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Interface Behavior of Water Saturated Limestone Rock Joints Using
Hollow Cylinder Testing
and
A Case History Regarding Mine Roof Stability:
Fort Hartford Mine Superfund Site

A Thesis
Presented for the
Master of Science
Degree
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ABSTRACT

Presented herein is a multi-part thesis prepared to partially meet the requirements for the Master of Science degree in Civil Engineering at the University of Tennessee. Part I provides a brief introduction to the two primary thesis topics that are presented in Parts II and III, respectively.

Part II presents findings from a series of tests performed with a hollow cylinder combined axial-torsional testing apparatus to study the effects of confining fluid pressure on the shear strength of artificial rock joints for Holston Limestone. Tests were performed for confining fluid pressures of 0.14 MPa to 0.55 MPa and effective joint normal stresses of 0.16 MPa to 0.65 MPa. Mohr-Coulomb failure criterion was used to interpret a joint effective friction angle for the Holston Limestone and the results were within the range of friction angle values given in published references. The combined effect of fluid pressure and mean stress on the joint interface shear strength was investigated for generalized stress conditions. It was found that an increase in intermediate principal stress resulted in measurable increases in joint interface shear strength, especially at lower normal stresses. Additionally, it was found that a simple linear relationship exists between the joint mean stress and the joint interface shear strength.

Part III is a case history regarding mine roof stability at the Fort Hartford Mine Superfund Site in Olaton, Kentucky. Specifically, mine roof instability at the Fort Hartford Mine Superfund Site has a number of potentially detrimental consequences including risks to mine personnel, subsidence damage, escape of hazardous gases from within mine, and contamination of the local groundwater system. Correspondingly, a

study was performed in 1993 to delineate areas in the mine with low, moderate, and high potential for mine roof deterioration. During the study, a mine roof stability model was developed using map overlaying techniques, whereby the combined impact of key parameters were evaluated. Mine roof stability has been monitored at the site for the past ten years using both mechanical instrumentation and visual inspection. Intensive roof and rib scaling was performed, and mitigative measures were implemented to repair unstable roof at several locations within the mine. Based on a decade of supporting data, the mine roof stability model has been recognized as a reliable tool for developing in-mine transportation plans and mitigative measures.

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PART I

Introduction

1. INTRODUCTION

Presented herein is a thesis prepared to partially meet the requirements for the Master of Science degree at the University of Tennessee. Specifically, the thesis has been prepared to present two separate primary topics. Part II presents findings from a series of tests performed with a hollow cylinder combined axial-torsional testing apparatus to study the effects of confining fluid pressure on the shear strength of artificial limestone rock joints. Part III is a case history regarding mine roof stability at the Fort Hartford Mine Superfund Site near Olaton, Kentucky.

PART II

Interface Behavior of Water Saturated Limestone Rock Joints Using Hollow Cylinder Testing

1. INTRODUCTION

The application of representative rock joint shear strength parameters is essential for the prudent design of many geotechnical and mining engineering structures.

Specifically, rock joint shear strength parameters are typically required in the design of dams, tunnels, underground mines and storage facilities, and rock slopes adjacent to many roadways and railways. It is critical during the design of these structures that the potential effects of rock joint fluid pressures and confining pressures be considered when developing the shear strength parameters. Laboratory methods such as a triaxial test and direct shear test are commonly used to estimate shear strength parameters for rock.

Triaxial testing is often used to develop shear strength parameters for intact rock whereas, the direct shear test is used for developing shear strength parameters for jointed rock media. Direct shear test is generally found to be adequate for estimating shear strength parameters for dry natural rock joints; however, it is limited in its ability to model the effects that fluid and confining pressures have on a rock joint. Further, it is difficult to evaluate three dimensional state of stress completely with the direct shear device. In order to overcome the inherent limitations with the conventional testing methods, a Hollow Cylinder Apparatus (HCA) has been developed specifically to perform shear strength testing of saturated artificial rock joints subjected to fluid pressures.

A series of tests have been performed with the HCA to investigate the interface shear strength for Holston Limestone using artificial rock joints. Two halves of a hollow cylinder specimen with prepared surfaces were brought into contact and subjected to a range of axial loadings, torque, and joint fluid pressures. It is to be noted that the joint fluid pressure corresponds to confining pressure around the interface, as well as to

intermediate principal stress during the application of shear stress. The results of the testing that are presented herein represent the effects of joint fluid pressure on the shear strength of saturated artificial limestone rock joints. Moreover, the testing demonstrates the advantages of using the HCA (i.e., compared to the direct shear device) for modeling the behavior of rock joints that are subjected to complex loadings conditions.

2. PRIOR RESEARCH USING HOLLOW CYLINDER DEVICES

A typical hollow cylinder device employs a compressive loading along the length, and a torsional loading about the axis of a thin-walled annular specimen. Equal confining pressure is generally applied during the test to the inner and outer cylinder walls. The favorable geometry exhibited in the hollow cylinder device permits the rotation of principal stresses during testing, as well as the variation of intermediate principal stresses (Lade, 1981; Hight et al., 1983; Saada, 1988). The method has historically been used for the testing of soils (Hvorslev, 1939; Bishop et al., 1971; Saada and Townsend, 1981); however, the inherent advantages of the hollow cylinder device have encouraged the development of similar devices for testing rock (Handin et al., 1967; Christensen et al., 1974; Santarelli and Brown, 1989; Lee et al., 1999; Lee et al., 2002). More specifically, hollow cylinder testing has been used to investigate the behavior of dry, unconfined rock joints (Kutter, 1974; Olsson, 1986; Xu and Freitas, 1988; Olsson, 1988). These studies have demonstrated that the primary advantages of rock joint testing with hollow cylinders (i.e., compared to the direct shear method) are: (1) the device's ability to uniformly distribute stresses along the rock joint during testing, and (2) the capability of the apparatus to monitor the complete state of stress within the joint throughout the test.

Previously, the HCA was used to test smoothened, unconfined, dry rock joints (Reardon et al., 1991). Results from the baseline HCA testing compared favorably with direct shear test results for similar materials. With the addition of the confining cell, the HCA is capable of applying intermediate principal stresses in the form of confining pressure, along with the ability to apply joint fluid pressures. Due to these advancements in the HCA, multiple stress paths can be investigated for a variety of geologic materials subjected to a range of joint fluid/confining pressures.

3. HCA STRESS STATE

Typical HCA setup involves bringing two halves of a hollow cylinder rock specimen into contact, thereby creating a rock joint oriented normal to the cylindrical axis. The specimen halves are subjected to a joint axial force, F_j , and a confining pressure, σ_c , which is applied to both the inner and outer walls of the cylinder. For the testing conditions presented herein, σ_c is applied in the form of a fluid pressure, u . The fluid pressure is also applied to the joint interface. Thereafter, the joint is subjected to a shear stress by applying a joint torsional loading, T_j , about the cylindrical axis. Figure 1^a shows a generalized schematic of the HCA loadings during a confined shear strength test of an artificial rock joint.

By examining an element from the lower rock specimen at the joint interface, an idealized state of stress can be developed for the rock joint as shown in Figure 2. Joint total normal stress and joint shear stress (i.e., σ_{zz} , and $\tau_{z\theta}$, respectively) are developed at

^a All tables and figures for Part II are located in the appendix.

the interface of the element as a result of the applied axial and torsional loadings that are transmitted to the joint. The inner and outer walls of the hollow cylinder element are subjected to an equal confining pressure, σ_c , which also corresponds to the radial and circumferential stress, σ_r and σ_θ , respectively. With the inclusion of the joint fluid pressure, u , a joint effective normal stress, σ'_{zz} is developed in accordance with the effective stress principal. Given the described loadings, the elemental joint stresses can be defined as shown in Eqs. (1), (2), and (3).

$$\sigma_{zz} = \frac{F_j}{\pi (r_o^2 - r_i^2)} + u \quad (1)$$

$$\tau_{z\theta} = \frac{3 T_j}{2\pi (r_o^3 - r_i^3)} \quad (2)$$

$$\sigma'_{zz} = \sigma_{zz} - u = \frac{F_j}{\pi (r_o^2 - r_i^2)} \quad (3)$$

Where r_o is the outer radius of the hollow cylinder specimen, and r_i is the inner radius of the hollow cylinder specimen.

The effective major and minor principal stresses can be defined for the rock joint interface as shown in Eqs. (4) and (5). As discussed earlier, the effective intermediate principal stress, σ'_2 equals the joint fluid pressure, u .

$$\text{Effective Major Principal Stress, } \sigma'_1 = \frac{\sigma'_{zz} + u}{2} + \sqrt{\left(\frac{\sigma'_{zz} - u}{2}\right)^2 + \tau_{z\theta}^2} \quad (4)$$

$$\text{Effective Minor Principal Stress, } \sigma'_3 = \frac{\sigma'_{zz} + u}{2} - \sqrt{\left(\frac{\sigma'_{zz} - u}{2}\right)^2 + \tau_{z\theta}^2} \quad (5)$$

Given the effective principal stresses, the mean joint effective stress can be defined by the first stress invariant, I_I , as shown in Eq. (6).

$$I_I = \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3} \quad (6)$$

4. DESCRIPTION OF HCA, SPECIMEN PREPARATION, TEST PROCEDURES

4.1. Description of the HCA

The hollow cylinder specimen used in the current research was obtained from limestone rock and was prepared with inside and outside diameters of 100 millimeters (4 inches) and 150 millimeters (6 inches), respectively. After the specimen was bisected into halves with lengths of about 75 to 100 millimeters (3 to 4 inches), its discontinuity surface (i.e., the contact plane) was ground and sandblasted. A photograph showing a typical hollow cylinder specimen is provided in Figure 3.

As demonstrated in Figure 4, two ends of a specimen are attached to the HCA's upper and lower aluminum end platens using high-strength epoxy grout, and then surrounded by a reinforced acrylic confining cell. This confining cell permits the application of confining pressure to the hollow cylinder specimen using air or water.

Axial load and torque are applied through the lower half of the specimen, and are transferred through the interface to the top half of the specimen, and are measured by an axial/torsional load cell mounted between the specimen top and the reaction frame. The axial and torsional loading is applied to the specimen via a Material Testing Systems (MTS) biaxial servo-hydraulic load frame, as shown in Figure 5. The load frame is designed with a high torsional and axial stiffness, and uses a 222 kN (50-kip) linear actuator for the axial loading, and a 226 kN-cm (20 inch-kip) rotary actuator for the torsional loading. The actuators are controlled with MTS 406 electronic controllers which are capable of testing in displacement, load, or strain control modes. Axial deformations are measured across the entire sample with an LVDT mounted on the axial load shaft of the MTS frame. The total rotation (i.e., shear deformation) of the specimens is recorded by an angular displacement transducer (ADT) in the load frame. Additionally, a pressure transducer is installed to monitor the fluid pressure surrounding the specimen.

4.2. Specimen Preparation

The rock joint strength tests as described herein were performed on a Holston Limestone specimen obtained from the Vulcan Materials quarry adjacent to the Holston River on Riverside Drive, east of Knoxville, Tennessee. The limestone sample, obtained in shot rock form, was approximately 0.028 cubic-meters (1 cubic-foot) and weighed roughly 70 kilograms (154 pounds). The grey limestone sample was smooth and massive with no apparent natural discontinuities or weathering.

In general, specimen preparation was consistent with procedures recommended in the literature (Brown, 1981; Xu and Freitas, 1988; Olsson, 1988) and is described herein.

The hollow cylinder specimen was taken from the limestone sample by first drilling a 100-millimeter diameter hole through the sample with a diamond drill bit. Without moving the sample, the hole was then overcored using a 150-millimeter diameter hollow diamond drill bit to create a concentric outer surface. The hollow cylinder specimen was then extracted from the limestone sample, and both ends were trimmed with a diamond rock saw. Thereafter, the specimen was bisected, thus creating an artificial saw-cut joint. The two halves of the hollow cylinder limestone specimen each measured approximately 75 to 100 millimeters (3 to 4 inches) in height, 150 millimeters in outside diameter, and 100 millimeters in inside diameter. Each specimen half was then attached to an HCA end platen and the rock joint surface was ground approximately flat and parallel to the surface of the end platen. Final preparation of the sample involved sandblasting the joint face of each half of the specimen to polish the joint surface.

After the initial specimen preparation was completed, a series of HCA tests were conducted to “run-in” the specimen halves (Olsson, 1988), and to investigate the dry, unconfined joint characteristics of Holston Limestone. Specifically, eighty-seven tests were performed to try to achieve a steady-state condition for the dry, unconfined limestone joint. The friction coefficient of the joint, μ , ranged from 0.47 to 0.88 during this phase of the testing. In spite of the steps taken to grind the joint surface and assure that the two specimen halves were properly mated, repeated testing was required until the joints became fully run-in and consistent strength results were obtained. These findings are consistent with those presented by Olsson. Ultimately, the steady-state friction coefficient for the dry, unconfined limestone rock joint was measured to be about 0.67.

4.3. HCA Test Procedures for Saturated, Confined Limestone Rock Joints

The assembled cell with hollow cylinder specimen was partially filled with water to reach a level above the artificial rock joint. A target fluid confining pressure was then applied to the system with the two halves of the specimen separated while the bottom loading ram was controlled to be in static equilibrium. This allowed measurement of the system response to the applied fluid confining pressure. It was found that every 0.138 MPa (20 psi) of applied fluid confining pressure introduced an additional 0.623 kN (140 pound force) increase in system normal force as measured by the MTS control system. The two halves of the specimen were then brought into contact, and the desired axial loading, F_j , was applied. The interface shear strength testing of the artificial rock joint was performed for a predetermined range of joint loadings and fluid pressures. Specifically, thirty-two tests were performed on the limestone specimen with fluid confining pressures ranging from 0.14 MPa to 0.55 MPa and effective joint normal stresses ranging from 0.16 MPa to 0.65 MPa.

5. RESULTS AND DISCUSSION

The primary data obtained during the testing included the joint rotation-joint shear stress response curves for varying effective normal stresses and confining pressure. Similar to findings published by Reardon (Reardon et al.; 1991), the observed response curves indicated a rapid increase in shear stress accompanied by a very small increase in rotation until a peak shear stress was mobilized. After reaching failure stress, large rotations were observed with a negligible amount of change in residual shear stress as

shown in Figure 6. Furthermore, the shear strength upon stress reversal was typically equal to that measured during the first loading.

The joint shear stress-joint rotation response curves were then used to develop effective stress failure envelopes for each applied fluid confining pressure based on the Mohr-Coulomb failure criteria. The joint shear strength based on the Mohr-Coulomb failure criterion can be defined as shown in Eq. (7).

$$\tau_{z\theta f} = c' + \sigma'_{zzf} \tan \phi' \quad (7)$$

Where $\tau_{z\theta f}$ is the joint shear stress at failure, σ'_{zzf} is the joint effective normal stress at failure, and ϕ' is the joint effective angle of internal friction. The joint effective cohesion, c' , was measured to be zero in this research; as such, the joint shear strength can be uniquely described in terms of either ϕ' or the mobilized friction coefficient (i.e., $\mu_f = \tan \phi' = \tau_{z\theta f} / \sigma'_{zzf}$).

As shown in Figure 7, Mohr-Coulomb envelopes have been developed for the Holston Limestone for the range of HCA testing conditions. A lower limit envelope has been developed from the dry, unconfined joint testing data; whereas, an upper limit envelope has been developed from the saturated joint test condition with 0.552 MPa applied confining pressure. Additionally, a composite Mohr-Coulomb envelope has been developed using all of the saturated, confined rock joint test data.

Using the Mohr-Coulomb envelopes, ϕ' has been determined for each of the HCA loading conditions. As shown in Table 1, ϕ' ranges from 33.7 degrees for the dry,

unconfined limestone rock joint to 37.0 degrees for the saturated joint with applied 0.552 MPa confining pressure. The composite Mohr-Coulomb failure envelope yields a ϕ' equal to 36.5 degrees. Table 1 also includes results from previous HCA and direct shear testing on dry, unconfined Imperial Black marble rock joints (Reardon, et al.; 1991), as well as friction angle values for limestone suggested in published sources (Schwartz, 1964; Schneider, 1974; Barton, 1976; Hoek and Bray, 1977; Goodman, 1989; Fang, 1991). Friction angle values developed for the limestone during this current research seem to be consistent with the previous HCA test data and the suggested values for limestone. Further, HCA test data shows that the shear strength of the smooth polished Holston Limestone rock joint is slightly greater for saturated conditions than for dry conditions (i.e., the friction angle is about 1.7 degrees to 3.3 degrees greater for saturated joint conditions than for dry joint conditions, as shown in Table 1). These results follow findings presented by Barton for smooth polished rock surfaces (Barton, 1976). Barton found that the shear strength of smooth polished rock surfaces, when subjected to low to medium stress levels, is unaffected or slightly increased when wet. Additionally, the increase in shear strength from dry, unconfined conditions to saturated, confined conditions can be attributed to the inclusion of confining pressure as described herein.

Figure 8 shows the variation of joint effective normal stress with mobilized friction, μ_f (i.e., $\tau_{z\theta f} / \sigma'_{zzf}$), for each applied effective intermediate principal stress. As shown in Figure 8, mobilized friction increases correspondingly with increase in joint effective normal stress for lower σ'_{zzf} values (i.e., from about 0.16 MPa to about 0.38 MPa). Additionally, the mobilized friction increases with increase in the effective intermediate principal stress in this test range. For larger σ'_{zzf} values (i.e., those ranging

from 0.38 MPa to 0.65 MPa), no significant increase in the mobilized friction is observed for any of the individual confining pressure curves.

The variation of effective intermediate principal stress with joint shear stress is demonstrated in Figure 9. Specifically, Figure 9 shows σ'_2 as a function of $\tau_{z\theta}$ for each applied joint effective normal stresses. As would be expected, the joint shear stress increases with an increase in joint effective normal stress. Figure 9 corroborates the findings demonstrated in Figure 8, in that there is a measurable increase in shear stress corresponding to an increase in effective intermediate principal stress.

Figure 10 shows the first stress invariant, I_1 (i.e., the mean joint effective stress) as a function of $\tau_{z\theta}$ for each of the applied fluid confining pressures. As shown in the figure, the mean joint effective stress is a linear function of the joint shear strength. The linear relationship can roughly be defined as shown in Eq. (8).

$$\tau_{z\theta} = 2.25I_1 + C \quad (8)$$

Where C is a constant that is dependent on the effective intermediate principal stress curve being assessed.

6. SUMMARY AND CONCLUSIONS

Rock interface modeling and testing is typically performed using conventional methods, namely the direct shear method. However, the direct shear method has significant limitations including: (1) the inability to determine the principal stresses except at failure, (2) non-uniform stress distribution within the rock interface and high

stress concentrations at its edges, and (3) difficulties in controlling and measuring applied fluid confining pressures in the rock interface. To overcome these inherent deficiencies associated with the direct shear method, the HCA has been developed specifically to model rock joint response for complex loading conditions and applied fluid pressures.

A series of tests have been performed using the HCA, whereby, a hollow cylinder Holston Limestone rock specimen with a smoothened artificial joint was subjected to various axial and torsional loadings and a range of fluid confining pressures. Initial run-in testing on the limestone specimen yielded joint friction coefficient values of 0.47 to 0.88 before a dry, unconfined steady-state joint friction coefficient of about 0.67 was obtained. Strength parameters developed for Holston Limestone during the HCA testing compare favorably with suggested parameters for limestone given in published sources. Moreover, joint behavior observed during the current research appears to be consistent with published findings developed during baseline hollow cylinder device testing on artificial rock joints (Olsson, 1988; Reardon, 1991).

Original findings were made during the current research regarding the joint response and its relationship to the applied confining pressures. These findings primarily included observations that:

- (1) Increases in confining fluid pressure (i.e., effective intermediate principal stress) results in measurable increases in joint shear strength, especially for lower values of joint normal stress.
- (2) A simple linear relationship exists between the mean joint effective stress and the joint shear strength.

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APPENDIX

Table 1. Angle of Internal Friction Values for Limestone

Limestone Formation	Test Description	Internal Friction Angle, degrees	Coefficient of Friction
Holston Limestone	HCA, dry, unconfined	33.7	0.67
Holston Limestone	HCA, $u = 0.138$ MPa ⁽⁶⁾	35.4 ⁽⁷⁾	0.71
Holston Limestone	HCA, $u = 0.276$ MPa ⁽⁶⁾	36.9 ⁽⁷⁾	0.75
Holston Limestone	HCA, $u = 0.414$ MPa ⁽⁶⁾	36.9 ⁽⁷⁾	0.75
Holston Limestone	HCA, $u = 0.552$ MPa ⁽⁶⁾	37.0 ⁽⁷⁾	0.75
Holston Limestone	HCA, Composite Data ⁽⁶⁾	36.5 ⁽⁷⁾	0.74
Imperial Black Marble ⁽¹⁾	HCA, dry, unconfined	33.5	0.66
Imperial Black Marble ⁽¹⁾	DST	36.0 – 37.0	0.73 – 0.75
limestone ⁽²⁾	Varies	27 – 50	N/A
Indiana Limestone ⁽³⁾	Triaxial	42.0	N/A
limestone ⁽⁴⁾	DST	33 - 40	0.65 – 0.84
limestone ⁽⁵⁾	DST	34	0.67

HCA = Hollow Cylinder Apparatus

DST = Direct Shear Test

u = Fluid Confining Pressure

- (1) Values obtained from Reardon, et al. (1991) for smooth artificial rock surfaces.
- (2) General values obtained from Fang (1991).
- (3) Values obtained from Goodman (1989) after Schwartz using triaxial testing on intact specimens.
- (4) Values obtained from Barton (1976) using DST on sand-blasted, rough-sawn and residual surfaces.
- (5) Values obtained from Schneider (1974) using DST on artificial rock joints.
- (6) Tests performed under saturated, confined conditions.
- (7) Values are effective angle of internal friction, ϕ' .

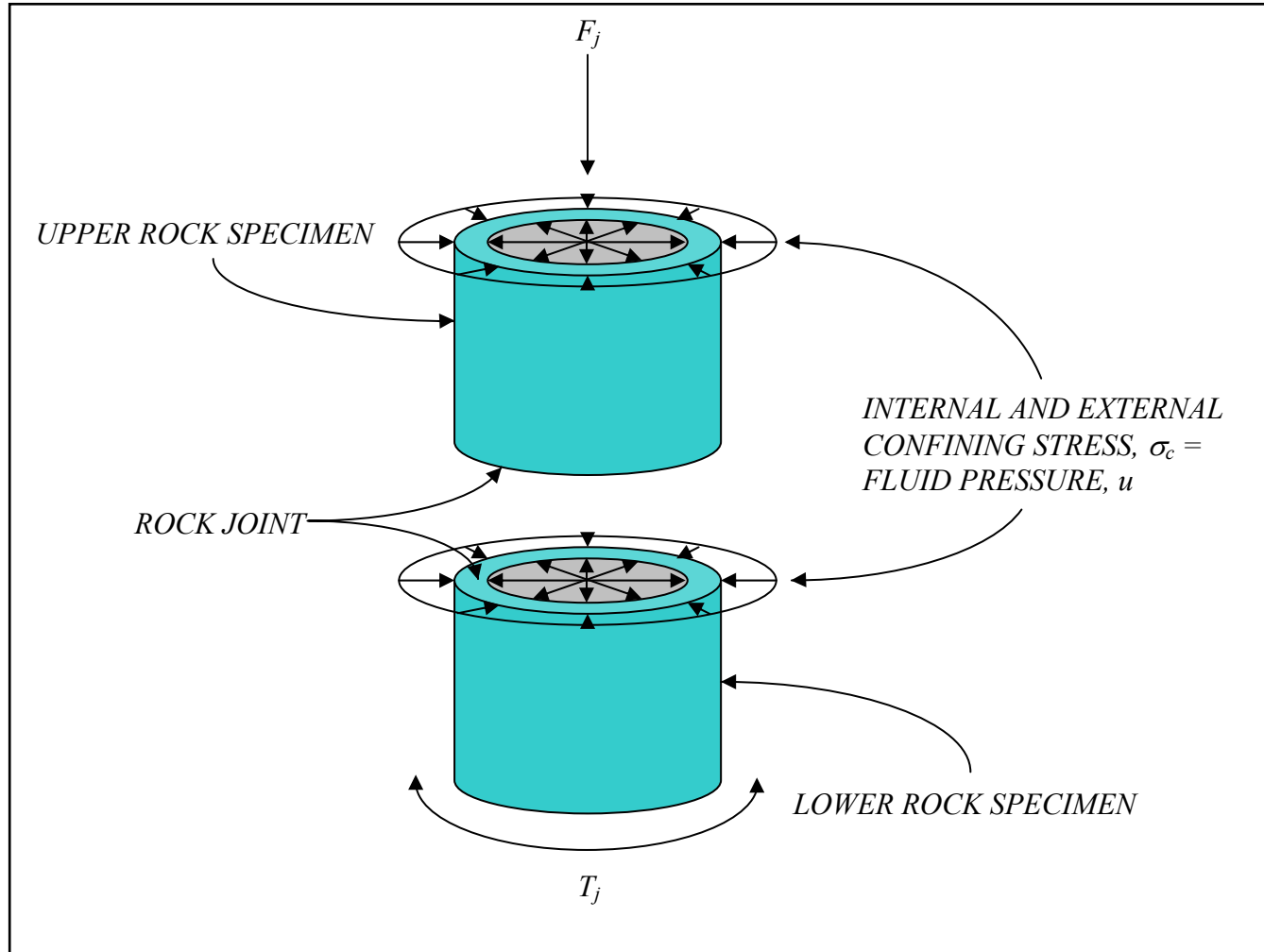


Figure 1. Generalized HCA Loadings for the Saturated, Confined Shear Strength Test.

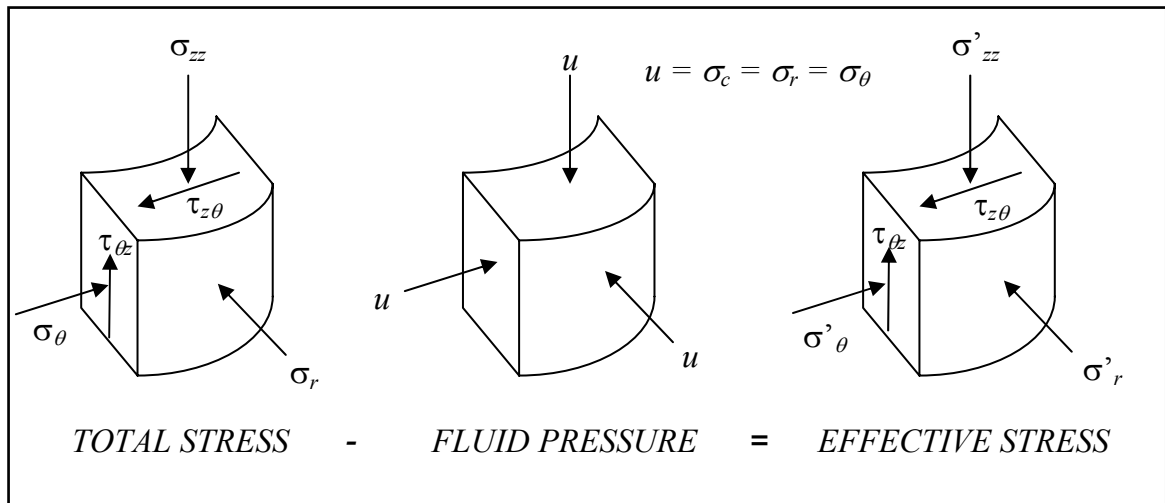


Figure 2. Joint Stress State for the HCA Saturated, Confined Shear Strength Test.



Figure 3. Typical HCA Sample (Scale in Inches).

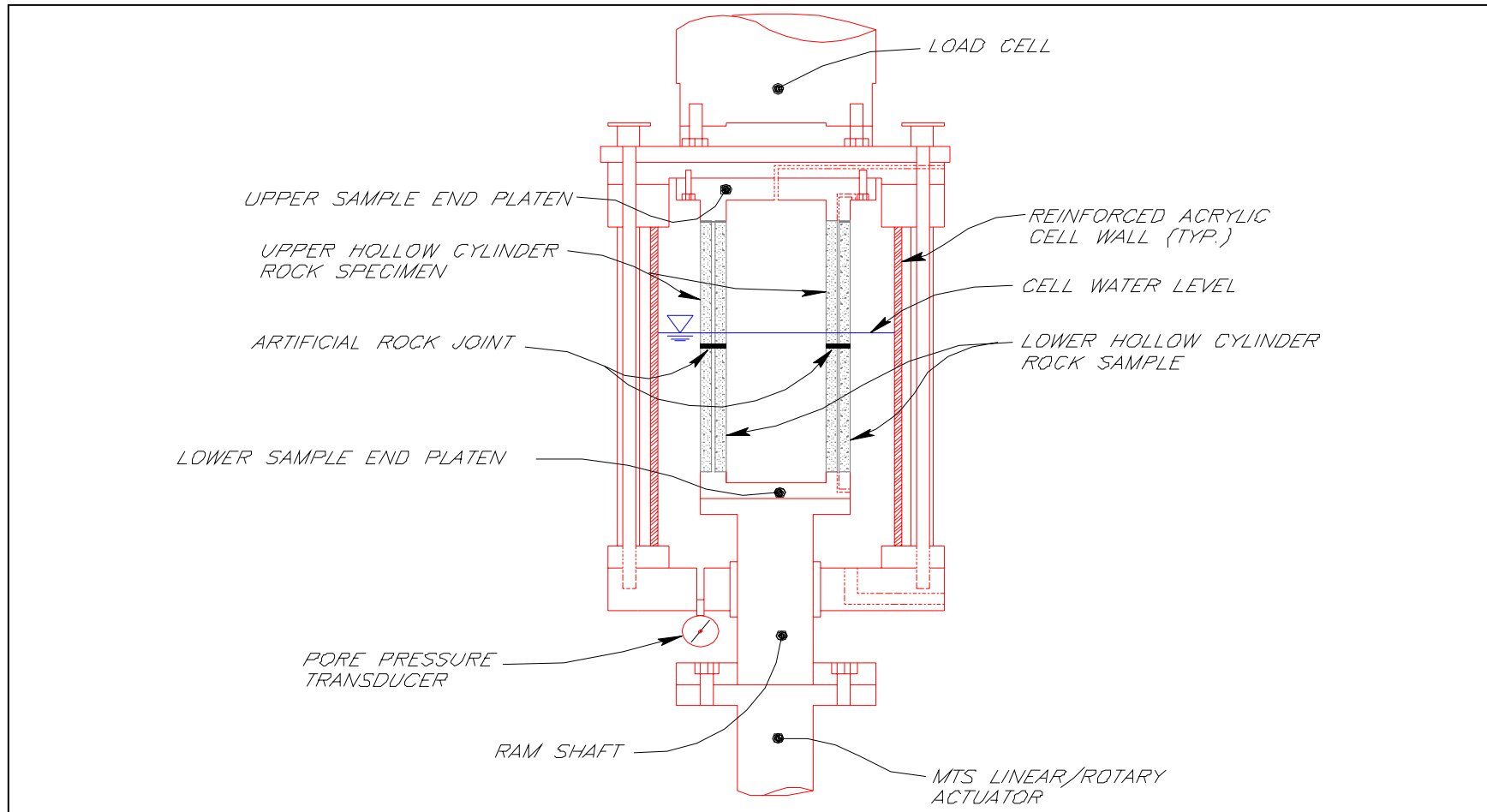


Figure 4. Typical Cross-Section through HCA.

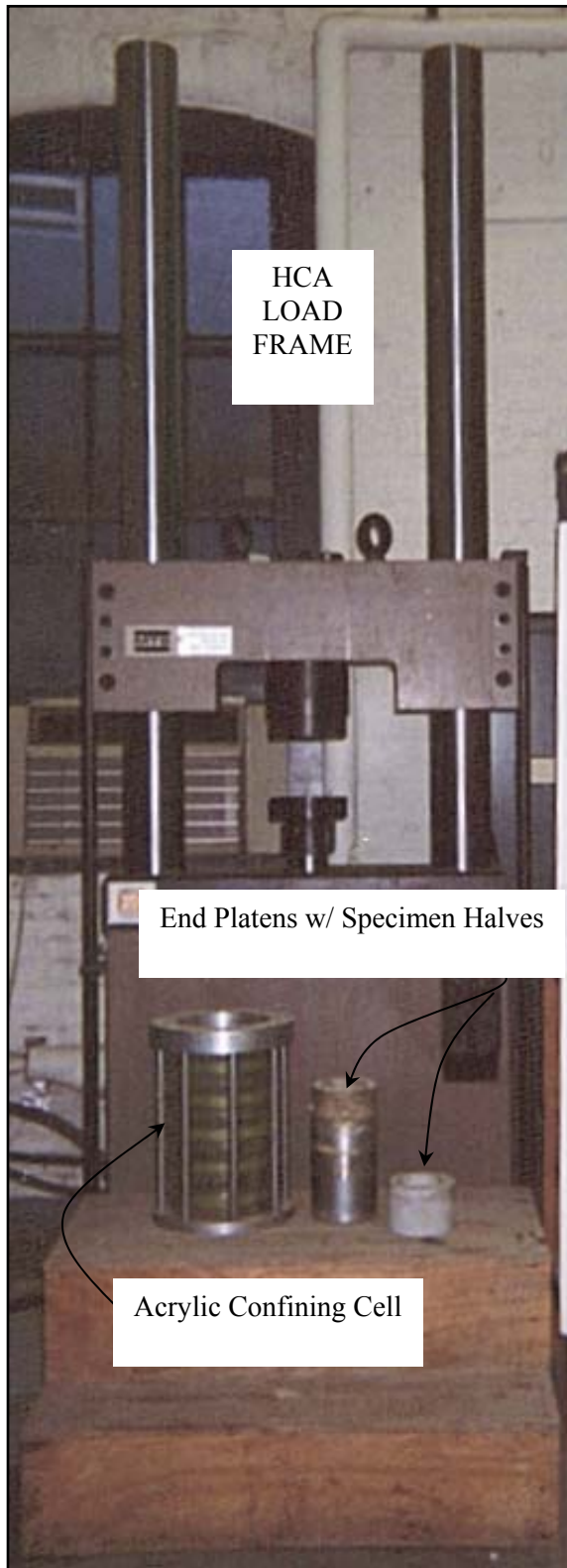


Figure 5. HCA MTS Load Frame.

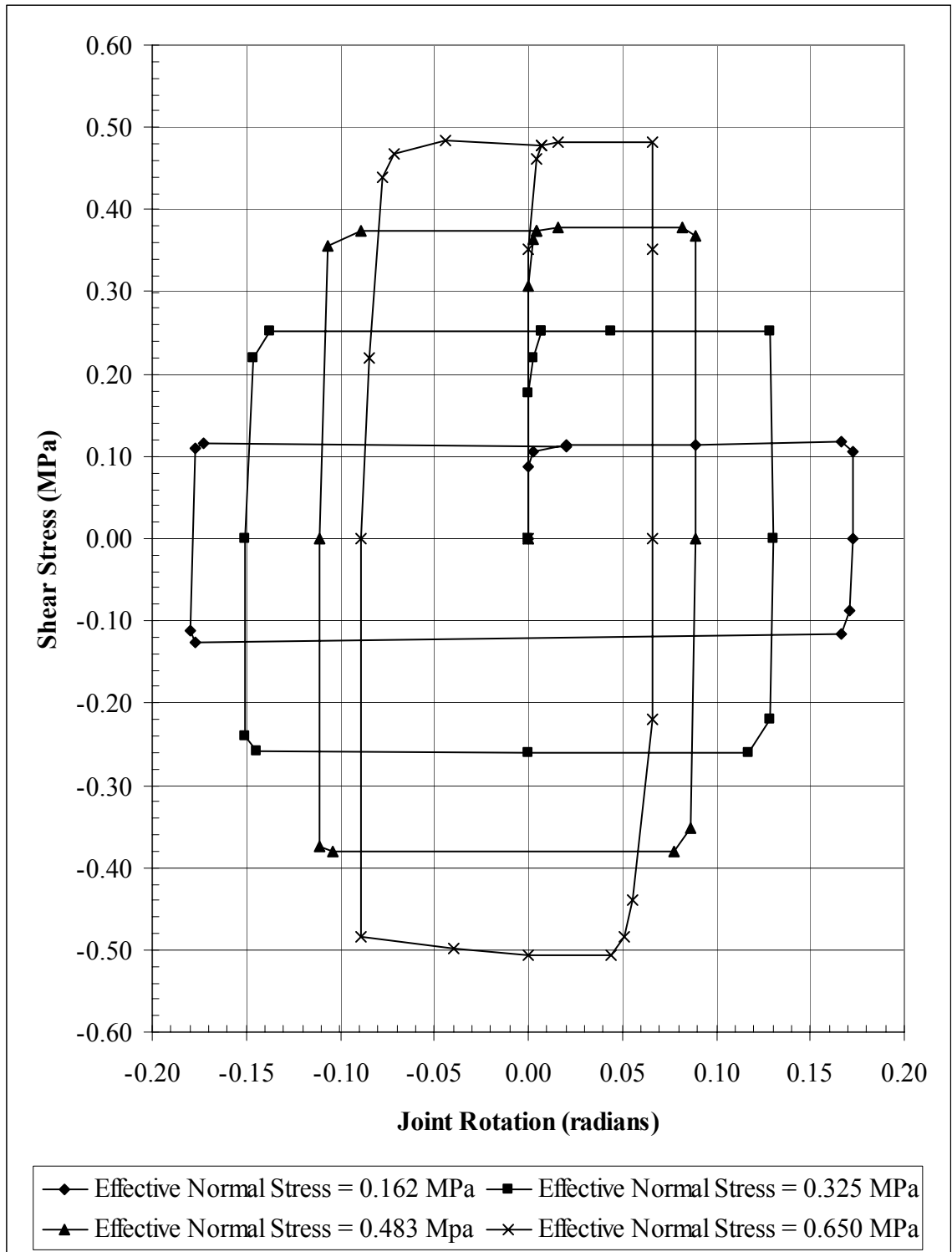


Figure 6. Typical Joint Rotation – Joint Shear Stress Response Curves from HCA Saturated, Confined Rock Joint Shear Strength Test with 0.276 MPa Confining Pressure / Joint Fluid Pressure.

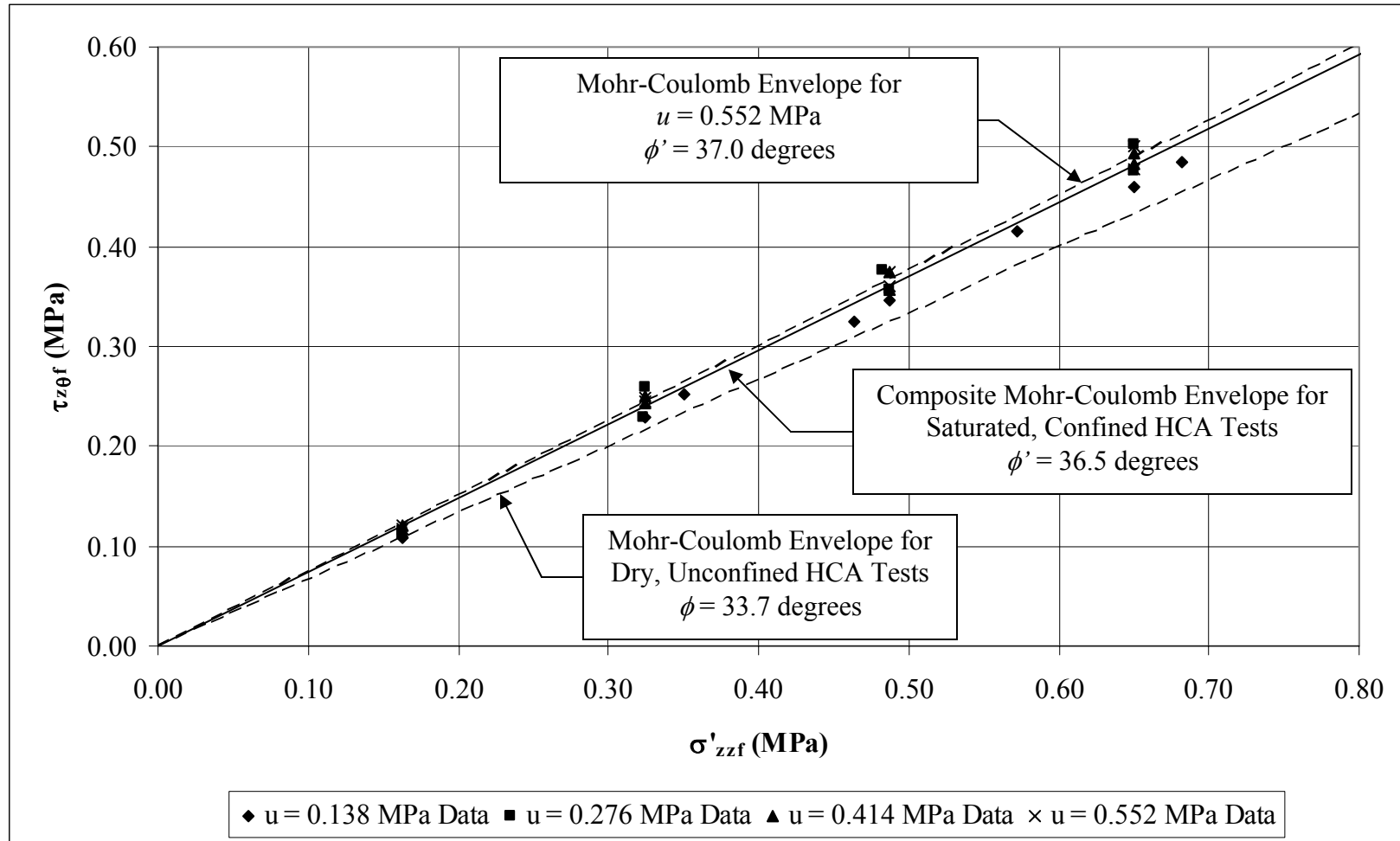


Figure 7. Mohr-Coulomb Failure Envelope Graph for HCA Dry, Unconfined and Saturated, Confined Rock Joint Shear Strength Tests on Holston Limestone.

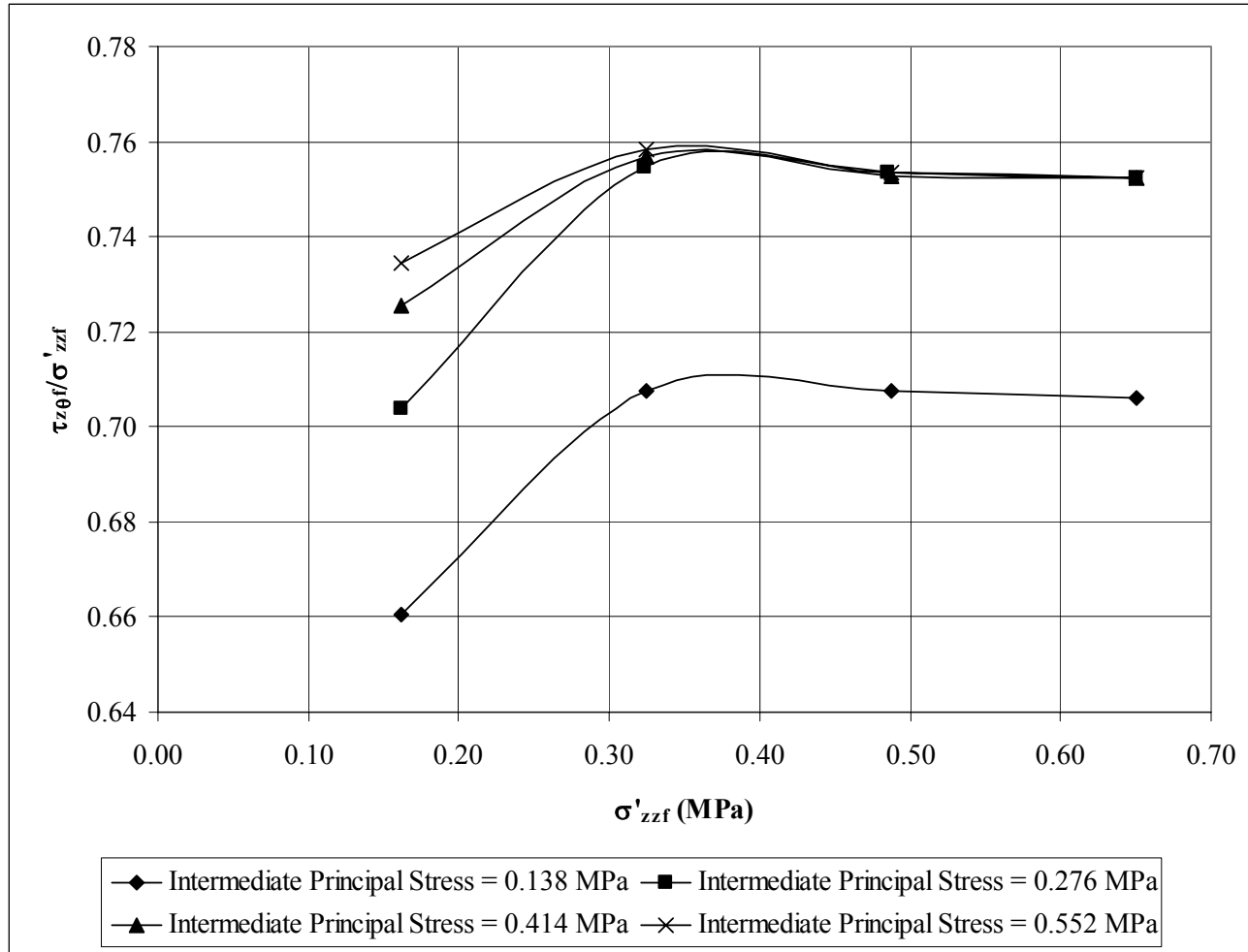


Figure 8. Joint Effective Normal Stress versus Mobilized Friction for HCA Saturated, Confined Rock Joint Shear Strength Tests on Holston Limestone.

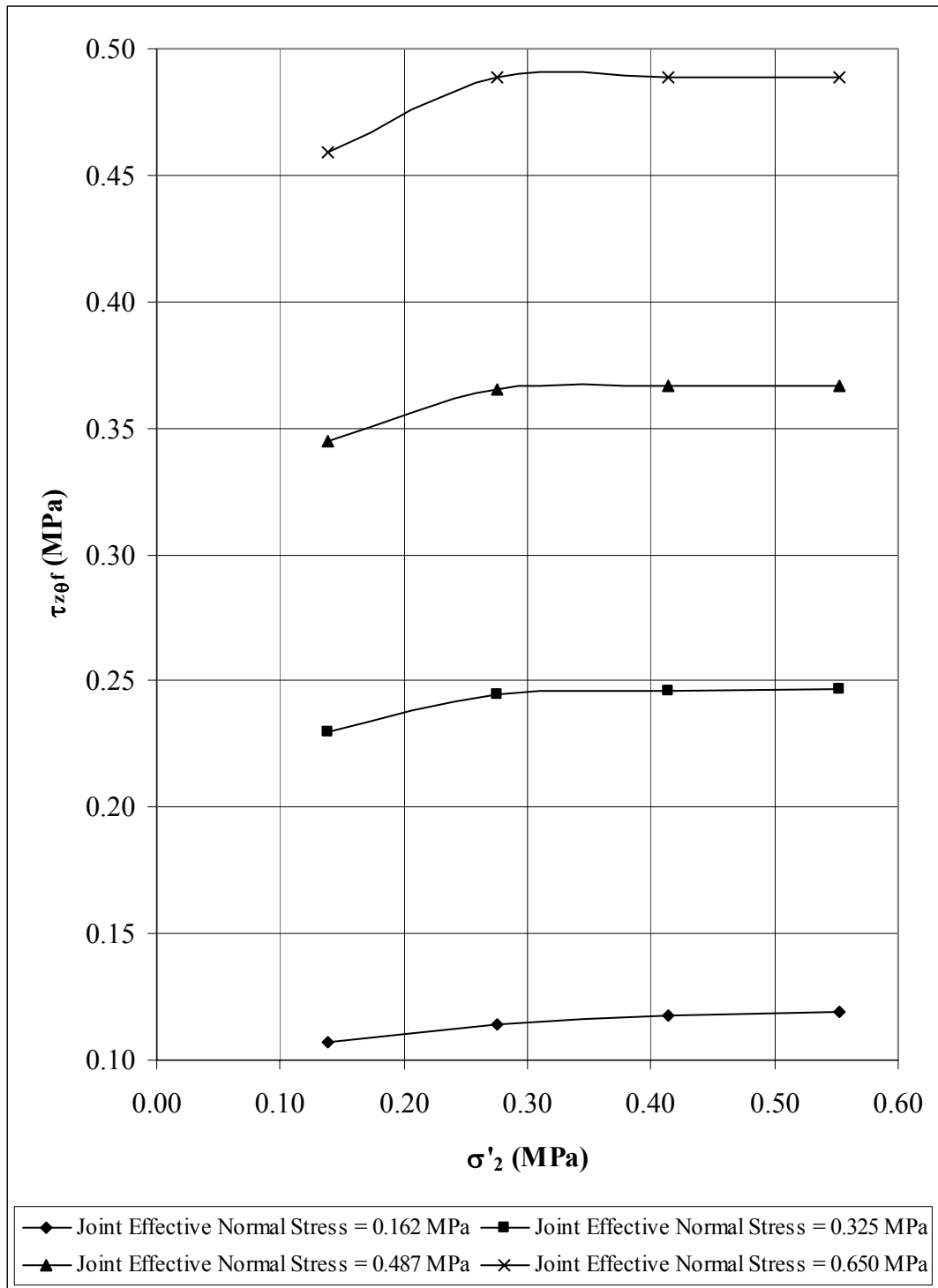


Figure 9. Effective Intermediate Principal Stress (i.e., Confining Pressure) versus Joint Shear Strength for HCA Saturated, Confined Rock Joint Shear Strength Tests on Holston Limestone.

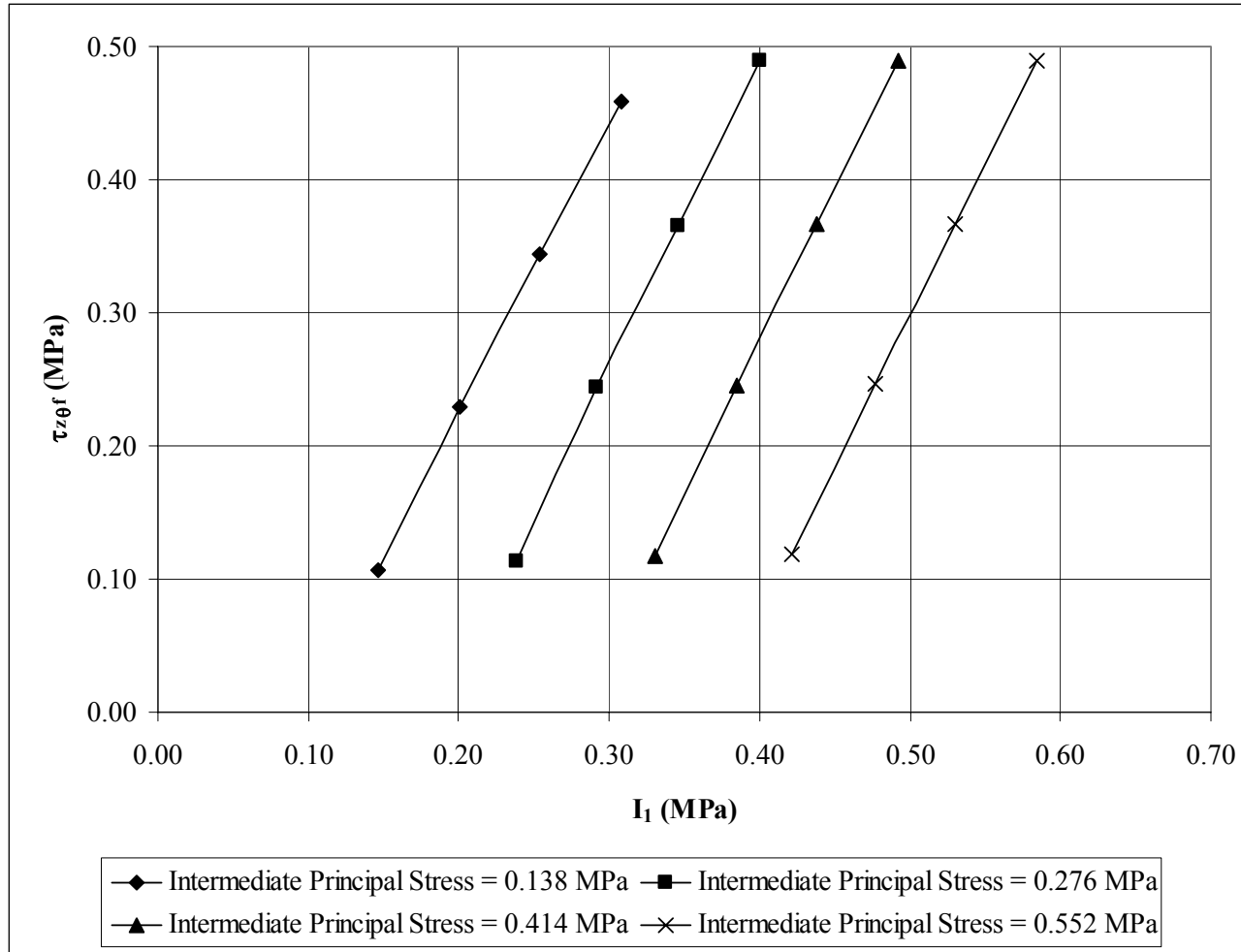


Figure 10. First Stress Invariant (i.e., Mean Joint Effective Stress) versus Joint Shear Strength for HCA Saturated, Confined Rock Joint Shear Strength Tests on Holston Limestone.

PART III

A Case History Regarding Mine Roof Stability: Fort Hartford Mine Superfund Site

1. INTRODUCTION

The Fort Hartford Mine Superfund Site is located on a 2.61 km² (645-acre) tract in a rural area of Ohio County, Kentucky. Underneath the site are approximately 0.49 km² (120 acres) of abandoned room and pillar limestone mine workings. Limestone mining was performed at the site from the late 1950's to the late 1970's to supplement the stone demands for local road and railway construction. The mine has two lobes known as the Rough River Lobe and the Caney Creek Lobe. Both lobes were first mined and partially second mined (i.e., pillars from the initial mining operation were subsequently reduced in size or removed). In 1981, Barmet Aluminum Company (now Commonwealth Aluminum Concast, Inc.) began using the mine as a storage facility for an aluminum recycling by-product known as "salt-cake fines". Between 1981 and 1991, approximately 12,455 MN (1.4 million tons) of salt cake fines were deposited into the mine. However, hydration of the salt-cake fines following ingress of groundwater and/or surface water runoff through mine roof fractures and breakthroughs resulted in chemical reactions that formed several gases, the most prominent being ammonia. Consequently, in 1990, the Environmental Protection Agency (EPA) designated the Fort Hartford Mine as a superfund site.

As part of the remedial action plan required by the EPA, pump stations were installed in the mine for removal of mine water, thereby reducing the potential for further hydration of the salt-cake fines. The remedial action plan also called for periodic in-mine maintenance and monitoring. However, certain areas of the mine exhibited unstable roof conditions that were potentially unsafe for maintenance personnel. In fact, roof instability presented a number of potentially detrimental consequences including: (1)

increased ingress of water into the mine through entryways created by roof collapses or through breakthroughs that propagated to the ground surface, (2) development of subsidence deformations and/or “sinkholes” at the ground surface above the mine, (3) the escape of hazardous gases through breakthroughs in the mine roof, in the vicinity of overlying residences, and (4) contamination of the local groundwater system.

The site is characterized by complex geologic conditions such as karst terrain and solution features, faulting and fracturing, and subsidence due to the mining operations. These varied geologic factors are believed to have contributed to numerous mine roof dropouts and breakthroughs. In 1993, Ogden Environmental and Energy Services (now AMEC) and the University of Tennessee Institute for Geotechnology performed a cooperative study to assess mine roof stability at the site (Ogden, 1993). Since 1995, ongoing monitoring, design, and implementation of mitigative measures addressing mine roof stability have been performed at the site by Geo/Environmental Associates, Inc. A discussion of the site geology, the mine roof stability assessment, instrumentation and monitoring, and mitigative measures performed at the Fort Hartford Mine Superfund Site is presented herein.

2. SITE GEOLOGY

2.1. General

The Fort Hartford Mine Superfund Site lies entirely within the Rough Creek fault zone in the north-central part of the Interior Low Plateaus Physiographic Province (Atherton, 1971). Within the Rough Creek fault zone, strata have been displaced as much as several hundred feet along a breached or faulted anticline. The regional trend of

the fault zone is generally in an east-west direction, and structural interpretations of the faulting cover the gamut from normal to strike-slip to reverse (Fenneman, 1938; Davis, 1984; Nelson and Lumm, 1984).

2.2. Stratigraphy

The United States Geological Survey (USGS) geologic quadrangle map for Olaton, Kentucky (See Figure 1)^a, as well as subsurface exploration data, shows that the mine site is encompassed by five members of the Mississippian-aged Chester Series. These primary stratigraphic members are detailed in Table 1. A typical stratigraphic cross-section of the site area is shown in Figure 2. It should be noted that the Tar Springs, the Glen Dean, and the Hardinsburg members, as described in Table 1, are locally absent in some areas of the mine.

2.3. Faults

As discussed earlier, the site lies completely within the Rough Creek fault zone. The USGS geologic quadrangle map shows the Rough Creek faults as normal faults bounding a series of horsts and grabens on the crest of a broad anticline with an east-west trending fold axis. Most of the faults that have been identified and field mapped in the study area are consistent with the USGS mapping, in that they are normal faults that strike approximately east-west.

2.4. Jointing

Field observations and measurements were performed in the portal areas of the mine to determine the primary and subsidiary joint sets at the site. Based on a data set of 52 measurements, the primary joint set was found to strike approximately N60°E and the

^a All tables and figures for Part III are located in the appendix.

subsidiary joint sets strike approximately north-south and N25°W, respectively. A strike rose diagram of the measured joint data is shown in Figure 3.

2.5. Weathering

The ingress of water into the mine through faults, fractures, joints, and previous roof collapses and breakthroughs has been observed at various locations throughout the mine, in areas of both high and low cover. The degree of weathering was noted to be severe in some locations where water ingress was observed. Many low cover areas at the site naturally correspond to topographic lows with saturated overburden soils. These saturated overburden soils contribute to water flow through the rock members above the mine, primarily in the shale of Hardinsburg Sandstone that lies directly above the Haney Limestone mine roof. As a result of the water flow/ingress through the Hardinsburg Sandstone, significant weathering was observed in the shale at various locations, as well as in the boring logs.

3. MINE ROOF STABILITY ASSESSMENT

3.1. General

A comprehensive study was performed in 1993 to assess mine roof stability at the Fort Hartford Mine Superfund Site. The study combined an extensive data collection/interpretation program with analytical methods to delineate zones of relative stability within the mine. Furthermore, map overlaying techniques were used to identify areas of low, moderate, and high potential for mine roof deterioration. The mapping generated during the study was used to establish the locations of the primary travel ways

in the mine and to select where and what types of instrumentation and potential mitigative measures should be implemented.

3.2. Data Collection and Interpretation

Much of the field data was collected inside the mine under hazardous conditions. Personnel working inside the mine were exposed to safety hazards related to mine roof instability, as well as high levels of potentially dangerous gases. Other portions of the mine were blocked off by previous roof falls or were filled with salt-cake fines. As such, field data acquisition was limited. Because of the obstacles involved in obtaining data from inside the mine, other sources such as mine maps, structure contour maps, subsurface exploration data, instrumentation data, and laboratory data were used to support the limited field data.

Based on the locations and types of previous roof collapses, results of the subsurface exploration program, and visual observations and measurements acquired during the site reconnaissance; certain parameters or combinations thereof were identified as being key to local roof stability. It appeared that the failure mechanisms and the parameters contributing to previous roof collapses varied spatially throughout the mine. In order to assess this apparent spatial variability, measurable parameters that could potentially have an impact on mine roof stability were collected and objectively evaluated.

As a base for mapping spatial variations of parameters within the mine, a north-south grid overlay indexed on an alphanumeric system was developed as shown in Figure 4. Each grid cell on the grid overlay measured approximately 61 meters by 61 meters (200 feet by 200 feet). Additionally, each grid cell was subdivided into quadrants

measuring approximately 30.5 meters by 30.5 meters (100 feet by 100 feet). The grid overlay was tied to the base map of the mine workings, as well as to the Kentucky state plane coordinate system. Thereafter, the following parameters were individually measured and/or assessed for each of the grid cells.

- (1) Span (pillar to pillar spacing). The maximum span was measured from the mine map for each grid cell quadrant.
- (2) Roof thickness. Thickness of the Haney Limestone mine roof was estimated by subtracting the elevation of the bottom of the Haney Limestone mine roof (developed from subsurface exploration data and in-mine surveys) from the elevation of the top of the Haney Limestone obtained from structure contour maps.
- (3) Thicknesses and sequence of individual roof layers. Individual roof layer thicknesses were measured at mine portals and from borehole logs. However, because of the high variability of this parameter and constraints preventing direct access to portions of the mine, consistent measurement of individual roof layers was not feasible.
- (4) Material properties. Material properties used in the study were estimated based on laboratory testing of core samples obtained during the subsurface exploration and from published values for comparable materials as shown in Table 2 (Hoek and Brown, 1980; Hoek and Bray, 1981; Goodman, 1989; and Fang, 1991). The primary material properties were (1) modulus of elasticity, E ; (2) Poisson's ratio, ν ; (3) unconfined compressive strength, q_u ; (4) tensile strength, T_o ; (5) permeability, k ; (6) density, γ ; and (7) the Hoek-Brown empirical frictional strength parameter, m , and empirical inherent strength parameter, s . It was assumed that there was not a

significant spatial variation in the material properties throughout the mine site.

Correspondingly, the material properties used in the study were applied as constant values.

- (5) Jointing. As discussed earlier, jointing data at the site were obtained using measurements made in the portal areas of the mine. Even though jointing data was not available for a large portion of the mine, there was no indication that jointing varied systematically with spatial location.
- (6) Proximity to faults. Proximity to faults was inferred from fault maps and field observations.
- (7) Proximity to fracture traces. Proximity to fracture traces was inferred from lineament mapping developed using remote sensing photographic imagery of the project site.
- (8) Amount of cover. The amount of cover (i.e., thickness of overburden) was obtained for each grid cell by subtracting the elevation of the top of the Haney Limestone, as obtained from the structure contour map, from the ground surface elevation.
- (9) Water conditions in the roof. The presence of water in the roof was observed at numerous locations throughout the mine, including areas of previous collapses and at backfilled or otherwise remediated breakthroughs. Based on direct observation and subsurface exploration data, it was apparent that the saturated conditions contributed to accelerated weathering of the shale in the Hardinsburg Sandstone. However, the spatial effects of this parameter were difficult to quantify.
- (10) Horizontal stress. No horizontal stress measurements were performed for the study. However, a fixed horizontal “field stress” was assumed based on observation and

equilibrium considerations, and was used in the stability analyses. Horizontal stress orientation was based on published sources (Zobeck et al., 1989 and 1991).

Each of the preceding parameters was independently assessed based on their physical significance as well as availability and reliability of the data for each. Specifically, parameters (1), (2), and (8) were measured directly from site maps (i.e., base topography, structure contour, and mine maps). Parameters (6) and (7) were inferred based on field observations and mapping. Parameters (3) and (9) were assumed to vary significantly throughout the mine based on observations at portals and subsurface exploration findings; however, because of access/safety constraints neither of these data could be consistently measured for a large portion of the mine. Parameter (5) was believed to be fairly consistent throughout the mine based on measurements at portals and boring log data; however, because of access/safety constraints the spatial variability of this parameter could not be confirmed. Parameters (4) and (10) were assumed to have very little spatial variability throughout the site and were applied as constants in the study. The parameters which could be directly measured and/or inferred from the site mapping and field measurements were ultimately selected as the primary parameters. Accordingly, the primary parameters were span and cover, roof thickness, proximity to faults, and proximity to fracture traces.

The impact of span and cover on mine roof stability was evaluated by use of the Hoek-Brown Stability Factor as described in Section 3.3. The impact of roof thickness on mine roof stability was correlated to previous roof collapses believed to be related to insufficient roof thickness, and is detailed in Table 2. The impacts of the mine roof

stability based on proximity to faults and fractures, respectively were assessed based on the criterion shown in Table 2.

3.3. Stability Analyses

3.3.1. Finite Element/Boundary Element Analyses

Mine roof stability analyses were performed using the computer program, *Phases* (plastic hybrid analysis of stress for estimation of support; Hoek, 1992). *Phases* is a 2-D finite element/boundary element computer program, that is typically used to estimate stresses and displacements associated with underground excavations. *Phases* was used for the Fort Hartford project to develop stability factor contours for a given range of mine roof span and cover, and an assumed constant mine roof thickness.

The first step of the *Phases* analysis was to develop representative cross-sections for the mine. Twenty-four cross-section combinations varying from low cover/low span conditions to high cover/high span conditions were modeled. A matrix of the model geometries is shown in Table 3. In all cases the cover was assumed to overlay a 3-meter thick Haney Limestone roof layer.

Material parameters used in the *Phases* analyses were estimated based on site specific laboratory test data and published values for similar materials (Hoek and Brown, 1980; Hoek and Bray, 1981; Goodman, 1989; and Fang, 1991). The rock units were modeled as Hoek-Brown isotropic, linear-elastic materials. Parameters for each of the cross-section materials used in the *Phases* analyses are shown in Table 3.

Given the specified cross-sectional geometries (i.e., cover and span), a finite element mesh was self-generated by the *Phases* program. The finite element mesh consisted of 3-noded and/or 6-noded triangular quadratic continuum elements. Boundary

conditions for the hybrid finite element/boundary element analyses were such that the side and bottom boundaries were assumed to have zero displacements, and the top boundary was treated as a free surface. Thereafter, a plane-strain finite element analysis was performed on each of the cross-sections.

In general, the *Phases* analyses demonstrated that the major principal stress direction above the center of the mine roof was approximately vertical for low cover conditions and subhorizontal under high cover conditions. In the case of low cover, the critical part of the mine roof with respect to stability was determined to be the outer extremity near the abutments. In the case of high cover and high span, the critical part of the mine roof was shown in the models to be the bottom of the roof beam, in the center of the span. These results were found to be in general agreement with data and theory presented for conventional methods such as elastic beam theory (Obert and Duvall, 1967; Stephansson, 1969) and voussoir beam theory (Beer and Meek, 1982).

3.3.2. Designation of Stability Factor

Based on field observations and the results of the finite element/boundary element analyses, it was clear that collapses under low cover conditions were largely due to low horizontal stresses that allowed shear failure of roof blocks at the abutments; whereas, the collapses under high cover/high span conditions were related to large vertical stresses and tensile strains that prompted roof beam bending and spalling. To further evaluate the impact of these presumed loading conditions, strength to stress ratios were developed for each of the modeled span and cover combinations. Specifically, the Hoek-Brown empirical stability factor for rock masses was used as the basis for comparing the strength to stress ratios.

The *Phases* computer program was used to calculate the Hoek-Brown stability factor at each integration point in the generated finite element mesh by determining the ratio of rock shear strength to the maximum predicted shear stress. The shear strengths and shear stresses were dependent on the specified values in Table 3 for m , s , and q_u , as well as the major principal stress, σ_1 , and the minor principal stress, σ_3 , as predicted by the *Phases* program. The Hoek-Brown stability factor was calculated using equations Eq. (1), Eq. (2), and Eq. (3). First, the Hoek-Brown shear strength, S_{max} , was defined as shown in Eq. (1).

$$S_{max} = \sqrt{\left(\frac{m q_u}{8}\right)^2 + \frac{m q_u (\sigma_1 + \sigma_3)}{8} + \frac{s q_u^2}{4}} - \frac{m q_u}{8} \quad (1)$$

The shear stress, S , was determined using the following stress invariant in Eq. (2).

$$S = \frac{\sigma_1 - \sigma_3}{2} \quad (2)$$

Therefore, the Hoek-Brown stability factor, SF , was calculated by the ratio shown in Eq. (3).

$$SF = \frac{S_{max}}{S} \quad (3)$$

Phases was used to develop contour plots of Hoek-Brown stability factor for each of the span and cover conditions described in Table 2. A typical plot showing Hoek-Brown stability factor contour distributions is shown in Figure 5. In order to quantify the Hoek-Brown stability factor, a weighted average was determined for the contoured ranges in the Haney Limestone mine roof for each modeled combination of span and cover. Specifically, the area of each plotted stability factor contour range was measured within the 3-meter thick zone of Haney Limestone roof directly above the mine excavation, and then the weighted range of the Hoek-Brown stability factor, $RWSF$, was determined using Eq. (4):

$$RWSF = \frac{\sum R_i A_i}{\sum A_i} \quad (4)$$

where R_i was the range of SF_i , and A_i was the area of the i^{th} region.

The mean weighted Hoek-Brown stability factors for each of the modeled span and cover conditions were then inserted as “z-coordinates” in span-cover space, and contoured. Figure 6 shows the contoured Hoek-Brown Stability Factor graph with hatched zones delineating low, moderate, and high relative stability for given span and cover conditions. Thereafter, each of the actual span and cover conditions measured within the grid cell quadrants were plotted on Figure 6.

3.4. Primary Stability Parameter Mapping and Parameter Stacking Model

The potential impact of each of the primary stability parameters was assessed for each grid cell quadrant within the study area. Based on this assessment, a relative

stability rating of low, moderate, or high was assigned to each grid cell quadrant according to the criteria described in Table 4. Maps were generated to show the relative stability ratings with respect to each of the primary parameters (See Figures 7 to 10).

To assess their combined impact, the relative stability maps developed for each of the primary parameters were “stacked” onto a composite map, as shown in Figure 11. Based on the “density” of the superimposed relative stabilities in a particular grid cell quadrant, a potential for mine roof deterioration rating was assigned as follows:

- (1) A low potential for mine roof deterioration was assigned to grid cell quadrants with four individual ratings of high relative stability.
- (2) A moderate potential for mine roof deterioration was assigned to grid cell quadrants with one rating of moderate relative stability and three ratings of high relative stability.
- (3) A high potential for mine roof deterioration was assigned to grid cell quadrants with two or more ratings of moderate relative stability or one or more ratings of low relative stability.

As shown on the parameter stacking model in Figure 11: (1) approximately 22 percent of the mine had a relatively high potential for mine roof deterioration, (2) approximately 24 percent of the mine had a relatively moderate potential for mine roof deterioration, and (3) approximately 54 percent of the mine had relatively low potential for mine roof deterioration. Figure 11 is now used as the primary tool when developing mine plans for access, abandonment, and mitigation at the Fort Hartford Mine Superfund Site.

4. INSTRUMENTATION AND MONITORING PLAN

In order to monitor the long-term mine roof stability at the Fort Hartford site, an instrumentation and monitoring plan was developed. The instrumentation and monitoring plan consisted of an annual surveillance program, installation and monitoring of borehole extensometers and installation of in-mine “drop-flag” monitors and crack monitors.

4.1. Annual Surveillance Program

The on-going annual surveillance program consists of reading the site instrumentation and performing in-mine inspections along the primary access routes. During the annual surveillance inspections, the mine roof is visually assessed for signs of instability and/or failure. In the past, findings from the annual surveillance program were used to develop remedial measures for roof control, as well for identifying particular areas of the mine that needed to be abandoned or restricted for usage due to roof instability.

During the annual surveillance performed between February 20, 2001 and March 8, 2001, indications of sinkhole development and ground surface movements were observed above the mine in the vicinity of borehole B-12. The location where the ground surface movements were occurring was above a previously identified roof fall in “G” Cut of the Caney Creek Lobe of the mine. Moreover, this area was previously assigned a high potential for mine roof deterioration based on the study performed in 1993. It was believed that overburden soils were migrating into the mine in this area through voids and fractures formed as a result of the previous roof collapse. The primary concern was that continued migration of soil into the mine would result in the formation of a sinkhole,

thereby allowing increased surface water flow into the mine. Currently, this area is under continued surveillance and remedial options are being considered.

4.2. Installation and Monitoring of Borehole Extensometers

An extensive site instrumentation program was initiated in October 1989 during the geotechnical investigation phase of the project and has been continued as part of the annual surveillance program. The instrumentation program included installation of thirteen Slope Indicator® multipoint stainless steel rod extensometers in the boreholes shown on Figure 4. The extensometers were installed so that the total and differential displacements of the primary stratigraphic units could be measured over time.

The extensometers were monitored extensively during the geotechnical investigation phase of the project, and they have since been monitored at least annually. Extensometer data is presented in Table 5. The displacements measured in the extensometers over time have been attributed primarily to environmental factors (i.e., temperature, rainfall, and groundwater effects in the Hardinsburg Sandstone). However, as shown in Table 5, there is an obvious trend of larger measured displacements corresponding to mine roof locations with high potential for mine roof deterioration (e.g. B-2 and B-5) and smaller measured displacements for areas with low potential for mine roof deterioration (e.g. B-4 and B-14). Figure 12 shows the time versus total displacement curves for the extensometers.

4.3. Installation of In-mine “Drop-Flag” Roof Monitors and Crack Monitors

Miner’s Helper drop-flag roof monitors were installed at critical locations inside the mine in order to warn personnel of real-time roof movements. The drop-flag roof monitors are single point extensometers combined with a drop-flag warning device. The

instruments were developed by Simplified Mining Instruments of Big Sandy, Texas. Localized differential movements can be measured in the Haney Limestone immediate roof beam using the *Miner's Helper*. Once a specified displacement has occurred, the *Miner's Helper* will eject a warning flag. Two *Miner's Helper* roof monitoring instruments have been installed in high usage areas with potentially unstable roof conditions. To date, no significant movements have been measured with the extensometer portion of the drop-flag roof monitors. Correspondingly, the drop-flags have not been activated.

In addition to the drop-flag warning devices, Avonguard “tell-tale” crack monitors were installed at selected locations inside the mine. Specifically, the crack monitors were installed to span across mine roof discontinuities, so that differential displacements across the discontinuities could be measured.

5. MITIGATIVE MEASURES RELATED TO MINE ROOF STABILITY

Several mitigative measures related to mine roof stability have been implemented at the Fort Hartford Mine Superfund Site since 1996. These mitigative measures included an extensive mine roof and rib scaling program, installation of a mine roof support system at the Caney Creek Lobe “A” Portal, relocation of the primary access road into the Caney Creek Lobe, and slope improvement at the ground surface above the Caney Creek Lobe “A” Portal. A brief description of these individual repairs follows.

5.1. Mine Roof Scaling Program

Mine roof scaling operations were conducted along the primary access routes within the mine between 1996 and 1998. Both manual and mechanical scaling was done

to remove potentially unstable roof and rib rocks/blocks. Figure 13 shows the limits of the scaling operations.

Where possible, the primary access routes within the mine were developed in areas which had low potential for mine roof deterioration. However, some areas that had moderate or high potential for mine roof deterioration could not be avoided due to the necessity to install pump stations in those locations or because alternate routes were blocked by salt-cake fines. In those locations, mine roofs were scaled extensively and roof inspections were/are performed by qualified personnel before entering the areas.

5.2. Caney Creek Lobe “A” Portal Mine Roof Support System

In April 1998, unstable roof conditions were exposed in the primary entryway to the Caney Creek Lobe of the mine, in the area known as the “A” Portal. As shown on Figure 13, the “A” Portal is an area which had a high potential for mine roof deterioration. Factors believed to have contributed to the unstable roof conditions at “A” Portal included: (1) relatively low cover conditions which were accompanied by low horizontal stresses, (2) weathered soil and rock above the immediate mine roof, (3) proximity to faulting, (4) vertical and cross jointing in the Haney Limestone mine roof, and (5) possible over-blasting during construction of the portal which may have damaged the lamination between the individual roof layers.

Between May and July 1998, a roof support system was designed and built to support the potentially unstable roof in the “A” Portal. The roof support system consisted of the following:

- (1) Four each 1.2-meter (4-foot) diameter, cast-in-place concrete columns were constructed to provide fixed-end type roof support (See Figure 14). The concrete

columns were approximately 6.6 meters (21.5 feet) in height and were supported by 2.4-meter by 2.4-meter (8-foot by 8-foot) concrete column pads.

- (2) Seventy-eight resin-anchored roof bolts were installed in the Haney Limestone mine roof to create a monolithic beam to span between the cast-in-place columns and to support the weak overburden materials (See Figure 15). Specifically, three individual roof beds ranging in thickness from about 0.7 meters to about 1.2 meters were bolted together to create one roof beam approximately 2.5 meters in thickness. Additionally, the perimeter of the roof bolt grid was “strapped” using C 200 x 17.1 (C 8 x 11.5) channel sections.

5.3. Caney Creek Lobe Primary Roadway Relocation

No additional roof stability problems have been experienced within the repaired section of “A” Portal. However, as a result of additional roof collapses in by the “A” Portal repair limits, the primary roadway through “A” Cut was abandoned and a new roadway was constructed in the location shown on Figure 13. It should be noted that these continued collapses occurred in areas that had a high potential for mine roof deterioration.

5.4. Slope Improvement above the Caney Creek Lobe “A” Portal

In December 2001, a landslide occurred above “A” Portal, thereby blocking access to the Caney Creek Lobe of the mine (See Figure 15). As shown in Figure 15, the landslide crushed the steel canopy at “A” Portal. Although the landslide was not directly related to the mine roof stability per se, it was a function of the complex geologic conditions previously discussed. Further, the landslide presented an obvious safety hazard for maintenance personnel.

In order to reestablish the entrance to the Caney Creek Lobe of the mine and to reduce the potential for future landslides, the slope above “A” Portal was graded and drainage improvements were made. Loose, saturated soil and rock materials were excavated from the slope and a bench was constructed. Moreover, a ditch was constructed on the bench to divert surface water off of the immediate slope above the “A” Portal. Manual scaling was done to remove loose rock from above the portal area. Figure 16 shows the repaired slope and “A” Portal canopy as of August 2002.

6. CONCLUSIONS

Mine roof instability at the Fort Hartford Mine Superfund Site has a number of potentially detrimental consequences including: (1) risks to monitoring/maintenance personnel working inside the mine, (2) ground surface subsidence effects related to roof collapses and breakthroughs, (3) roof collapses leading to the escape of potentially hazardous gases from within mine to the surrounding area, and (4) contamination of the local groundwater system. A study was performed in 1993 to delineate areas in the mine with low, moderate, and high potential for mine roof deterioration. The study was performed using map overlaying techniques, whereby the combined impact of key parameters with respect to mine roof stability were evaluated. According to the study: 22 percent of the mine area had a high potential for mine roof deterioration, 24 percent of the mine area had a moderate potential for mine roof deterioration, and 54 percent of the mine area had a low potential for mine roof deterioration.

Mine roof stability has been monitored at the site for the past ten years using both mechanical instrumentation (i.e., borehole extensometers and drop-flag monitors) and

visual inspection. Borehole extensometer data shows that the instruments with the largest observed displacements (i.e., 4.47 centimeters in B-5 and 4.14 centimeters in B-2) correspond to areas with high potential for mine roof deterioration, and that an instrument with one of the least observed displacements (i.e., 0.23 centimeters in B-4) is anchored into an area with low potential for mine roof deterioration. Moreover, the findings of the visual inspections show that significant cases of roof instability have occurred in areas with high potential for mine roof deterioration (e.g., “A” Portal roof collapses, and G-Cut roof fall and associated subsidence), whereas no notable cases of roof instability have occurred in areas with low potential for mine roof deterioration. Based on the monitoring observations and additional findings showing consistent correlations between the 1993 study results and actual field conditions, the mine roof stability model has been recognized as a reliable tool for developing in-mine transportation plans and mitigative measures.

Intensive roof and rib scaling has been performed along the primary access ways, and mitigative measures have been implemented to repair unstable roof at several locations that had high potential for mine roof deterioration. The mitigated areas constitute only a small portion of the total area that exhibits moderate to high potential for mine roof deterioration. As such, continued mine roof maintenance and monitoring will occur at the site, and future mitigative measures will be implemented on an as need basis.

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APPENDIX

Table 1. Primary Stratigraphic Formations/Members

Formation	Symbol	Thickness (m)	Description
Tar Springs Sandstone ⁽¹⁾	Mts	21.2	Fairly competent sandstone strata
Glen Dean Limestone	Mgd	11.6-16.6	Fairly competent limestone strata
Hardinsburg Sandstone	Mh	6.6-10.8	Relatively weak shale with interbedded lenses of limestone, sandstone, siltstone, and claystone
Haney Limestone	Mgh	9.8-13.0	Subhorizontally bedded competent limestone strata which forms the mine itself
Big Clifty Sandstone	Mgbc	16.7-21.8	Massive, competent sandstone strata that typically forms the mine floor

- (1) Only one boring was cored through the total thickness of the Tar Springs Sandstone.

Table 2. Phases Model Cross-Section Geometries

Cover (m)	Stratigraphy	Spans (m)			
6	6 m Mh	9	15	21	25
10	10 m Mh	9	15	21	25
20	10 m Mh 10 m Mgd & Mts	9	15	21	25
30	10 m Mh 20 m Mgd & Mts	9	15	21	25
48	10 m Mh 38 m Mgd & Mts	9	15	21	25
60	10 m Mh 50 m Mgd & Mts	9	15	21	25

Table 3. Material Properties

Property	Haney Limestone	Hardinsburg Shale	Glen Dean & Tar Springs
Unit Weight, γ , (MN/m ³)	0.027	0.022	0.0245
Young's Modulus, E , (MPa)	60000	4500	12000
Poisson's Ratio, ν	0.29	0.26	0.29
Compressive Strength, q_u , (MPa)	50	30	50
Hoek-Brown, m -parameter	0.7	0.05	0.3
Hoek-Brown, s -parameter	0.004	0.00001	0.0001

Table 4. Relative Stability Assignments for Grid Cell Quadrants

Primary Parameter(s)	Relative Stability Assignment		
	Low	Moderate	High
Span & Cover ⁽¹⁾	Mean $RWSF \leq 1.10$ (Low Cover) Mean $RWSF \leq 1.17$ (High Cover)	$1.10 < \text{Mean } RWSF < 1.17$ (For Cover < 20 m)	$RWSF > 1.17$
Mine Roof Thickness, t ⁽²⁾	$t \leq 1.5$ m	$1.5 \text{ m} < t < 2.1$ m	$t \geq 2.1$ m
Proximity to Fractures ⁽³⁾	2 or more fractures in a grid cell quadrant	1 fracture in a grid cell quadrant	No fractures in a grid cell quadrant
Proximity to Faults ⁽⁴⁾	2 or more faults or 1 primary fault in a grid cell quadrant	1 subsidiary fault in a grid cell quadrant	No faults in a grid cell quadrant

(1) Based on Hoek-Brown Mean Weighted Stability Factor contours shown in Figure 6.

(2) Based on correlations with previous roof collapses.

(3) Based on fracture trace study provided in the 1993 Ogden report.

(4) Based on field observations and mapping.

Table 5. Extensometer Data.

Borehole No.	Depth to Mine Roof (meters)	Potential for Mine Roof Deterioration	Total Displacement (cm)	Differential Displacement (cm)
B-1	15.0	Moderate	0.16	0.07
B-2	11.4	High	4.14	1.07
B-3	26.1	Moderate	0.71	0.15
B-4	40.0	Low	0.23	0.14
B-5 ⁽¹⁾	6.9	High	4.47	N/A
B-7	30.4	Moderate	0.77	0.41
B-8	19.6	High	0.59	0.19
B-11	17.1	High	0.21	0.03
B-12	22.3	High	1.07	0.21
B-13	22.2	Low	1.25	0.03
B-14	26.0	Low	0.56	0.02
B-15	30.0	High	1.81	0.13
B-16	17.1	Moderate	0.35	0.11

(1) Only one anchor was installed in B-5, therefore differential displacements are not applicable.

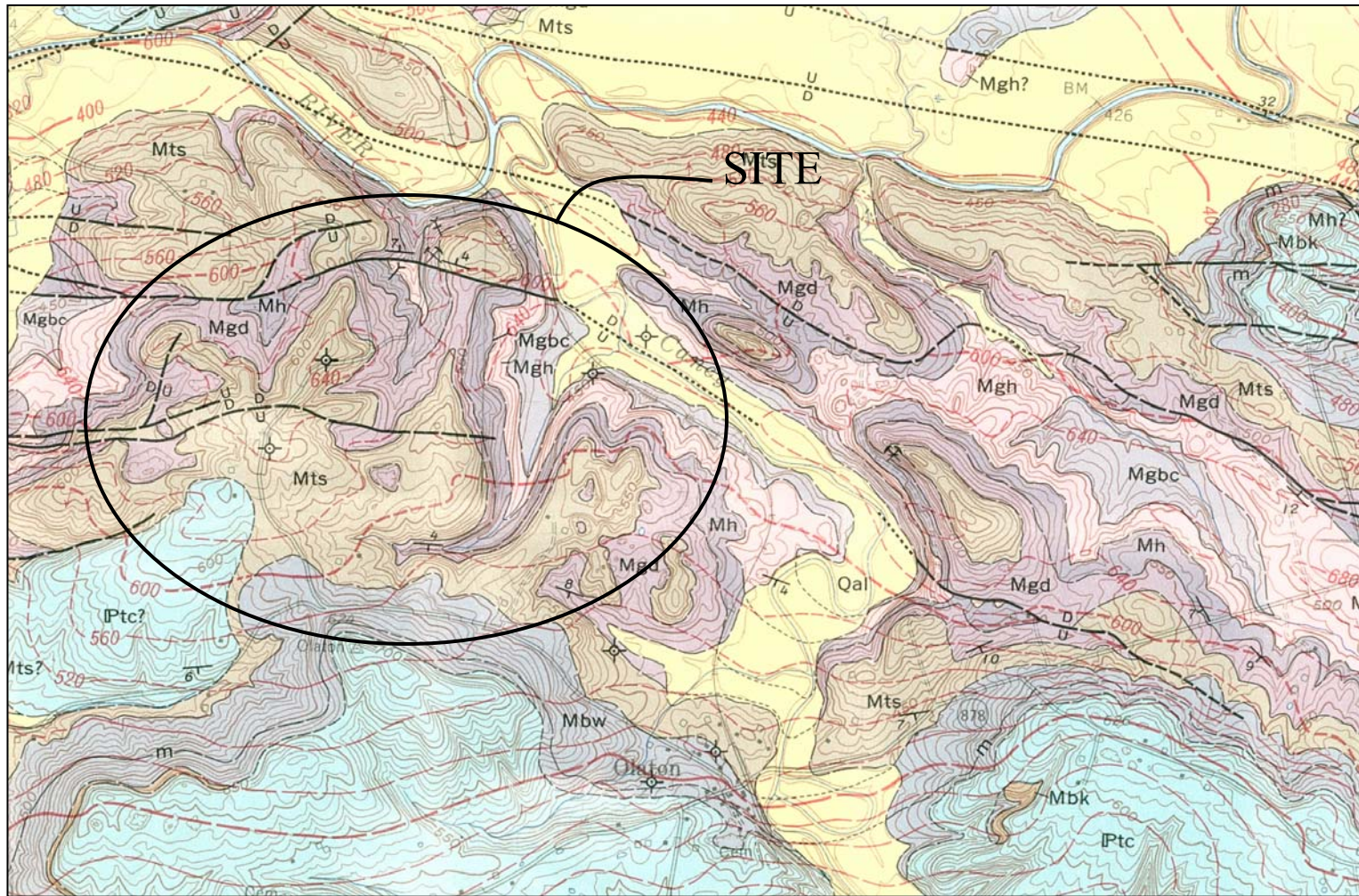


Figure 1. Geologic Map of Fort Hartford Mine Superfund Site (USGS Geologic Quadrangle; Olaton, Kentucky; 1968)

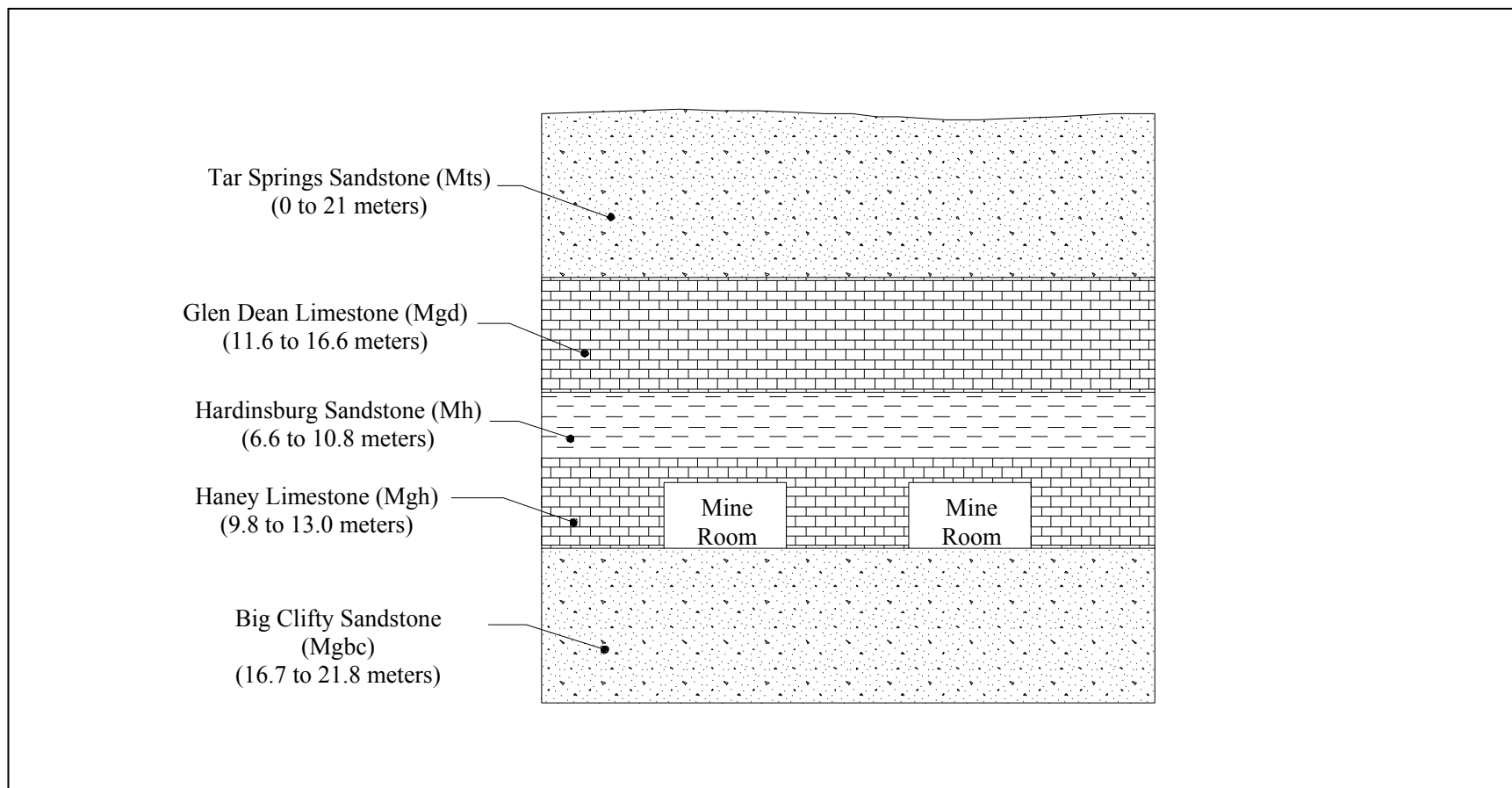


Figure 2. Typical Cross-Section of Fort Hartford Mine Superfund Site Showing Stratigraphy

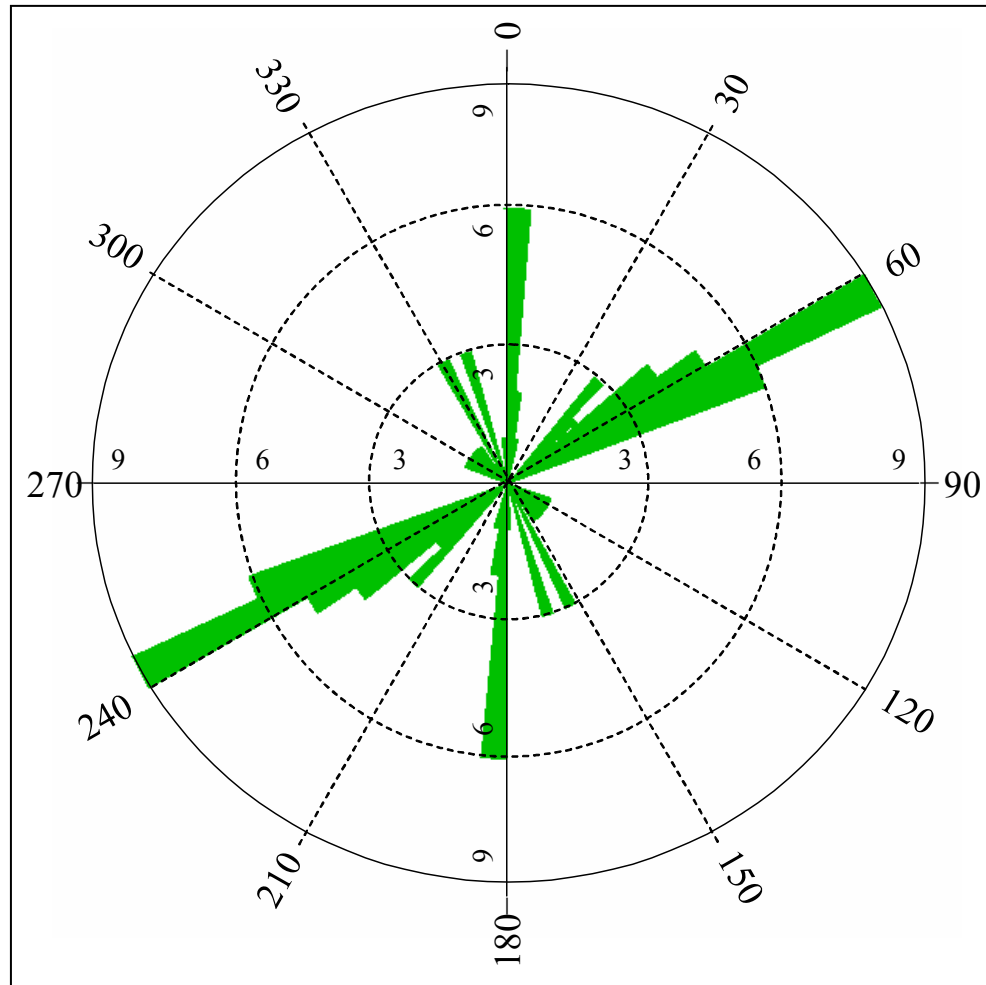


Figure 3. Strike Rose Diagram of Fort Hartford Mine Superfund Site Joint Sets (Based on 52 Data Points)

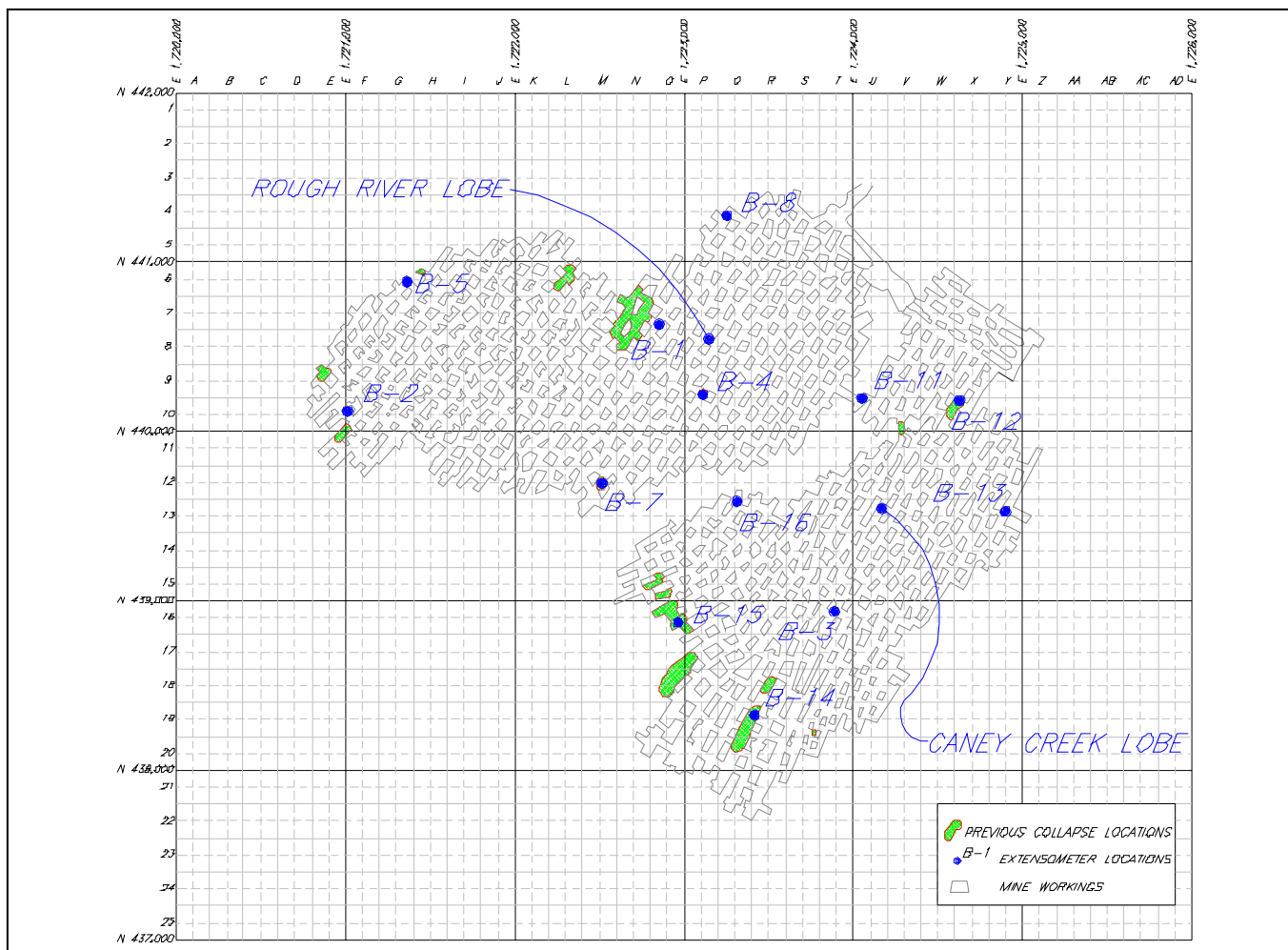


Figure 4. Fort Hartford Mine Superfund Site Base Map with Alphanumeric Grid Overlay and Borehole Locations

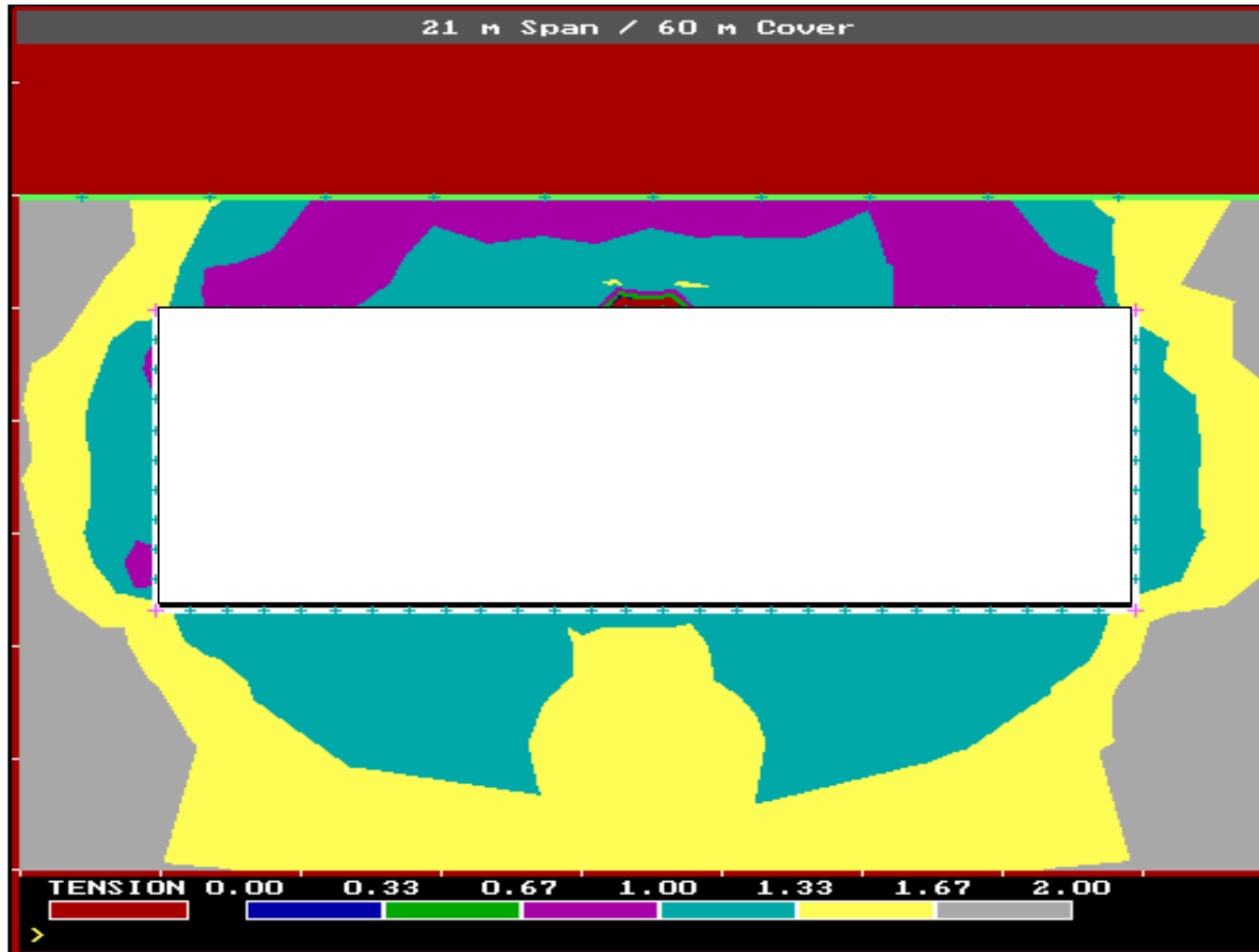
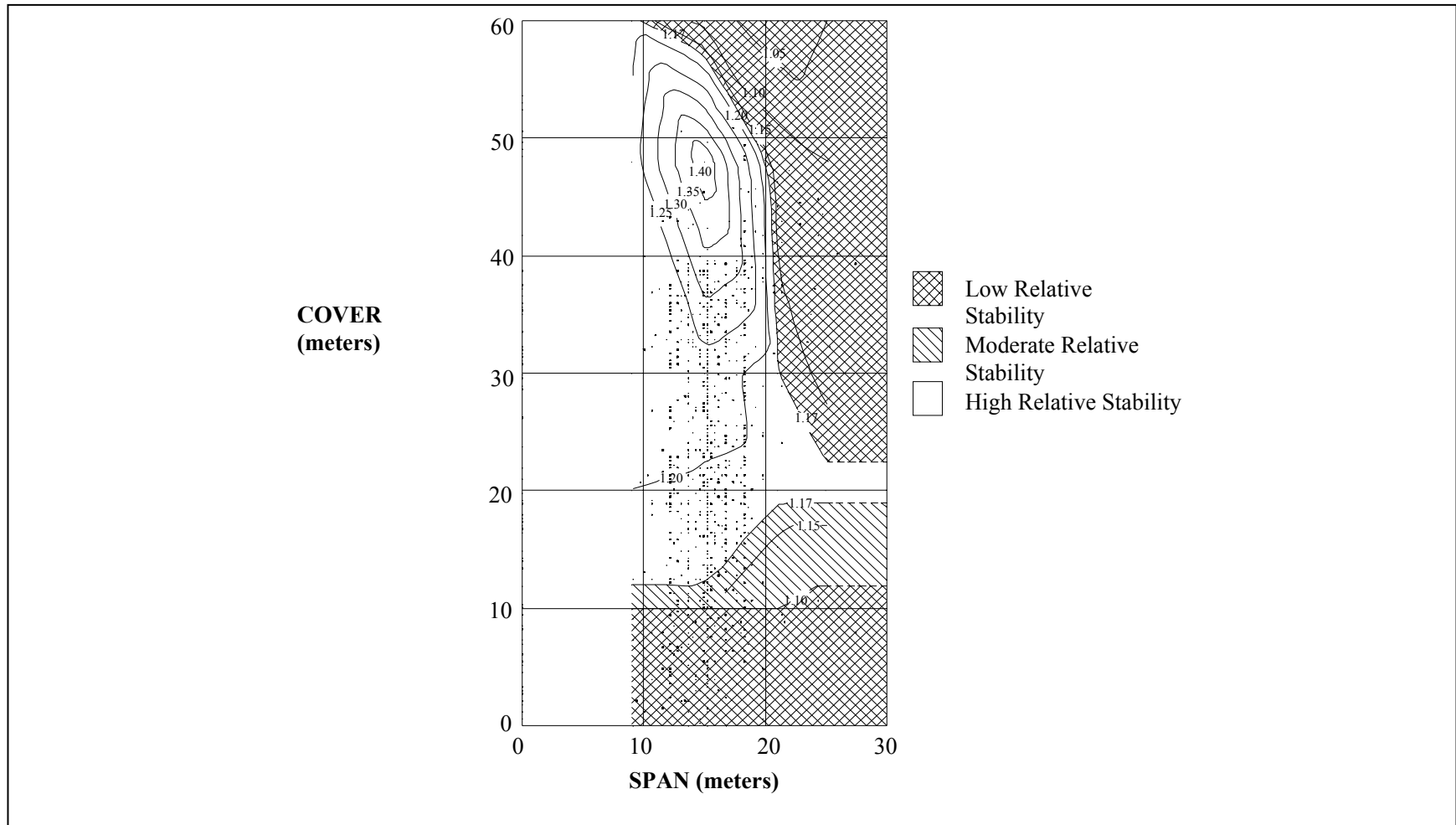


Figure 5. *Phases* Plot showing Hoek-Brown Stability Factor Contours



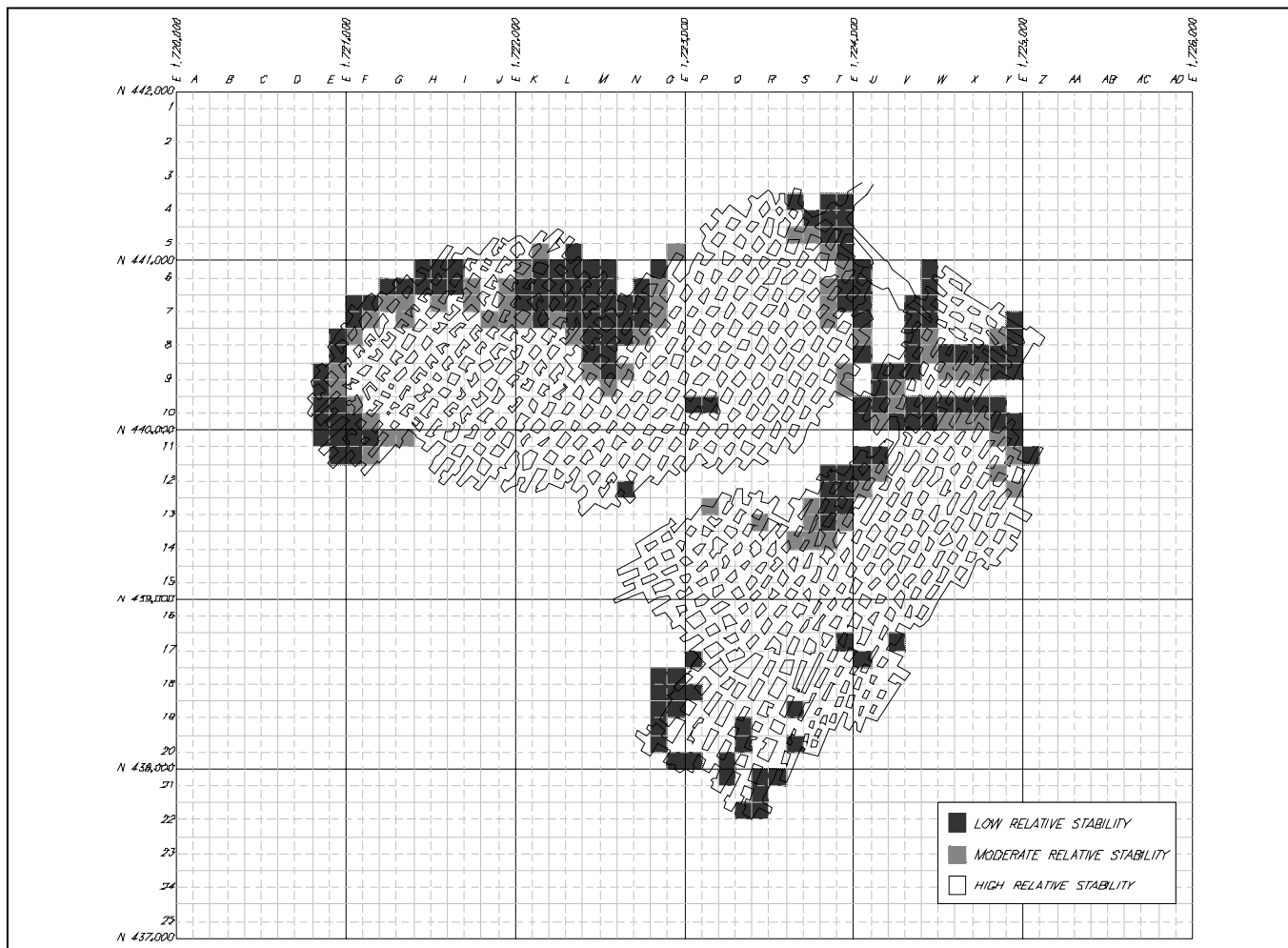


Figure 7. Fort Hartford Mine Superfund Site Relative Stability Map Based on Span and Cover

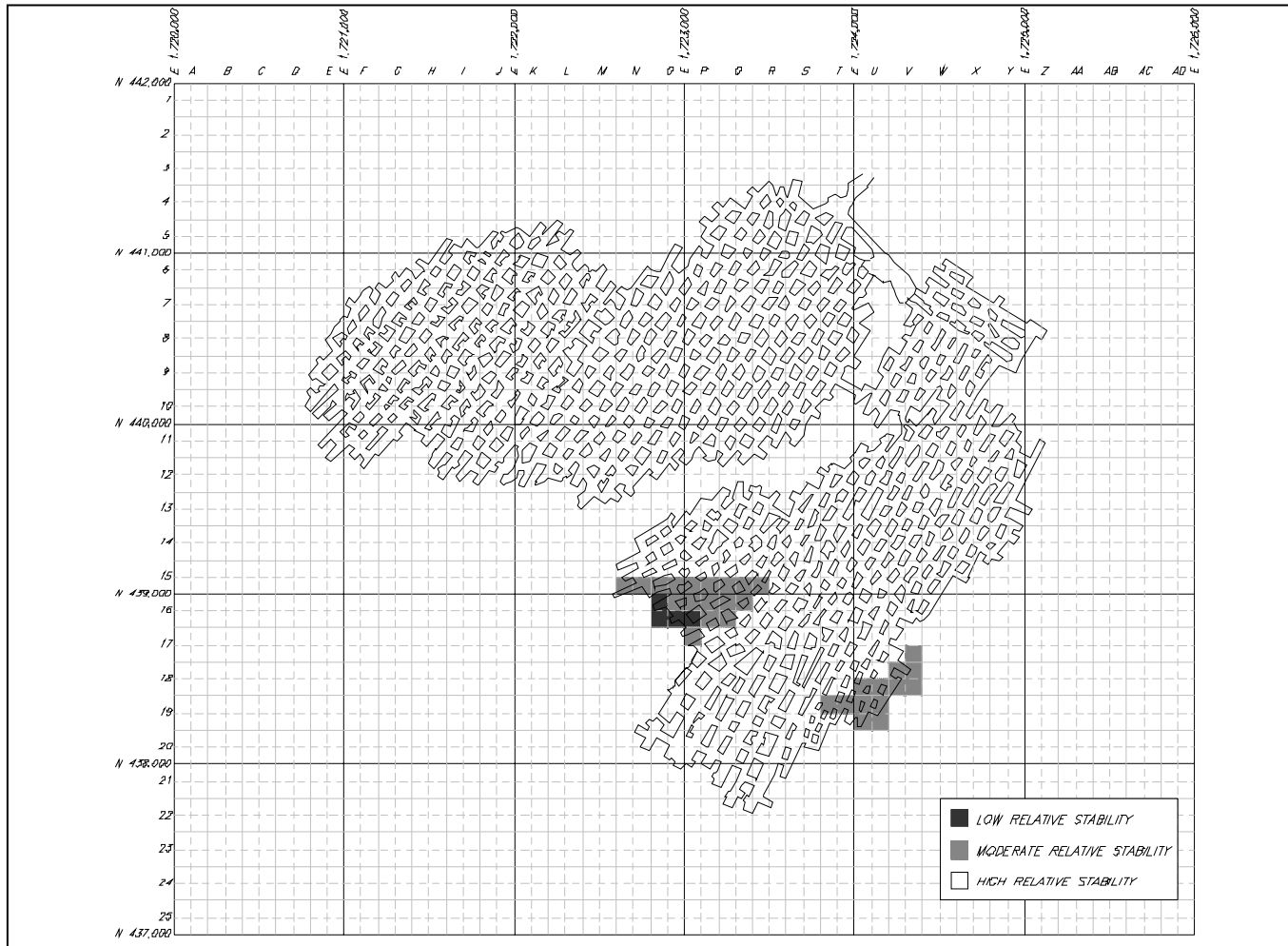


Figure 8. Fort Hartford Mine Superfund Site Relative Stability Map Based on Mine Roof Thickness

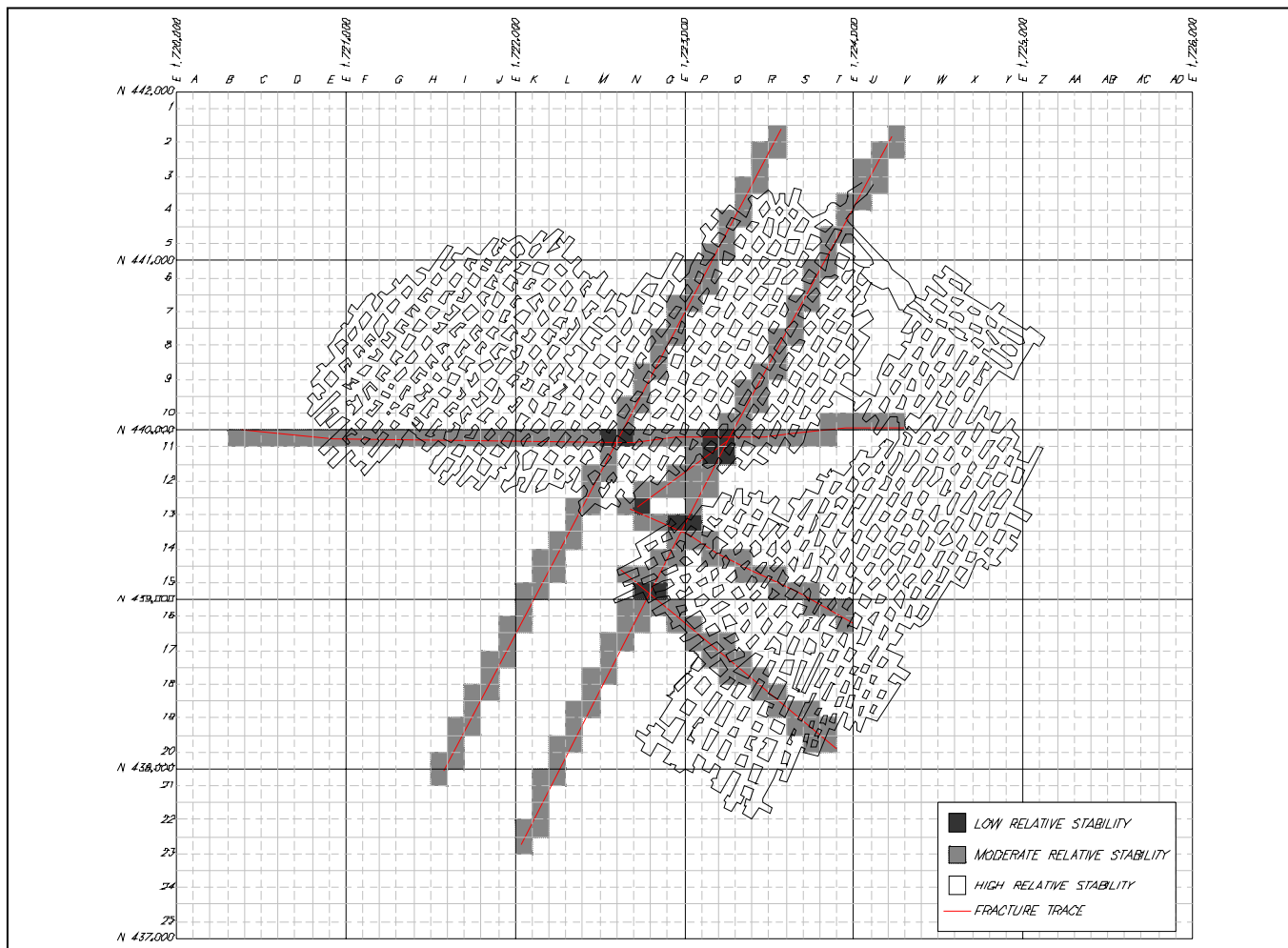


Figure 9. Fort Hartford Mine Superfund Site Relative Stability Map Based on Proximity to Fracture Traces

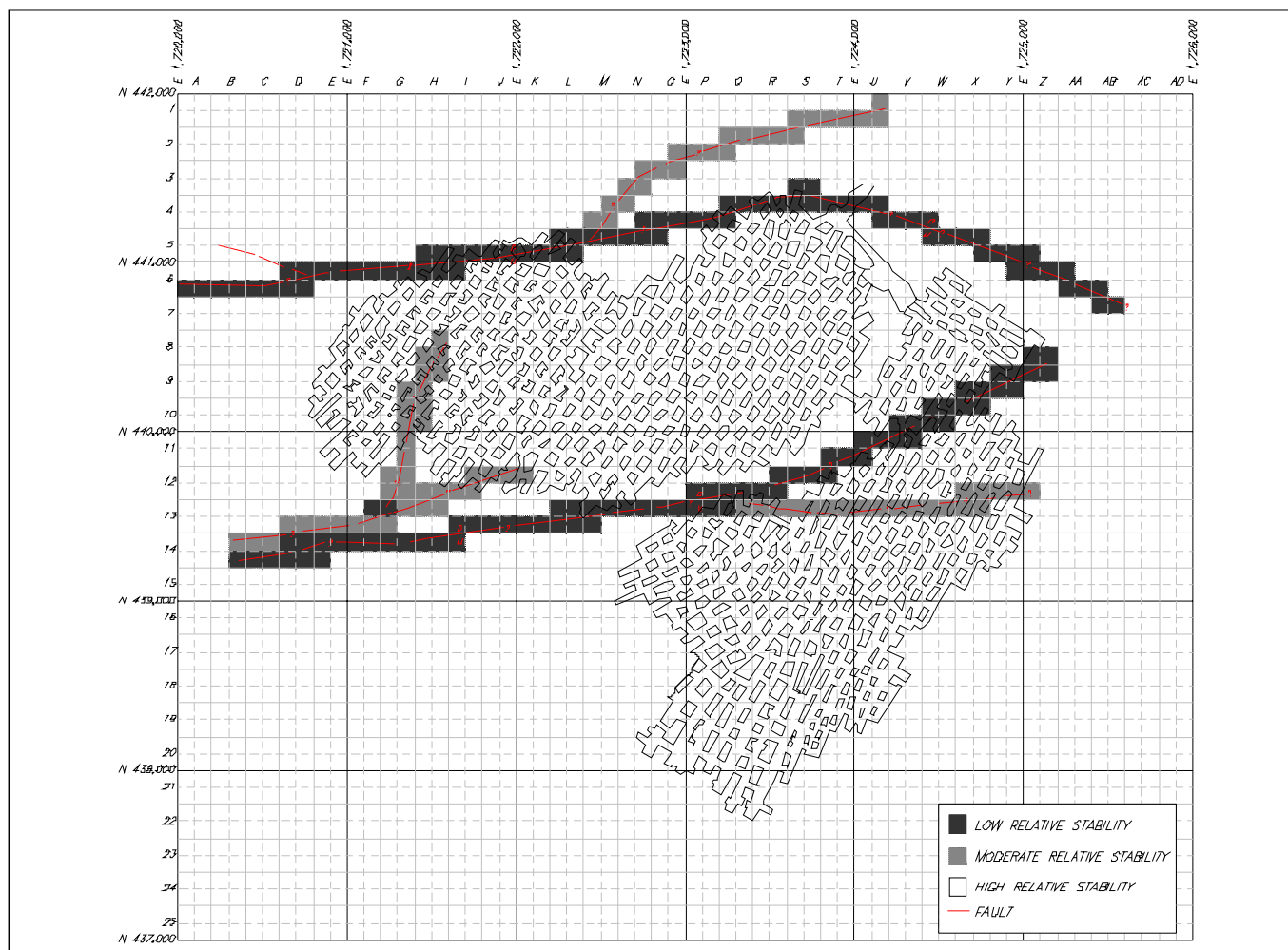


Figure 10. Fort Hartford Mine Superfund Site Relative Stability Map Based on Proximity to Faults

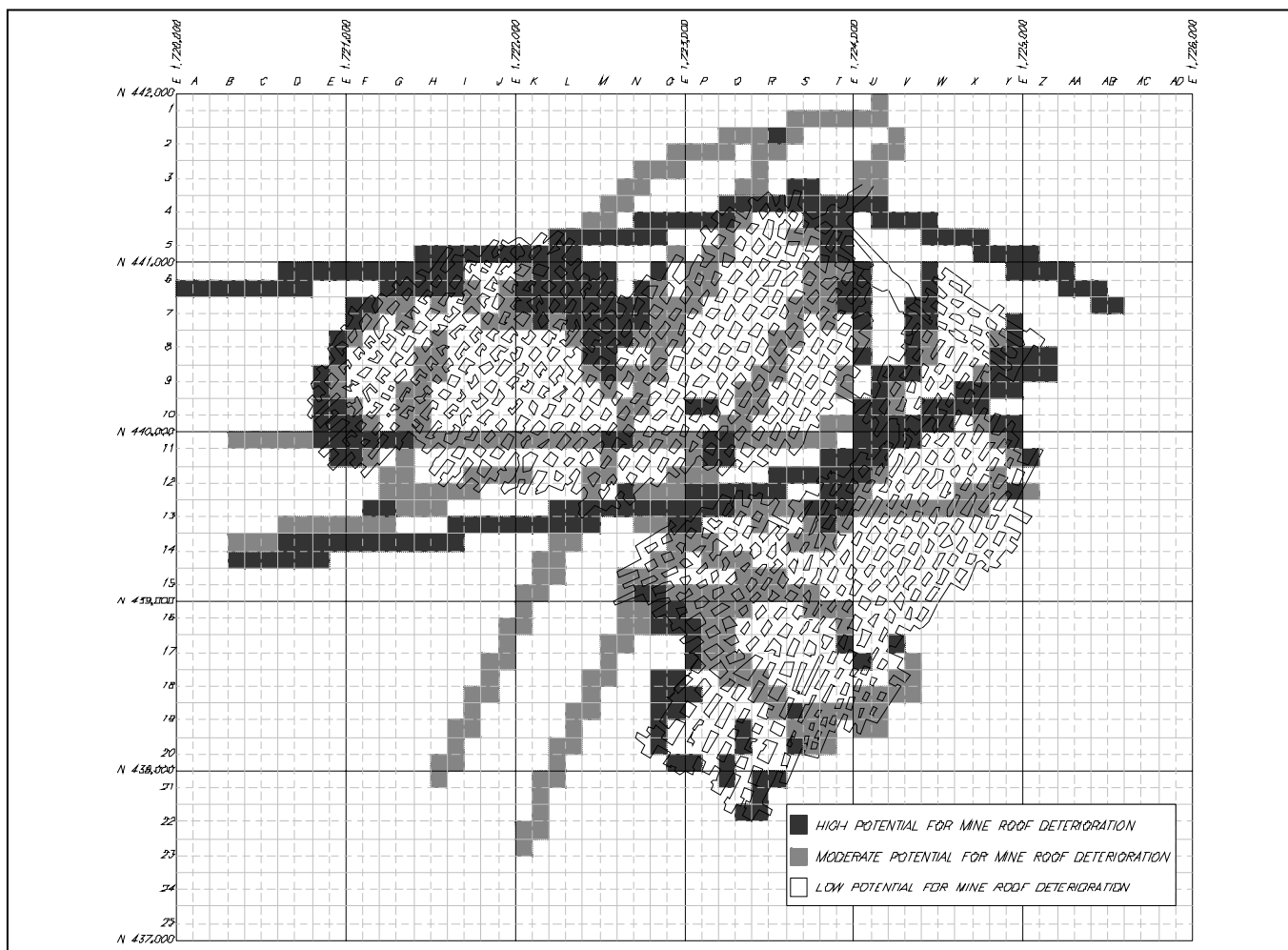


Figure 11. Fort Hartford Mine Superfund Site Composite Map Showing Potential for Mine Roof Deterioration

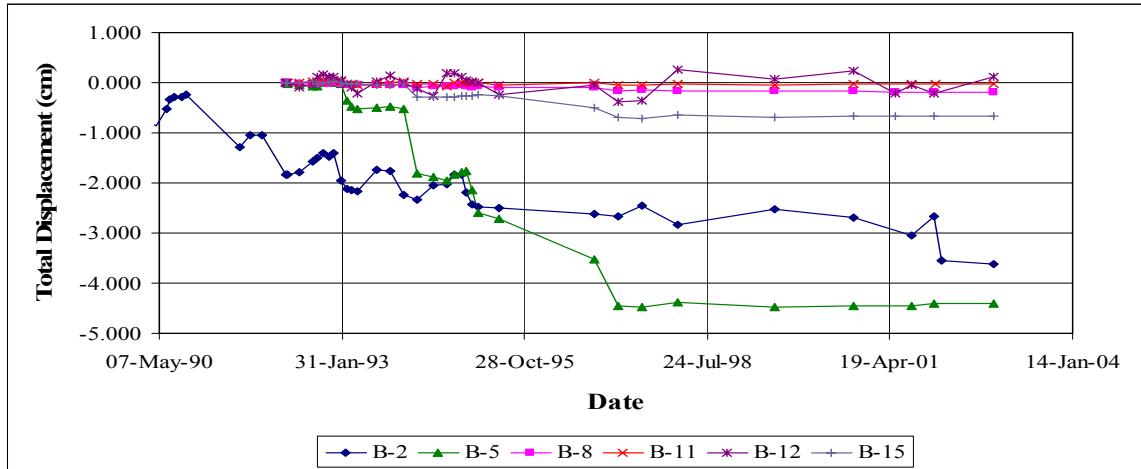


Figure 12a. Extensometers in areas with high potential for mine roof deterioration.

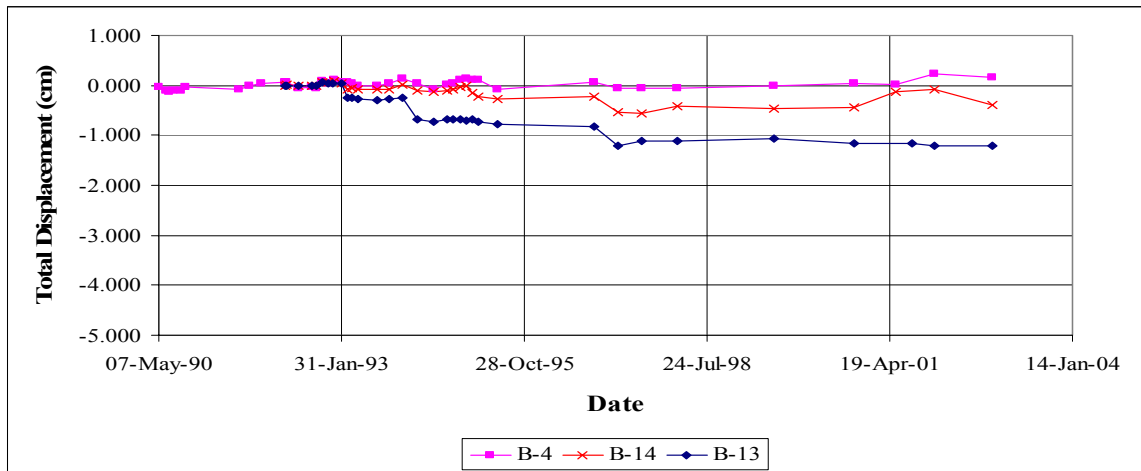


Figure 12b. Extensometers in areas with moderate potential for mine roof deterioration.

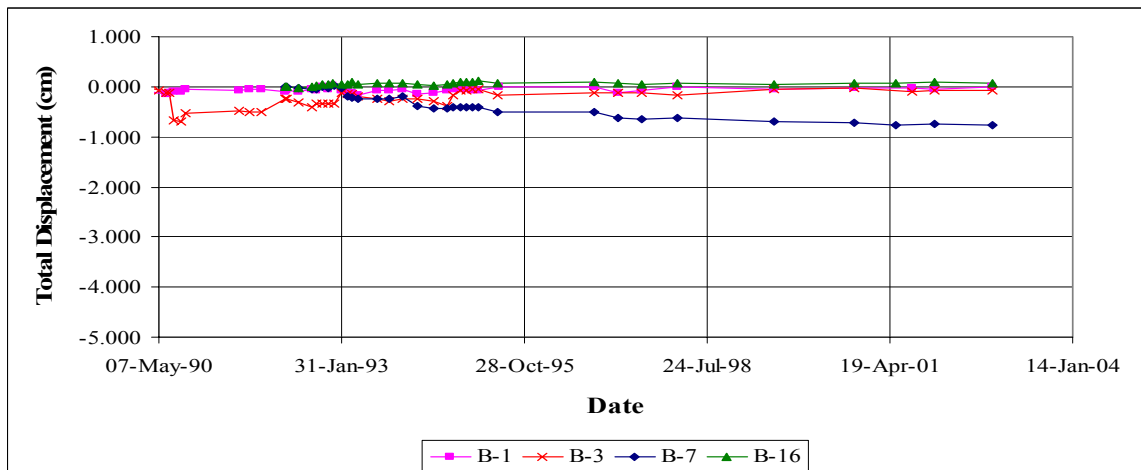


Figure 12c. Extensometers in areas with low potential for mine roof deterioration.

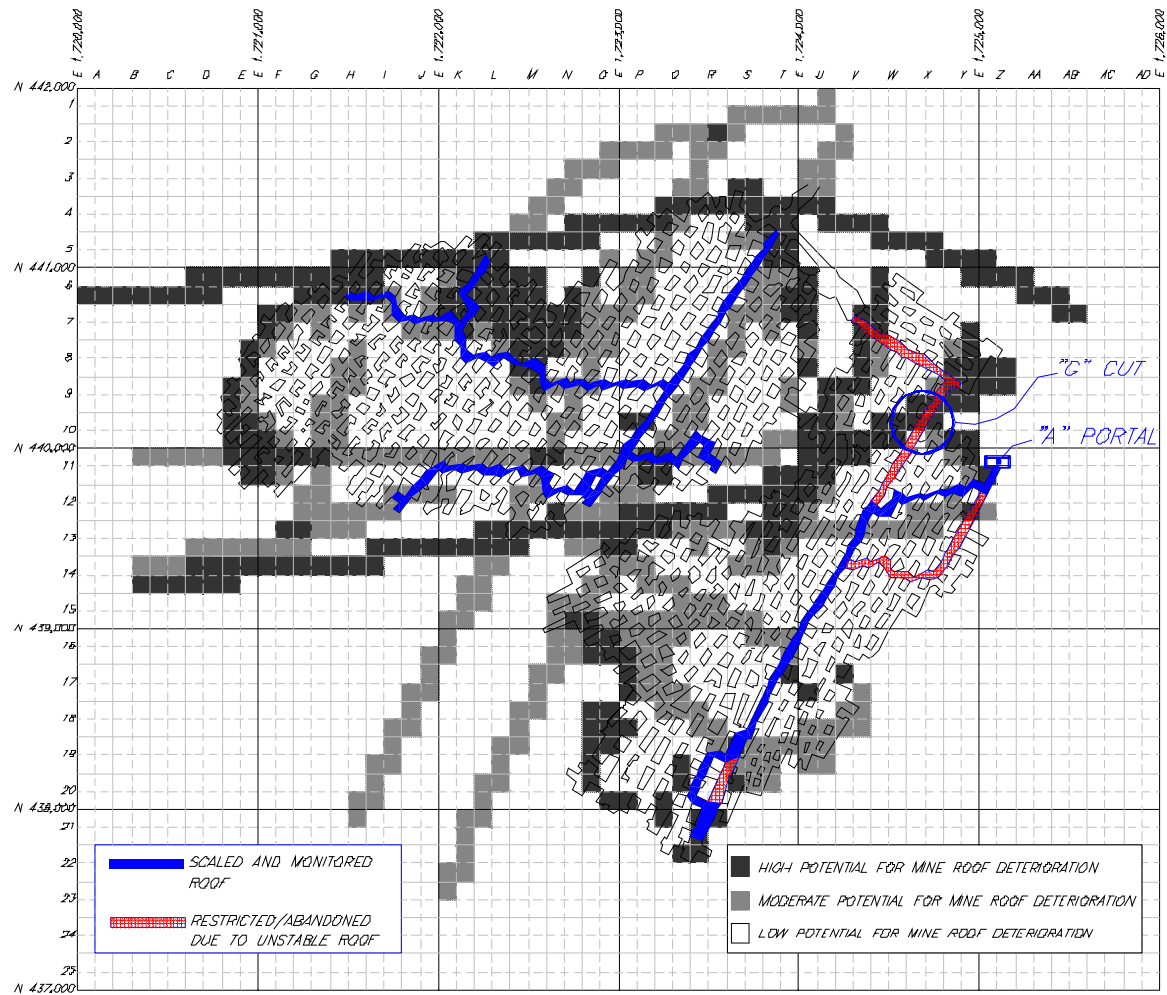


Figure 13. Fort Hartford Mine Superfund Site Mine Roof Scaling and Mitigation Location Map



Figure 14. Fort Hartford Mine Superfund Site Cast-In-Place Concrete Columns at “A” Portal



Figure 15. Fort Hartford Mine Superfund Site Installation of Roof Bolts at “A” Portal



Figure 16. Fort Hartford Mine Superfund Site “A” Portal Landslide



Figure 17. Fort Hartford Mine Superfund Site Repaired Slope at “A” Portal

VITA

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