{T}{2}} = \frac{\frac{1(T^3)(12^3)}{12}}{\frac{T(12)}{2}} = \frac{144T^2}{6}$$

$$f'_{cl} = 5\phi\sqrt{f'_c} = 5(0.65)\sqrt{2500} = 162.5 \text{ psi}$$

$$f'_{cl} = 150 = \sigma = \frac{M_u c}{I} = \frac{M_u}{S} = \frac{868500}{\frac{144T^2}{6}} = \frac{36190}{T^2}$$

$T = \sqrt{\frac{36190}{162.5}} = \sqrt{222.7} = 14.9$ ft thick masonry bulkhead is required for worst-case rib to rib fixed explosion-isolation bulkhead.

$$\sigma_s = \frac{M_u}{S} = \frac{M_u}{\frac{144T^2}{6}} = \frac{868500}{\frac{144(12^2)}{6}} = \frac{868500}{3456} = 251.3 \text{ psi}$$

$$FS = \frac{f'_{cl}}{\sigma_s} = \frac{162.5}{251.3} = \mathbf{0.65}$$

Appendix D. Exhaust Drift explosion-isolation wall design calculations (Continued)

Therefore, 12-ft thick cement mortared masonry block wall, worst-case griped at both ribsides of the 14-ft wide Exhaust Drift, is NOT acceptable as an explosion-isolation bulkhead.

Masonry bulkhead deep beam griped at Exhaust Drift rock salt roof and floor (best-case)

$$M_n = \frac{\omega \ell^2}{24} = \frac{69120(12^2)}{24} = 414{,}700 \text{ ft·lb}$$

$$M_u = \frac{M_n}{0.65} = \frac{414700}{0.65} = 638{,}000 \text{ ft·lb}$$

$$S = \frac{I}{c} = \frac{\frac{bT^3}{12}}{\frac{T}{2}} = \frac{\frac{1(T^3)(12^3)}{12}}{\frac{T(12)}{2}} = \frac{144T^2}{6}$$

$$f'_{cl} = 5\phi\sqrt{f'_c} = 5(0.65)\sqrt{2500} = 162.5 \text{ psi}$$

$$f'_{cl} = 162.5 = \sigma = \frac{M_u c}{I} = \frac{M_u}{S} = \frac{638000}{\frac{144T^2}{6}} = \frac{26580}{T^2}$$

$$T = \sqrt{\frac{26580}{162.5}} = \sqrt{177.2} = 13.3\text{-ft thick concrete block masonry bulkhead is required for}$$
best-case roof to floor fixed beam bulkhead.

$$\sigma_s = \frac{M_u}{S} = \frac{M_u}{\frac{144T^2}{6}} = \frac{638000}{\frac{144(12^2)}{6}} = \frac{638000}{3456} = 184.6 \text{ psi}$$

$$FS = \frac{f'_{cl}}{\sigma_s} = \frac{162.5}{184.6} = \mathbf{0.88}$$

Therefore, 12-ft thick cement mortared masonry block wall, best-case griped at roof and floor of the 12-ft high, Exhaust Drift is NOT acceptable as an explosion-isolation bulkhead.

APPENDIX E. INTAKE DRIFT CONSTRUCTION-ISOLATION WALL DESIGN
              CALCULATIONS (ROOF FALL PRESSURE)

Notation:

C = compressive bending force (lb)

D = dead load $\left(\frac{lb}{ft}\right)$

FS = factor of safety

$\sqrt{f'_c}$ = square root of f'_c

f'_s = masonry shear strength $\left(2\sqrt{f'_c} = 100\ psi\right)$

H = depth below surface (2150 ft)

I = moment of inertia (in^4)

ℓ = Intake Drift width (20 ft)

M_n = nominal beam moment (ft·lb)

S = section modulus (in^3)

U = required strength $\left(\frac{lb}{ft}\right)$

V_n = nominal shear force (lb)

V_s = shear stress (psi)

ω = uniform bulkhead load $\left(\frac{lb}{ft}\right)$

ρ_d = dynamic pressure head (0.035 psi)

γ_c = masonry density (151PCF)

γ_s = salt density (140 PCF)

ϕ = plain concrete strength reduction factors
    0.65 plain concrete flexure, compression
        shear and bearing

c = centroidal distance (in)

F = fluid load $\left(\frac{lb}{ft}\right)$

f'_c = masonry comp strength (2,500 psi)

f_{cl} = masonry tensile strength $\left(5\phi\sqrt{f'_c}\ psi\right)$

$\left[5(0.65)\sqrt{2500} = 162\ psi\right]$

h = Intake Drift height (13 ft)

L = live, dynamic, load $\left(\frac{lb}{ft}\right)$

M = bending moment (ft·lb)

M_u = factored beam moment (ft·lb)

T = overall bulkhead thickness (4 ft)

V_c = concrete shear strength (lb)

V_u = factored shear force (lb)

W = bulkhead load (lb)

p = design pressure head (0.070 psi)

ρ_g = pressure gradient $\left(\frac{psi}{ft}\right)$

γ_w = water density (62.4PCF)

σ_s = flexure stress (psi)

Load factors (ACI 318-95, Sec 9.2.1)
        Static fluid load factor (F) = 1.4;
        Live (dynamic) load factor (L) = 1.7

Load factor (DOE, 1996, Appendix PCS: 2.2.3.1)
        Live (dynamic) load factor (L) = 2

Allowable pressure gradient:

        Low pressure grouting of masonry/rock salt contact but not rock salt, gradient allowable
        = 41 psi/ft (Garrett & Campbell-Pitt, 1958, Chekan, 1985, p11), with factor of safety of 4

        Intake Drift bulkhead, 10 psf design dynamic pressure head = $L\rho_d$ = 2(0.035) = 0.070 psi

Appendix E. Intake Drift construction-isolation wall design calculations (Continued)

Required bulkhead thickness with low pressure grouting on masonry/rock salt bulkhead contact:

$$T = \frac{\rho}{41} = \frac{0.070}{41} = 0.0002 \text{ ft}$$

Pressure gradient along bulkhead thickness T = 4 ft

$$\rho_g = \frac{\rho}{T} = \frac{0.070}{4} = 0.0175 \text{ psi/ft}$$

Factor of Safety against leakage of explosion gasses along masonry/rock salt contact around 28-ft effective bulkhead thickness is:

$$\underline{FS} = \frac{41}{0.0175} = >2300$$

Allowable masonry shear on Intake Drift perimeter:

$$f'_s = 2\sqrt{f'_c} = 2\sqrt{2500} = 100 \text{ psi} \qquad \text{(ACI 318-95, Sec 11.3.1.1)}$$

$$T = \frac{\rho_a h \ell}{2(h+\ell)f'_s}$$

$$\rho_a = \frac{2T(h+\ell)f'_s}{h\ell} = \frac{2(4)(13+20)100}{13(20)} = \frac{26400}{260} = 101.5 \text{ psi}$$

$$W = \rho_d h\ell = 0.070(13)20(144) = 2621 \text{ lb}$$

$$V_s = \frac{W}{[2(h+\ell)]T(144)} = \frac{2621}{[2(13+20)]4(144)} = \frac{2621}{38020} = 0.0689 \text{ psi}$$

Factor of Safety against masonry shear failure $= \frac{f'_s}{V_s} = \frac{100}{0.0689} = >1450$

Allowable rock salt shear force along masonry/rock salt contact, Intake Drift constructon-isolation bulkhead:

Rock salt cohesion (C_{rs}) approximately 1070 psi (See appendix F.)

Length of minimum shear path in rock salt (L_{rs}) adjacent to masonry/rock salt contact:

$$L_{rs} = 4 \text{ ft}^2/\text{ft of perimeter}$$

Minimum rock salt perimeter = 2(14+21) = 70 ft
(6-in inset in roof, walls and floor)

Total effective bulkhead shear area = 280 ft^2

Appendix E. Intake Drift construction-isolation wall design calculations (Continued)

Maximum rock salt shear resistance = $2(14+21)12(1070)144 = 129,400$ lb
based on perimeter hitched 6-in into roof, ribs and floor of Intake Drift

Maximum shear force on masonry/rock salt contact potentially opened by roof fall overpressure:

Face area = $14(21) = 294$ ft^2

Design maximum thrust = $(294)(0.070)144 = 2,964$ lb

Factor of Safety against rock salt shear failure = $\frac{129,400}{2,964} = 43.7$

Masonry construction-isolation beam bending stress design, Intake Drift (ACI 318-95, Sec 9.3.5, Sec 10.5; ACI318.1-89, Sec 6.2.1)

for 0.070 psi design dynamic roof fall pressure head:

$$\omega = U = 2\rho_d(144) = 2(0.035)144 = 10.1 \left(\tfrac{lb}{ft^2}\right)$$

Simply supported 4-ft thick bulkhead beam supported at rock salt ribs by contact grout pressure (worst-case)

$$M_n = \frac{\omega\ell^2}{8} = \frac{10.1(21^2)}{8} = 556.8 \text{ ft·lb}$$

$$M_u = \frac{M_n}{0.65} = \frac{556.8}{0.65} = 856.6 \text{ ft·lb}$$

$$S = \frac{I}{c} = \frac{\frac{bT^3}{12}}{\frac{T}{2}} = \frac{\frac{1(T^3)(12^3)}{12}}{\frac{T(12)}{2}} = \frac{144T^2}{6}$$

$$f'_{cl} = 5\phi\sqrt{f'_c} = 5(0.65)\sqrt{2500} = 162.5 \text{ psi}$$

$$f'_{cl} = 150 = \sigma = \frac{M_u c}{I} = \frac{M_u}{S} = \frac{856.6}{\frac{144T^2}{6}} = \frac{35.69}{T^2}$$

$$T = \sqrt{\frac{35.69}{162.5}} = \sqrt{0.220} = 0.47 \text{ ft thick masonry concrete bulkhead is required for}$$
worst-case rib to rib simply-supported construction-isolation bulkhead

$$\sigma_s = \frac{M_u}{S} = \frac{M_u}{\frac{144T^2}{6}} = \frac{856.6}{\frac{144(4^2)}{6}} = \frac{856.6}{384.0} = 2.23 \text{ psi}$$

$$FS = \frac{f'_{cl}}{\sigma_s} = \frac{162.5}{2.23} = >73$$

Appendix E. Intake Drift construction-isolation wall design calculations (Continued)

Therefore, 4-ft thick cement-mortared concrete block masonry bulkhead, worst-case, simply-supported at both ribsides of the hitched in 21-ft wide Intake Drift, is acceptable as a construction-isolation bulkhead.

Simply supported 4-ft thick bulkhead beam supported at roof and floor by contact grout pressure (best-case)

$$M_n = \frac{\omega \ell^2}{8} = \frac{10.1(13^2)}{8} = 213.4 \text{ ft·lb}$$

$$M_u = \frac{M_n}{0.65} = \frac{213.4}{0.65} = 328.3 \text{ ft·lb}$$

$$S = \frac{I}{c} = \frac{\frac{bT^3}{12}}{\frac{T}{2}} = \frac{\frac{1(T^3)(12^3)}{12}}{\frac{T(12)}{2}} = \frac{144T^2}{6}$$

$$f'_{cl} = 5\phi\sqrt{f'_c} = 5(0.65)\sqrt{2500} = 162.5 \text{ psi}$$

$$f'_{cl} = 162.5 = \sigma = \frac{M_u c}{I} = \frac{M_u}{S} = \frac{328.3}{\frac{144T^2}{6}} = \frac{13.68}{T^2}$$

$$T = \sqrt{\frac{13.68}{162.5}} = \sqrt{0.0842} = 0.29 \text{ ft thick masonry bulkhead is required for best-case rib to}$$
rib simply-supported construction-isolation bulkhead

$$\sigma_s = \frac{M_u}{S} = \frac{M_u}{\frac{144T^2}{6}} = \frac{328.3}{\frac{144(4^2)}{6}} = \frac{328.3}{384.0} = 0.855 \text{ psi}$$

$$FS = \frac{f'_{cl}}{\sigma_s} = \frac{162.5}{0.855} = >190$$

Therefore, 4-ft thick cement-mortared concrete block masonry bulkhead, best-case simply-supported at roof and floor of the hitched 13-ft high Intake Drift, is acceptable as a construction-isolation bulkhead.

APPENDIX F. TRIAXIAL PROPERTIES OF PERMIAN EVAPORITES

Source, Cycle etc. and testing lab or reference	Unconfined Compression Strength (psi)	Angle of Internal Friction (degs)	Cohesion (psi)	Confining Pressure Range (psi)	Number Samples Tested
AEC-7 & ERDA-9 boreholes, rock salt from 1900 to 2800 ft, Carlsbad, NM (GCR, Chapter 4)	3230	29.6°	940	0-3000	8
Mississippi Chemical Corp., Carlsbad, NM					
Cycle 7, Potash salt (CSM Lab, 1982)	3520	46.6°	700	0-1500	64
Cycle 5, Potash salt (CSM Lab, 1991)	2880	36.9°	720	0-1500	34
Cycle 5, Roof rock salt	3170	39.3°	750	0-2000	25
Cycle 5, Floor clayey rock salt (CSM '92)	3580	38.1°	870	0-2000	20
PCS Mining, Ltd., Rocanville Div., Saskatchewan (Molavi & Wooley, 1986)					
Esterhazy potash salt mbr.	2300	27.3°	700	Unknown	??
RE/SPEC, Inc., Paradox Basin, UT (Pfeifle, Mellegard & Senseny, 1982)					
Cycle 6, Rock salt	4970	33.7°	1330	0-2200	4
Cycle 6, Carnalite	5660	32.0°	1570	0-2200	4
Cycle 7, Rock salt	5880	33.5°	1580	0-2200	4
RE/SPEC, Inc., Permian Basin, TX (Pfeifle, Mellegard & Senseny, 1982)					
Cycle 5, Rock salt	5000	31.2°	1410	0-2200	10
Cycle 4, Rock salt	4150	30.8°	1180	0-2200	4
Dettin Well, Rock salt, 1900-ft depth	4640	33.8°	1240	0-2200	6
Dettin Well, Rock salt, 2250-ft depth	4280	36.0°	1090	0-2200	6
Zeeck Well, Rock salt, 28-2900-ft depth	4750	33.4°	1280	0-2200	9
Carey Salt Co., Hutchinson, KS (CSM Lab, 1985)					
Rock salt	2900	49.1°	540	0-1000	24
Independent Salt Co., Kanapolis, KS (CSM Lab, 1987)					
Roof rock salt	4330	31.2°	1220	0-3000	16
Averages	4080	35.2°	1070		
Standard Deviations	1040	5.9°	330		

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APPENDIX G.   TRIAXIAL PROPERTIES OF SELECTED EVAPORITES

Source, Cycle etc. and testing lab or reference	Unconfined Compression Strength (psi)	Angle of Internal Friction (degs)	Cohesion (psi)	Confining Pressure Range (psi)	Number Samples Tested
Minerales Para La Industria, Sal-Gema, Hidalgo, N.L., Mexico (CSM & Adv Terra Testing Labs 1992)					
Grueso Brecha Domo Sal, Barreno30	3630	48.4°	690	0-2000	9
Intermedio Brecha Domo Sal, Bar 30	3180	49.3°	590	0-2000	7
Fino Brecha Domo Sal, Barreno 30	3150	48.8°	600	0-2000	9
All Barreno 30 Brecha Sal Pruebas	3340	48.7°	630	0-2000	25
Upper Brecha Sal, Barreno 36	7510	44.9°	1560	0-2000	5
Lower Brecha Sal, Barreno 36	5230	47.8°	1010	0-2000	5
All Barreno 36 Brecha Sal Pruebas	6350	46.4°	1270	0-2000	10
ALL BRECHA SAL PRUEBAS, Bars 30-36	4110	48.4°	780	0-2000	35
Minerales Para La Industria, Sal-Gema, Hidalgo, N.L., Mexico (ATT Lab 1992)					
Yeso, Caliza y Anhidrita Brecha Barreno 36	5050	47.2°	990	0-2000	5
Minerales Para La Industria, Sal-Gema, Hidalgo, N.L., Mexico (ATT Lab 1992)					
Red Brecciated Salt, Barreno 20	2260	20.0°	790	0-2000	3
AEC-7 & ERDA-9 boreholes, rock salt from 1900 to 2800 ft, Carlsbad, NM (GCR, Chapter 4)	3230	29.6°	940	0-3000	8
Cote Blanche Mine, Cote Blanche, LA (Hansen, 1977)					
Cote Blanche, dome salt	4130	40.1°	960	0-1500	9
Cote Blanche, yellow dome salt	6840	34.5°	1800	0-1500	3
German Democratic Republic (Menzel, et al, 1972)					
Rock salt	5590	34.8°	1460	0-4250	8
"Hartsalz"	6950	35.5°	1790	0-4250	8
Carnolite	4730	30.6°	1350	0-4250	8
"Sylinit"	6730	34.0°	1790	0-4250	8

APPENDIX G. TRIAXIAL PROPERTIES OF SELECTED EVAPORITES (Continued)

Source, Cycle etc. and testing lab or reference	Unconfined Compression Strength (psi)	Angle of Internal Friction (degs)	Cohesion (psi)	Confining Pressure Range (psi)	Number Samples Tested
Mississippi Chemical Corp., Carlsbad, NM					
Cycle 7, Potash salt (CSM Lab, 1982)	3520	46.6°	700	0-1500	64
Cycle 5, Potash salt (CSM Lab, 1991)	2880	36.9°	720	0-1500	34
Cycle 5, Roof rock salt	3170	39.3°	750	0-2000	25
Cycle 5, Floor clayey rock salt (CSM Lab, 1992)	3580	38.1°	870	0-2000	20
PCS Mining, Ltd., Rocanville Div., Saskatchewan (Molavi & Wooley, 1986)					
Esterhazy potash salt mbr.	2300	27.3°	700	Unknown	??
RE/SPEC, Inc., Paradox Basin, UT (Pfeifle, Mellegard & Senseny, 1982)					
Cycle 6, Rock salt	4970	33.7°	1330	0-2200	4
Cycle 6, Carnalite	5660	32.0°	1570	0-2200	4
Cycle 7, Rock salt	5880	33.5°	1580	0-2200	4
RE/SPEC, Inc., Permian Basin, TX (Pfeifle, Mellegard & Senseny, 1982)					
Cycle 5, Rock salt	5000	31.2°	1410	0-2200	10
Cycle 4, Rock salt	4150	30.8°	1180	0-2200	4
Dettin Well, Rock salt, 1900-ft depth	4640	33.8°	1240	0-2200	6
Dettin Well, Rock salt, 2250-ft depth	4280	36.0°	1090	0-2200	6
Zeeck Well, Rock salt, 28-2900-ft depth	4750	33.4°	1280	0-2200	9
RE/SPEC, Inc., LA salt domes (Pfeifle, Mellegard & Senseny, 1982)					
Richton Dome	3280	35.7°	840	0-2200	4
Vacherie Dome	3340	36.6°	840	0-2200	4
RE/SPEC, Inc., Avery Island, LA (Hansen & Mellegard, 1979)					
Dome rock salt	3640	35.4°	940	0-3000	7
Meadowbank Mine, Cheshire, Great Britain (Sen, 1962)					
Rock salt	4640	40.9°	1060	0-1000	3

APPENDIX G. TRIAXIAL PROPERTIES OF SELECTED EVAPORITES (Continued)

Source, Cycle etc. and testing lab or reference	Unconfined Compression Strength (psi)	Angle of Internal Friction (degs)	Cohesion (psi)	Confining Pressure Range (psi)	Number Samples Tested
Carey Salt Co., Hutchinson, KS (CSM Lab, 1985)					
Rock salt	2900	49.1°	540	0-1000	24
TG Chemicals, Inc., Granger, WY (CSM Lab, 1985)					
Bed # 21, Trona	3900	43.9°	830	0-2000	49
General Chemical Corp., Green River, WY (CSM Lab, 1987)					
Bed # 17, Trona	5860	42.8°	1280	0-3000	72
Independent Salt Co., Kanapolis, KS (CSM Lab, 1987)					
Roof rock salt	4330	31.2°	1220	0-3000	16
Morton Salt Co., Fairport Harbor Mine, Painesville, OH (CSM Lab, 1987)					
Rock salt	4020	39.0°	960	0-4000	30
Gold Bond Building Products Co., Shoals Mine, IN, (CSM Lab 1988-89)					
Gypsum	2880	34.0°	765	0-1300	22
Averages	4390	36.2°	1100		
Standard Deviations	1320	6.3°	350		

APPENDIX C

Waste Isolation Pilot Plant, Review of Proposed Panel Closure Enhancements
AMEC Earth and Environmental Limited

WASTE ISOLATION PILOT PLANT
REVIEW OF PROPOSED PANEL
CLOSURE ENHANCEMENTS

Prepared for:

Mining and Environmental Services LLC
Idaho Springs, Colorado

Prepared by:

AMEC Earth & Environmental Limited
Burnaby, BC

10 July 2001

VA06080

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EXECUTIVE SUMMARY

The panel closure system for the Waste Isolation Pilot Plant (WIPP) Project near Carlsbad, New Mexico, is comprised in part of a mass concrete plug, together with installation of a grout to fill any voids between the concrete plug and host halite rock. The specification for the project provided a prescription based formulation for the *Salado Mass Concrete* for the closure panel. The specification also required the use of a crushed quart aggregate. With respect to the grout, the specification required the use of a fresh-water based grout.

AMEC has been asked to review and comment on three proposed *Panel Enclosure Enhancements:*

(a) A change to the specifications to allow the use of a well-rounded carbonate-based coarse aggregate in lieu of the originally specified crushed quartz aggregate in the mass concrete.
(b) A change in the grout used from a fresh-water grout to a salt-based grout.
(c) A change for the mass concrete requirements from a prescription-based specification (which gives a recipe for the *Salado Mass Concrete*) to a performance-based specification which requires the Contractor to formulate the mass concrete mixture design and meet a prescribed set of performance specifications.

In brief, AMEC concurs in principle with the above-proposed *Panel Closure Enhancements* as detailed in the report, which follows. AMEC has, however, provided a number of recommendations as to issues which should be addressed by the specifiers with respect to the proposed changes. In particular, with respect to the proposed change to a *well-rounded carbonate-based rock*, it is recommended that it be demonstrated that:

- The carbonate-based rock is physically and chemically suitable for it's intended purpose, i.e. it should be shown to not be susceptible to deleterious alkali-carbonate reactivity, or salt-induced chemical degradation, and should display satisfactory thermal properties.
- The coarse aggregate should have suitable bonding characteristics to the past fraction (the use of particles with a partial *crush-count* should be considered).

With respect to the proposed use of a salt-based grout, AMEC concurs with this proposed conge, since it will counteract the tendency for dissolution (and hence void formation) of fresh-water based grouts. Also, review of the literature indicates enhanced long-term chemical stability of salt-saturated grouts placed in salt formations, compared to fresh-water grouts.

Finally, with respect to the proposal to change from a prescriptive *Saslado Mass Concrete* specification to a *Generic Salt-Based Concrete* specification for the closure panel concrete, AMEC concurs with this recommended change, since it gives the Contractor more freedom to adjust the mass concrete mixture properties to optimize the concrete construction process. Also, it places responsibility for performance of the concrete with the Contractor. This is contractually preferable for the Owner. The project specification should, however, then be written in rigorous performance-based specification language. In addition, more specifics should be provided in the specification regarding permissible constituents materials for the mass concrete for components such as the salt (type and saturation level required) and shrinkage compensating materials.

1.0 INTRODUCTION

AMEC Earth & Environmental Limited (AMEC), was retained by Mining and Environmental Services LLC (MES) to review certain proposed panel closure enhancements for the Waste Isolation Pilot Plant (WIPP) near Carlsbad, New Mexico USA. Specifically AMEC was provided with a *Panel Closure Enhancement* (1) report by MES and asked to:

(a) Review the proposed change to the specifications to allow the use of well-rounded carbonate coarse aggregate in the panel closure concrete, in lieu of the crushed quartz aggregate specified in the *detailed design report for an operational phase panel closure system.*

(b) Review the proposed change to the specifications to allow the use of a salt-based grout, in lieu of the fresh water grout specified in the initial design report.

(c) Comment briefly on the proposed change to the specification to permit the use of a generic salt-based concrete in the panel closure concrete in lieu of the Salado Mass Concrete specified in the initial design report. (This was an addition to the initial terms of reference)

In addition to reviewing the report detailing the proposed *Panel Closure Enhancements* (1) provided by MES, AMEC conducted a literature search on the subject and reviewed relevant papers including Sandia National Laboratories Reports provided to AMEC by the Environmental Evaluation Group in Albuquerque, New Mexico. AMEC also communicated with a grouting specialist, Alex Naudts of ECO Grouting Specialists Ltd. in Ontario, Canada, with experience in the use of salt-based cement grouts in potash mines and other salt formations.

The report, which follows, provides a brief review of the preceding proposed panel closure enhancements. This report is written by Dr. D. R. Morgan, P.Eng, a civil engineer with particular experience in concrete technology, including concrete and grout mixture designs and construction monitoring and testing for civil and mining projects. While Dr. Morgan is not a chemist, he has considerable experience with respect to the hydration and durability of portland cement based systems, as affected by the addition of chemical admixtures, supplementary cementing materials and ingress of external aggressive agents. He also has experience in the design and construction of mass concrete structures in civil and mining applications. This report is written from this perspective.

2.0 LITERATURE REVIEW

AMEC conducted a literature search using key words: brine + salt + concrete + grout + potash + Salado in various combinations. Cited references are listed in Appendix A. A general list of references is provided in Appendix B. Readers wanting more details regarding any of the technical issues discussed in this report are invited to examine the cited references and if necessary general references. This reference list should not be considered as all-inclusive. Many more references are contained in the cited and listed references. It does, however provide a listing of key publications on the subjects of interest.

With respect to the proposed use of salt-based grout and generic salt-based panel closure concrete, it should be recognized that the performance of such products is likely to be very *salt-specific*. Thus, while there is value in examining the results of test on salts and performance of concretes and grouts from locations other than the Waste Isolation Pilot Plant (WIPP), most emphasis should be placed on WIPP tests and observations. From review of the literature, it is apparent that the majority of the testing and evaluation for the WIPP project has been conducted by the Sandia National Laboratories in Albuquerque, New Mexico. Most emphasis should thus be placed on the findings of these reports.

3.0 CHANGE IN GENERAL AGGREGATE SPECIFICATIONS

3.1 General

A proposed enhancement to the panel closure is to expand the specification for the coarse aggregate to be used in the mass concrete to allow for the use of well-rounded natural carbonate materials in place of the specified crushed quartz aggregate. The main reason for this proposed change is to improve the workability of the concrete in general, and pumpability in particular. A second reason for the proposed change is the fact that crushed quartz is not available within a 50-mile radius of the WIPP site, whereas natural rounded carbonate aggregates are locally available.

It is reported that the carbonate aggregate mineralogy is such that it should not adversely affect properties. This should be verified by:

(a) Examination of a petrographic analysis of the aggregate (ASTM C295) (2)
(b) Examination of conformance of the aggregate to the ASTM C33-99a *Standard Specification for Concrete Aggregates* (3)

In particular, the aggregate should be demonstrated to conform to the requirements in ASTM C33-99a, Tables 2 and 3 for:

- Gradation, including percent passing the 75 μm (No. 200) sieve;
- Deleterious particles (e.g. clay clumps, friable particles, chert, coal, etc).
- Magnesium sulphate soundness loss.

3.2 Alkali Aggregate Reactivity

Evaluation of the alkali aggregate reactivity (AAR) susceptibility of the aggregate should be carried out. More specifically it is recommended that records be produced demonstrating that the carbonate aggregate is non-reactive when evaluated against the:

- ASTM C 586-99 *Standard Test Method for Potential Reactivity of Carbonate Rocks as Concrete Aggregates (Rock-Cylinder Method)* (4) ;
- ASTM C1105-95 *Standard Test Method for Length Change of Concrete Due to Alkali-Carbonate Rock Reaction* (5) (12 month prism test).

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Additional guidance with respect to evaluation and testing of carbonate-based aggregates for AAR can be found in CSA A 23.1-00, *Concrete Materials and Methods of Concrete Construction, Appendix B Alkali Aggregate Reaction* (6) and CSA A 23.2-27A *Standard Practice to Identify Degree of Alkali-Reactivity of Aggregates and Measures to Avoid Deleterious Expansion in Concrete* (7).

3.3 Chemical Stability in Brine

It is reported by Nowak et al (8) and Wakeley et al (9) that a certain dolomitic aggregate near Carlsbad, NM, has shown vulnerability to chemical alteration by reaction with brines in concrete. (Note: Dolomite, $MgO.CO_3.CaO.CO_3$.). It should be demonstrated that the carbonate aggregate selected for use is not susceptible to such deterioration.

3.4 Thermal Considerations

In addition to the above-recommended tests to demonstrate the suitability of the carbonate-based aggregate for it's intended use in the mass concrete closure panels, consideration should be given to the differences in thermal properties of the carbonate-based aggregates compared to the previously approved quartz-based aggregates. There are three thermal properties of aggregate that may be significant in the performance of the mass concrete: coefficient of thermal expansion, specific heat and conductivity. It is recommended that these thermal properties of the carbonate-based aggregate be determined and compared against those of the original crushed quartz aggregate specified. The design engineers for the closure panels should then evaluate the significance of any differences between the thermal properties of the two aggregate types on the expected behaviour of the mass concrete closure panels.

It is not proposed in this brief review report to elaborate on the test methods for the determination of thermal properties of the aggregates (or concrete) and the significance of these thermal properties with respect to the short and long term performance of the mass concrete closure panels. Good guidance in this regard can however, be found in publications such as: Cook, *Thermal Properties* (10,11) and Neville, *Properties of Concrete* (12).

A few thermal considerations are nevertheless worth pointing out:

- The coefficient of thermal expansion of the aggregate influences the coefficient of thermal expansion of the concrete containing such aggregate.
- It is desirable to have a coefficient of thermal expansion in the coarse aggregate which does not differ too much from the coefficient of thermal expansion of the hydrated portland cement paste in the concrete. Serious differences in the coefficients of thermal expansion have been reported to occur with aggregates with very low expansion, such as certain granites, limestones, and marbles (12). In such concretes, a large change in temperature (e.g. such as induced by the heat of hydration of the concrete) may introduce differential movement between the aggregate particles and paste, sufficient to break bond. This could result in microcracking, sufficient to impact on the durability of the concrete.

Thus for the carbonate aggregates proposed for use in the mass concrete closure panels at WIPP, it is recommended that the coefficient of thermal expansion of the aggregate be determined, to verify that it is suitable for use in it's intended application.

3.5 Workability/Pumpability

The prime reason for the proposed change from crushed quartz aggregate to a well-rounded carbonate-based rock is to enable production of a concrete with enhanced *workability* (mixing, pumping, placing, and consolidation characteristics) for the mass concrete closure panels. While quarried crushed rock is used to produce concrete with acceptable *workability* in several parts of North America (particularly the Eastern USA and Eastern Canada), natural rounded fluvial or glaciofluvial gravels typically produce concretes with superior *workability*, which are easier to mix, pump, place, consolidate and finish.

It should, however, be noted that most gravels used in concrete production are usually partially crushed, i.e. have a certain *crush-count*. There are certain advantages to having partially fractured faces in a sufficient percentage of the aggregate particles, including enhanced compressive, flexural and tensile strength development in the concrete made with such particles, compared to concrete made with natural rounded particles only. This observation, however, only applies if the particle crushing process produces aggregate particles with suitable shape i.e. particles that are more *equant* (cubical to round), as opposed to excessive quantities of particles that are flat, platy or elongated.

With respect to the proposed change to the use of natural rounded aggregates it should be cautioned that aggregate particles with very dense, smooth (polished) surfaces will typically have lower bond strengths than aggregates with rougher surface texture. Partial crushing of natural rounded aggregates is beneficial in that fracture faces on the aggregate particles typically have greater surface roughness. This enhances bond strength to the paste and consequently improves compressive, flexural and tensile strength development in the concrete.

To summarize, the proposed enhancement to use natural well-rounded aggregates to improve *workability* (including pumpability) of the closure panel concrete is considered appropriate. Consideration should, however, be given to producing an aggregate with a partial *crush-count*, to enhance paste to aggregate bond and consequent physical concrete properties.

4.0 SALT-BASED GROUT

It is understood that the *Environmental Protection Agency (EPA) Certified Closure* for the panel closure design approved the use of a *fresh water grout*. The proposed enhancement is to use a salt saturated grout instead. The purpose of the grout is to fill any voids that develop at the *back* (overlying roof) between the panel closure concrete, and the host halite formation as a result of sedimentation and bleeding, etc. in the mass concrete.

The concern with respect to using a *fresh water grout* is twofold:

(a) The liquid grout has the potential to dissolve the host halite material during placement, which is counterproductive to it's intended void sealing function;

(b) There is potential for a reduction in the long term stability and durability for the hardened grout because of the *chemical gradient* developed between the fresh water grout and the host halite formation; i.e. magnesium and other ions moving from the halite formation into the grout can result in calcium dissolution and depletion, which in turn can cause a loss of strength and degradation of the grout. Details regarding this dissolution mechanism can be found in publications by Wakeley and Burkes (13), Tumidajski and Chan (14), Wakeley et al, (15), Lambert et al (16) and Pool et al, (17).

Thus the proposed enhancement to use a salt-based grout is intended to negate the above concerns with using a *fresh water grout*. The use of a suitable salt solution (typically sodium chloride) as a mixing fluid in the grout minimizes the potential for the fresh grout to dissolve the host halite formation during placement and reduces the chemical gradient between the host halite formation, hardened grout, and proposed salt-based concrete (13, 14, 15, 16,17). In addition, Wakeley et al (15) report that salt-free grouts, when placed in contact with halite form no bond with the host rock. By contrast, a salt-saturated cementitious material bonds well with the halite.

There is precedence for the use of salt-saturated mixing water in cement-based grouts for void sealing of salt formations. Details can be found in publications such as Eyermann et al (18), Al-Manaseer et al (13) and others.

In addition to a review of the literature on this subject, Dr. Morgan of AMEC spoke with Alex Naudts at ECO Grouting Specialists Ltd., in Ontario, Canada, who has considerable experience in grouting in potash and salt mines. At AMEC's request, he provided a brief *capability statement* (20) of his experience in this regard. A copy of this statement is attached in Appendix B.

Notable examples of the use of salt-saturated grouts for void sealing purposes in salt formations include:

- Rocanville, Potash Mine, Saskatchewan, 1985 (19, 20)
- Esterhazy (K2) Mine, Saskatchewan, 1986 (19,20)
  (The writer was involved in the design and construction of one of the four bulkheads constructed on this project).
- Kali & Salz Potash Mine, Kassel, Germany (20)
- Potocan Potash Mine, New Brunswick (20)

Naudts (20) drew the conclusion that: *Most parties agree that it is not appropriate to use fresh water in grouts in contact with salt or potash ore (halite, carnalite, etc.) because of the migration of sodium, calcium or magnesium ions into the gelling grout, leaving a porous matrix near and at the contact zone.*

Review of the literature indicates that great care should be exercised in formulating salt-based portland cement grouts. Sodium chloride typically acts as a set retarder and water reducer. Chemical admixtures which work well in conventional portland cement-based grouts (without

salt addition) may however, not be compatible with salt-based portland cement grouts. For example, certain water reducers and retarders are reported to not be suitable for use in salt-based grouts because of problems such as very high air contents, foaming, excessively rapid rate of slump loss and excess set retardation (9, 20, 21, 22). Also, bentonite is reported to not be compatible with salt-based grouts (20).

By contrast, supplementary cementing materials, such as fly ash (type F or type C) are reported to be beneficial with respect to improving both the plastic and hardened properties of the grout. The addition of fly ash results in a stable suspension, with reduced bleeding and hence provides a more homogenous grout. Similarly, silica fume is reported to have been beneficially used for such purposes (20, 22). In addition, it is reported that hardened grouts with fly ash addition displayed enhanced bond to halite (13) and superior *long term* performance, compared to grouts with no fly ash, with respect to parameters such as compressive and flexural strength and modulus of elasticity, when the grouts were submerged in containers with brine at various confining pressures (21).

An argument can thus be made that the salt-based grouts should be formulated with a supplementary cementing material, such as fly ash (and / or silica fume) for the WIPP project.

An example of a formulation for a salt-based grout with fly ash addition, is the Grout BCT-1F described below, which is given in the *WIPP Project Specification Section 02722 Section 2.1* (23).

Table 1: Salt-Saturated Grout (BCT-1F)

Component	Percent of total Mass (wt.)
Class H cement	48.3
Class C fly ash	16.2
Cal Seal (plaster – from Halliburton)	5.7
Sodium Chloride	7.9
Dispersant	0.78
Defoamer	0.02
Water	21.1

The specification states: *The following formulation is suggested to the contractor as an initiation point for selection of the grout mix.* The specification does, however, not provide any performance requirements for the grout. As such, the specification is likely to be interpreted as a *prescription specification* by the contractor. This creates potential contractual conflicts if the contractor simply adopts the BCT-1F formulation and the grout does not perform as intended. It is thus recommended that WIPP develop a set of *performance specifications* for the grout, and require the contractor to demonstrate conformance of the grout to these *performance specifications.*

Parameters of interest with respect to *performance specifications* for the grout could include:

- Chloride saturation, including a statement on what type of salt is permissible. Note: While halite is comprised mainly of sodium chloride, it can contain lesser amounts of salts such as calcium sulphate, calcium chloride and magnesium chloride (28);
- Water /cementing materials ratio (likely variable, depending on size of voids to be filled and distance grout has to travel);
- Viscosity, as measured by the Marsh Flow Cone Test e.g. API RP 13B-1;
- Early age volume change e.g. ASTM C 287-95a *Standard Test Method for Change in Height at Early Ages of Cylindrical Specimens for Cementitious Mixtures* (24).
- Volume change at later ages (1, 14, 28 days) as measured by CRD C 621 (25).
- Bleeding and Expansion e.g. ASTM C 940-98a, *Standard Test Method for Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate Concrete in the Laboratory* (26).
- Setting time e.g. ASTM C953 *Standard Test Method for Time of Setting of Grouts for Preplaced-Aggregate Concrete in the Laboratory* (27).
- Specific Gravity e.g. API RP 13B-1.

It is recommended that appropriate testing methodologies be selected and a set of performance requirements be established for the salt-based grout(s). This approach would be consistent with the proposed *Panel Closure Enhancement* (1) for the *Generic Salt-Based Concrete*, which represents a change from a prescription to a performance-based specification.

Finally the Panel Closure Enhancement (1) Section 3.6 statement that: *No strength specifications for grout is appropriate, since any grout injected would serve in a lithostatic stress state and not compromise a structural element of the barrier*, is noted. This reviewer concurs with this statement and so setting compressive strength performance requirements for the grout would not appear to be warranted.

5.0 SALT-BASED CONCRETE

AMEC has been asked to comment briefly on the proposed *Panel Closure Enhancement* (1) to permit the use of a generic salt-based concrete in the panel closure concrete in lieu of the Salado Mass Concrete specified in the initial design report. This was an additional item to the original terms of reference, and while we have conducted a literature search on the subject (and done pertinent background reading on the subject) this review is brief, because of time and budget constraints.

In principle, AMEC concurs with this proposed change to the specifications. It is recognized that considerable research has been conducted by the Sandia National Laboratories (8, 9, 15, 16, 17, 18) and others in developing the Salado Mass Concrete for the WIPP Project.

The Panel Closure Cast-in-Place Concrete Specification, Section 03300, provides a list of so-called *Target Properties of the Concrete Mix*, in clause 2.5. For convenience of referral, this clause is produced below.

2.5 Target Properties of the Concrete Mix

The Contractor shall develop and proportion a salt-saturated mix for use in constructing the concrete barrier. The Contractor shall demonstrate by trial mix that the proposed concrete meets the following properties:

Table 2: Target properties for Barrier Concrete

Property	Comment
4-hr working time	Indicated by 8-inch slump (ASTM C 142) after 3-hr intermittent mixing, or an appropriate measure of pumpability.
Less that 25°F heat rise prior to placement	Difference between initial condition and temperature after 4 hr.
4,000 psi compressive strength (f'$_c$)	At 56 days after casting (ASTM C 39)
Volume stability	Length change between +0.05 percent and-0.02 percent (ASTM C 490)
Minimal entrained air	2 percent to 3 percent air

The Contractor shall provide certified copies of test data from an approved laboratory demonstrating compliance with the above target properties.

In addition to the target properties the Contractor shall provide certified test data for the trial mix for the following properties:

- *Heat of hydration*   ASTM C-186
- *Concrete set*   ASTM C-403
- *Thermal Diffusivity*   USACE CRD-C36
- *Water Permeability*   USACE CRD-C43

The specifications then provide what amounts to a *prescription* formulation for the Salado Mass Concrete, as detailed below.

An example of initial proportioning for the concrete is the salt-saturated concrete shown below:

Table 3 : Salt-Based Concrete Mixture Proportions

Component	Percent of Total Mass
Class H cement (API 10)	*4.93*
Chem Comp III (ASTM C-845 Type K)	*2.85*
Class F fly ash (ASTM C-618)	*6.82*
Fine aggregate	*33.58*
Coarse aggregate	*43.02*
Sodium chloride	*2.18*
Defoaming agent	*0.15*
Sodium citrate	*.009*
Water	*6.38*

The specification then goes on to say: The Contractor shall prepare a trial mix and provide certified test data from an approved testing laboratory for slump, compressive strength, heat rise, heat of hydration, concrete set time, thermal diffusivity, and water permeability.

Despite the wording in the specifications, the above approach is tantamount to a *Prescription Specification*, i.e. the Contactor is being told what mass concrete mixture proportions to use. There are potential contractual concerns with this approach, since if the concrete supplied failed to meet the required performance parameters, the Contractor could argue, with some justification that it is not his responsibility, but that if the specifying authority.

In AMEC's view, it is preferable to write the specification as a *Performance-Based Specification*, with the responsibility for the concrete mixture proportioning residing with the Contractor. The specifier should write a rigorous performance-based specification, including provision of specifics regarding all constituent materials permitted to be used in the *generic salt-based concrete*. In particular, specifics should be provided regarding the salt that is permitted to be added to the concrete, and which shrinkage compensative materials will be permitted to be added.

The Specifier should make available to the Contractor pertinent material regarding *Salado Mass Concrete* developed for the WIPP Project. This material can then provide the Contractor with a starting point for his *generic salt-based mixture proportioning*. In this way, the Contractor can adjust the mixture design, if required during construction, to facilitate the construction process. The responsibility for performance of the concrete would however, reside with the Contractor.

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6.0 Limitations and Closure

This report has been prepared for the exclusive use of Mining and Environmental Services LLC for the purpose described in the report. Any use which a third party makes of this report, or any reliance on or decisions made based on it, are the responsibility of such third parties. AMEC accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report. It has been prepared in accordance with generally accepted materials engineering practices. No other warranty, expressed or implied, is made.

AMEC thanks you for this opportunity to have been of service. We trust that this report satisfies your current requirements. Should you have any questions, please contact this office.

Yours Truly,

AMEC Earth & Environmental   Reviewed by:

Original signed by   *Original signed by*

D. R. Morgan, Ph.D., P. Eng   John Laxdal, P. Eng
Chief Materials Engineer   Regional Manager

APPENDIX A :

CITED REFERENCES

CITED REFERENCES

(1) *Panel Closure Enhancements,* Report provided to AMEC by Mining and Environmental Services LLC

(2) ASTM C 295-98, *Guide for Petrographic Examination of Aggregates for Concrete.*

(3) ASTM C33-99a, *Standard Specification for Concrete Aggregates.*

(4) ASTM C586-99, *Standard Test Method for Potential Reactivity of Carbonate Based Rocks as Concrete Aggregates (Rock-Cylinder Method).*

(5) ASTM C1105-95, *Standard Test Method for Length Change of Concrete due to Alkali – Carbonate Rock Reaction.*

(6) CSA A23.1-00, *Concrete Materials and Methods of Concrete Construction, Appendix B, Alkali Aggregate Reaction,* Canadian Standards Association, 2000.

(7) CSA A23.2-27A, *Standard Practice to Identify Degree of Alkali Aggregate Reactivity of Aggregates and Measures to Avoid Deleterious Expansion in Concrete,* Canadian Standards Association, 2000.

(8) Nowak, E.J., Wakeley, L.S., Poole, T.S., and Burkes, J.P., *Investigations of Deteriorated Concrete from the Liner of the Waste Isolation Pilot Shaft,* SAND 91-2733.UC-721.

(9) Wakeley, L.D., Ernzen, J.J., Neeley, B.D. and Hansen, F.D., *Salado Mass Concrete: Mixture Development and Preliminarily Characterization,* SAND93-7066.UC-721.

(10) Cook, H.K., *Thermal Properties,* ASTM STP 169-A, Significance of Tests and Properties of Concrete-Making Materials, 1966, pp. 476-486.

(11) Cook, H.K., *Thermal Properties* ASTM STP 169-B, Significances of Test and Properties of Concrete and Concrete-Making Materials, 1978, pp. 695-707.

(12) Neville, A.M., *Properties of Concrete,* 4th Edition, 1996, J. Wiyley & Sons, pp. 148-149.

(13) Wakeley, L.D. and Burkes, J.P., *Distribution of Chloride in a Salt-Saturated Grout in Contact with Halite Rock,* Cement and Concrete Research, 1986, Vol. 16, pp. 267-274.

(14) Tumidajski, P.J., and Chan, G.W., *Durability of High Performance Concrete in Magnesium Brine,* Cement and Concrete Research, 1996, Vol. 26, No. 4, pp. 557-565.

(15) Wakeley, L.D., Harrington, P.T., and Weiss, C.A., *Properties of Salt-Saturated Concrete and Grout After Six Years In Situ at the Waste Isolation Plant,* SAND93-7019.

(16) Lambert, S.J., Novak, E.J., Wakeley, L. and Poole, T.S., *Interactions Between concrete and Brine at the Waste Isolation Plant (WIPP) Site, New Mexico,* SAND91-1421•UC-721.

(17) Poole, T.S., Wakeley L.D. and Young, C.L., *Individual and Combined Effects of Chloride, Sulfate and Magnesium Ions on Hydrated Portland Cement Paste*, SAND93-7040.

(18) Eyermann, T.J., Sambeek, L.L., and Hansen, F.D, *Case Studies of Sealing Methods and Materials Used in the Salt and Potash Mining Industries*, SAND95-1120•UC-721.

(19) Al-Manaseer, A.A., Haug, M.D. and Wong, L.C., *Microstructure of Cement-Based Grouts*, American Concrete Institute, SP-132, 1992, pp. 635-654.

(20) Naudts, A., *Personal Communication Regarding the use of Salt-Based Grouts prepared for Dr. Morgan at AMEC Earth & Environmental*, July, 2001.

(21) Al-Manaseer, A.A. and Haug, M.D., *Long Term Performance of Grout Containing Fly Ash and Brine*, American Concrete Institute, STP-153, pp. 109-123.

(22) Naudts, A., *Curtain Grouting from 900 Metres Deep Donut Shaped Tunnel to Dry Up Leaking Shaft at IMC's K2 Mine*, ATTI Proceedings, Soil and Rock Improvements in Underground Works, Milano, March, 1991.

(23) Specification for Panel closure Construction at WIPP. (Cover sheet not provided; thus date of preparation of document not known)

(24) ASTM C827-95a *Standard Test Method for Change in Height at Early Ages of Cylindrical Specimens from Cementitious Mixtures.*

(25) CRD C 621, *Corps of Engineers Specifications for Non Shrink Grout*, US Army Corps of Engineers.

(26) ASTM C940-98a, *Standard Test Method for Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate in the Laboratory.*

(27) ASTM C953-87 (1997), *Test Method for Time of Setting of Grouts for Preplace-Aggregate Concrete in the Laboratory.*

(28) Read, H.H., *Rutley's Elements of Mineralogy*, 24th Edition, 1947, Thomas Murby & Co., pp. 216-218.

APPENDIX B :

GENERAL REFERENCES

GENERAL REFERENCES

(1) Pavement and Structures Monitoring, Pavement Instrumentation, and Drainage Systems Evaluation

(2) Transportation Research Record 1596, 1997.

(3) Resistance to Chemical Attack, Tuthill, L. 1978 Significance of Tests and Properties of Concrete and Concrete-Making Materials. ASTM STP 169B. p. 369-387.

(4) Mechanism of Sseawater Attack on Cement Paste, 1970. Kalousek, G. and Benton, E.J. in ACI Journal, Proceedings Vol 67, Feb. 1970.

(5) Effects of Seawater on Concrete, 1966. Mather, B. Highway Research Record No. 113, Highway research Board, Washington, D.C.

(6) Physico-Chemical Studies of Cement Pastes, Mortars, and Concretes exposed to Sea-Water, 1980. Regourd, M. Performance of Concrete in a Marine Environment, SP-65, american Concrete Institute, detroit, MI.

(7) Mechanism of Cement Paste Degration Due to Chemical and Physical factors, Proceedings, 8th In'tl symposium on Chemistry of Concrete, Rio de Janeiro, Vol. 1, 1986.

(8) Concrete and its Chemical Behavior, 1987. Eglington, M.S. London: Thomas Telford.

(9) Salt-Saturated Concrete Strength and Permeability,T. W. Pfeifle, Member, ASCE, F. D. Hansen and M. K. Knowles, pp. 173-182 in Materials for the New Millennium (Proceedings of the Fourth Materials Engineering Conference held in Washington, D.C., November 10-14, 1996) Ken P. Chong, (editor) New York: ASCE

(10) Mechanism of Sea-Water Attack on Cement Mortar, 1980. Conjeaud, M.L.SP-65, Performance of Concrete in a Marine Environment, ACI, Detroit, MI.

(11) Durability of Concrete in Seawater, 1975. Mehta, P.K. and Haynes, H.H. ASCE Journal of the Structural Division, Aug., 1975.

(12) Durability of Concrete to the marine Environment, 1991. Beslac, J. SP-126, Durability of Concrete, ACI, Detroit, MI.

(13) Action of Chloride Ions Hydrated cement Pastes: Influences of the Cement type and Long Time Effect of the Concentration of Chlorides, 1992. Geogout, P., Revertegat, E., and Moine, G. Cement and Concrete Research, Vol. 22.

(14) A Study of the Effect of Some Ions Contained In Seawater on Hydrated Cement Compounds, 1987. Ftikos, C. and Parissakes, G. Concrete Durability, SP-100, ACI Detroit, MI.

(15) Long-time Durability of Concrete in Seawater, 1971. Gjorv, O.E. Journal ACI, Vol. 68.

(16) Mixture Proportions and Thermomechanical Properties of Salado Mass Concrete. Sandia National Laboratory Report SAND93-7066, Sandia National Laboratory, NM.

(17) Variability in Properties of Salado Mass Concrete. Wakeley L. D., Harrington P. T., and Hansen F. D. 1995. SAND94-1495. Sandia National Laboratories, Albuquerque, NM. WPO 22744.

(18) The Waste Isolation Pilot Plant: A Potential Solution for the Disposal of Transuranic Waste (1996)

(19) Commission on Geosciences, Environment and Resources, National Academy Press.

(20) Durability of Concrete Materials in High Magnesium Brine, 1994. Wakely, L.D., Poole, T.S. and J.P. Burkes. SAND 93-7073. Sandia National Laboratories, Albuquerque, NM.

(21) Data Report on the Small-Scale Seal performance Tests, Series F Grouting Experiments, 1995. Ahrens, E.H. SAND 93-1000. ibid.

(22) The Advantages of a Salt/Bentonite Backfill for waste Isolation Pilot Plant Disposal Rooms, 1991. Butcher, B.M., Novak, C.F., Jercinovic, M. SAND 90-3074. Sandia National Laboratories, Albuquerque, NM.

(23) Criteria for the Certification and Recertification of the Waste Isolation Pilot Plant's Compliance with the Disposal Regulations, 1998. Federal Register, May 18, Vol. 63 #95 p. 27353-27406.

(24) Degradation of Portland Cements Exposed to Evaporite Brine at Hydrothermal temperatures, 1991. Krumhansl, J.L. and Lambert, S.J. in Advanced Cementitious Systems, proceedings of the Symposium, p. 123-128.

(25) Summary of WIPP Room B Heater Test: Brine and Backfill Material Data, 1991. Krumhansl, J.L., Stein, C.L., Jarrell, G.D. and Kimball, K.M., SAND90-06267.

(26) Sealing Concepts for the Waste Isolation Pilot Plant Site, 1982. Christensen, C.L., et al. SAND 81-2195. Sandia National Laboratories, Albuquerque, NM

(27) Interactions Between Concrete and Brine at the Waste Isolation Pilot Plant Site, 1992. Lambert, S.J., Nowak, E.J. et al. Mat. Res. Soc. Symp. Proc., Vol 245, p. 111-116.

(28) Laboratory-Scale Sodium-carbonate Concrete Aggregate Interactions, 1983. Westrich, H.R., Cygan, R.T. et al. Sandia National Laboratories Report.

(29) Conditions and Technical Issues Related to the EPA's Proposed Certification Decision on the waste Isolation Pilot Plant, 2001. Betsy Forinash and Scott Monroe. http://www.wmsym.org/wm98/htmq/sess21/21-03/21-03.htm.

(30) Salt-Structure Interaction of Underground Nuclear Waste Storage, 1983. Li, W.T., Antonas, N.J., and Wu, C.L. Waste management '83, ed. Roy Post, p. 393-398.

(31) Mecahnical Behavior of Concrete Seals in Rock Salt, 1988. Van sambeek, L.L. in Key Questions in Rock Mecahnics, p. 665-672.

(32) Long-term Leaching Experiments of Full-scale Cemented Waste Forms, 2000. Kienzler, Bernard, et al. Nuclear Technology, v.129, n.1 p. 101-118.

(33) Cover materials for Potash tailings Piles, 1991. Proc. of the 44th Canadian geotechnical Conference, Part 2. p. 66/1-66/9.

(34) Field Testing Results for the Strategic Petroleum reserve Pipeline Corrosion Control Program, 1998. Buchheit, R.G., Maestas, L.M., and Hinkebein, T.E. Sandia National Laboratories, Albuquerque, NM.

(35) Investigations of Deteriorated Concrete from the Liner of the Waste Isolation Pilot Plant waste Shaft, 1992. Nowak, E.J., Wakely, L.D., Poole, T.S., Burkes, J.P. SAND91-2733. Sandia National Laboratories, Albuquerque, NM.

(36) Waste Handling Shaft Concrete Liner Degradation Conclusions and Recommendations, 1992. Westinghouse Electric Corp., Carlsbad, NM. 213 p.

(37) WIPP Small-scale Seal Performance Tests, 1991. Finley, R.E. and Tillerson, J.R. SAND 91-2247C. Sandia National Laboratories, Albuquerque, NM.

(38) Coupled Mechanical/ Hydrological Model for WIPP Shaft Seals, 1991. Ehgartner, B. SAND91-1421C. Sandia National Laboratories, Albuquerque, NM.

(39) Studies of the Geochemical Stability of a Salt-Saturated Expansive Grout, 1987. Wakely, L.D. and Poole, T.S. Army Corps of Engineers Waterway Experiment Station. 80 p.

(40) Evaluation of Accessory Materials for Desalination Plants, 1973. Jones, B.V., Kuehn, V.L., and Morrison, W.R. Bureau of Reclamation, Denver, CO. 154 p.

(41) Research into the Mechanism of Brine Movement Through Clay Seams around Bulkheads in Saskatchewan Potash Mines, 1990. Haug, M.D. and Wong, L.C. U of Saskatchewan Transp. and Geot. group. 59 p.

(42) Microstructural Investigation of Deteriorated Portland Cement Concretes, 1996. Grattan-Bellew, P.E. Construction and Building Materials, Vol. 10, Issue 1, p. 3-16.

(43) Impact of Confining Pressure on Long-term Performance of Chemical Grout in salt Water, 1998. Haug, M.D., Al-Manaseer, A.A. and coode, A.M. J. of the Materials in Civil Engineering, Vol. 10, Issue 2. p. 70-75.

(44) Ten Years of Continous Monitoring in a Mining Panel, 1997. De Souza, E., Mottahed, P., Coode, A. and Sellers, B. Int'l J, of Rock Mechanics and Mining Sciences, Vol. 34, Issue 3-4. p. 622.

(45) Compressive Strength of Concrete Containing Fly-Ash, Brine, and Admixtures, 1989. ACI Materials J. Vol. 86, No. 1.

(46) Long-Term Brine Migration Through an Engineered Shaft Seal System, 1997. D. G. Fryar, J. A. Beach, V. A. Kelley, and M. K. Knowles. Computing in Civil Engineering, ASCE. p. 372-379

(47) Conclusions Regarding Geotechnical Acceptability of the WIPP Site, 1983. Weart, W.D. Waste Management '83, ed. Roy Post. p. 373-376.

(48) Potash resources at WIPP Site, New Mexico, 1999. New Mexico Bureau of Mines Circular. 92 p.

(49) The Waste Isolation Plant Experimental Program, 2001. Mark Matthews and Reisner, Anita. http://www.wmsym.org/wm98/htmq/sess09/09-14/09-14.htm.

(50) US Army Corps of Engineers Technical Manual # 5-818-6/AFM 88-32, Section 3, Grouts, 1988.

APPENDIX C :

PRIVATE COMMUNICATION FROM A. NAUDTS

ECO GROUTING SPECIALISTS LTD

ECO

293199 8th Line, RR 1, Grand Valley, ON, Canada L0N 1G0   Phone: (519) 928-5949 Fax: (519) 928-5968
e-mail: ecogrout@ecogrout.com   Home Page Internet Address: ecogrout.com

July 3, 2001

ATTN: Dr. Rusty Morgan
AMEC Earth and Environmental Ltd.
2227 Douglas Road
Burnaby, BC
V5C 5A9

amec
Earth & Environmental

JUL 05 2001

RECEIVED

Dear Dr. Morgan:

Re : Capability statement regarding cement based suspension grouts using saturated brines
as carrier for the particles, to be used in salty environments

This note is further to our conversation of this afternoon. It was good to hear from you after
all those years.

ECO has done extensive work developing and testing cement based suspension grouts to be
used in saline environments using brine as carrier for the particles.

Most parties agree that it is not appropriate to use fresh water in grouts in contact with salt
or potash ore (halite, carnalite, etc), because of migration of sodium, calcium or magnesium
ions into the gelling grout, leaving a porous salt matrix near and at the contact zone.

The following is an oversight of some of the work we did in potash and salt mines
(formulating and testing):

1. We first developed cement based suspension grouts for Kali & Salz in Germany for a
potash mine near Kassel during the late seventies. We established basic fluid and set
characteristics. The use of additives and admixtures such as styrene butadieen, clay-
phyllosilicates and first generation superplasticisers were novelties at the time. We used a
saturated locally found mine brine as the carrier.

We were aiming at low matrix permeability and high strength with the highest fluidity
possible. From our on-going relationship with this client, we have learned that the
performance of this type of grouts has been satisfactory over the life time. The tests
involved bleed-tests, viscosity tests (marsh) and specific gravity test (Mud balance) as well
as unconfined compressive strength tests and I believe we also did bond-strength tests from
grout to the potash ore.

2. During the eighties we performed lab testing for PCS during the Rocanville flooding. Our design for the plug was implemented. The plug is still standing up under a pressure of 1200 psi (since March 1985). We formulated both regular cement based grouts, microfine cement based grouts and solution grouts using the brine from the mine as carrier. We found that some of the water reducing agents were not compatible with the brine or some of the additives and admixtures. We predominantly used the following additives and admixtures to the brine: super plasticiser, type F-flyash, silica fume (first time we used this) and slag. We found out that bentonite was not compatible with brine. (we should have pre-hydrated the bentonite in fresh water and added this to the mix). We also made samples of grouts and bubbled hydrogen sulphide through the suspension and let the suspension grouts cure in this saturated environment. Three years ago a sealed jar with this smelly gas and cured grout broke when we moved to our present lab-location. The (slag based microfine cement) grout looked "perfect" after 13 years. I remember that some grouts never reached final set and only made it to "somewhere between initial and final set" (Vicat needle).

3. During the IMC potash inflow-crisis in 1986, ECO was asked by this organisation to perform a series of tests to develop "durable cement based grout formulations" for a variety of applications. The tests were actually conducted in the lab of BBT in Saskatoon.

We used similar admixtures as during the tests in Rocanville, but included a portland cement based microfine cement based grout. We also used type C fly-ash (not aware at the time of the critical maximum contents BWOC) and kiln dust. A series of formulations were developed and reports were made. Some of the formulations were used in various phases of the project.

4. ECO performed lab-testing for the grouting around shaft plugs in an abandoned salt mine in Dearborn (Michigan). The actual grouting was never performed because the permitting to use this abandoned mine as a landfill site was never approved.

5. During 1992 ECO performed extensive lab-testing for the installation of a pilot grout-curtain in an abandoned limestone mine in Acron (Ohio), which was to be deepened to be used for the power house for a power generating system (pump storage). The water bearing zones in the limestone were saturated with hydrogen sulphide and brine. A variety of grouts were developed with different rheology, viscosity, cohesion, resistance against pressure filtration, thixotropy, gel times, set times etc.

At the time we used much more sophisticated mixes, using admixtures and additives such as metha glycol cellullose to curtail run-away situations, F-fly-ash, de-airing agents, super plasticisers, slag, pumice and bentonite slurries. The tests were also more sophisticated and included resistance against pressure filtration tests, initial and final gelation and set tests, cohesion tests etc.

6. In 1997, for Potocan (and its German part-owner, Kali & Salz) ECO performed a substantial grout formulation and testing program for one of the biggest grouting ever performed. From over 200 formulations, six formulations were retained with distinctive fluid and set characteristics. The formulations contained in addition to most of the

aforementioned admixtures and additives also modern day bio-polymers and the third generation of super plasiticisers and for the first time we used iso-propyl alcohol to delay the curing of the grout. Brine from this New Brunswick potash mine was used as the carrier of the particles.
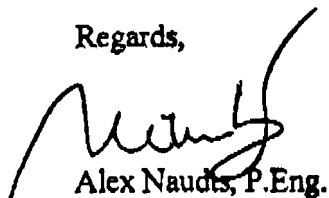
7. ECO is involved in an elaborate seal-grouting project for the Werra potash mine in the former East Germany for its owner Kali & Salz. This is a group effort involving an international panel of specialists, all with different expertise. We established grout formulations and test procedures both for regular and microfine cement based grouts. This project is ongoing.

8. ECO has done several grout test programs for manufacturers of microfine cement based grouts. Some of these formulations were made for use in saline environments. Brine saturated solutions were typically used for these tests.

After all those years of testing and evaluating performance of grouts, I must admit that there is still a lot to be learned about the application of suspension grouts in saline environments, especially if magnesium brines are present.

We would be very interested to participate in testing programs or review information by others. You can find more information about our firm on our site: www.ecogrout.com. We are completely independent and have no links or ties with manufacturers nor contractors nor distributors. Thank you for contacting us. We look forward to contribute to this project
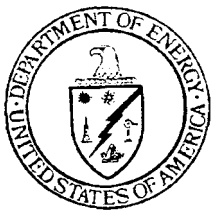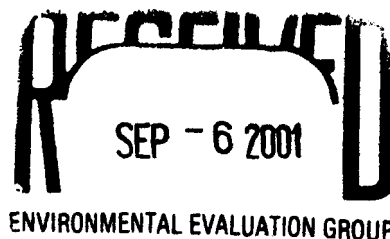
Regards,

Alex Naudts, P.Eng.

APPENDIX D

September 4, 2001 letter, Triay to Marcinowski

Department of Energy
Carlsbad Field Office
P. O. Box 3090
Carlsbad, New Mexico 88221

September 4, 2001

Mr. Frank Marcinowski
Office of Radiation and Indoor Air
U.S. Environmental Protection Agency
401 M. Street, S. W.
Washington, DC 20460

Dear Mr. Marcinowski:

On April 17, 2001 we submitted a request to you seeking approval of several proposed enhancements to the WIPP panel closure system. Based on communications with your staff during our recent working meetings, we respectfully withdraw our request for approval of those modifications. In view of the approval of our panel 1 utilization request and our current and projected throughput rates, we wish to re-evaluate our panel closure strategy.

At the conclusion of our evaluation, we expect to revisit this topic with the EPA. We appreciate the effort already expended by your staff in the review of our April submittal. If you or your staff have any questions regarding this matter, please contact Mr. Daryl Mercer at (505) 234-7452.

Sincerely,

Dr. Inés R. Triay
Manager

cc:
D. Huizenga, EM
S. Monroe, EPA
S. Ghose, EPA
C. Byrum, EPA
N. Stone, EPA
S. Zappe, NMED
M. Silva, EEG

CBFO:ORC:DMM:VW:01-1340:UFC:5400

APPENDIX E
LIST OF EEG REPORTS

LIST OF EEG REPORTS

EEG-1   Goad, Donna, <u>A Compilation of Site Selection Criteria Considerations and Concerns Appearing in the Literature on the Deep Disposal of Radioactive Wastes</u>, June 1979.

EEG-2   <u>Review Comments on Geological Characterization Report, Waste Isolation Pilot Plant (WIPP) Site, Southeastern New Mexico SAND 78-1596, Volume I and II</u>, December 1978.

EEG-3   Neill, Robert H., et al., (eds.) <u>Radiological Health Review of the Draft Environmental Impact Statement (DOE/EIS-0026-D) Waste Isolation Pilot Plant, U.S. Department of Energy</u>, August 1979.

EEG-4   Little, Marshall S., <u>Review Comments on the Report of the Steering Committee on Waste Acceptance Criteria for the Waste Isolation Pilot Plant</u>, February 1980.

EEG-5   Channell, James K., <u>Calculated Radiation Doses From Deposition of Material Released in Hypothetical Transportation Accidents Involving WIPP-Related Radioactive Wastes</u>, October 1980.

EEG-6   <u>Geotechnical Considerations for Radiological Hazard Assessment of WIPP. A Report of a Meeting Held on January 17-18, 1980</u>, April 1980.

EEG-7   Chaturvedi, Lokesh, <u>WIPP Site and Vicinity Geological Field Trip. A Report of a Field Trip to the Proposed Waste Isolation Pilot Plant Project in Southeastern New Mexico, June 16 to 18, 1980</u>, October 1980.

EEG-8   Wofsy, Carla, <u>The Significance of Certain Rustler Aquifer Parameters for Predicting Long-Term Radiation Doses from WIPP</u>, September 1980.

EEG-9   Spiegler, Peter, <u>An Approach to Calculating Upper Bounds on Maximum Individual Doses From the Use of Contaminated Well Water Following a WIPP Repository Breach</u>, September 1981.

EEG-10   <u>Radiological Health Review of the Final Environmental Impact Statement (DOE/EIS-0026) Waste Isolation Pilot Plant, U. S. Department of Energy</u>, January 1981.

EEG-11   Channell, James K., <u>Calculated Radiation Doses From Radionuclides Brought to the Surface if Future Drilling Intercepts the WIPP Repository and Pressurized Brine,</u> January 1982.

EEG-12   Little, Marshall S., <u>Potential Release Scenario and Radiological Consequence Evaluation of Mineral Resources at WIPP</u>, May 1982.

EEG-13   Spiegler, Peter, <u>Analysis of the Potential Formation of a Breccia Chimney Beneath the WIPP Repository</u>, May, 1982.

EEG-14   Not published.

EEG-15   Bard, Stephen T., <u>Estimated Radiation Doses Resulting if an Exploratory Borehole Penetrates a Pressurized Brine Reservoir Assumed to Exist Below the WIPP Repository Horizon - A Single Hole Scenario</u>, March 1982.

EEG-16   <u>Radionuclide Release, Transport and Consequence Modeling for WIPP. A Report of a Workshop Held on September 16-17, 1981</u>, February 1982.

EEG-17   Spiegler, Peter, <u>Hydrologic Analyses of Two Brine Encounters in the Vicinity of the Waste Isolation Pilot Plant (WIPP) Site</u>, December 1982.

EEG-18   Spiegler, Peter and Dave Updegraff, <u>Origin of the Brines Near WIPP from the Drill Holes ERDA-6 and WIPP-12 Based on Stable Isotope Concentration of Hydrogen and Oxygen</u>, March 1983.

EEG-19   Channell, James K., <u>Review Comments on Environmental Analysis Cost Reduction Proposals (WIPP/DOE-136) July 1982</u>, November 1982.

EEG-20   Baca, Thomas E., <u>An Evaluation of the Non-Radiological Environmental Problems Relating to the WIPP</u>, February 1983.

EEG-21   Faith, Stuart, et al., <u>The Geochemistry of Two Pressurized Brines From the Castile Formation in the Vicinity of the Waste Isolation Pilot Plant (WIPP) Site</u>, April 1983.

EEG-22   <u>EEG Review Comments on the Geotechnical Reports Provided by DOE to EEG Under the Stipulated Agreement Through March 1, 1983</u>, April 1983.

EEG-23   Neill, Robert H., et al., <u>Evaluation of the Suitability of the WIPP Site</u>, May 1983.

EEG-24   Neill, Robert H. and James K. Channell, <u>Potential Problems From Shipment of High-Curie Content Contact-Handled Transuranic (CH-TRU) Waste to WIPP</u>, August 1983.

EEG-25   Chaturvedi, Lokesh, <u>Occurrence of Gases in the Salado Formation</u>, March 1984.

EEG-26   Spiegler, Peter, <u>Proposed Preoperational Environmental Monitoring Program for WIPP</u>, November 1984.

EEG-27   Rehfeldt, Kenneth, <u>Sensitivity Analysis of Solute Transport in Fractures and Determination of Anisotropy Within the Culebra Dolomite</u>, September 1984.

EEG-28   Knowles, H. B., <u>Radiation Shielding in the Hot Cell Facility at the Waste Isolation Pilot Plant: A Review</u>, November 1984.

EEG-29   Little, Marshall S., <u>Evaluation of the Safety Analysis Report for the Waste Isolation Pilot Plant Project</u>, May 1985.

EEG-30   Dougherty, Frank, Tenera Corporation, <u>Evaluation of the Waste Isolation Pilot Plant Classification of Systems, Structures and Components</u>, July 1985.

EEG-31   Ramey, Dan, <u>Chemistry of the Rustler Fluids</u>, July 1985.

EEG-32   Chaturvedi, Lokesh and James K. Channell, <u>The Rustler Formation as a Transport Medium for Contaminated Groundwater</u>, December 1985.

EEG-33   Channell, James K., et al., <u>Adequacy of TRUPACT-I Design for Transporting Contact-Handled Transuranic Wastes to WIPP</u>, June 1986.

EEG-34   Chaturvedi, Lokesh, (edi.), <u>The Rustler Formation at the WIPP Site</u>, February 1987.

EEG-35   Chapman, Jenny B., <u>Stable Isotopes in Southeastern New Mexico Groundwater: Implications for Dating Recharge in the WIPP Area</u>, October 1986.

EEG-36   Lowenstein, Tim K., <u>Post Burial Alteration of the Permian Rustler Formation Evaporites, WIPP Site, New Mexico</u>, April 1987.

EEG-37   Rodgers, John C., <u>Exhaust Stack Monitoring Issues at the Waste Isolation Pilot Plant</u>, November 1987.

EEG-38   Rodgers, John C. and Jim W. Kenney, <u>A Critical Assessment of Continuous Air Monitoring Systems at the Waste Isolation Pilot Plant</u>, March 1988.

EEG-39   Chapman, Jenny B., <u>Chemical and Radiochemical Characteristics of Groundwater in the Culebra Dolomite, Southeastern New Mexico</u>, March 1988.

EEG-40   <u>Review of the Final Safety Analyses Report (Draft), DOE Waste Isolation Pilot Plant, December 1988</u>, May 1989.

EEG-41   <u>Review of the Draft Supplement Environmental Impact Statement, DOE Waste Isolation Pilot Plant</u>, July 1989.

EEG-42 Chaturvedi, Lokesh, <u>Evaluation of the DOE Plans for Radioactive Experiments and Operational Demonstration at WIPP</u>, September 1989.

EEG-43 Kenney, Jim W., et al., <u>Preoperational Radiation Surveillance of the WIPP Project by EEG 1985-1988</u>, January 1990.

EEG-44 Greenfield, Moses A., <u>Probabilities of a Catastrophic Waste Hoist Accident at the Waste Isolation Pilot Plant</u>, January 1990.

EEG-45 Silva, Matthew K., <u>Preliminary Investigation into the Explosion Potential of Volatile Organic Compounds in WIPP CH-TRU Waste</u>, June 1990.

EEG-46 Gallegos, Anthony F. and James K. Channell, <u>Risk Analysis of the Transport of Contact Handled Transuranic (CH-TRU) Wastes to WIPP Along Selected Highway Routes in New Mexico Using RADTRAN IV</u>, August 1990.

EEG-47 Kenney, Jim W. and Sally C. Ballard, <u>Preoperational Radiation Surveillance of the WIPP Project by EEG During 1989</u>, December 1990.

EEG-48 Silva, Matthew, <u>An Assessment of the Flammability and Explosion Potential of Transuranic Waste</u>, June 1991.

EEG-49 Kenney, Jim, <u>Preoperational Radiation Surveillance of the WIPP Project by EEG During 1990</u>, November 1991.

EEG-50 Silva, Matthew K. and James K. Channell, <u>Implications of Oil and Gas Leases at the WIPP on Compliance with EPA TRU Waste Disposal Standards</u>, June 1992.

EEG-51 Kenney, Jim W., <u>Preoperational Radiation Surveillance of the WIPP Project by EEG During 1991</u>, October 1992.

EEG-52 Bartlett, William T., <u>An Evaluation of Air Effluent and Workplace Radioactivity Monitoring at the Waste Isolation Pilot Plant</u>, February 1993.

EEG-53 Greenfield, Moses A. and Thomas J. Sargent, <u>A Probabilistic Analysis of a Catastrophic Transuranic Waste Hoist Accident at the WIPP</u>, June 1993.

EEG-54 Kenney, Jim W., <u>Preoperational Radiation Surveillance of the WIPP Project by EEG During 1992</u>, February 1994.

EEG-55   Silva, Matthew K., <u>Implications of the Presence of Petroleum Resources on the Integrity of the WIPP</u>, June 1994.

EEG-56   Silva, Matthew K. and Robert H. Neill, <u>Unresolved Issues for the Disposal of Remote-Handled Transuranic Waste in the Waste isolation Pilot Plant</u>, September 1994.

EEG-57   Lee, William W.-L, Lokesh Chaturvedi, Matthew K. Silva, Ruth Weiner, and Robert H. Neill, <u>An Appraisal of the 1992 Preliminary Performance Assessment for the Waste Isolation Pilot Plant</u>, September 1994.

EEG-58   Kenney, Jim W., Paula S. Downes, Donald H. Gray, Sally C. Ballard, <u>Radionuclide Baseline in Soil Near Project Gnome and the Waste Isolation Pilot Plant</u>, June 1995.

EEG-59   Greenfield, Moses A. and Thomas J. Sargent, <u>An Analysis of the Annual Probability of Failure of the Waste Hoist Brake System at the Waste Isolation Pilot Plant (WIPP)</u>, November 1995.

EEG-60   Bartlett, William T. and Ben A. Walker, <u>The Influence of Salt Aerosol on Alpha Radiation Detection by WIPP Continuous Air Monitors</u>, January 1996.

EEG-61   Neill, Robert, Lokesh Chaturvedi, William W.-L. Lee, Thomas M. Clemo, Matthew K. Silva, Jim W. Kenney, William T. Bartlett, and Ben A. Walker, <u>Review of the WIPP Draft Application to Show Compliance with EPA Transuranic Waste Disposal Standards</u>, March 1996.

EEG-62   Silva, Matthew K., <u>Fluid Injection for Salt Water Disposal and Enhanced Oil Recovery as a Potential Problem for the WIPP: Proceedings of a June 1995 Workshop and Analysis</u>, August 1996.

EEG-63   Maleki, Hamid and Lokesh Chaturvedi, <u>Stability Evaluation of the Panel 1 Rooms and the E140 Drift at WIPP</u>, August 1996.

EEG-64   Neill, Robert H., James K. Channell, Peter Spiegler, Lokesh Chaturvedi, <u>Review of the Draft Supplement to the WIPP Environmental Impact Statement, DOE/EIS-0026-S-2</u>, April 1997.

EEG-65   Greenfield, Moses A. and Thomas J. Sargent, <u>Probability of Failure of the Waste Hoist Brake System at the Waste Isolation Pilot Plant (WIPP)</u>, January 1998.

EEG-66   Channell, James K. and Robert H. Neill, <u>Individual Radiation Doses From Transuranic Waste Brought to the Surface by Human Intrusion at the WIPP</u>, February 1998.

EEG-67   Kenney, Jim W., Donald H. Gray, and Sally C. Ballard, <u>Preoperational Radiation Surveillance of the WIPP Project by EEG During 1993 Though 1995</u>, March 1998.

EEG-68   Neill, Robert H., Lokesh Chaturvedi, Dale F. Rucker, Matthew K. Silva, Ben A. Walker, James K. Channell, Thomas M. Clemo, <u>Evaluation of the WIPP Project's Compliance with the EPA Radiation Protection Standards for Disposal of Transuranic Waste</u>, March 1998.

EEG-69   Rucker, Dale, <u>Sensitivity Analysis of Performance Parameters Used In Modeling the Waste Isolation Pilot Plant</u>, April 1998.

EEG-70   Bartlett, William T. and Jim W. Kenney, <u>EEG Observations of the March 1998 WIPP Operational Readiness Review Audit</u>, April 1998.

EEG-71   Maleki, Hamid, <u>Mine Stability Evaluation of Panel 1 During Waste Emplacement Operations at WIPP</u>, July 1998.

EEG-72   Channell, James K. and Robert H. Neill, <u>A Comparison of the Risks From the Hazardous Waste and Radioactive Waste Portions of the WIPP Inventory</u>, July 1999.

EEG-73   Kenney, Jim W., Donald H. Gray, Sally C. Ballard, and Lokesh Chaturvedi, <u>Preoperational Radiation Surveillance of the WIPP Project by EEG from 1996 - 1998</u>, October 1999.

EEG-74   Greenfield, Moses A. and Thomas J. Sargent, <u>Probability of Failure of the TRUDOCK Crane System at the Waste Isolation Pilot Plant (WIPP)</u>, April 2000.

EEG-75   Channell, James K. and Ben A. Walker, <u>Evaluation of Risks and Waste Characterization Requirements for the Transuranic Waste Emplaced in WIPP During 1999</u>, May 2000.

EEG-76   Rucker, Dale F., <u>Air Dispersion Modeling at the Waste Isolation Pilot Plant</u>, August 2000.

EEG-77   Oversby, Virginia M., <u>Plutonium Chemistry Under Conditions Relevant for WIPP Performance Assess, Review of Experimental Results and Recommendations for Future Work</u>, September 2000.

EEG-78   Rucker, Dale F., <u>Probabilistic Safety Assessment of Operational Accidents at the Waste Isolation Pilot Plant</u>, September 2000.

EEG-79   Gray, Donald H., Jim W. Kenney, Sally C. Ballard, <u>Operational Radiation Surveillance of the WIPP Project by EEG During 1999</u>, September 2000.

EEG-80   Kenney, Jim W., <u>Recommendations to Address Air Sampling Issues at WIPP</u>, January 2001.

EEG-81   Gray, Donald H. and Sally C. Ballard, <u>EEG Operational Radiation Surveillance of the WIPP Project During 2000</u>, October 2001.

EEG-82   Allen, Lawrence E., Matthew K. Silva, James K. Channell, John F. Abel, and Dudley R. Morgan, <u>Evaluation of Proposed Panel Closure Modifications at WIPP</u>, December 2001.