HIGH CAPACITY MICROPILES IN MINED GROUND FOR BRIDGE SUPPORT: A CASE HISTORY OF SITE INVESTIGATION, GROUND CHARACTERIZATION AND THE EVOLUTION OF FOUNDATION DESIGN AND TREATMENT CONCEPTS

Norman I. Norrish[1], John Szturo[2], Marcelo Chuaqui[3],

ABSTRACT

The Missouri Department of Transportation is completing the Route 249 – US 171 interchange on an abandoned underground lead-zinc mining site near Joplin, Missouri. The interchange required the construction of 5 major bridges connecting massive earth embankments and involved the construction of 23 bents, 3 culverts and 3 MSE walls.

The geotechnical characteristics of the project site are the product of the original limestone and chert bedrock geology, the tectonic processes leading to brecciation, folding and paleo karst, and the anthropogenic (manmade) activities associated with mining. These processes resulted in a site that exhibited critical geotechnical attributes:

- Highly variable rock parameters (strength, deformability and hydraulic conductivity)
- Very low level of predictability or correlation between subsurface explorations
- Locally modified hydrologic regime due to the mining disturbance
- Presence of residual mining voids in both the upper “confused” zone and the lower “sheet” zone
- Potential for ground loss beneath foundation elements
- Presence of heterogeneous mine fillings including compressible clay
- Presence of vertical shafts with random and partial filling

These attributes led the design team to the selection of a foundation design concept that required bent specific geologic characterization based on subsurface exploration and geotechnical testing; pre design utilizing stress analyses and other analytical approaches to predict the interaction of ground quality, voids and imposed loading; systematic confirmatory exploration during construction; and ground treatment and foundation design modifications as required by the confirmatory site engineering work. The basic premise of this approach was that there were no foreseeable ground conditions at the project site that could not be adequately improved to provide foundation support for the proposed structures.

---

[1] Principal, Wyllie & Norrish Rock Engineers Inc. 17918 NE 27th St, Redmond, WA 98052 Ph 425.861.7327, Fax 425.861.7327, nnorrish@msn.com

[2] Senior Geologist, HNTB Corporation 715 Kirk Drive Kansas City, MO 64105 Ph 816.527.2275, Fax 816.472.5013, JSZTURO@HNTB.com

[3] General Manager, Monir Precision Monitoring Inc. 2359 Royal Windsor Drive Unit 25 Mississauga, ON L5J 4S9 Ph 905.822.0090, Fax 905.822.0044, marcelo@monir.ca

This paper describes the historical research, site investigation and design methodologies leading to the selection of spread footings at 7 of the bents and the 220 micropiles (827-1,891 kN DWL) used to provide deep foundations for 16 of the bents. Pregrouting of the rock mass involved 7,000 m^3 of LMG and 350 m^3 HMG and was conducted to confirm design assumptions at all bents and as a site preparation for those bents to be micropiled. A separate program of mine shaft location and treatment was also required.

PROJECT DESIGN ELEMENTS

The Missouri Department of Transportation (MoDOT) is constructing a five bridge interchange in Jasper County, Missouri connecting Route 249 and US 171. HNTB with project team sub consultants Geosystems, L.P., Monir Precision Monitoring Inc. and Wyllie & Norrish Rock Engineers Inc. was retained by MoDOT to provide geotechnical exploration assistance as well as design of all bridges, retaining walls and major box culvert structures for the entire interchange.

Figure 1 shows the bridges, retaining walls and box culverts locations for the interchange project. The structure numbers of the five bridges are A6140, A6148, A6149, A6150 and A6165. Bridge A6140 is along Route 249 North Bound Lane (NBL) and spans the MNA Railroad. Bridge A6148 is along Route 249 South Bound Lane (SBL) and spans the MNA Railroad. Bridge A6165 is along Route 249 SBL and spans EB Bus. 171. Bridge A6149 is along Ramp #3 and spans EB Bus 171, SBL and the MNA Railroad. Bridge A6150 is along Ramp #4 and spans the MNA Railroad. The bridges range from two spans to eight spans with lengths ranging from 67 m to 444.2 m. Approach fill heights at the bridge locations range from 7 to 21.5 m. The three Mechanically Stabilized Earth (MSE) walls are located at End Bent 1 of Bridge A6149 (Wall A7265), and End Bent 1 and End Bent 3 of Bridge A6165 (Walls A7263 and A7264). Wall heights range from 6.5 to 8 m. The three box culverts will carry the existing Mine Branch Creek through embankments at the south end of the project. Box structure numbers are A7260, A7261 and A7262. Embankment heights over the boxes range from 13 to 23 m.

SITE HISTORY

The Tri-State mining district, so named for its location at the junction of Missouri, Kansas and Oklahoma, was formerly one of the largest lead and zinc producing districts in the world. Major minerals mined were sphalerite (zinc sulfide) and galena (lead sulfide). Mining began in the project area in the 1850's with excavation of the shallow "upper ground" deposits of insoluble residual minerals in the upper clay or in incompetent zones of broken, unconsolidated chert. These deposits were dug by hand in typically small claims of 30 to 60 square meters. Mines were dug as deep as practical by hand; typically the limiting depth was the groundwater table, usually 15 meters or less below the surface. Many shafts were sunk at close intervals due to the unstable nature of the upper ground and the broken rock that prohibited drifting or horizontal mining. The gangue material was randomly placed as backfill in the mined areas. Ore was crudely smelted on site with wood fires.

In the period near 1900, advances in mining along with capitalization brought mechanization, milling, hoisting, explosives and pumps with the ability to dewater the mines and lower the groundwater. Shafts were deepened and horizontal drifts began in the lower more competent rock layers. This began the era of large scale company mining on larger tracts of ground. The deeper shafts began mining veins or runs of the larger, extensive, lower grade "sheet-ground" ore bodies. Horizontal mining, or drifts, were highly irregular in size, shape and elevation, as the excavation followed the irregular shaped ore deposits. Only the economically feasible ground was mined. Mining references suggest a few of the openings may have been as large as 18 to 21 meters. More common openings found in the project explorations size the openings in the sheet ground at 1 to 2 meters or less. The sheet ground deposits were mined by irregular pattern room and pillar methods.

Mining flourished during the World War I period in the Webb City Carterville Area. The Cornfield Mine associated with the project was thought to have been worked during the period 1910 – 1920. At the end of World War I mineral prices declined and operations ceased in this area with much of the equipment moved to the richer, more profitable Pitcher Field of Oklahoma. Lead and zinc were again in demand during World War II and, while not documented at this location, it was typical practice for a small mining operation to reopen some of the mines and scavenge any readily available ore. Another common practice was to rob the pillars supporting the mine. It was also during this period the vast mine tailings piles "chat" were reworked for additional recovery of minerals.

After the mines stopped operating, they filled with groundwater and are believed to be filled with water today. Also, since the closure of the mines, the enormous tailings piles have been transported off site for use as aggregate and mineral fillers. The Environmental Protection Agency has designated the tailings as a hazard due to the presence of heavy metal particles. Special material handling and encapsulation will be required for the proposed highway project. Present day evidence of mining on the right of way for the proposed project include, chat piles, mine shafts closed by Missouri Department of Natural Resources, and occasional surface depressions.

GEOLOGIC SETTING

The project area is situated within the Ozark Plateau physiographic province, a gently uplifted plateau of nearly horizontal sedimentary rocks. As the area is on the far west flank of the Ozark Dome, the dip is gently to the west – northwest at about 3 meters per kilometer. The plateau has been eroded to form a topography of rolling hills.

Structurally, the area is controlled by northwest – southeast trending Joplin Anticline and parallel east adjacent Webb City Syncline. References indicate the mineralization of the area appears to be confined to the synclinal areas.

Bedrock is of the Lower Pennsylvanian and Mississippian Age (Figure 2). The lowermost rock is the Reeds Springs Formation composed of nearly equal parts of chert and limestone. The chert is bluish to tan, nodular and irregularly bedded. Chert can make up one third to two thirds of the formation. The formation averages 30 to 45 meters thick in the project area. While not included in the modern nomenclature, the Grand Falls Chert Member, or Elsey Formation is usually included in the Reeds Springs formation, but this rock layer is prominent in the study area and is composed wholly or heavily predominant beds of chert. Above the Reeds Springs is the Burlington – Keokuk Formation, 20 to 30 meters of coarsely crystalline limestone with layers and nodules of chert common. The Short Creek Member is found near the top of the Burlington Keokuk Formation. The Short Creek is a persistent 1 to 3 meter thick layer of oolitic limestone. The undifferentiated Pennsylvanian Carterville and Cherokee Member Shale found in the study area is the result of paleo karst activity. The shale and sometimes sandstone and coal material is found occupying depressions in the Mississippian formations, the result of deep seated solutioning and collapse.

The predominant controlling feature of the geology of the site is the brecciation of the bedrock and the "Cornfield Bar" (Figure 3). The basal breccias are the "confused" or "broken" ground and consist of broken, angular chert lying on the slopes and bottom of the formerly solutioned, collapsed valleys. The chert is the residual component of the solutioned cherty limestone. It is within this porous, confused ground zone that most of the mineralization of the area has occurred. Areas of confused ground can extend nearly throughout the rock column of the project area, from the bedrock surface to over 35 meters deep near the top of the Grand Falls Chert Member.

In addition, many areas of brecciated, intact rock were noted in the borings. These areas are known as "sheet" breccias and are characterized as chert crushed in place and recemented by chert. The brittle chert layers were broken in place by horizontal stresses while the more elastic limestone escaped brecciation. The stresses occurred as a result of minor faulting, solution adjustment, warping, and horizontal thrust. During mineralization, the chert was re-cemented.

The "Cornfield Bar" feature has a large influence on the project, controlling the location of the broken and confused ground as well as location of the shale bedrock. The confused ground reaches nearly down to the sheet ground in the area of the Bar, so called because it is barren of mineralization. The width of the bar varies from 15 to 90 meters, with the location of the bar being in direct relationship to the location of the Cherokee Shale.

The synclinal bar is characterized by flanking bedrock dipping into the trough. The trough filled with a confused mixture of limestone and chert mixed with weathered and broken shale. All this lies unconformably on the broken sheet ground. The bar is believed to have been formed by overlying strata slowly sinking into a solution cavity after Mississippian Time.

SITE INVESTIGATIONS

Between 1996 and 1998 during the initial mine study phase, MoDOT drilled borings near selected known or suspected mine features. Subsequently in 2000 and 2001 MoDOT forces drilled 152 sample and core borings plus additional pattern auger borings during the bridge preliminary design phase.

During the bridge final design phase in 2004, MoDOT forces drilled 134 sample and core borings plus additional pattern auger borings. Standard MoDOT operating procedures for the retention and documentation of rock core was to take the core to the drilling and geotechnical office in Jefferson City, photograph, and then dispose of the rock core. Due to changes in the bridge configurations, additional borings were required at new substructure locations, box culverts and retaining walls. MoDOT Drill Crews drilled 8 additional borings for the bridges, 30 borings for the addition of 3 box culverts, and 24 borings for the addition of 3 retaining walls to the project in 2005. The borings were drilled during the period April 26th – May 18th, 2005. MoDOT forces utilized truck mounted Failing 1500, Mobil B - 31, CME – 45, and track mounted CME – 850 drill rigs to accomplish the drilling. Core was taken at the bridge substructure locations with NX double tube barrels. SPT samples were taken in select borings for the box culverts and retaining walls. The remainder of the borings were augered to refusal to obtain inferred top of rock profiles.

The HNTB exploration program consisted of 23 borings drilled during the period November 9, 2004 and January 17, 2005. Boring locations were staked by MoDOT surveyors utilizing GPS methods. The borings were planned by HNTB and logged in the field by geologists and geotechnical engineers from HNTB. The borings were drilled under a subcontract with Boart Longyear of Wytheville, Virginia. Borings were generally drilled at bridge foundation locations. Depths of borings ranged from 28.22 to 66.20 meters.

Borings were drilled with either a Longyear Model LF 70 trailer mounted, or Longyear Model 44, truck mounted drill rig. The borings were advanced through the surficial materials with tricone roller rock bits utilizing water and polymer additive drilling fluid to flush cuttings and stabilize the borehole. Casing was then seated a short distance into bedrock after it was deemed corable.

Coring of the bedrock was accomplished with triple tube core barrels (double swivel tube with a set of split inner tubes). Three diameters of cores (PQ-3, HQ-3, NQ-3) were taken, with larger diameters starting at the surface, switching to smaller diameters if drilling difficulties were encountered. Many different types of impregnated bits with varying diamonds and matrices were used. Water with polymer “easy mud” additives as necessary, were used as the fluid during coring. Standing surface waters and Mine Branch were used as sources. Much of the coring was accomplished with little or no water return to the surface.

The drilling was observed and rock core logged in the field with items such as percent recovery, RQD, lithology, physical characteristics, drilling action, and drilling fluid loss noted and recorded. Additional structural geologic logs were also recorded and field point load testing accomplished at the core storage facility in Granby, Missouri. The geologic structural logs further described items such as core loss, areas of RQD calculation, breaks per foot, rock type, color, weathering, grain size in general accordance with ISRM(1981). Description of and graphic log of discontinuities such as bedding planes, fractures and filling were also recorded. Point load test, both axial and diametral were performed generally at 3 meter intervals throughout all testable core. All core was photographed.

Borehole videos, acoustic televiewer (ATV), and borehole caliper diameters, were also taken at selected locations. The work was performed under subcontract to Boart Longyear by Geological Logging Systems of Bluefield, Virginia. The borehole video produced movies of drill hole sidewalls in digital format available on DVD. The Acoustic Televiewer provided an orientated, full 360 degree view of the borehole sidewall, detecting not only the presence of fractures, but also the dip angles and direction of the fractures and bedding planes. The ATV produced a hard copy log of the borehole. The caliper tool measured the diameter of the borehole and produced a graphical record.

GROUNDWATER

A total of six groundwater observation wells were installed at selected intervals along the proposed bridge alignments. The observation wells were installed in the cored borings. The holes were generally drilled 28 to 66 meters in depth. The standpipe consisted of a 3.05 meter length of 25mm I.D., Schedule 40, 0.01-inch factory slotted PVC screen pipe connected to a remaining length of 25mm, solid Schedule 40 PVC pipe to reach approximately 1 meter above the existing ground surface. No. 3 filter sand was then placed in the annular space surrounding the slotted screen and slightly above by means of a tremie and water flush. An approximately one meter seal constructed of bentonite pellets was placed on the filter sand layer. The remaining annular space of the borehole was filled to the ground surface with a cement-bentonite grout placed with a tremie pipe. A 4-inch square, galvanized steel, hinged top, protective guard was set in the grout. Observation wells were developed using a standpipe and airlift method. Compressed air was forced down to the bottom of the well with a hose. Water was forced up from the observation well until it appeared clean. Groundwater levels in the observation wells were taken throughout the exploration period and shortly thereafter, utilizing an electronic water level meter. Samples of the water were taken during the well development by Pace Analytical Services, Inc under subcontract to Boart Longyear. Analysis of the water included

- Field pH
- Field Dissolved Oxygen
- Sulfide
- Nitrate
- Sulfate
- Chloride
- Bromide
- Fluoride
- Alkalinity (includes Bicarbonate, Mg, Na, K, Ca)

SURFACE WATER

The historic USGS maps indicated what appeared to be an area of constant standing water to the north and east of borings B49-7 and B49-8. The water appeared to be impounded by the railroad embankment and was witnessed during the entire period of drilling between November, 2004 and January 2005. Shortly after the end of drilling during a site visit, the water had been drained and is no longer known to impound. Also the area between borings B-50-2 and B50-3 has been known to impound water; the area was pumped dry during drilling in the area by MoDOT and remained dry during summer period of 2004. It subsequently refilled and impounded water. While not specifically known, it is speculated the depression could be the result of shallow surface mining of small coal beds during the mining era.

GEOTECHNICAL CHARACTERIZATION

Overview

The following general material types were present at the project site:

Overburden:

Chat: Loose gravel and sand, crushed limestone and chert fragments

Tailings: Loose silt and sand, localized deposits

Soil: Primarily medium stiff to stiff clay developed from bedrock weathering

Bedrock (with typical ISRM strength classification and typical fracture spacing)

Limestone: Strong to very strong, close to moderate spacing

Chert: Very strong to extremely strong, close to moderate spacing

Breccia: Medium strong to very strong, very close to close spacing

Shale: Extremely weak to very weak, very close to extremely close spacing

Sandstone: Weak to strong, very close to close spacing

The above tabulation underscores the inherent variability of the bedrock units.

Mining voids were known to be present in both the upper “confused” zone and the deeper “sheet” zone. Based on published information the upper zone voids were reported to be irregular and variable in shape and extent. Typical sizes were 15 to 50 m in horizontal dimension and 2 to 8 m in vertical dimension. The lower voids exhibit lateral continuity with horizontal dimensions greater than 100 m but with vertical dimension limited to less than about 3m.

Groundwater data indicated the present water table is at or near the top of bedrock surface. The extensive presence and use of granular backfill of chat in the project area may also promote a locally perched water table above rather impervious materials such as residual fat clay or shale. Measured groundwater elevations within the bedrock at the site ranged from 295 to 298 m corresponding to a depth below ground surface of about 5m. The head difference between piezometers in the upper and lower mining horizons was less than 0.5 m indicating a slight downward gradient. For the two overburden piezometers, one has reported essentially dry conditions and the second a groundwater elevation of about 299 m (4 m below ground elevation).

Chemical analyses of groundwater samples from the bedrock indicated neutral pH with minimal potential for dissolution of limestone.

ROCK MECHANICS TESTING

Methods

Rock mechanics testing determines the physical properties of cylinders of core that represent "intact rock", that is; rock devoid of major defects such as joints and bedding. As such, the physical properties so determined have a bias to higher quality rock that is able to withstand the rigors of coring and recovery handling to retain an intact cylindrical aspect. This unavoidable testing bias must be recognized in the development of geotechnical parameters for design and by others who interpret such data for construction (e.g. drilling).

For the Route 249 Interchange project, programs of both laboratory testing and field testing were performed. The rock mechanics testing results summarized herein were developed during 2004 and 2005 in conjunction with the HNTB design study. The laboratory program included strength, elastic properties and density testing while the field program consisted only of strength testing.

Laboratory Testing Program

Uniaxial Compressive Strength (symbol: UCS units: MPa)

A total of 24 samples were tested for uniaxial compressive strength by a specialty laboratory under subcontract to Wyllie & Norrish. These tests were performed in accordance with ASTM D2938-95 and the results are summarized in Table 1. In addition, MoDOT performed additional uniaxial testing at the request of HNTB for the 2005 holes.

Elastic Properties (modulus symbol: E_r units: MPa)
(Poisson's symbol: v units: dimensionless)

A total of 12 samples were tested for deformation modulus and Poisson's ratio in accordance with ASTM D3148-93. The results are summarized in Table 1. For comparative purposes, the elastic properties were reported at a uniaxial stress level of 10 MPa for all samples.

Density (symbol: γ units: kg/m^3)

For all test procedures requiring preparation of right cylinders, density determinations were made in accordance with ASTM D2216-92. The results for the 24 density determinations are summarized in Table 1.

Table 1. Laboratory Testing Summary

Bore Hole	Depth Interval from	to	Core Size	Rock Type (based on bore hole log descriptions)	γ (kg/m^3)	I_{s50} (a) (MPa)	I_{s50} (d) (MPa)	σ_u (MPa)	E_r (MPax10^3)	ν	σ_t (MPa)
40-1	39.4	40.2	HQ3	*Limestone* , mg, sl wx, strong	2546		2.07	59.5	8.6	0.12	
40-1	61.3	62.2	HQ3	*Limestone* , shaly, md wx, weak to v weak	2487		1.75	38.2			
40-1	93.6	94.6	HQ3	*Limestone* , cherty, sl wx, strong	2656		5.42	124.7	20.8	0.39	6.5
40-3	9.0	10.0	PQ3	*Limestone* , chert nod, sl wx, strong	2667	3.98		65.3			
40-3	48.3	49.0	PQ3	*Limestone* , sl wx, strong	2637	4.89		78.9			
48-3	15.1	16.2	HQ3	*Chert* , fresh, strong	2412		5.28	144.2	11.9	0.09	
48-3	74.9	75.5	HQ3	*Chert* , fresh, strong	2402	6.57		114.6			
49-2	45.4	46.3	PQ3	*Limestone* , fresh, strong	2521	1.56		37.2			
49-2	159.0	159.8	PQ3	*Limestone* , cg, fresh, strong	2639	2.67		78.0	13.1	0.14	6.0
49-3	29.2	29.9	PQ3	*Limestone* , fresh, strong	2589	7.70		134.1	15.9	0.09	
49-3	54.0	54.8	PQ3	*Limestone* , wx, weak	2185	0.28		8.3			
49-3	166.6	167.4	HQ3	*Limestone* , fg, fresh, strong	2681		2.34	119.8	20.7	0.28	5.6
49-3	194.3	195.2	HQ3	*Chert, sl wx, v strong*	2533		8.34	182.5	26.1	0.21	19.8
49-4	16.1	17.1	HQ3	*Limestone* , chert nod, sl wx, strong	2681		4.11	103.9			
49-4	44.5	45.3	HQ3	*Limestone* , 45%chert nod, sl wx, strong	2661		3.14	96.8	16.4	0.17	
49-5	25.2	25.8	HQ3	*Limestone* , cg, md wx, weak	1982	0.44		3.5			
49-5	42.7	43.4	HQ3	*Limestone* , fresh, strong	2610		3.63	77.3	12.1	0.21	
49-5	103.2	103.9	HQ3	*Chert* , sl wx	2442	4.96		67.8			
49-6	56.8	57.4	HQ3	*Limestone* , h wx, weak	2391	1.78		34.1			
49-6	64.1	65.1	HQ3	*Limestone* , m wx, med strong	2643		3.74	113.8	19.0	0.21	
49-8	15.2	16.2	PQ3	*Limestone* , cherty, sl wx, strong	2414	4.09		89.2	18.7	0.14	
49-8	39.5	40.2	PQ3	*Limestone* , cherty, sl wx, strong	2177	3.09		44.0			
49-8	87.4	88.7	PQ3	*Limestone* , cherty, fg, sl wx, strong	2626	1.84		107.7	51.0	0.16	5.7
65-2	62.5	63.2	HQ3	*Limestone* , cg, md wx, med strong	2486		2.16	35.7			
				Mean Test Result:	2503	3.37	3.82	81.6	19.5	0.18	8.72
				No. of Tests:	24	13	11	24	12	12	5

Key to Testing Protcol:

Strength	Elastic	Full		Test Type	Test Method
x	x	x	γ	Density (kg/m^3)	ASTM D2216-92
x	x	x	I_{s50}	Point load strength (MPa), a=axial, d=diam	ISRM, 1972
x	x	x	UCS	Uniaxial Compressive Strength (MPa)	ASTM D2938-95
	x	x	E_r	Deformation modulus (MPa)	ASTM D3148-93
	x	x	ν	Poisson's ratio	ASTM D3148-93
		x	σ_t	Indirect or Uniaxial tensile strength (MPa)	Brazilian Disc

Tensile Strength	(symbol: σ_t units: MPa)

Five tests for tensile strength were performed using the indirect Brazilian disc method (Table 1).

Point Load Strength	(axial test symbol: $I_{s50(a)}$	units: MPa)
	(diametral test symbol: $I_{s50(d)}$	units: MPa)

The point load test is a standardized strength index test procedure by the International Society of Rock Mechanics (ISRM, 1985). The test is performed with portable equipment and with minimal sample preparation, thereby making the point load test suitable for field applications as an adjunct to core logging.

The test consists of loading a length of core in either an axial or diametral orientation between conical loading platens. The tensile stress imparted on the sample leads to a tensile type of failure. The calculated point load strength can be correlated with more conventional strength determinations such as uniaxial compressive strength.

In order to develop project specific correlations between uniaxial compressive strength and point load strength, for each of the UCS tests performed in the laboratory point load tests were performed in either the axial or diametral orientations on the core immediately adjacent to the UCS location. An analysis of the correlations for the data in Table 1 provided the following relationships for the project:

$$UCS = 18 * I_{s50(a)}$$

$$UCS = 25 * I_{s50(d)}$$

These correlation coefficients are within the typical range of published values for this type of test. The difference between the axial and diametral values is an indication of strength anisotropy of the intact rock.

Field Testing Program

Field testing was performed by HNTB and was limited to point load testing of the core at approximately 3 m intervals to develop strength profiles.

Table 2. Composite Rock Mechanics Testing Summary

Rock Type: CHERT	INTACT ROCK PARAMETER*						
	γ	$I_{s50\,(a)}$	$I_{s50\,(d)}$	UCS	E_r	ν	σ_t
TESTING PROGRAM	(kg/m^3)	(MPa)	(MPa)	(MPa)	(GPa)		(MPa)
W&N							
No of Tests	3	1	2	3	2	2	1
Mean	2462	4.96	6.81	131.5	19.0	0.15	19.8
Minimum	2412	4.96	5.28	67.8	11.9	0.09	19.8
Maximum	2533	4.96	8.34	182.5	26.1	0.21	19.8
HNTB							
No of Tests		29	54				
Mean		5.04	5.78				
Minimum		0.63	0.75				
Maximum		9.94	14.45				
ALL TESTING PROGRAMS							
No of Tests	3	30	56	3	2	2	1
Mean	2462	5.04	5.82	131.5	19.0	0.15	19.8
Minimum	2412	0.63	0.75	67.8	11.9	0.09	19.8
Maximum	2533	9.94	14.45	182.5	26.1	0.21	19.8

Rock Type: LIMESTONE	INTACT ROCK PARAMETER*						
	γ	$I_{s50\,(a)}$	$I_{s50\,(d)}$	UCS	E_r	ν	σ_t
TESTING PROGRAM	(kg/m^3)	(MPa)	(MPa)	(MPa)	(GPa)		(MPa)
W&N							
No of Tests	21	12	9	21	10	10	4
Mean	2509	3.24	3.15	74.5	19.6	0.19	6.0
Minimum	1982	0.28	1.75	3.5	8.6	0.09	5.6
Maximum	2681	7.70	5.42	134.1	51.0	0.39	6.5
MoDOT							
No of Tests				98			
Mean				58.7			
Minimum				17.4			
Maximum				188.0			
HNTB							
No of Tests		101	161				
Mean		4.19	4.07				
Minimum		0.15	0.08				
Maximum		13.86	11.29				
ALL TESTING PROGRAMS							
No of Tests	21	113	170	119	10	10	4
Mean	2509	4.09	4.02	61.5	19.6	0.19	6.0
Minimum	1982	0.15	0.08	3.5	8.6	0.09	5.6
Maximum	2681	13.86	11.29	188.0	51.0	0.39	6.5

Table 2 (cont'd). Composite Rock Mechanics Testing Summary

Rock Type: SANDSTONE / SILTSTONE	INTACT ROCK PARAMETER*						
	γ	$I_{s50\,(a)}$	$I_{s50\,(d)}$	UCS	E_r	ν	σ_t
TESTING PROGRAM	(kg/m^3)	(MPa)	(MPa)	(MPa)	(GPa)		(MPa)
MoDOT							
No of Tests				1			
Mean				0.41			
Minimum				0.41			
Maximum				0.41			
HNTB							
No of Tests		4	4				
Mean		0.83	0.81				
Minimum		0.49	0.29				
Maximum		1.58	1.74				
ALL TESTING PROGRAMS							
No of Tests		4	4	1			
Mean		0.83	0.81	0.41			
Minimum		0.49	0.29	0.41			
Maximum		1.58	1.74	0.41			

Rock Type: SHALE	INTACT ROCK PARAMETER*						
	γ	$I_{s50\,(a)}$	$I_{s50\,(d)}$	UCS	E_r	ν	σ_t
TESTING PROGRAM	(kg/m^3)	(MPa)	(MPa)	(MPa)	(GPa)		(MPa)
MoDOT							
No of Tests				13			
Mean				0.37			
Minimum				0.05			
Maximum				1.48			

Rock Type: BRECCIA	INTACT ROCK PARAMETER*						
	γ	$I_{s50\,(a)}$	$I_{s50\,(d)}$	UCS	E_r	ν	σ_t
TESTING PROGRAM	(kg/m^3)	(MPa)	(MPa)	(MPa)	(GPa)		(MPa)
HNTB							
No of Tests		13	23				
Mean		3.33	3.21				
Minimum		0.62	0.45				
Maximum		9.78	6.61				

*Note: * See Table 1 for test abbreviations.*

Intact Rock Properties

Table 2 summarizes the rock mechanics testing performed in conjunction with the 2004 / 2005 HNTB design study on the basis of the primary rock types. The number of tests, mean value, minimum and maximum value are reported for each rock parameter by rock type. The results indicate a high degree of variability. For the limestone, this variability reflects a gradation of composition from nominally pure limestone to limestone with increasing percentages of chert nodules or chert layers. The end member of this spectrum is the nominally pure chert. Variability is also imparted through recementation of breccias and by near surface weathering.

For baseline purposes the mean values were deemed appropriate with the recognition of the inherent variability discussed above. For foundation design, intact rock strength was estimated on the basis of proximal testing for the strata of interest and was not based on rock type, per se.

PREDICTED GROUND BEHAVIOR

The capacity of the geologic strata to support the proposed structure loads was judged to be dependent on:

1. The rock mass strength and deformability for the strata, especially the "roof beam" between the foundation loads and possible voids.
2. The presence and size of void space (or compressible void infilling).

Rock Mass Properties

Rock mass behavior is a function of the ***intact*** rock properties (as developed through laboratory testing) and the frequency, nature and orientation of the discontinuities (joints, bedding planes) that intersect the rock mass. Scale restrictions preclude direct testing for rock mass properties using laboratory methods. Consequently, empirical approaches have been developed to enable estimation of rock mass engineering properties from more readily available information. For the purposes of this project the approach developed by Hoek et al (2002) has been followed. This method incorporates a Rock Mass Rating (RMR)[4] value and an intact rock strength value to calculate rock mass shear strength, rock mass uniaxial strength, rock mass tensile strength and rock mass deformability.

The rock mass characterization procedure estimated the probable zone in which bearing capacity would be developed for each foundation unit. The stratigraphy was idealized into engineering units for which representative values for Rock Quality Designation (RQD), fracture frequency (f/m) and **intact** rock strength were assigned. From these values and the nature of the discontinuity surfaces recorded in the structural

---

[4] For the Route 249 project the RMR system was adopted rather than the more recent Geological Strength Index (GSI) system. The GSI and RMR values are nominally identical for the range of rock qualities on this project.

core logging, the RMR value for each idealized engineering unit was calculated. Through correlations of rock properties such as shown in Figure 4 the engineering units were idealized into three typical rock qualities:

Rock Mass	Typical RMR	Typical Intact Strength (MPa)
Good Quality	65	130
Fair Quality	50	75
Poor Quality	30	30

Figures 5 through 7 illustrate the character of the three rock mass quality designations. For each of these rock mass categories, shear strength, uniaxial strength, tensile strength and elastic properties were calculated using the program RocLab, Version 1.01 (RocScience, 2004). This procedure led to the following engineering properties for the three typical rock mass qualities:

Rock Mass	Φ (deg)	c (MPa	σ_u (MPa)	σ_t (MPa)	E (GPa)
Good Quality	36.5	8.7	18.5	-0.77	23.7
Fair Quality	32.1	3.9	4.52	-0.14	8.66
Poor Quality	26.1	1.1	0.52	-.013	1.73

Key:

Φ = friction angle
c = cohesion
σ_u = uniaxial compressive strength (rock mass)
σ_t = tensile strength
E = modulus of deformation

These parameters were used for generic design analyses to demonstrate the sensitivity of geotechnical and structural parameters and to thereby focus the subsequent bent-by-bent foundation analyses to the critical structures.

Interpretation of Void Size

Core drilling intersected multiple voids in both the upper "confused" mining zone and the lower "sheet" mining zone. These void intersections included core loss zones, broken rock and very soft clay infilling (Figure 8). Previous studies showed that remote sensing methods (geophysics) were inadequate to positively determine the size and location of voids. Consequently a trio of indirect approaches was employed:

1. Historical Records

Smith and Siebenthal (1907) make reference to the size of mine openings as follows:

Upper Mining Horizon
"Runs": Length = "few hundred feet"
Width = 300 feet maximum, 10 to 50 feet typical

Lower Mining Horizon
"Sheet": Length = ¾ mile
Width = 50 to 300 feet
Face = 10 feet
Depth = 170 feet (corresponds to 248m elevation)

2. Recent Sinkhole Development

Two recent collapse features are located adjacent to the highway alignment to the south of the interchange (Figure 9). The size and shape of these features led to the conclusion that they were not shaft-related but rather represented roof collapse features over shallow voids. Measurements of the surface depressions indicate that the causative voids probably had a horizontal dimension in the range of 20 to 35 m.

3. Borehole Correlations

Single Bent Cross Sections:
A6149 Bent 7: 6 m continuity 2 holes @ 273 m elevation
A6149 Bent 8: 6 m continuity 3 holes @ 283 m elevation

Between Bents:
A6140 Bent 1 to A6149 Bent 8:
Possible 72 m continuity @ 290 m elevation

In summary, the indirect evidence on horizontal continuity of voids was:

Literature: 15 to 30 m (physical dimensions)

Sinkholes: 20 to 35 m (surface expression)

Boreholes: 6 to 72 m (inferred correlations)

Therefore, based on the above indirect evidence, it was concluded that a **horizontal** void continuity of 20 to 60 m should be used for foundation design purposes.

The drilling programs and borehole logging provided direct information on the vertical continuity of voids and mine features (Figure 10). An analysis of this data indicated the upper mining voids had vertical continuity typically less than 5 m although two features were intersected with apparent vertical continuities greater than 20 m. For the lower mining horizon (sheet ground) the drilling indicated vertical void continuity in the range of 0.5 to 3m. Two features were intersected with vertical continuities of about 6m. For design purposes the respective **vertical** continuities for voids were:

- Upper “confused” zone: 5m
- Lower “sheet” zone: 3m

FOUNDATION BEHAVIOR

Mechanisms

It was recognized that both short and long term processes could affect the performance of bridge foundation units:

Short Term:

- Bearing failure due to imposed loading

Long Term:

- Ground loss into shafts or mine voids
  - erosion of shaft plugs
  - dissolution of limestone
  - roof cave
- Increased loading related to regional groundwater lowering

The long term processes were discounted based on either direct mitigation (e.g. permanent shaft closures), institutional controls (e.g. regional groundwater effects) or perceived low risk (dissolution and roof cave). Foundation design was therefore based on short term bearing failure for which two mechanisms were considered for analysis:

- A “punching” failure of a spread footing or micro pile group situated on a rigid stratum (e.g. limestone) overlying a compressible unit (void or mine infilling) as shown in Figure 11.
- A flexural “beam” bending of a rigid stratum (e.g. limestone) overlying a compressible unit (void or mine infilling).

The objective of the analyses was to determine the thickness and quality of rock mass required to support the foundation loads assuming the structures were located over voids with the horizontal continuity as developed above. The punching failure was analyzed using an equilibrium formula while the “beam” behavior was modeled using both an analytical solution and finite element stress analysis.

Analytical Approach

Initial analyses confirmed that the flexural “beam” bending mechanism was more critical than the “punching” failure. The analytical approach to predict ground behavior was to perform a sensitivity analysis in which rock quality, void width and roof beam depth (i.e. thickness) were varied. The finite element software Phase**2** Ver 5.048 distributed by Rocscience Inc. was used to perform the sensitivity analyses. For the analytical model the twin bearing pads were replaced with a single circular footing with radius of 4.8m and with same total load and bearing pressure. This footing simplification enabled an axisymmetric model – 3D model that is rotationally symmetric about the line of loading to be used. Note that this model represents a void by half of its true width.

The piezometric data indicated the groundwater surface at the project site was just below the top-of-rock (elevation +/- 295m). This discrepancy from the historical records in which the water table was reported at a depth of about 15m was attributed to the mining disturbance and extensive drilling that has locally modified the groundwater regime. For the purposes of foundation design, the stratigraphic section was assumed to be saturated below the top-of-rock. This was incorporated in the stress analyses for foundation design by assuming buoyant unit weight for all strata beneath top-of-rock.

Sensitivity Analysis

The initial stress analyses attempted to calibrate the model by determining the combinations of rock quality, void width and beam depth that would replicate the observed sinkhole development shown in Figure 9. An example of this analysis is shown in Figure 12 in which it is shown that Fair Quality rock over a 40 m void at 10 m depth would have marginal stability.

Having demonstrated the applicability of the model, a series of analyses were performed that included foundation loads (Figure 13) and foundation and embankment loads (Figure 14). For the generic sensitivity analyses the foundation load was set at 0.5 MPa and the embankment load at 0.196 MPa. Figure 15 summarizes the entire set of sensitivity results developed from the stress analyses. The combinations of rock quality, void width and beam depth highlighted in red or yellow were predicted by the model to have unacceptable stability.

The finite element model was compared to a closed form solution for plate bending (Figure 16). The calculated Factors of Safety (FS) were subjectively assigned to the stability categories in keeping with typical engineering practice. As shown in Figure 17 the closed form solution was slightly more conservative than the stress analysis but agreed reasonably well.

Application to Foundation Design

The analytical results, site specific ground conditions and actual foundation loading were used to develop bent-by-bent summaries for selection of foundation types and for foundation design. In order to simplify the extensive quantity of data, for each foundation location an idealized interpreted stratigraphy with associated geotechnical parameters was produced. For the load bearing strata, the intact rock strength (UCS), RQD, fracture frequency (FF), and rock mass rating (RMR) were provided. The UCS value were assigned based on rock mechanics testing and point load testing, suitably discounted to a design value based on engineering judgment. The RQD and fracture frequency values were weighted averages for the idealized layers derived from the geotechnical core logging. These idealized interpreted stratigraphy and geotechnical parameters were then used to select the intensity of production exploration/treatment and the foundation type at that location. Figure 18 shows two examples of these summaries which also included the recommended depth for ground treatment by grouting prior to foundation construction.

SELECTION OF FOUNDATION CONCEPTS AND TREATMENT

General

The highly variable nature of the geology required a systematic approach be implemented to reduce the risk associated with variability between assumed and actual ground conditions at any specific location. To reduce this risk, the project team developed a foundation design concept that incorporated the following basic principles:

1. The *exact nature* of the rock mass under each foundation would be identified and verified during production drilling.
2. At each foundation location productions holes would be drilled to verify the ground conditions and also to treat the ground to limit subsequent micropile grout takes and/or to improve the mechanical properties of the rock mass.
3. The actual foundation built at each location would be responsive to the information obtained from the production drilling.
4. No bent (or wall) would be built over the location of a mineshaft that had not been remediated in some definite fashion in advance.

Bridge Foundation Types

Multiple foundation types were evaluated including drilled shafts, H piles, spread footings, and micropiles. Drilled shaft foundations were not recommended due to the variability of rock conditions, presence of underground mining and anticipated difficulty and costs of advancing to significant depths within the chert layers. H piles were not recommended for support of heavy loads due to the variability of rock conditions, presence of underground mining, and the possible necessity of significant high cost predrilling. Thus either spread footings or micropiles were the preferred foundations for the support of each bent.

In evaluating the 23 bent foundation locations, two geotechnical/geostructural categories of foundation conditions were defined to simplify foundation design:

- **Ground Type 1 consisted of competent, non mined limestone extending to at least 40 meters below ground surface.**

- **Ground Type 2 consisted of all other conditions, included voided, collapsed, solutioned or highly fractured ground.**

Spread footings on rock were recommended at bent locations where competent limestone was shallow and geologic conditions were interpreted to be in an area of Ground Type 1.

Micropiles were recommended at bent locations where geologic conditions were interpreted to be of Ground Type 2. Table 3 summarizes the final foundation design parameters and recommended foundation type for each bent.

Table 3. Foundation Types by Bridge Bent

Number	Foundation Type	Treatment Intensity	Locations
7	Spread Footing	Low	A6140 - EB3 A6148 – EB1, B2, B3, EB4 A6149 – EB9 A6150 – EB1
6	Micropile	Medium	A6149 – B3, B6 A6150 – B2 A6165 – EB1, B2, EB3
10	Micropile	High	A6140 – EB1, B2 A6149 – EB1, B2, B4, B5, B7, B8 A6250 – B3, EB4

Table 3 indicates 7 bents were anticipated to have spread footings, while 16 bents would have micropiles.

Footings at each bent location would therefore either be cast on a spread footing keyed 6 inches minimum into competent limestone or be supported on micropiles. Micropile design would be in general accordance with FHWA “Micropile Design and Construction Guidelines”, Publication No. FHWA-SA-97-070.

The micropile permanent steel casing was designed to extend approximately 3 m into rock based on the lowest interpreted elevation or to a minimum elevation as required for lateral stability with the bond zone designed below the bottom of casing. Bond zones below the cased length of 5 to 8 m were foreseen in competent, strong limestone, and of 10 to 15.5 m in more chaotic horizons. Micropiles were designed to support axial compression loads in side friction along the bond zone length and lateral loads through a combination of battered piles and bending.

Four preconstruction "performance piles" were installed and tested to geotechnical bond failure or to at least twice the anticipated average bond stress to verify overall design assumptions prior to production piles being installed. A minimum of one production pile from each bent was selected for a proof testing to at least 120% of the design working load.

Production Exploration and Treatment Principles

As noted above, the exact nature of the rock mass under each foundation was verified during production drilling. These exploratory holes were also used to treat the ground to limit subsequent micropile grout takes and/or to improve the mechanical properties of the rock mass.

The general approach to exploring each bent location was uniform and consistent, but the foreseen amount and type of drilling and grouting at each bent was to variable based on the preproduction understanding of local foundation conditions (i.e. Ground Type 1 or Ground Type 2 as a base) and the footing geometry. This approach featured the concept of "intensity" of the treatment conducted at each bent prior to construction of the spread footings or micropiles; namely low, medium and high, as illustrated conceptually in Figure 19. Each exploratory hole was drilled vertically to the target depth. Core drilling was not specified but each hole had to be logged during drilling in accordance with automated Monitoring While Drilling (MWD) principles. Minimum hole diameter was set at 100 mm in rock. The grout type was varied with the severity of the conditions.

Low Mobility Grout (LMG) was used in voided conditions (apertures greater than 100 mm). High Mobility Grout (HMG) with or without sand was used in tighter ground conditions as illustrated in Figure 20.

Based on the ground water chemistry, Type II cement was recommended for use in both the HMG and LMG grouts. LMG was specified to have a low slump (less than 150 mm), high internal friction and 28 day strength in excess of 4 MPa. The HMG was specified to have a Marsh Cone Viscosity of 40-50 seconds, be stable and have 28 day strength in excess of 4 MPa. In the event of excessive take in any one location, it was foreseen that sand would be added to the HMG or the viscosity would be modified.

Depending on the actual conditions found in the field at each bent the level of treatment could escalate i.e. additional holes might be required to explore and treat the bent. Conversely, it was not anticipated that there would be cause to reduce the intensity of treatment at any bent under this program.

It was anticipated that ascending stage grouting principles could be used for both the LMG and HMG operations. However, particularly severe ground conditions could require downstage grouting. For bidding purposes it was estimated that up to 25% of the drilling would require downstage grouting due to variable rock quality.

The plans and specifications included the foreseen ground treatment program and quantities for each bent location. Modifications to the treatment program and foundation construction were made primarily in the field based on actual field conditions encountered and on the judgment of the monitoring and design personnel.

MSE RETAINING WALL FOUNDATION EXPLORATION and TREATMENT

At three locations, Mechanically Stabilized Earth (MSE) walls were selected to reduce the overall bridge lengths and associated costs. MSE walls were selected as they are considered the most economical and can accommodate a variety of subsurface conditions. It was recommended that maximum wall heights be kept to about 9 m.

The three large panel MSE walls were planned to retain approach embankments at end bent locations of two bridges at the following locations; End Bents 1 and 3 of Bridge A6165, and End Bent 1 of Bridge A6149, designated with structure numbers A7263, A7264, and A7264, respectively.

The major foundation concern for these structures was the potential for loss of ground into, and loss of support from, underlying voids or mine shafts. As for the bents, all known or suspected shafts were to be pretreated.

It was recommended that the top of leveling pad and the bottom of wall be exposed and inspected first to verify the actual foundation material conditions. As Type 2 ground conditions were assumed, a line of primary treatment holes at 6 meter centers was drilled to a depth of 30 meters and treated with LMG if Type 2 conditions were found or inferred.

It was noted that depending on the local conditions, additional “closure”, or secondary treatment holes may be needed. With respect to grouting, this phase of exploration and treatment was intended to locate and fill shafts or other major voids. Systematic fissure grouting was not deemed necessary, and so only LMG was specified.

Additionally, one treatment hole approximately every 36 m^2 was planned to be drilled to a maximum depth of 20 meters below ground surface in a regular pattern under the footprint of the reinforced mass for the MSE wall if Type 2 conditions were found or inferred. These treatment holes would be treated with LMG. It was recommended that should significant features be encountered, additional “closure”, or secondary treatment holes may be needed.

BOX CULVERT FOUNDATION EXPLORATION and TREATMENT

Three box culverts were planned at the southern portion of the project. Structure numbers of the box culverts were A7260, A7261 and A7262. The box culverts will carry the Mine Branch Creek below the approach roadway embankments of Rte. 249 NBL, Rte. 249 SBL, Ramp 3 and Ramp 4.

The major foundation concern for these structures is the potential for loss of ground into, and loss of support from, underlying voids or mine shafts. As for the bents and walls, all known or suspected shafts were pretreated.

Even though Type 1 ground conditions were foreseen, it was recommended that one primary treatment hole approximately every 36 m^2 be drilled to a maximum depth of 20 meters in a regular pattern under the footprint of the box foundation slab. These holes were treated with LMG. It was noted that depending on the local conditions, additional “closure”, or secondary treatment holes may be needed. With respect to grouting, this phase of exploration and treatment was intended to locate and fill shafts or other major voids. Systematic fissure grouting was not deemed necessary, and so only LMG was foreseen.

MINE SHAFT/OPEN FEATURE CLOSURE PHILOSOPHY

The general philosophy was that each known or suspected mine shaft or open feature location on the site should be explored and treated in addition to the specific actions to be conducted at individual bent, MSE wall, and box culvert locations. The locations of the major structural elements of the project including the bridge foundations, MSE walls, and box culverts had already been adjusted during preliminary design iterations to avoid known mine shafts or open features. It was noted that several suspected features were located on MNA Railroad property.

Two categories of mine shafts/open features were defined based on their proximity to the proposed structures and grading limits. Each had a different treatment method and intensity:

"Type 1 Shaft" - Mine shafts/open features located within approximately 15 meters of a major structural element of the project or within approximately 5 meters of the footprint of an embankment or cut footprint.

"Type 2 Shaft" - Mine shafts/open features located beyond approximately 15 meters of a major structural element of the project and beyond approximately 5 meters of an embankment or cut footprint.

Based on review of mining maps, literature, site drilling and reconnaissance, some 26 potential mine shaft/open feature locations were identified within the proposed interchange. A number of locations were investigated further by performing test pit excavations. As a result, seven potential feature locations showed no indication of a shaft; therefore, no further investigation or treatment was recommended at these locations. One mine feature location, J-7, was located beneath the MNA Railroad tracks and no investigation or treatment by MoDOT was recommended. Of the remaining 18 feature locations, six locations were confirmed as mine shafts either open or previously plugged by the Missouri Department of Natural Resources (MDNR). These six shafts were recommended to be closed either as Type 1 or 2 closures.

Twelve unconfirmed shaft locations were identified for investigation and possible closure. Exploratory inspection excavations were recommended at these locations to determine if mine features are present. If so, closure would be required either by Type 1 or 2 procedures. The actual number of Type 1 and 2 closures would therefore be determined during the exploration phase.

It was noted that the excavations for the exploration and closure of mine features might encounter groundwater and that excavations on or near the railroad right-of-way may necessitate the use of temporary shoring to control the excavation and maintain the railroad tracks.

For Type 1 shafts, the recommended treatment involved full penetration by drilling with an initial treatment hole to confirm the shaft base elevation, filling it with LMG and verification of thoroughness of treatment by a minimum of 2 additional treatment holes. These additional treatment holes would be within the mine shaft limits.

Therefore, each Type 1 shaft would require a minimum of three treatment holes drilled to the bottom of the feature. It was noted that the drilling might encounter obstructions and/or other complexities in the backfill such as timber, metal, concrete, reinforcing steel, etc.

Regarding the LMG volume, it was anticipated that not all the shaft space was void, but that there would be workings leading off the shaft which may still be open, filled, or collapsed.

In addition, wherever Type 1 shafts were encountered, it was recommended that at least three vertical treatment holes shall be drilled at 3 meter centers to a depth of 30 meters in a line running transverse to the direction of the shaft and any structure that was within the critical distance. These treatment holes were intended to verify that no open shallow lateral workings still existed between the shafts and the interchange structures.

It was recommended that these activities should be conducted under the utmost safety standards. Drilling equipment should operate from either frames/platforms or be vertically suspended from remotely located leads. Prior excavation to top of rock would provide visual evidence of the in situ geometry of the feature, indication of the required of safety measures, anticipated quantities of LMG, and the precise location of the treatment holes.

Type 2 shaft closure involved partial excavation to top of rock, temporarily plugging the throat with polyurethane foam, and then casting a reinforced concrete plug over the top.

REFERENCES

Bieniawski Z.T., 1989. Engineering Rock Mass Classifications. publ Wiley, New York. 251 pp

Blair, Kevin P., Others, 1992, Structure-Contour Maps on the Top of the Mississippian Carbonates and on the Top of the Upper Cambrian and Lower Ordovician Arbuckle Group, Joplin 1º x 2º Quadrangle, Kansas & Missouri, U.S. Geological Survey.

Erickson, R. L., Others, 1990, Summary Geochemical Maps of the Joplin 1º x 2º Quadrangle, Kansas & Missouri, U.S. Geological Survey.

Grisafe, David A. and Rueff, Ardel W., 1991, Maps Showing Industrial Mineral Resources of the Joplin 1º x 2º Quadrangle, Kansas & Missouri, U.S. Geological Survey.

Hoek, E., Carranza-Torres, C., and Corkum, B. (2002). Hoek-Brown Failure Criterion – 2002 Edition. Proceedings 5th North American Rock Mechanics Symposium, Toronto, July 2002, 267-273.

International Society of Rock Mechanics (ISRM), 1981. “Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses”, Pergamon Press, Oxford, U.K.

ISRM, 1985. “Suggested Method for Determining Point Load Strength”, International Journal of Rock Mechanics, Vol 22, No 2, April 1985.

Laas, Virginia, Ritchie, E., and Stewart, D., 1989, An Introduction to the Tri-State Mineral Museum, 25 p.

Lefchik, Thomas, 1998, Manual for Abandoned Underground Mine Inventory and Risk Assessment, State of Ohio-Department of Transportation, 168 p.

McCafferty, Anne E. and Cordell, Lindrith E., 1992, Geophically Inferred Structural and Lithologic Map of the Precambrian Basement in the Joplin 1º x 2º Quadrangle, Kansas & Missouri, U.S. Geological Survey.

McCracken, Mary H., 1971, Structural Features of Missouri, Missouri Geological Survey and Water Resources, Rept. Inv. 49, vi + 100 p., 1 pl., 11 figs.

McFarland, Michael C., 1982, Underground Mines and Shafts-Webb City Quadrangle, Missouri, Missouri Department of Natural Resources.

McFarland, Michael C. and Brown Jr., James C., 1982, Open Shafts, Pits, and Subsidences-Webb City Quadrangle, Missouri, Missouri Department of Natural Resources.

Pratt, Walden P., Others, 1993, Assessment of the Joplin 1º x 2º Quadrangle, Kansas & Missouri, For Mississippi Valley-Type Deposits and Other Minerals, U.S. Geological Survey.

Ritchie, Everett, 1986, Guide Book to the Tri-State Mineral Museum, 82 p.

RocScience, 2004. RocLab, Version 1.01, Toronto, Ontario.

Smith, W. S. T. and Siebenthal, C. E., 1914, Geologic Atlas of the United States: Joplin District Folio No. 148, U.S. Geological Survey, 20 p., 13 figs.

Thompson, Thomas L., 1986, Paleozoic Succession in Missouri, Part 4-Mississippian System, Missouri Department of Natural Resources, Division of Geology & Land Survey, Report of Investigations 70, 189 p., 110 figs., 2 tables.

Thompson, Thomas L., 1995, The Stratigraphic Succession in Missouri (Revised 1995), Missouri Department of Natural Resources-Division of Geology and Land Survey, Volume 40 (2nd Series) Revised, 188 p., 42 figs., 1 table.

Wyllie, Duncan C., 1999, Foundations on Rock, pub E&FN Spon, London, 401pp.

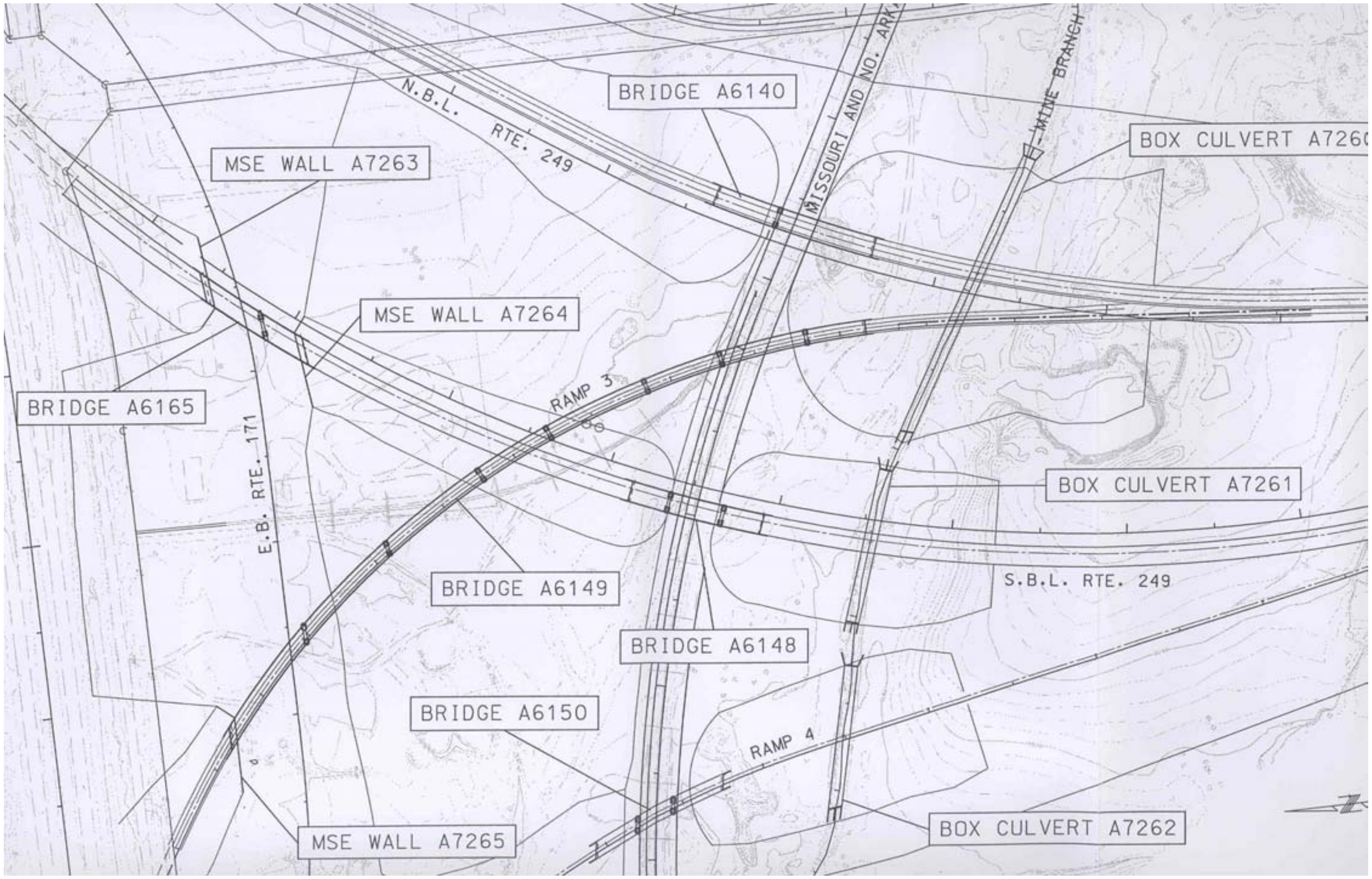


Figure 1. Site Plan.

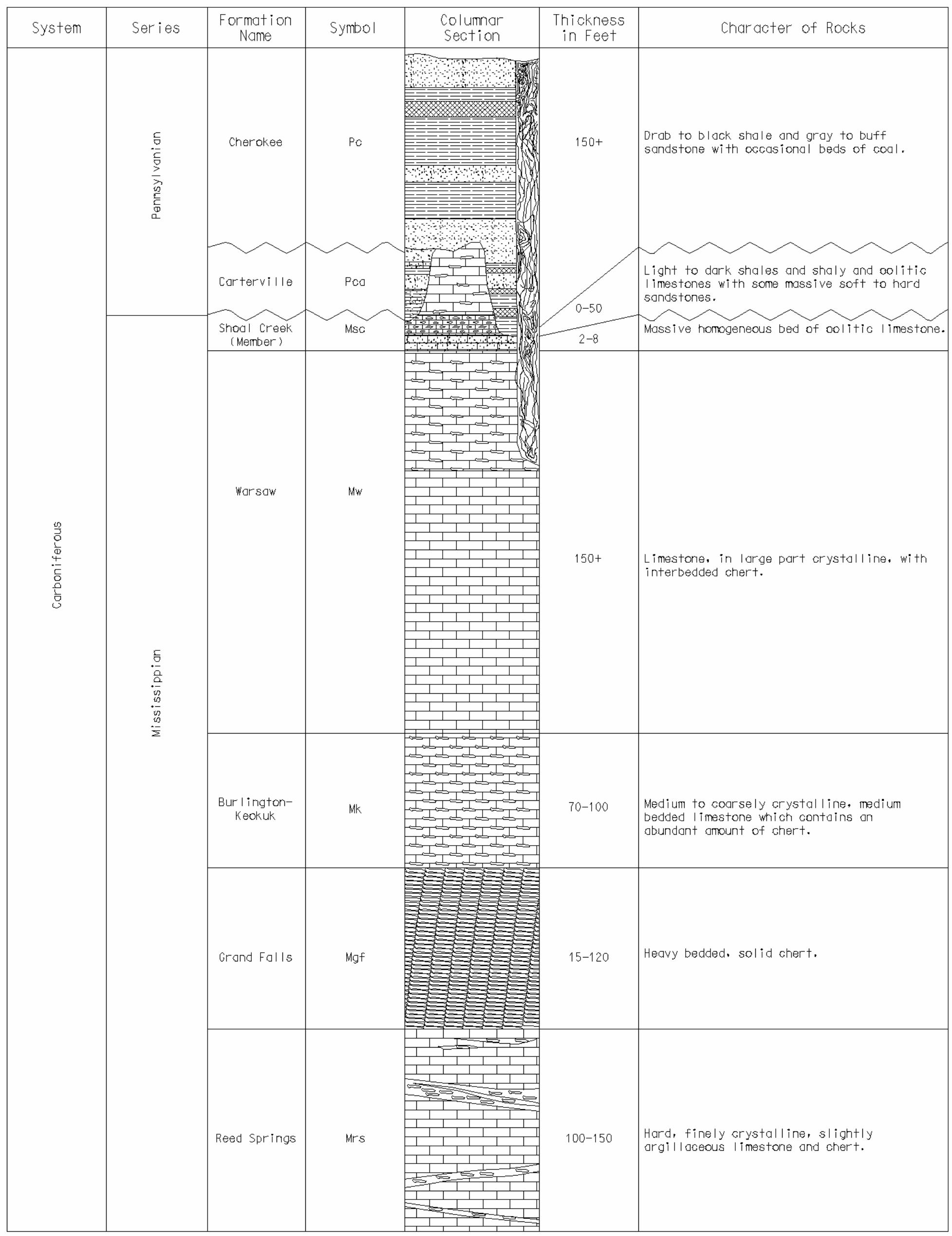


Figure 2. Generalized Geologic Column for Joplin, Missouri

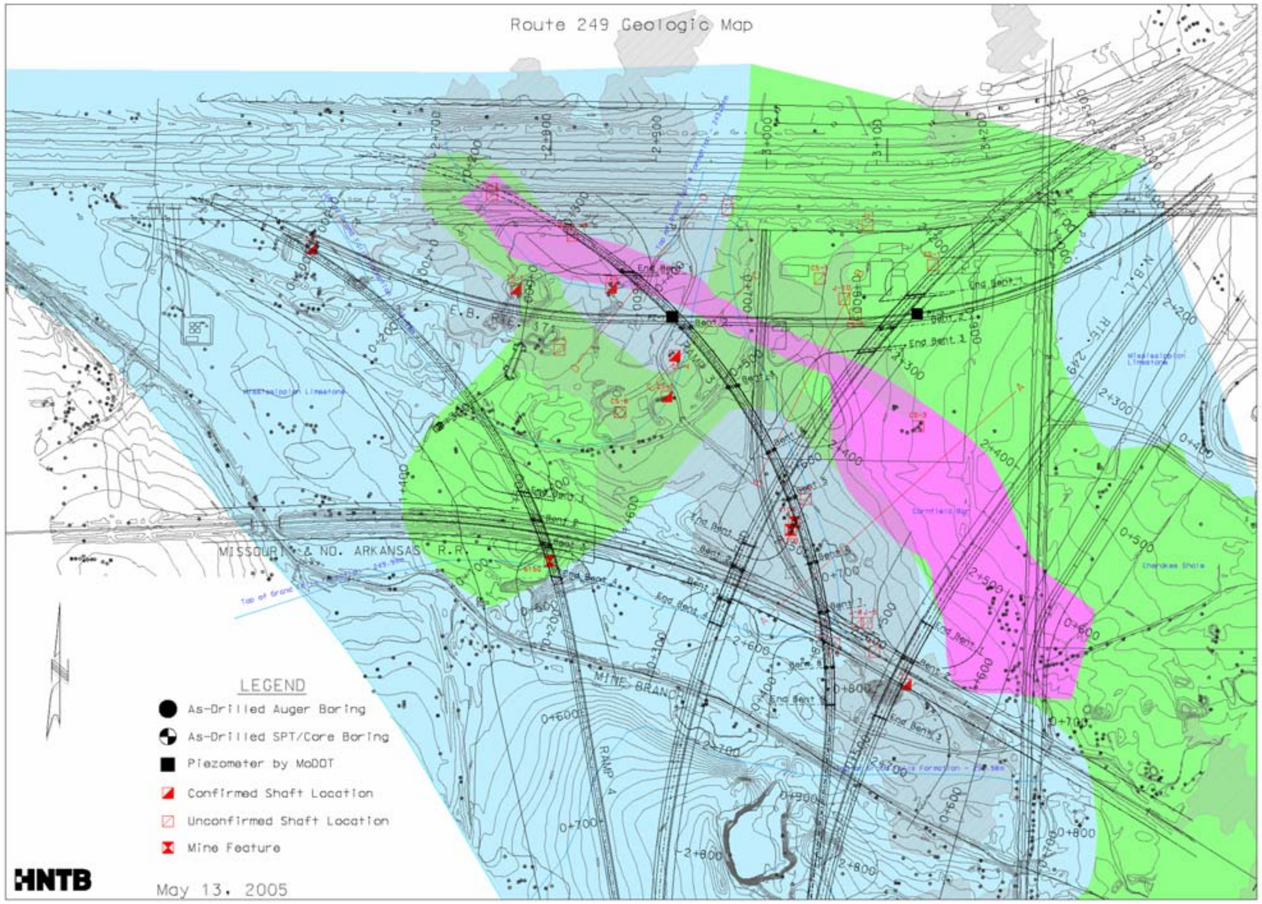


Figure 3. Geologic Map Cornfield Bar.

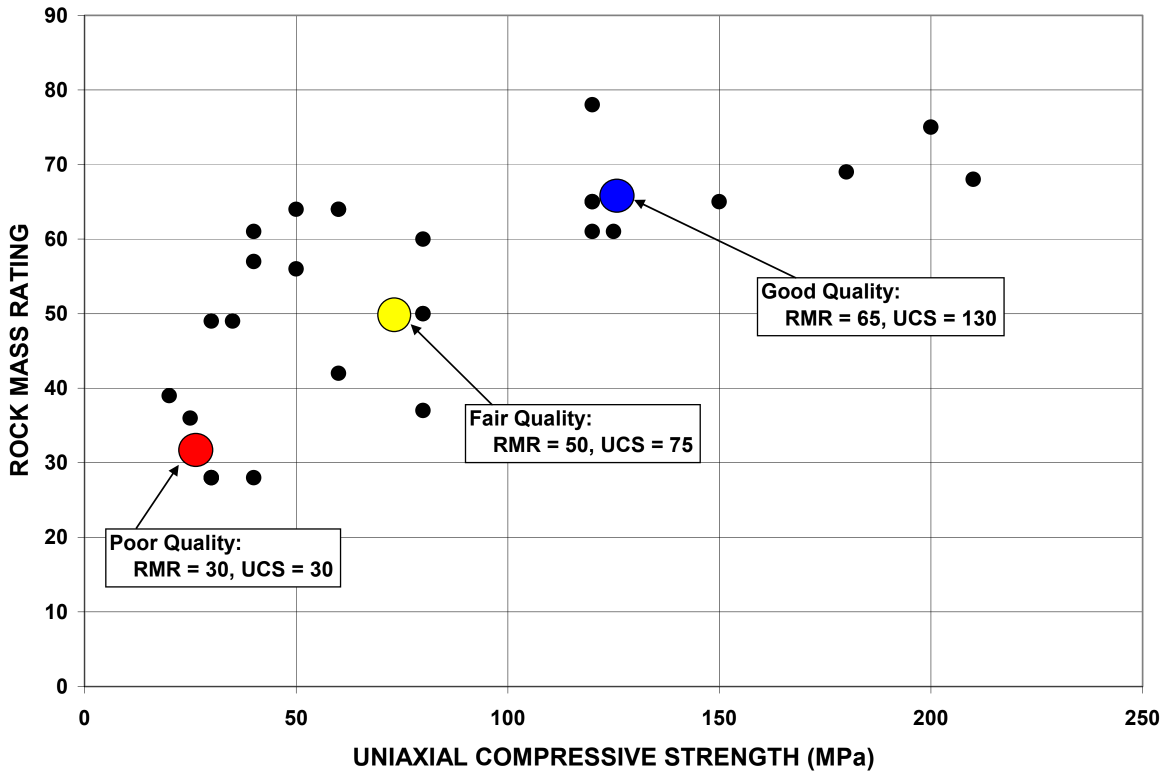


Figure 4. Rock Mass Ratings for Load Bearing Zones ("Roof Beam")

Figure 5. Example of “Good Quality” Rock Mass

Figure 6. Example of “Fair Quality” Rock Mass

Figure 7. Example of "Poor Quality" Rock Mass

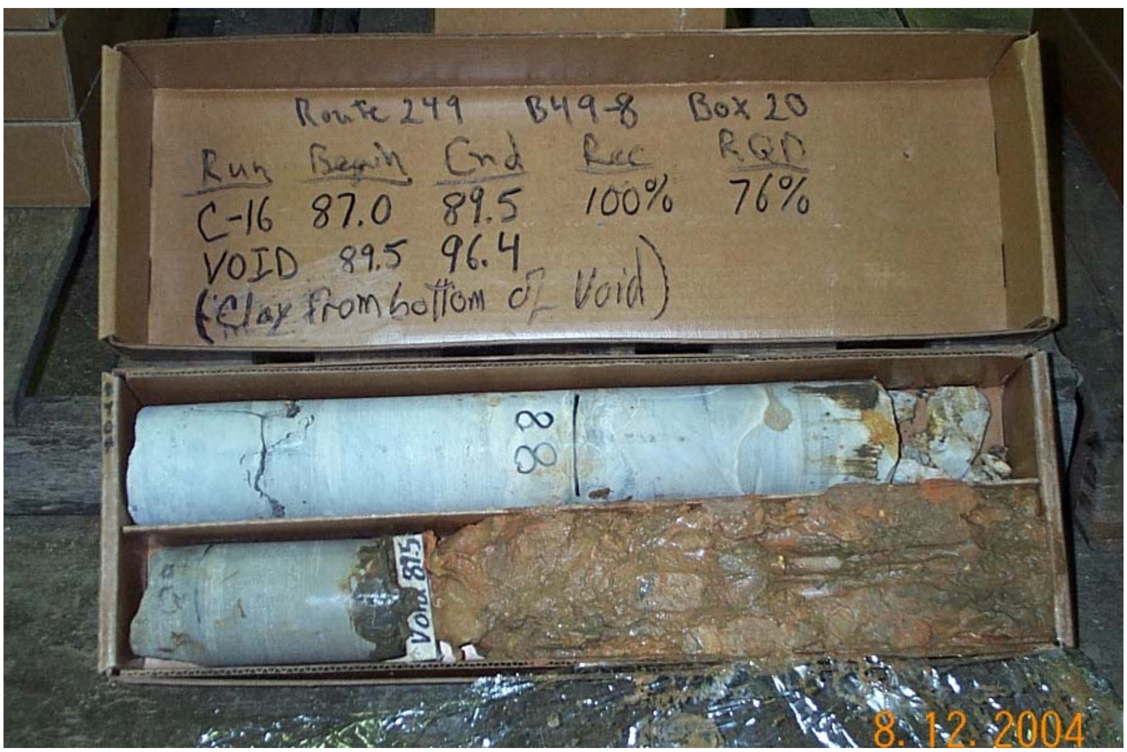


Figure 8. Character or Voids – Bridge A6149 Bent 7

Figure 9. Horizontal Continuity Estimation of Voids

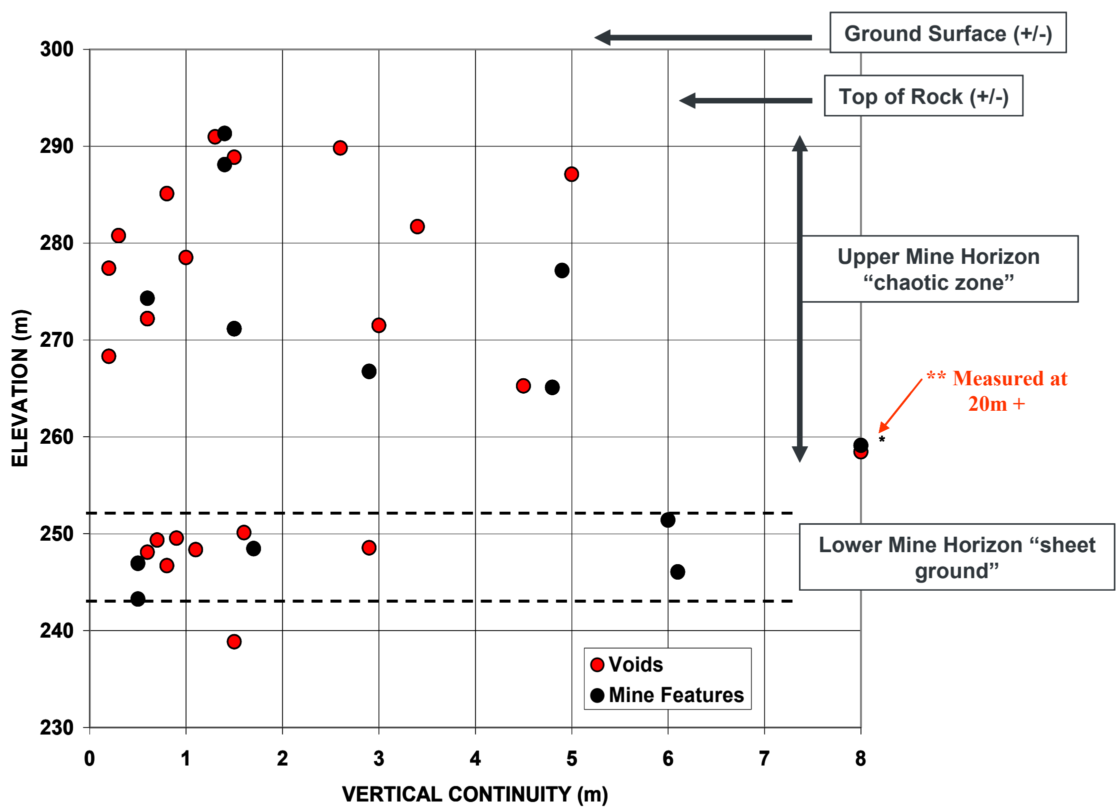
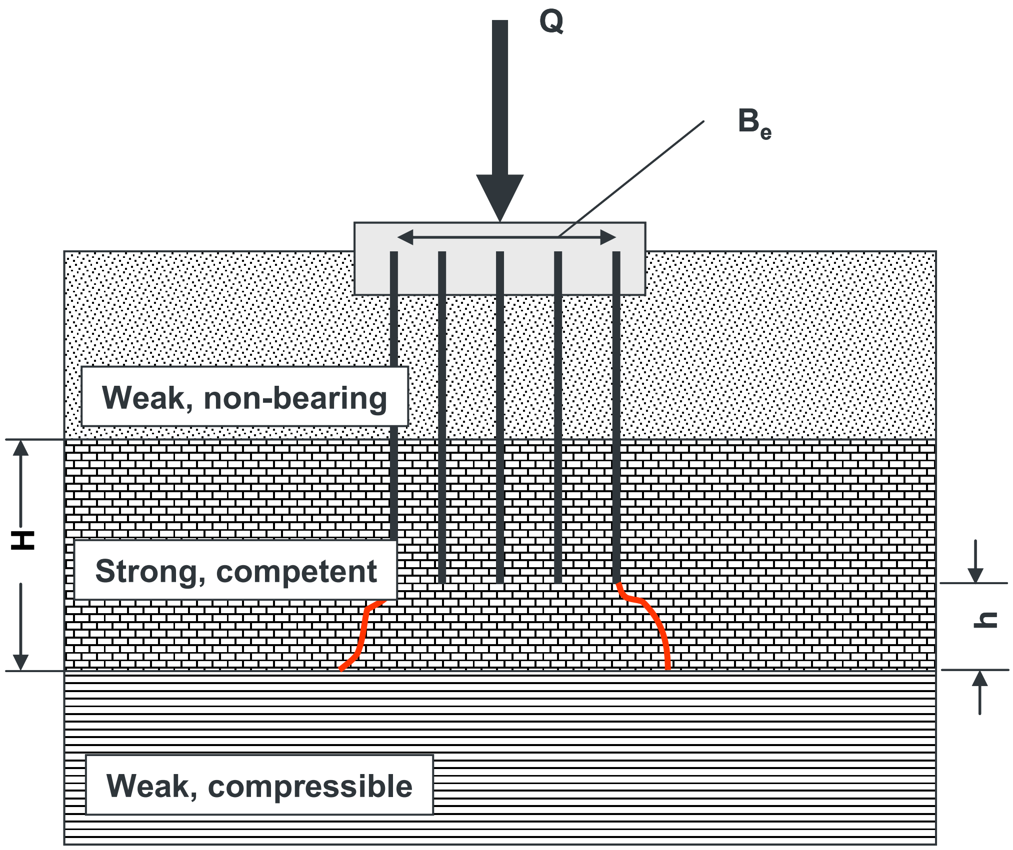


Figure 10. Vertical Continuity Estimation of Voids



Punching Failure:

FS = Resisting Force / Driving Force

$$= \frac{\pi B_e H (c + \sigma_N(\tan\Phi))}{Q + W_o + W_r}$$

Where:

c = cohesion
σ_N = normal stress
Φ = friction angle
W_o = weight overburden
W_r = weight rock

Figure 11. Punching Failure Analysis

FAIR QUALITY

10 m beam depth
20 m void width
Self weight

FAIR QUALITY

10 m beam depth
40 m void width
Self weight

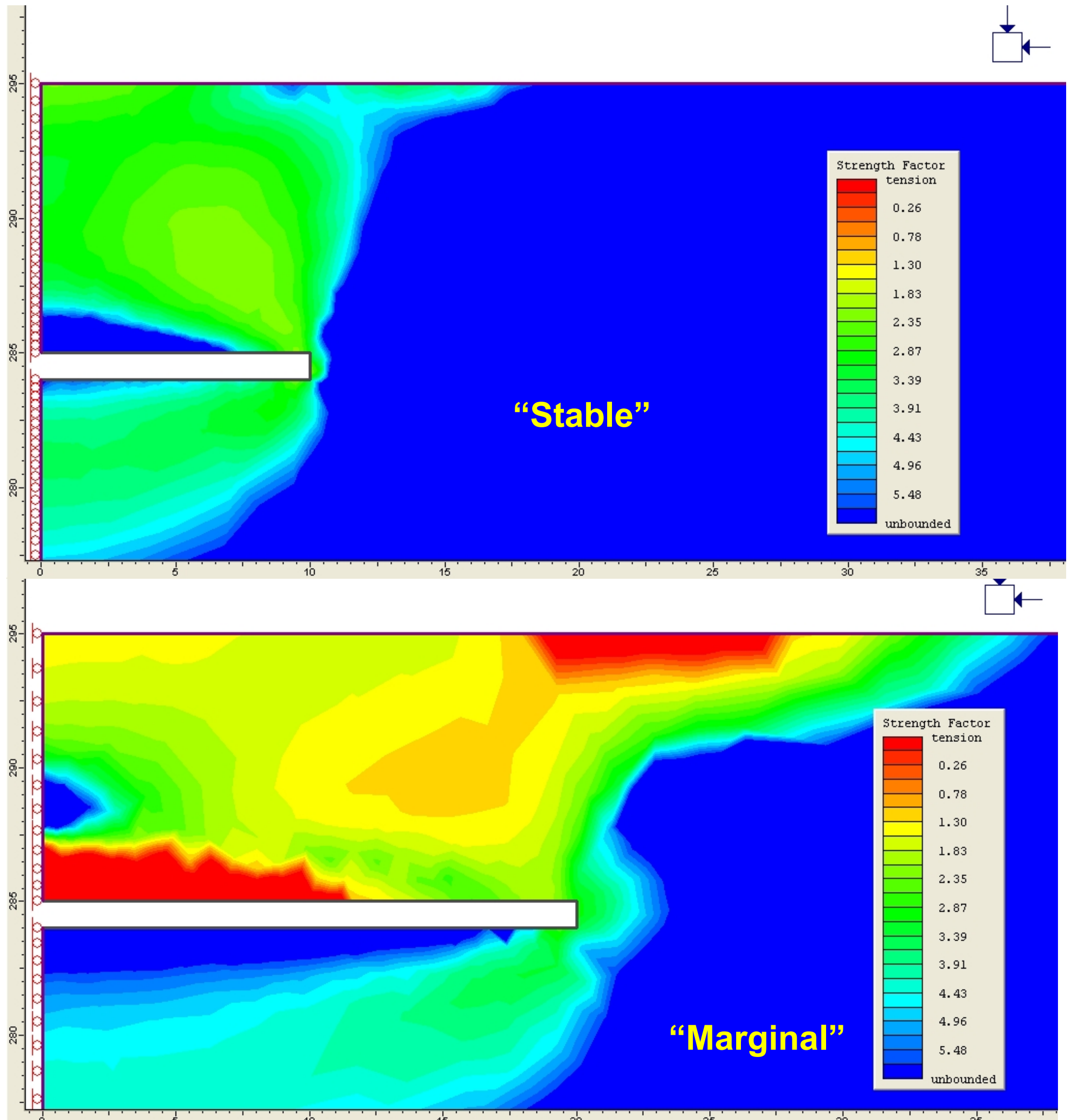


Figure 12. Finite Element Analyses – Roof Stability over Void

FAIR QUALITY

10 m beam depth
40 m void width
Foundation Load

FAIR QUALITY

10 m beam depth
60 m void width
Foundation Load

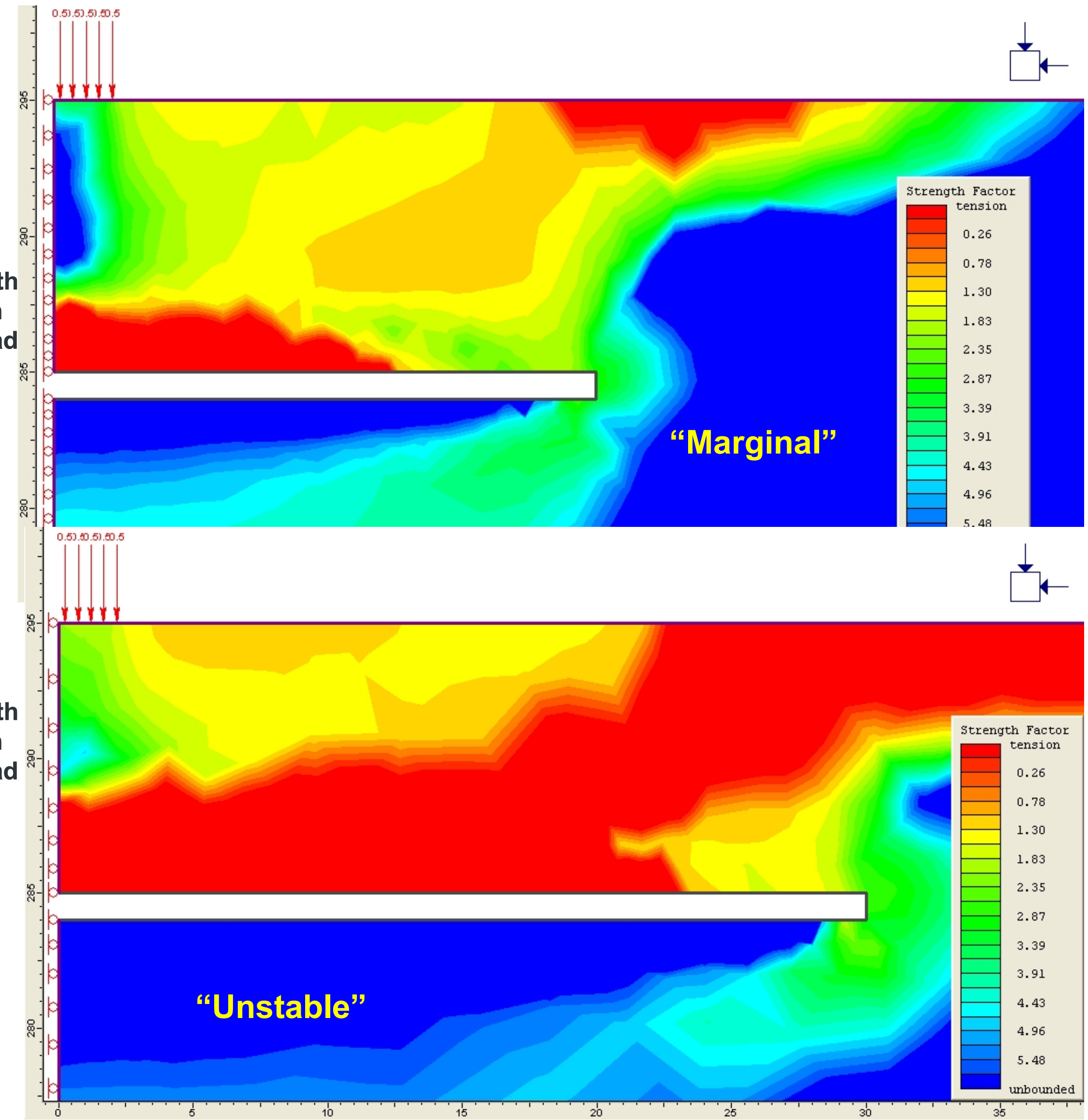


Figure 13. Finite Element Analyses – Roof Stability over Void with Foundation Load

GOOD QUALITY

5 m beam depth
60 m void width
Foundation &
Embankment Load

GOOD QUALITY

10 m beam depth
60 m void width
Foundation &
Embankment Load

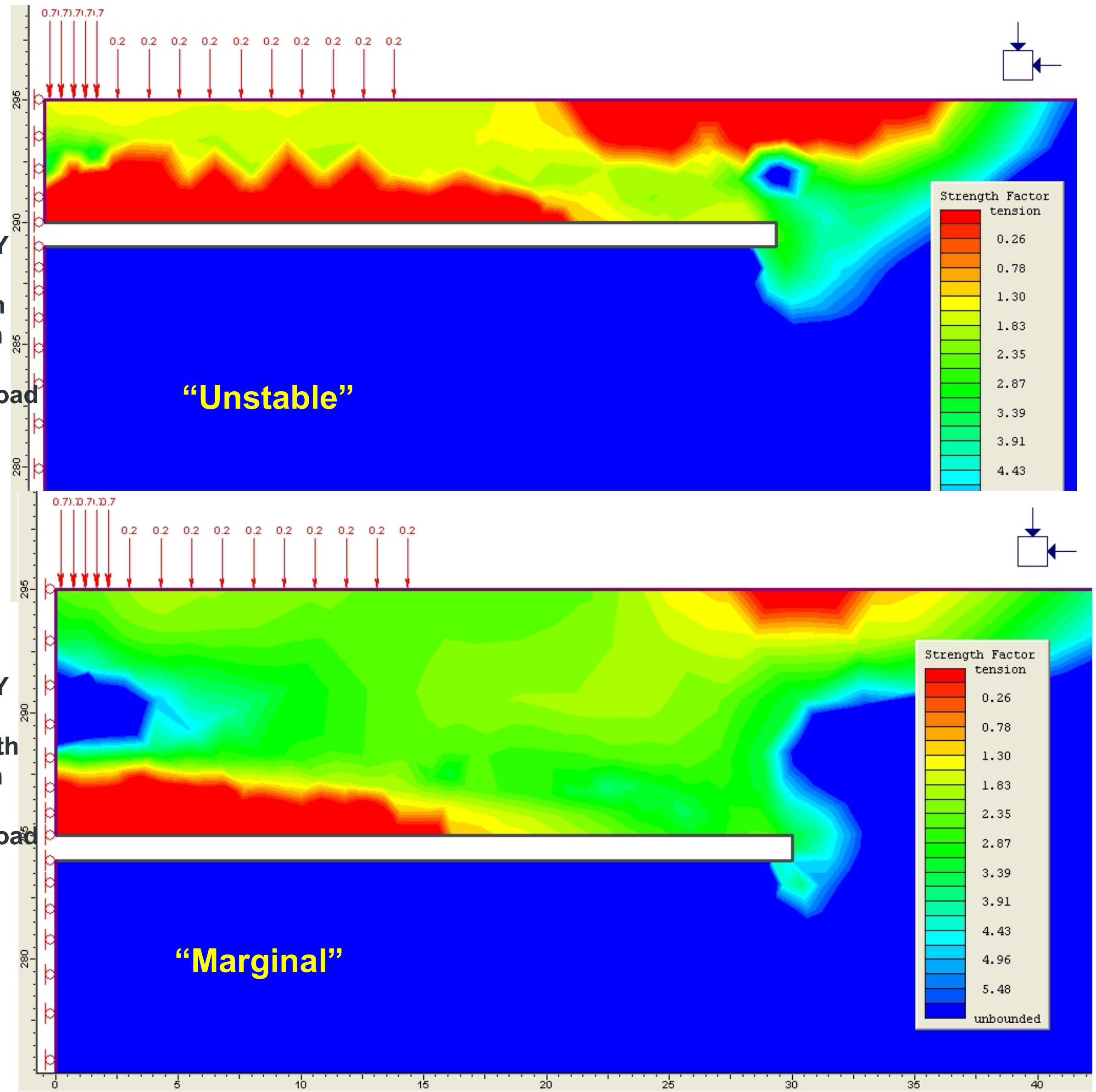


Figure 14. Finite Element Analyses – Roof Stability over Void with Foundation and Embankment Load

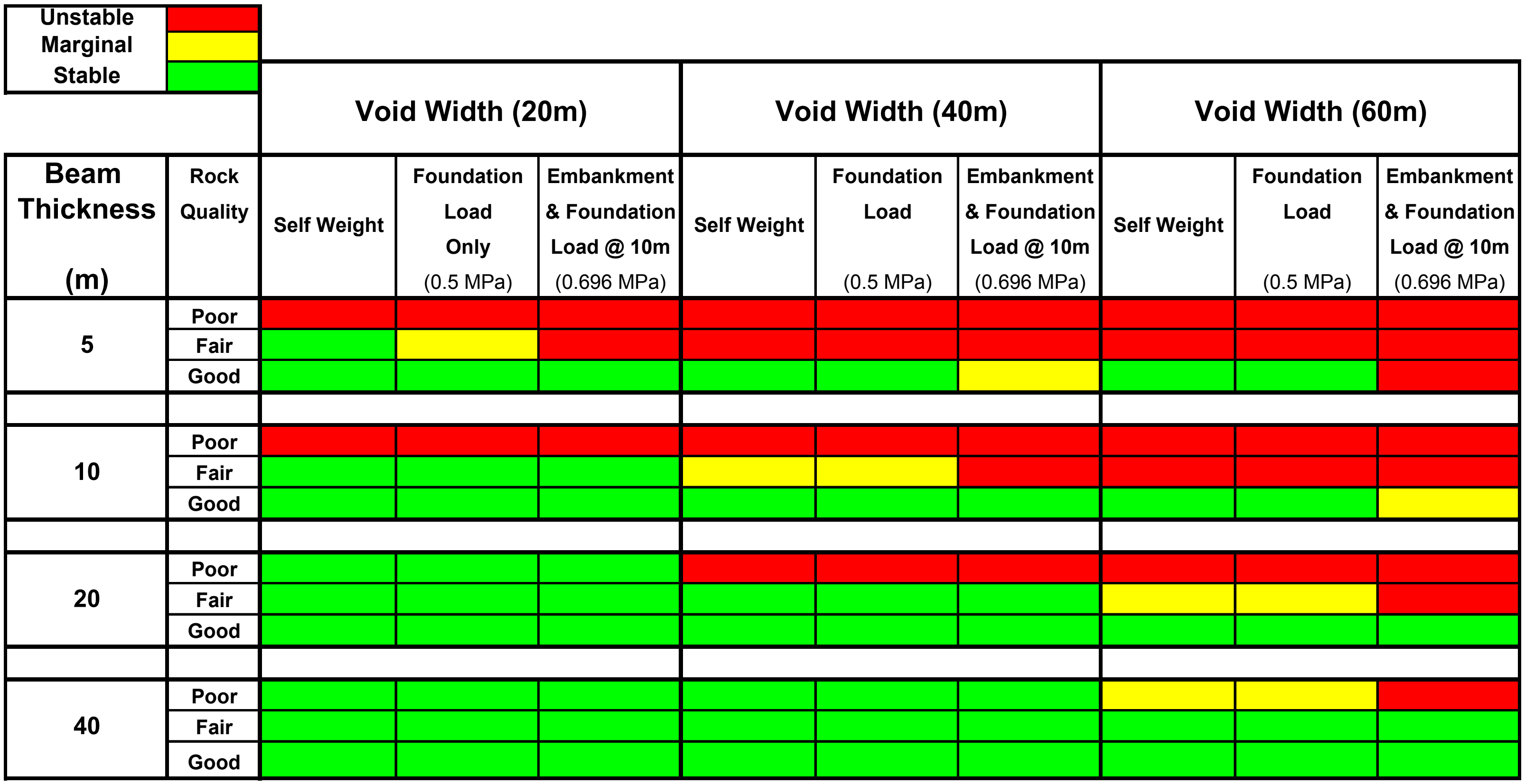
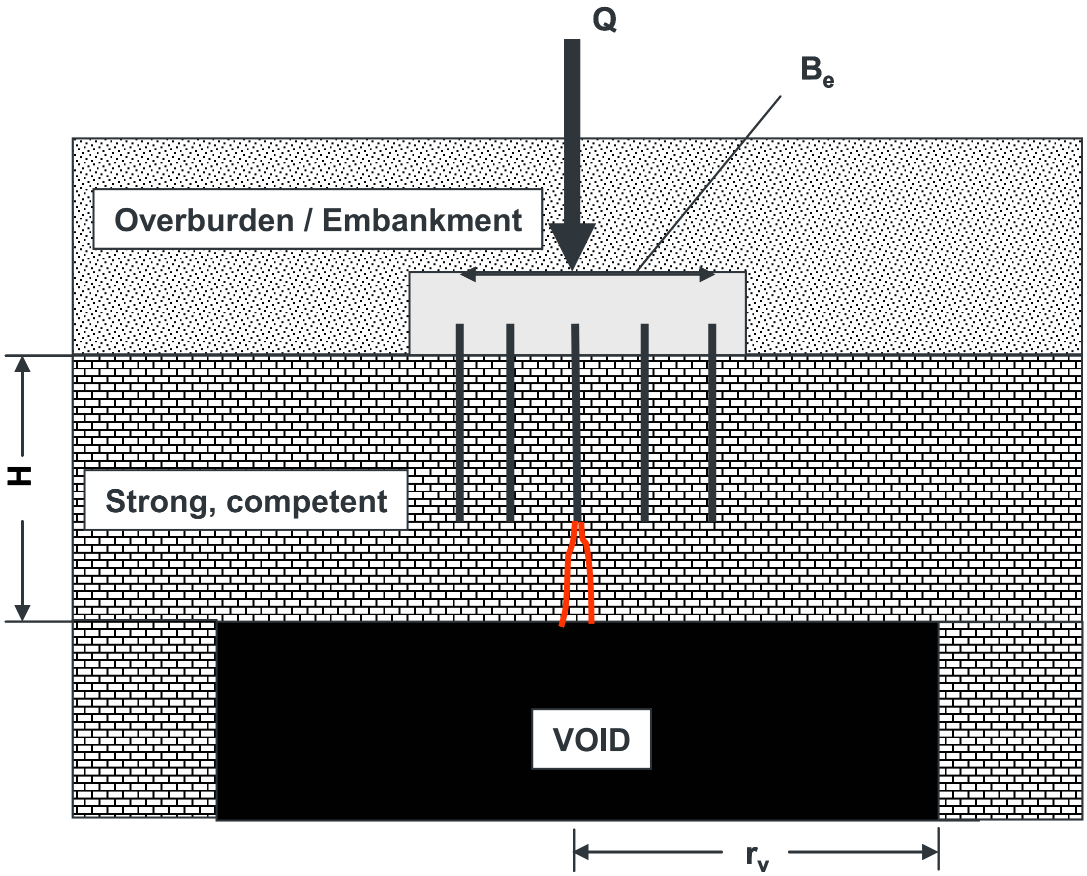


Figure 15. Rock Beam – Void Dimension Sensitivity based on Finite Element Analyses



Bending Failure:

FS = Resisting Rockmass Tensile Strength / Induced Tensile Stress (σ_t)

Where:

$$\sigma_t = 6M/H^2$$

$$M = \frac{(Q + W_o + W_r/2)}{4\pi} [(1+v)\log_e(r/r_0)+1], \qquad B>H, \text{ then } r_o = B/2$$

$$B<H, \text{ then } r_o = [1.6(B/2)^2+H^2]^{0.5} - 0.67H$$

v = Poisson's ratio, r_v = radius of void, W_o = weight of overburden, W_r = weight rock

Figure 16. Closed Form Plate Bending Analysis
(after Wyllie, 1999)

Legend	
Unstable	
Marginal	
Stable	

Phase2 Finite Element Analysis

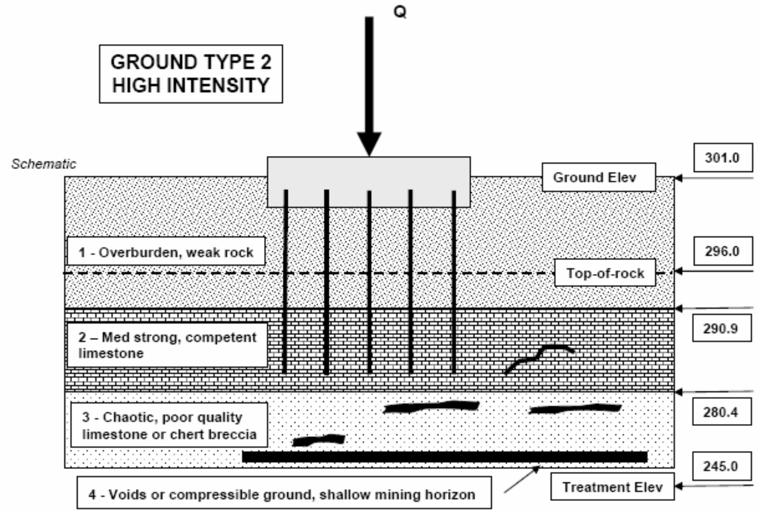
Beam Thickness (m)	Rock Quality	Void Width (20m) Self Weight	Void Width (20m) Foundation Load Only (0.5 MPa)	Void Width (20m) Embankment & Foundation Load @ 10m (0.696 MPa)	Void Width (40m) Self Weight	Void Width (40m) Foundation Load (0.5 MPa)	Void Width (40m) Embankment & Foundation Load @ 10m (0.696 MPa)	Void Width (60m) Self Weight	Void Width (60m) Foundation Load (0.5 MPa)	Void Width (60m) Embankment & Foundation Load @ 10m (0.696 MPa)
5	Poor									
	Fair									
	Good									
10	Poor									
	Fair									
	Good									
20	Poor									
	Fair									
	Good									
40	Poor									
	Fair									
	Good									

Legend	
Unstable	FS<1.0
Marginal	FS<2.5
Stable	FS>2.5

Closed Form Plate Bending Analysis

Beam Thickness (m)	Rock Quality	Void Width (20m) Self Weight	Void Width (20m) Foundation Load Only (0.5 MPa)	Void Width (20m) Embankment & Foundation Load @ 10m (0.696 MPa)	Void Width (40m) Self Weight	Void Width (40m) Foundation Load (0.5 MPa)	Void Width (40m) Embankment & Foundation Load @ 10m (0.696 MPa)	Void Width (60m) Self Weight	Void Width (60m) Foundation Load (0.5 MPa)	Void Width (60m) Embankment & Foundation Load @ 10m (0.696 MPa)
5	Poor		0.03	0.02		0.02	0.01		0.02	0.01
	Fair		0.28	0.21		0.22	0.16		0.19	0.14
	Good		1.53	1.12		1.16	0.85		1.02	0.74
10	Poor		0.12	0.09		0.08	0.06		0.07	0.05
	Fair		1.29	0.96		0.93	0.69		0.8	0.6
	Good		>5	>5		5	3.73		4.3	3.21
20	Poor		0.61	0.47		0.39	0.3		0.32	0.24
	Fair		>5	>5		4.27	3.29		3.52	2.71
	Good		>5	>5		>5	>5		>5	>5
40	Poor		4.39	3.55		1.9	0.77		1.43	1.15
	Fair		>5	>5		>5	>5		>5	>5
	Good		>5	>5		>5	>5		>5	>5

Figure 17. Comparison of Analytical Approaches

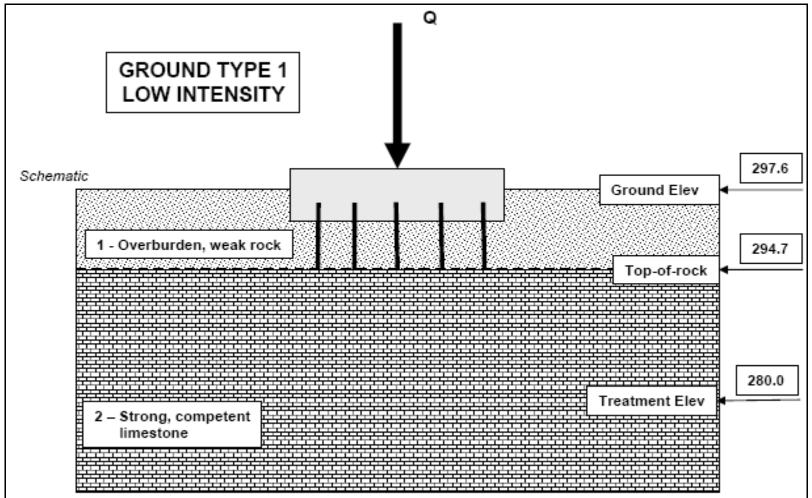


FOUNDATION DESIGN DATA

Layer	Top Elev	Bot Elev	UCS (MPa)	RQD (%)	FF (fr/m)	RMR
1 – O/B	302.0	290.9	n/a	n/a	n/a	n/a
2 – LS	290.9	280.4	30	60	11.5	49
3 – Poor LS	280.4	268.2	n/a	17	16.2	n/a
4 – Void	290	285	n/a	n/a	n/a	n/a

Primary Data Boreholes: H-00-23, H-00-17, H-00-22, H-00-26, H-00-27

Notes: No boreholes deeper than 280m elevation, heterogeneous rock quality with shallow voids
Assume same ground conditions as for borehole B40-1.
Coalescence of upper and lower mine workings.



FOUNDATION DESIGN DATA

Layer	Top Elev	Bot Elev	UCS (MPa)	RQD (%)	FF (fr/m)	RMR
1 – O/B	297.6	294.7	n/a	n/a	n/a	n/a
2 – LS	294.7	283.6	90	86	3.4	60
3 – Void	n/a	n/a	n/a	n/a	n/a	n/a

Primary Data Boreholes: B50-1.

Notes: Rock mechanics parameters apply to top 10 m of high quality rock.
No voids identified to elevation 262.5 m.

Figure 18. Presentation of Ground Conditions for Foundation Design

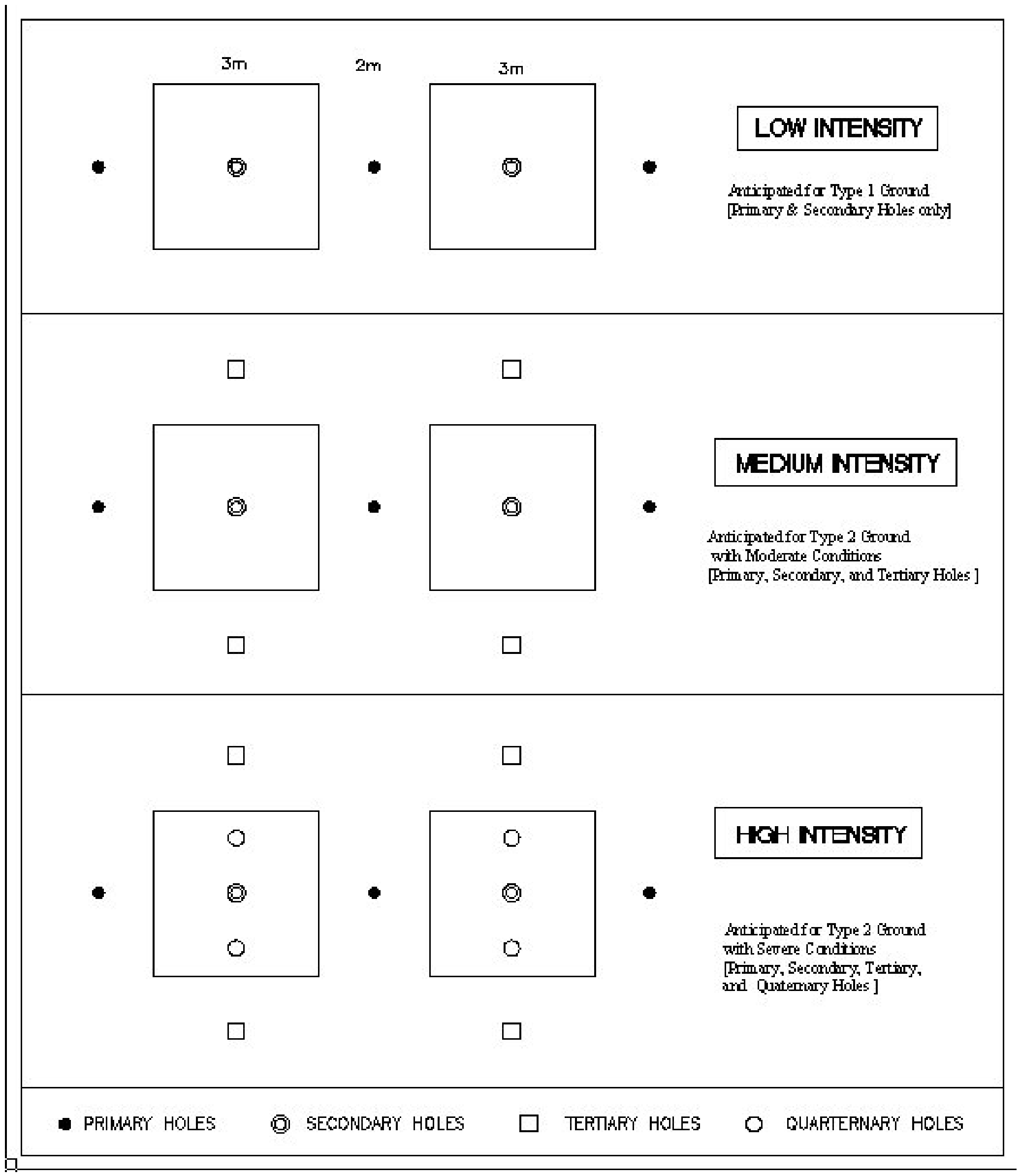


Figure 19. Treatment Intensity

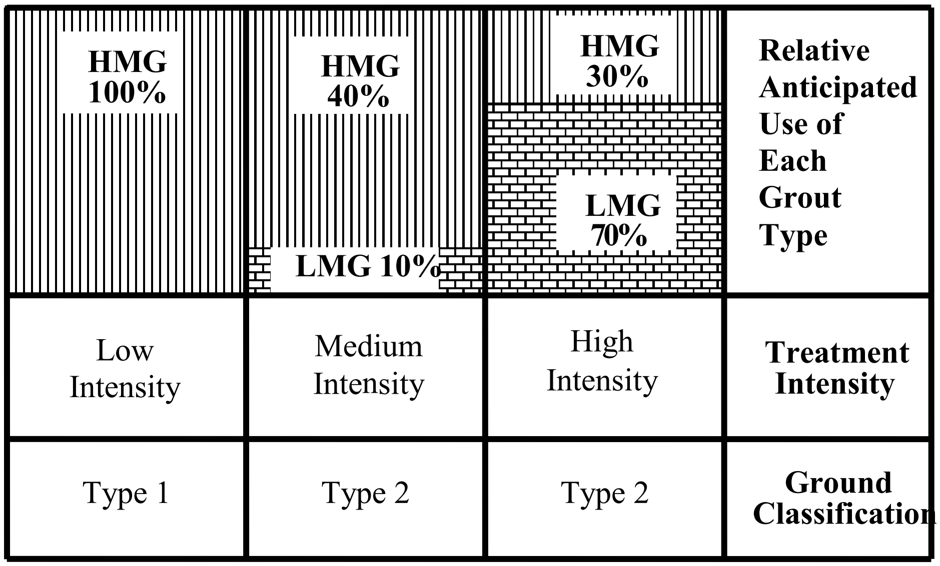


Figure 20. Anticipated Grout Type Use Versus Ground Classification and Intensity of Treatment.