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STABILITY ANALYSIS OF TABAS COAL MINE ROADWAY USING EMPIRICAL AND NUMERICAL METHODS

Ali Sahebi[1], Hossein Jalalifar[1], Mohammad Ebrahimi[1] and Ali Abdolrezaee[2]

ABSTRACT: Tabas coal mine is located south east of Tabas city, in Iran. The mine is the first fully mechanized coalmine in Iran that produces 4000 tonnes coal per day. Method of extraction is retreat longwall. One of the main problems in this mine, is the stabilization of entry roadways. In this research, five different methods were used to calculate potential rock loading on roadways, and according to the predicted rock load two types of section arches; V29 and V36, were considered for stabilization. Finally, the designed support system was numerically evaluated. From the numerical analyses, it was concluded that the roadway East1 Maingate could reach to the stabilised using V29 section arch.

INTRODUCTION

Tabas Coal Mine No.1 is located in a remote rugged desert environment approximately 85 km south of town of Tabas in Yazd province in mid Eastern Iran. In 1998, the National Iranian Steel Company (NISCO) issued an international tender for Tabas Coal Mine and NISCO has selected the Joint Venture Partnership of Iran International Engineering Company (IRITEC) and IRASCO as the preferred bidder. At the time, IRITEC/IRASCO as a contractor excavated the East 1 Main and Tail Gate to commission the 1st retreat longwall coal face (the East 1 Panel) and produce 1.5 million tonne coal annually. The mine is working seam C1. The seam gradient is 1 in 5 to 1 in 2 (11 ^{o} to 26°) in initial mining area. In E1 MG panel, the gradient has been observed to be between 19 ^{o} and 29 ^{o}. At E1 panel the seam thickness varied from 1.8 m to 2 m. The C1 seam coal has a uniaxial compressive strength of less than 6 MPa. There are some other seams C2 and D1 above and B1 and B2 below the C1 seam (IRITEC 1992). Figure 1 shows Mine No.1 and other districts of Tabas coal mine with location of the exploration shafts.

Table 1 - Core data and RMR parameters (Taghipoor 2008)

Depth into roof(m)	Rock Type	UCS (MPa)	RQD (%)	Discontinuity Spacing (m)	Discontinuity Condition	Ground Water	Discontinuity orientation	RMR
0 - 2.12	Sandy Siltstone	**32**	**18**	**0.06-0.2**	Slightly rough, separation<1mm	Dripping to dry	Very unfavourable	Class III–IV (30 – 41)
Rating		**4**	**3**	**8**	**23**	**4 - 15**	**-12**	**35.5**
2.12-3.35	Silty Sandstone	**73**	**26**	**0.06-0.2**	Slightly rough, separation<1mm	Dripping to dry	Very unfavourable	Class III–IV (38 – 49)
Rating		**7**	**8**	**8**	**23**	**4 - 15**	**-12**	**43.5**
3.35 -3.8	Sandy Siltstone	**32**	**49**	**0.06-0.2**	Slightly rough, separation<1mm	Dripping to dry	Very unfavourable	Class III -IV (35 – 46)
Rating		**4**	**8**	**8**	**23**	**4 - 15**	**-12**	**40.5**
3.8- 4.75	Sandstone	**73**	**43**	**0.06-0.2**	Slightly rough, separation<1mm	Dripping to dry	Very unfavourable	Class III -IV (38 – 49)
Rating		**7**	**8**	**8**	**23**	**4 - 15**	**-12**	**43.5**

A 4.75 m long roof core taken in E1MG panel revealed that the roof strata was made of layers of siltstone, sandy siltstone and silty sandstone above the roof of the MG. According to Table 1, the sequence of the stratification above the coal seam and other details are as indicated. For simplicity in modelling, all sandy siltstone and silty sandstone were considered as siltstone and sandstone,

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respectively. The use of TH section arches are being considered for the roadway at Tabas coal mine and it is understood that both V29 and V36 section arches are under consideration. These notes examine the use of TH arches in this situation. Support of the immediate portal area, i.e. the first few arches set under the excavation lip is considered in a separate note.

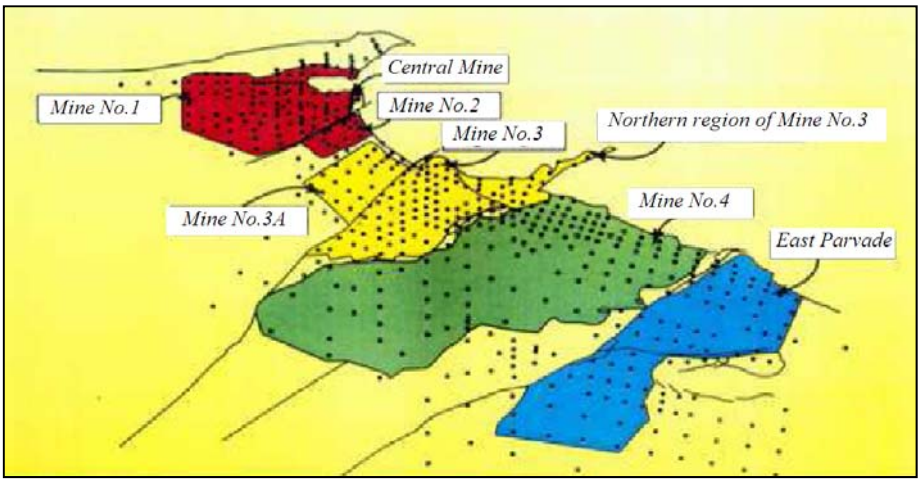

Figure 1 - Districts of Tabas coal mine and location of exploration shafts (not to scale, IRITEC 1992)

ROCK LOADING

Five different methods were used to calculate potential rock loading on roadways. These were as follows:

- Airey loosened zone approach
- Geomechnics rock mass classification system
- National Coal Board (NCB) loosened zone approach
- Terzaghi design method
- Whittaker and Hodgkinson loosened zone approach

In all methods, a rock density of 2.6 (tonnes/m^3) was assumed. Rock / Support interaction analysis was considered as a possible method of estimating support requirements but this was not pursued due to the lack of reliable geological / geotechnical data. If such data becomes available, estimates can be made then this approach may be re-considered as it offers a good system of design for standing supports.

Airey loosened zone approach

This assumes that a loosened zone exists above a mine roadway, created by the roof strata fracturing into a triangular shaped loosened zone governed by the angle of friction of the rock mass (Final report ECSC 1982).
Figure 2 shows the general principle of Airey Triangular Loosened Zone and this gives the following rock loads.

- Angle of friction (F1) = 23 ^{o}.
- RMR = 40

In Table 2 result was shown.

$$Hp = \frac{\left(\frac{w}{2}\right)}{\tan F_1} \quad (1)$$

$$P = \left(\frac{w}{2}\right) \times Hp \times \gamma \quad (2)$$

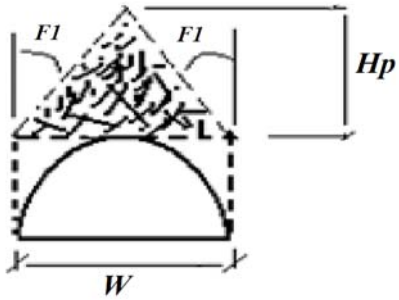

Figure 2 - Airey triangular loosened zone method

Table 2 - Rock loads calculated by Airey Method

Roadway Width (m)	Rock Load (tonnes/m)
4.5	31
5	38.28
5.6	48.02

Geomechanics rock mass classification system

The Geomechanics rock mass classification system allows a RMR to be determined for the given rock mass. One of the outputs from this system is a method of determining rock load, P (Bieniwski 1989). This is given as follows:

$$Hp = \left(\frac{100 - RMR}{100}\right) \times W \quad (3)$$

$$P = Hp \times W \times \gamma \quad (4)$$

Where;
W = Roadway Width (m)
γ = Rock Density (tones / m^3)

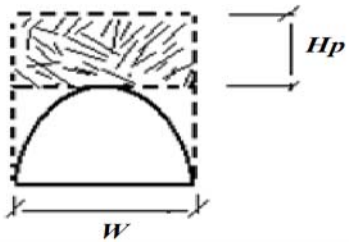

Figure 3 - Geomechanics system rock load height method

Table 3 - Rock loads calculated, GRMC method

Roadway Width (m)	Rock Load (tonnes/m)
4.5	31.59
5	39
5.6	48.92

National Coal Board (NCB) loosened zone approach

This assumes that a triangular loosened zone exists above the mine roadway which loads the stand in supports. Figure 4 shows general principle. For ease of calculation, the loosened zone is assumed to be triangular with a height of 1 to 1.5 times roadway width (National Coal Board MRDE 1970)

$$Hp = (1 \text{ to } 1.5)W \quad (5)$$

$$P = \left(\frac{W}{2}\right) \times Hp \times \gamma \quad (6)$$

Terzaghi design method

Using a combination of modal tests and observations of load on steel arch supported roadways, Terzaghi proposed a rock load classification system for steel arch supported roadways. He subdivided his classification into nine categories to cater for a variety of conditions from "Hard and Intact" to "Swelling" rock. The category chosen for this estimation is "Very Blocky and Seamy" as it is the most appropriate of the categories to suit the anticipated conditions at Tabas coal mine (IRITEC 1992). For this condition, the rock load height, Hp is given as follows:

$$Hp = (0.35 \text{ to } 1.1) \times (\text{Roadway Width} + \text{Roadway Height}) \quad (7)$$

$$P = W \times Hp \times \gamma \quad (8)$$

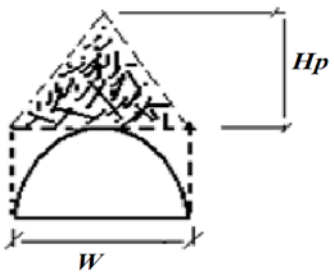

Figure 4 - National coal board triangular loosened zone method

Table 4 - Rock loads calculated, NCB method

Roadway Width (m)	Rock Load (tonnes / m)	
	H=W	H=1.5W
4.5	26.32	39.48
5	32.5	48.75
5.6	40.76	61.15

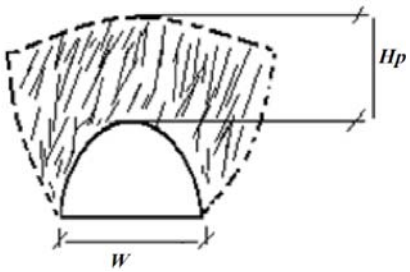

Figure 5 - Terzaghi loosened zone method

Table 5 - Rock loads calculated Terzaghi method

Roadway Width (m)	Roadway Height (m)	Min. Load (tonnes/m)	Max. Load (tonnes/m)
4.5	3.5	32.76	102.96
5	3.5	38.61	121.55
5.6	3.5	46.3	145.74

Whittaker and Hodgkinson loosened zone approach

This is similar to the National Coal Board approach but assumes that the loosened zone is semi elliptical in shape and extends to a height equivalent to the width of the roadway. Figure 6 shows the general principle. (Whittaker and Hodgkinson 1971)

$$Hp = W \quad (9)$$

$$P = \left(\frac{W}{2}\right) \times \left(\frac{\pi}{2}\right) \times Hp \times \gamma \quad (10)$$

ESTIMATION OF ROCK LOADS

After using five different methods to calculate potential rock loading on different width roadways, the results of rock loads are summarized in Table 7.

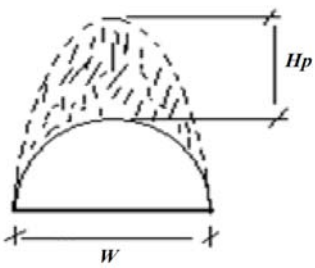

Figure 6 – Whittaker and Hodgkinson loosened zone method

Table 6 - Rock loads calculated W&H method

Roadway Width (m)	Rock Load (tonnes/m)
4.5	41.35
5	51.05
5.6	64.03

Table 7 - Estimation of rock load based on various method

Roadway Width (m)	Rock load for given Design Method (tonnes/m)							
	Airey	Geomech. (GRMC)	NCB (1)	NCB (2)	Terzaghi (1)	Terzaghi (2)	Whittaker & Hodgkinson	Mean
4.5	31	31.59	26.32	39.48	32.76	102.96	41.35	**33.94**
5	38.28	39	32.5	48.75	38.61	121.55	51.05	**41.91**
5.6	48.02	48.92	40.76	61.15	46.3	145.74	64.03	**52.57**

Note on the above table:

- Only the Airey and geomechanics approach take geotechnical parameters into consideration.
- The Airey or geomechanics approach can be seen to give a good agreement with the mean and should be used if a quick approximation is required.
- NCB(1) – Triangle height equals roadway width.
- NCB(2) - Triangle height equals 1.5 times roadway width.
- Terzaghi(1) – Hp equals 0.35 times (Roadway width plus height).
- Terzaghi(2) – Hp equals 1.10 times (Roadway width plus height).
- Mean does not include Terzaghi loads as they are clearly outside the parameters given by the other methods.

ROCK LOADS FOR DESIGN PURPOSES

Apart from the immediate portal area which may be subject to dead loading, and is discussed elsewhere, there are two distinct areas along the declines to consider. These are the seismic zone and the remaining length inbye of this section (normal zone). A review of the methods for design of roadways in seismic active areas revealed that it is common practice to allow 15% addition to the static rock load in order to cater for seismic events. The following Table 8 gives rock loads for design purposes and incorporates this recommendation. It should be noted that no Factor of Safety has been incorporated into these rock loads.

Table 8 - Rock loads for design purpose

Roadway Width (m)	Rock load for given section of the decline (tonnes /m)	
	Normal	Seismic
4.5	33.31	38.3
5	40.43	46.5
5.6	49.86	57.33

Table 9 - Collapse loads for sections of TH arch

Arch type	Collapse load for Design purposes of given TH section (tonnes)
V29	35
V36	52.9

Table 10 - Recommended spacing of the TH

Arch Type	Recommended Spacing (m)	
	Seismic Section	Normal Section
V29	0.58	0.66
V36	0.87	1

REQUIRED SUPPORT

The use of TH section arches are being considered for the declines at Tabas and it is understood that a support with 15.5 m^2 cross section is preferred. Both V29 and V36 section TH arches are considered here with a base width, internal, of 5 m giving a maximum excavated width of 5.6 m. The collapse loads for TH arches are summarized in appendix1. These have been compiled from theoretical work and the results of actual laboratory tests. From appendix 1, the collapse loads for design purposes are as follows for these sections of TH arch. From the above information, the recommended spacing of the TH arches is as follows.

It should be noted that this spacing are the theoretical values and in practice conventional spacing would most probably be used (e.g. 0.5 instead of 0.58 etc.) although it would not be inconceivable to manufacture special struts for this project.

DISCUSSION

TH arches are of the yielding type and the load capacities quoted from the test results are obtained by ensuring that the yield clamps do not slip. This is usually achieved by welding them together. In underground use, of course, the clamps can slip and this type of arch is designed to close (i.e. reduce its internal cross section) as load is applied. Yielding arches can accept a higher strata movement than conventional rigid arches. However, high lateral movement or eccentric loads can result in the clamps locking which can lead to early failure of support. An even load distribution around the arch is critical if optimum performance is to be achieved with a TH arch. It could be argued that a long life decline is the place where yield and hence closure cannot be tolerated. In this case, a strong, rigid arch would be preferable. The report concluded that in drill and blast excavated roadways rigid arches provided better support and roadways conditions, with the possible exception of floor heave, than yielding TH arches of comparable size in conventional gate roadways. In general, the TH arches exhibited about 30% greater vertical closure than rigid arches. This trial also showed that TH arches had a slight advantage over rigid arches in machine cut conditions provided that the roof strata was strong enough to retain the cut profile and eliminate point loads.

The use of TH arches in the "seismic section" could be an advantage due to the yielding nature of the arch gives greater flexibility it their application. The arch will require to be well packed to the strata in order to function well, but this applies to any section of roadway supported by TH arches, particularly in a drill and blast section.

NUMERICAL MODELING

Usual support system of coal mines in Iran is steel arches of type TH section, so it was decided to design a suitable support system of this kind for E1MG roadway. Calculation the support pressure, were used to construct a model using the FLAC ^{2D} software. The model results showed that; steel arch V29 with spacing of 1.0 m is the best support system for this type of roadway. Figure 7 shows the roadway East 1 main gate profile that is supported by steel arch V29.

ROCK MASS PROPERTIES

To provide input parameters (rock mass properties) for the numerical simulation, Roclab program (based on GSI classification, GSI=RMR-5) was used (Rocscience, 2002). Table 11 displays the intact rock and rock mass properties.

Table 11 - Intact Rock and Rock Mass Parameters (IRITEC 1992)

Depth into Roof (m)	Intact Rock			Rock Mass							
	UCS (MPa)	m*	GSI	m	s	C (MPa)	Φ (deg.)	Tensile Strength (MPa)	Compressive Strength (MPa)	E (GPa)	υ
Coal	5	1	25	0.1	0.0002	0.1	20	0	0.5	0.7	0.26
0 - 2.12	32	7	30.5	0.58	0.0004	0.5	23	0.02	0.5	1.7	0.26
2.12 - 3.35	73	13	38.5	1.45	0.0011	1	44	0.05	2.2	3.5	0.25
3.35 - 3.8	32	7	35.5	0.7	0.0008	0.6	31	0.03	0.8	2.7	0.25
3.8 - 4.75	73	13	38.5	1.45	0.0011	1	44	0.05	2.2	3.5	0.25

IN SITU STRESS AND SIMULATION

No field measured value for the in situ stress was available. The E1 MG was at a depth of about 210 m around the coring position. Then a vertical stress of about 5.7 MPa and the ratio of horizontal to vertical stress K=0.33 was considered for the site, according to tectonic history of the region (Taghipoor , 2008)

The numerical modelling package, FLAC ^{2D}, was used to conduct the numerical simulations. The code is restricted to 2dimensional problems, hence only cross sections through the roadway are presented. These problems were analyzed on the assumption of plane strain along the axis normal to the plane of the model. Figures 8 and 9 show the tunnel convergence and shear strain around the tunnel respectively. As it can be seen from the Figures 8 to 11 and Tables 13 and 14, the value of displacements and shear strain increment (especially in roof and floor of roadway) around the roadway are high. It means roadway needs to be supported.

NUMERICAL ANALYSIS

To investigate the tunnel stability the Sakurai method and et al. (1994) was used. The method evaluates the critical strain in the elastic region. Since the rock mass is under triaxial stress, it is logical to use the maximum critical strain for investigation of roadway stability. They suggested following equation [Lotfi, 1999]:

$$log\ \varepsilon_c = -0.25\, log\ E - 1.22 \tag{11}$$

$$\gamma_c = (1+\nu)\,\varepsilon_c \tag{12}$$

Where;

E = Young's modulus of intact rock $\left({}^{kgf}\!/_{cm^2} \right)$

ε_c = critical strain in UCS state

γ_c = critical strain

ν = Poisson's ratio

Critical displacement values based on the critical strain are obtained by following equation (Lotfi. 1999)

$$\varepsilon_c = \frac{U_c}{a} \tag{13}$$

Where;

U_c = Allowable displacement;

a = radius of the roadway;

The maximum horizontal and vertical displacements around the tunnel before the steel arch installation are shown in Table 13. Table 14 shows the critical strain values around the tunnel. As it shows, the strain value is more than the allowable strain values, which causes the instability of roadway. Table 12 shows the properties of V29 steel arch.

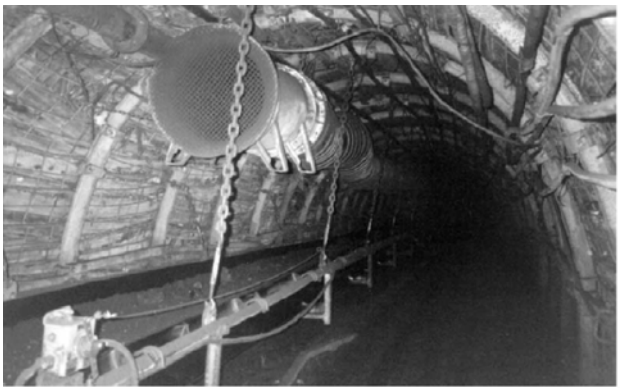

Figure 7 - The roadway East 1 main gate profile

Table-12 - V29 steel arch properties

Wx (cm^3)	Ix (cm^4)	Area (cm^2)	Width (mm)	Height (mm)	Weight (kg/m)
94	616	37	151	124	29

Table 13 - Horizontal and vertical displacement of E1MG (mm)

Model	Roof	Floor	Right hand Rib	Left hand rib
No Support	18.9	17.3	51.3	66
TH Arch V29	4.5	5.98	3.3	8.96
Critical Displacement	10.2	10.2	18.3	18.3

Table 14 - Maximum shear strain increment around E1MG (10^{-3})

Model	Roof	Floor	Right hand Rib	Left hand rib
No Support	4.35	40.6	55.3	94.4
TH Arch V29	1.26	2.16	2.93	1.31
Sakurai Shear Strain	4.54	4.54	6.48	6.48

The numerical results after steel arch installation showed that TH Arch V29 is suitable to support the E1MG. After installation it was observed that the critical strain values on roadway walls and roof were less than the permitted value s which demonstrated the roadway stability. Figure 11 displays the vertical displacement after installation of steel arch V29 in E1MG, which shows that there is a good agreement with the experimental result.

CONCLUSIONS

From the empirical and analytical methods and numerical simulations following conclusions can be inferred.

- Horizontal and vertical displacements appeared to be high which showed roadway needs to be supported.
- Elastic strains are good indications to show the roadway instability. It means Sakoraei method is quite applicable to predict the tunnel instability.
- The numerical simulations indicated that there is a good agreement between empirical, numerical and field monitoring data.

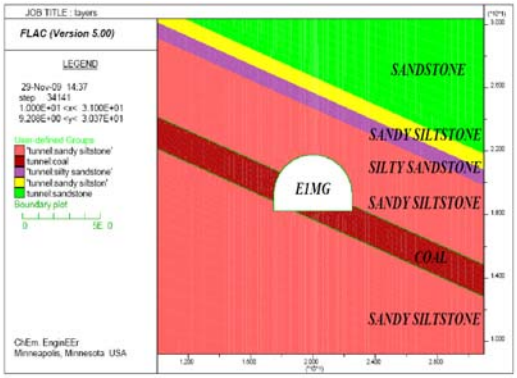

Figure 8 - Geological Sequence of strata used in the model

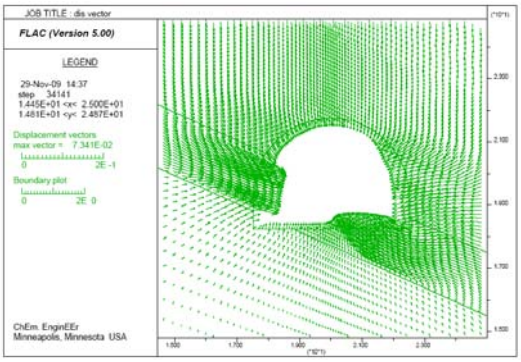

Figure 9 - Total displacement around the radway

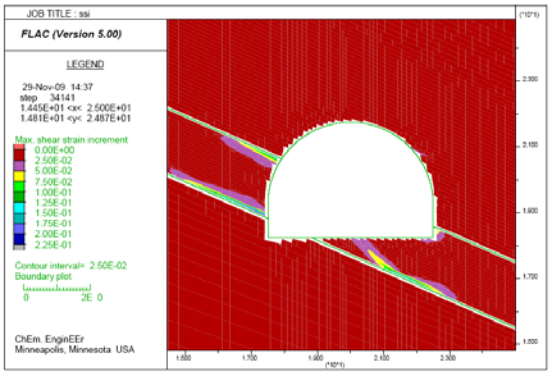

Figure 10 - Maximum shear strain increment before steel arch V29 was installed

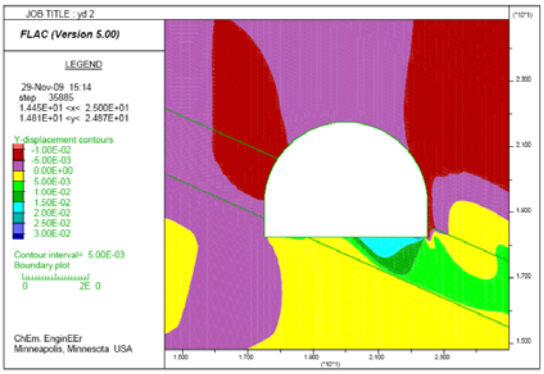

Figure 11 - Vertical displacement after steel arch V29 was installed

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APPENDIX 1: COLLAPSE LOADS FOR TH TYPE ARCHES

Arch Type	Arch Dia (m)	Theoretical Collapse Load (tonnes)	Collapse Loads from Practical Tests (tonnes)		Likely Collapse Load For Design Purposes (tonnes)
			Sadler (1984)	British Steel	
V29	5	41.3	32.5 to 44.2	22	35
V36	5	57.75	43.6 to 75 .3	35	52.9

Note on the above table:

- Theoretical loads given by equation, load = 2.2 Z / D where Z = section Modulus (cm^3), D = Arch dia (m) and is taken(Sadler,1984)
- British steel tests are given in the form of tables issued in about 1986. These tests were carried out for British Steel by the National coal Board (subsequently British coal). Some results quoted were extrapolated from tests on other sections using the section Modulus as the main criteria.