Pillar strength in underground stone mines in the United States

G.S. Esterhuizen *, D.R. Dolinar, J.L. Ellenberger

National Institute for Occupational Safety and Health, PO Box 18070, Pittsburgh, PA 15236, USA

A B S T R A C T

Stone mines in the Eastern and Midwestern United States make use of the room-and-pillar method of mining to extract relatively flat-laying sedimentary formations. A survey of pillar performance was carried out to identify potential modes of instability. Pillars were found to have been successful in providing support to the overburden, but a small number of individual failed pillars were observed. Failure of the pillars was observed to be related to spalling of the hard brittle rocks, shearing along pre-existing angular discontinuities or progressive extrusion of soft infill materials on bedding planes. A method of estimating the pillar strength and selecting a safety factor for design was developed based on observations of stable and failed pillars, supplemented by numerical models. The developed pillar strength equation can be used to design stable stone mine pillars provided the rock conditions are similar to those included in the study.

1. Introduction

There are more than 120 operating underground stone mines in the United States that use the room-and-pillar method of mining. Mining is typically conducted in flat laying or gently undulating deposits ranging from highly siliceous limestone to chemical-grade dolomite and limestone. Large mechanized equipment, such as 50-ton dump trucks and 5-m^3 wheel loaders, are used to achieve the required economy of scale. The mining equipment requires adequately sized openings for effective operation. Where the thickness of the deposit allows it, second pass bench-mining of the floor may be carried out.

The function of pillars in room-and-pillar mines is to provide both local and global stability. Local stability is defined as the provision of stable ribs (pillar walls) and stable roof conditions, allowing safe access to working areas. Falls of ground from the roof and pillar ribs account for about 15% of all injuries in underground stone mines [1] and are related to local instabilities. Regional stability is defined as the need to support the weight of the overburden rocks up to the ground surface. Inadequate regional stability can result in the collapse of multiple pillars over large sections of a mine, which can result in significant safety hazards [2,3]. A review of the literature revealed that two cases of multiple pillar failures have occurred in stone mines in the United States [3], both without injury or fatalities. One of the cases was assessed to be related to punching of the pillars into a weak floor stratum, while the other case appears to have been the collapse of irregular sized pillars [4]. Insufficient data are available to estimate the pillar stress at the time of failure of the latter case.

The design of slender pillars in hard brittle rocks has received considerable attention in recent years [5–8]; however, a widely accepted design method does not exist for pillars in stone mines. The research described in this paper had the objective to reduce the rock fall hazard and the potential for multiple pillar collapse in stone mines by providing a method for designing pillars. The research was carried out by first conducting a detailed survey of mining practices and pillar performance in operating stone mines in the eastern and midwestern United States. Data were collected on the mining dimensions, pillar and roof stability conditions, spacing and orientation of discontinuities and the rock mass conditions using the rock mass rating (RMR) system of Bieniawski [9]. Rock samples were collected for laboratory strength testing. The field data together with numerical modeling results were used to develop a method for estimating the pillar strength and selecting an appropriate safety factor for design.

2. Geological setting

The stone mines included in this study are concentrated in the Interior Plains and the Appalachian Highlands physiographic regions [10]. Stone deposits located in the Interior Plains region are generally flat laying or only gently dipping and include rocks ranging across most of the Paleozoic Era, from the Ordovician Age to the Pennsylvanian. Overall, the rocks encountered in the Appalachian Highlands region are similar in age to those found in the Interior Plains region. They differ in that they have been transformed through mountain building processes to consist of

* Corresponding author. Tel.: +1 412 386 5207; fax: +1 412 386 6891.
E-mail address: eee5@cdc.gov (G.S. Esterhuizen).

elongated belts of folded and faulted sedimentary rocks. Mines located in strata dipping greater than 10° were excluded from the study.

3. Rock mass characteristics

The characteristics of the rock mass were evaluated at ninety-two investigation sites in the thirty-four different operating mines. Rock samples were collected for strength testing at the National Institute for Occupational Safety and Health (NIOSH) laboratory in Pittsburgh, Pennsylvania. Uniaxial compressive strength (UCS) tests were conducted on nominally 50-mm-diameter cores drilled from the rock samples. Cores were prepared and tested in accordance with ASTM standards [11]. The results are summarized in Table 1, grouped into three strength categories. It can be seen that there is considerable variation in the intact rock strength of the stone formations being mined, the average strength ranging from 44 MPa for the weakest formations to 301 MPa in the strongest.

Discontinuities in the stone formations were categorized into bedding-related discontinuities, which are nearly horizontal, and joints which are typically steeper-dipping discontinuities. The bedding discontinuities are the most prominent feature in most of the mines. The average bedding discontinuity spacing was 0.98 m with 42% of the observed strike lengths falling in the range of 3–10 m. In some cases, persistent beds could be traced over more than 100 m. Bedding discontinuities were typically very rough and tight or coated with thin carbonaceous materials. Occasionally, the bedding discontinuities contained weak infill materials such as indurated clay minerals, calcite or carbonaceous matter. Extrusion of weak materials in the beds was observed to contribute to instability of the pillar ribs.

Two or more steeply-dipping joint sets were observed to be present at all the sites investigated. The majority of the joints are nearly vertical, tight with no infilling or weathering, and the average spacing between joints is 0.41 m. 94% of the joint trace lengths fall in the range of 15 cm–3.0 m, with an upper limit of about 10 m. Joints are typically tight, rough, planar with no infilling or weathering. Isolated cases were observed where joints were weathered and contained soft calcitic or clayey infill, usually at locations near the surface outcrop. The steeper joints did not appear to have an impact on pillar stability, while angular joints were observed to cause block release from pillar ribs.

Large, widely spaced discontinuities that extend from the roof to the floor or across the width of an excavation were observed in about 40% of the locations visited. These large discontinuities were logged separately because of their infrequent occurrence and their potential impact on excavation stability. The average dip was 82° and the average spacing was 7.9 m, with 72% of the spacings less than 12 m. The distribution of measured spacings follows the shape of a negative exponential distribution. The discontinuity surfaces are rough, planar and can have thin soft infill materials. These large angular discontinuities can cause a safety hazard where they intersect the roof or the pillar ribs. Also, they can have a significant impact on the strength of a pillar, particularly if they bisect a tall, slender pillar, such as the one shown in Fig. 1.

Groundwater conditions were generally dry with damp or dripping conditions observed only in six out of the 92 locations mapped. Damp conditions are thought to have contributed to pillar instability where weak bedding infill materials were softened by the presence of moisture.

Rock mass classification was carried out at each underground data collection site. Two perpendicular rock faces, typically two sides of a pillar, were classified using the 1989 version of the rock mass rating (RMR) system [9]. The RMR makes use of the rock quality designation (RQD) [12] which is best obtained from drill core logging. Since drill core was not available at the mine sites, the combined RQD and joint spacing rating was determined using the fracture frequency approach proposed by Laubscher [13]. The mean and standard deviation of the various classification parameters and their ratings are summarized in Table 2. The combined discontinuity spacing and RQD rating is shown as the "discontinuity frequency" rating out of a possible 40 points.

Fig. 1. Example of a pillar that is bisected by a large angular discontinuity.

Table 2
Summary of rock mass component ratings.

Parameter	Rating mean	Rating standard deviation	Maximum possible rating
UCS (MPa)	11.6	1.79	15
Discontinuity frequency	23.0	3.91	40
Discontinuity length	4.0	1.23	6
Separation	5.5	1.14	6
Roughness	4.7	1.27	6
Infilling (gouge)	5.7	0.82	6
Weathering	6.0	0.1	6
Groundwater	14.4	2.1	15
Total rating	74.9	–	100

Table 1
Uniaxial compressive strength of stone mine rocks grouped by strength.

Group	Average (MPa)	Range (MPa)	Samples tested	Representative limestone formations
Lower strength	88	44–144	50	Burlington, Salem, Galena-Plattesville
Medium strength	135	82–207	100	Camp Nelson, Monteagle, Plattin, Vanport, Upper Newman, Chickamauga
High strength	220	152–301	32	Loyalhanna, Tyrone

The table shows that the calculated mean RMR is 74.9, which is classified as "Good" rock according to the RMR system [9]. The distribution of RMR values is shown in Fig. 2 which shows that the RMR varied between a value of 60 and 85 out of a possible 100. The fact that only about half of the mines need to regularly install roof reinforcement in the 10–15-m wide openings is evidence of the quality of the rock mass being mined.

The rock classification data were collected within the mining horizon and are therefore representative of the material forming the pillars. The floor and overlying roof strata were not classified. The overlying and underlying strata can be similar in strength and composition to the strata being mined, or may consist of weaker sedimentary rocks such as shale or siltstone that is strongly bedded [14]. Although low strength roof or floor strata can have an impact on pillar performance, it did not appear to have an impact on pillar stability in any of the mines included in the study. However, roof and floor strength should not be excluded from geotechnical investigation because one of the reported pillar collapses [4] was assessed to have resulted from pillars punching into a weak floor stratum.

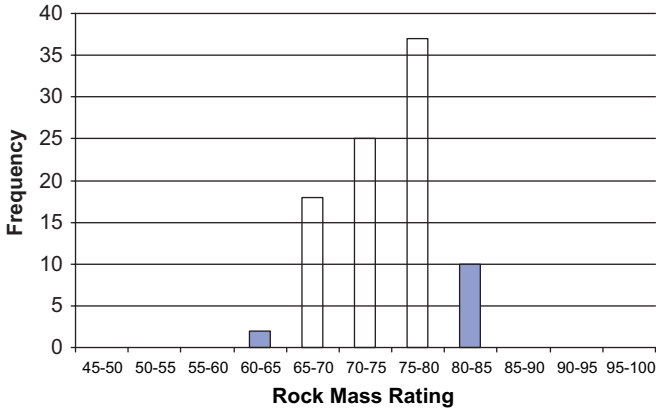

Fig. 2. Distribution of rock mass rating [9] values in underground stone mines.

Table 3
Summary of mining dimensions and cover depth of mines included in study.

Dimension	Average	Minimum	Maximum
Pillar width (m)	13.1	4.6	21.5
Pillar height (m)	11.1	4.8	40.0
Width-to-height ratio	1.41	0.29	3.52
Room width (m)	13.5	9.1	16.1
Cover depth (m)	117	22.9	670

4. Observed pillar performance

A survey of stable and unstable pillars in underground stone mines within the Eastern and Midwestern United States identified the causes of pillar instability to provide data for estimating pillar strength [15]. Mines that were likely to have unstable pillars owing to their depth of working or size of pillars were identified as targets for the survey. Data were collected that included both the intended design dimensions and the actual pillar and room dimensions in the underground workings. In older areas of mines, where the original design dimensions were unknown, the measured dimensions were assumed to adequately represent the intended design. Table 3 summarizes the dimensions and cover depth of the pillar layouts that were investigated. The approximate number of pillars in each layout was recorded to the nearest order of magnitude and the depth of cover determined from surface topography and mine maps. The LaModel stress analysis software [16] was used to estimate the average pillar stress in cases where the tributary area method was considered inappropriate. Data from one abandoned limestone mine, which was not observed as part of this study, was added to the records owing to its great depth and reported stable conditions [17].

All the pillar layouts surveyed were considered to be successful in providing global stability because they all provided satisfactory support of the overburden weight up to the ground surface. However, not all the pillar layouts were fully successful in providing local stability in the form of stable roof spans and pillar ribs. A total of eighteen cases of individual unstable pillars among otherwise stable pillars were observed. Table 4 presents a summary of the relevant dimensions, estimated stress, rock strength and notes related to the failed pillars. The instability resulted in hazardous local conditions requiring the area to be barricaded, and in some cases mining operations had to be halted or re-directed to avoid the hazardous situations. Therefore, the pillars can be considered to have failed in their function to provide local stability. These failed pillars are a small percentage of the more than 1000 pillars that were assessed.

Table 4
Summary characteristics of failed pillars.

Case	Width (m)	Height (m)	*W*:*H* ratio	Pillar stress (MPa)	UCS (MPa)	Notes
1	10.7	18.3	0.58	9.0	215	Partially benched pillar, contains angular discontinuities
2	10.7	18.3	0.58	9.4	215	Partially benched pillar, contains angular discontinuities
3	10.7	18.3	0.58	10.3	215	Partially benched pillar, contains angular discontinuities
4	15.2	27.4	0.56	12.6	153	Pillar fully benched to 27 m height causing reduced width-to-height ratio
5	10.7	18.3	0.58	12.8	215	Benched pillar, contains angular discontinuities
6	12.2	27.4	0.44	17.2	150	Partly benched pillar
7	8.5	15.8	0.54	17.2	150	Large steep dipping discontinuity and elevated stress ahead of benching
8	12.2	27.4	0.44	17.3	150	Partly benched pillar
9	7.9	9.8	0.81	19.0	160	Thin weak beds in limestone, pillar undersized causing elevated stress
10	12.8	7.3	1.73	17.4	160	Thin weak beds in pillar causing progressive spalling
11	12.5	15.2	0.82	17.8	160	Thin weak beds in pillar and moist conditions, pillar collapsed
12	6.1	12.2	0.49	19.0	160	Benched pillar is undersized causing elevated stresses
13	6.7	12.2	0.54	20.0	160	Benched pillar is undersized causing elevated stresses
14	3.7	8.5	0.43	24.1	215	Undersized pillar subject to elevated stress
15	8.2	9.1	0.9	25.0	160	Thin weak beds in pillar caused progressive slabbing
16	5.5	7.3	0.75	27.0	160	Undersized pillar subject to elevated stress
17	12.2	15.8	0.77	8.4	165	Partially benched pillar, contains angular discontinuities
18	12.2	15.8	0.77	7.6	165	Partially benched pillar, contains angular discontinuities

The survey further showed that the following factors can contribute to pillar failure or instability:

(a) Large angular discontinuities that typically extend from roof-to-floor in a pillar. Sliding can occur along these discontinuities that can significantly weaken these slender pillars [18]. Of the eighteen unstable pillars observed, seven were affected by these large angular discontinuities. Fig. 3 shows a pillar that is weakened by two angular discontinuities that contributed to failure of the pillar at a relatively low pillar stress.
(b) Weak bands within pillars that can extrude resulting in progressive spalling of the pillar ribs [19]. Fig. 4 shows a pillar that has been severely compromised by this mechanism of failure. It appears that moisture present in the weak beds was a contributing factor in this failure. Pillars in the immediate surrounding area appeared to be unaffected.
(c) High pillar stress caused by deep cover or high extraction ratios can cause spalling of the pillar ribs [5,20–22]. It was found that spalling can initiate when the average pillar stress exceeds about 10% of the uniaxial compressive strength of the pillar material. Pillars tend to take on an hourglass shape when spalling initiates. Fig. 5 shows a pillar that has failed and taken an hourglass shape due to rib spalling. Fig. 6 shows stable pillars at one of the deeper stone mines in which minor spalling has resulted in concave pillar ribs.

The failed pillars were typically surrounded by pillars that appeared to be stable, showing minimal signs of disturbance. These observations led to the conclusion that the failed pillars represent the low end of the distribution of possible pillar strengths, and not the average strength. As a result, the average safety factor of the layouts containing the failed pillars can be expected to be substantially higher than that of the individual failed pillars. However, from a design and safety point of view, it is important to identify these low strength conditions and take them into account when designing a proposed pillar layout.

Fig. 3. Partially benched pillar that failed along two angular discontinuities. Width-to-height ratio is 0.58 based on full benching height and average pillar stress is about 4% of the UCS.

Fig. 4. Pillar that had an original width-to-height ratio of 1.7 failed by progressive spalling. Thin weak beds are thought to have contributed to the failure. Average pillar stress is about 11% of the UCS.

Fig. 5. Partially benched pillar failing under elevated stress at the edge of bench mining. Typical hourglass formation indicating overloaded pillar.

5. Pillar strength estimation

The stability of a pillar can be evaluated by calculating a factor of safety (FOS), which is the ratio of the pillar strength to the average stress in the pillar. The average stresses in pillars that are of similar size and are located in a regular pattern can be estimated with relative ease using the tributary area approach, which assumes that the overburden weight is equally distributed among the pillars. This provides an upper limit of the pillar stress and does not consider the presence of barrier pillars or solid abutments that can reduce the average pillar stress. The conditions where the tributary area method is not valid, such as

Fig. 6. Stable pillars in a limestone mine at a depth of cover of 275 m. Slightly concave pillar ribs formed as a result of minor spalling of the hard, brittle rock.

irregular pillars, limited extent of mining or variable depth of cover, numerical models can be used to estimate the average pillar stress.

Estimating pillar strength is more difficult and has been the subject of much research in the mining industry [5–7,18–26]. Owing to the complexity of pillar mechanics, empirically based pillar strength equations, which are based on the statistics of failed and stable pillar systems, have found wide acceptance [27]. Very few pillar failures have occurred in stone mines, and consequently the field data alone are inadequate to develop a statistically based pillar strength equation. An empirically developed pillar strength estimation method that was originally developed by Roberts et al. [7] was used as a starting point. The method is based on the observation of a large number of collapsed and stable pillars in the Missouri Lead Belt mines, where the lead mineralization is hosted in dolomitic limestone rocks [20]. The rock strength, rock mass rating and mining dimensions are similar to those found in stone mines [15]. The method predicts the pillar strength that can be expressed as a power equation incorporating the *UCS* of the rock, the pillar width (w) and the pillar height (h) as follows:

$$S = 0.65 \times UCS \times \frac{w^{0.30}}{h^{0.59}} \quad (1)$$

Eq. (1) was used to calculate the average pillar strength and safety factors for the limestone mine dataset to determine whether reasonable results would be obtained. In this calculation, the minimum width and the tallest face of the pillars were used as the width and height in the calculations. This implies that the strength of long rectangular pillars and partly benched pillars might have been under-estimated. The six cases of failed pillars that were weakened by large angular discontinuities were excluded from the calculation because their strength is dominated by the properties of the discontinuities and should be evaluated separately.

The results are presented in Fig. 7, which shows the distribution of the FOS for the successful pillar layouts and the individual failed pillars. It can be seen that the FOS values of

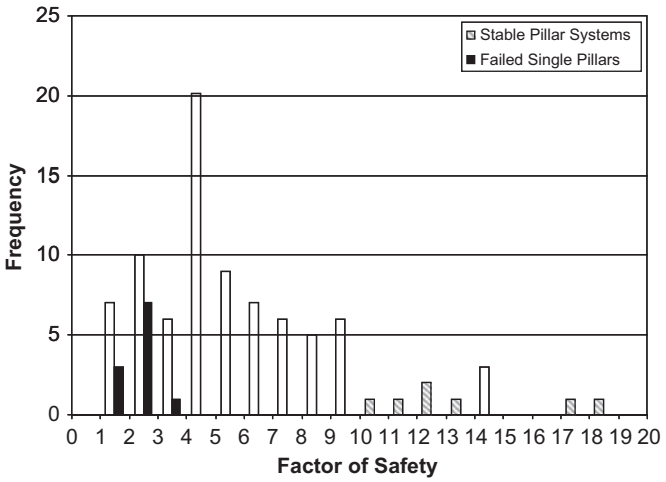

Fig. 7. Distribution of factors of safety of successful pillar systems and failed single pillars using Eq. (1). Failed pillars that were intersected by large angular discontinuities were excluded from the chart.

the successful pillar layouts are all greater than 1.0, as expected, with the largest concentration of FOS values falling in the range of 4.0–5.0. The minimum FOS of the successful cases was 1.5, which is considered to be reasonable, since no instances were observed in which large numbers of pillars showed signs of distress and overloading, as one might expect if the average FOS approaches 1.0. When calculating the FOS of the failed pillars, the actual pillar dimensions and local mine layouts were simulated using the LaModel [16] program to obtain improved estimates of the pillar stress. However, the average rock strength for each mine site was used in the calculations. The FOS of the failed pillars can be seen to fall in the range of 1.0–4.0 with an average value of 2.35. The relatively high value of FOS is not entirely unexpected, since the failed pillars represent the weakest members in the array.

It was concluded that Eq. (1) provides reasonable agreement with the observed stable and failed pillar performance in limestone mines. However, the observations showed that large angular discontinuities and weak bedding bands can have a significant impact on pillar stability and should be incorporated explicitly in the pillar strength equation. In addition, several of the limestone mines are making use of rectangular pillars to assist with roof control and ventilation control. These rectangular pillars can be expected to be stronger than square pillars and should also be accommodated in the pillar design equation.

Adjustments to Eq. (1) are presented that take into consideration the presence of large angular discontinuities and the effect of rectangular pillar shapes. The impact of weak bedding bands was not included as a modification because the rare occurrence of this failure mode made it difficult to generalize regarding the particular conditions leading to failure and it would be difficult to verify a suggested adjustment. Should weak bedding bands be identified at a proposed mine site, a detailed geotechnical study of the character of the weak beds and their likely impact on pillar strength is recommended.

6. Adjustment for large angular discontinuities

An adjustment for the presence of large discontinuities in pillars should account for both their inclination and spacing. Large discontinuities can be widely spaced and do not necessarily intersect each pillar in a layout. The two-dimensional UDEC [28] software for geotechnical analysis was used to assist in investigating the potential effect of a single large discontinuity on the

strength of pillars with width-to-height ratios of 0.5–1.5. In these models, the discontinuities were assumed to be smooth and planar, having a friction angle of 30° and negligible cohesion. The intact rock was modeled to simulate a typical limestone formation displaying brittle spalling at low confinement. The two-dimensional models simulated rib-pillars in which the strike of the discontinuities were parallel to the pillar edges, and the discontinuities were assumed to pass through the centers of the pillars, producing conservative results.

Various analyses were carried out in which the dip of the discontinuity was varied from 30° to 90° and the strength of the pillar was determined by simulating the gradual compression of the pillar until it reached its peak resistance and started to shed load. A series of curves were fitted to the peak resistance of each modeled pillar and are shown in Fig. 8. The results show firstly that as the discontinuity dip increases from 30° to about 60°, its impact on the pillar strength increases. When the discontinuity dip is greater than 70°, the effect starts to diminish. A vertical joint through the center of a pillar is seen to have a relatively small impact on pillar strength. These trends in the relationship between pillar strength and discontinuity dip are similar to the results obtained when testing laboratory specimens with inclined planes of weakness [24].

The width-to-height ratio is also shown to be a significant factor affecting the impact of large discontinuities. The chart shows, for example, that a pillar with a width-to-height ratio of 0.5 will suffer a 95% reduction in strength if it is intersected by a 60° dipping joint while a pillar with a width-to-height ratio of 1.0 would only suffer a 34% reduction in strength. The observation that slender pillars that are intersected by large angular discontinuities can fail when the average stress is only about 5% of the intact rock strength confirms that these large strength reductions do occur in the field and are similar to those predicted by the numerical models.

Table 5 summarizes the effect of the dip of large angular discontinuities on pillar strength, based on the numerical model results, expressed as a factor that relates the strength of a pillar intersected by a single large discontinuity to the undisturbed pillar strength. This factor is called the discontinuity dip factor (*DDF*) which can be used by designers to estimate the strength of pillars that are intersected by these large angular discontinuities.

The *DDF* values in Table 5 are applicable for a single pillar intersected by a single large discontinuity. However, these factors would be conservative for assessing the strength of pillars in a layout because the large discontinuities are typically widely spaced and may not necessarily intersect every pillar. Therefore, a

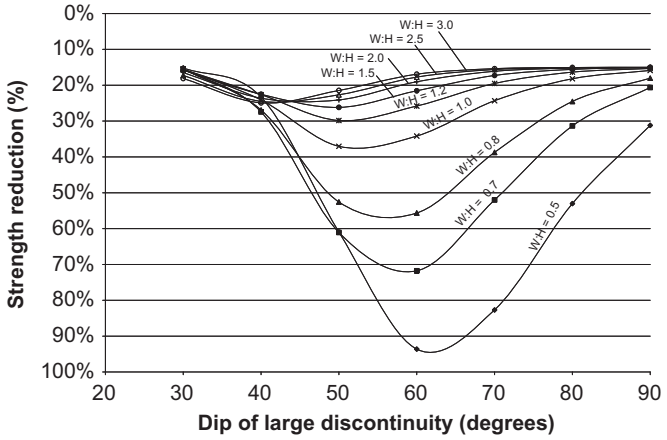

Fig. 8. Chart showing the impact of large angular discontinuities on the strength of pillars, based on numerical model results.

Table 5
Discontinuity dip factor (DDF) representing the strength reduction caused by a single discontinuity intersecting a pillar at or near its center, used in Eq. (3).

Dip (°)	Pillar width-to-height ratio								
	≤0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	>1.2
30	0.15	0.15	0.15	0.15	0.16	0.16	0.16	0.16	0.16
40	0.23	0.26	0.27	0.27	0.25	0.24	0.23	0.23	0.22
50	0.61	0.65	0.61	0.53	0.44	0.37	0.33	0.30	0.28
60	0.94	0.86	0.72	0.56	0.43	0.34	0.29	0.26	0.24
70	0.83	0.68	0.52	0.39	0.30	0.24	0.21	0.20	0.18
80	0.53	0.41	0.31	0.25	0.20	0.18	0.17	0.16	0.16
90	0.31	0.25	0.21	0.18	0.17	0.16	0.16	0.15	0.15

Table 6
Frequency factor (*FF*) used in Eq. (3) to account for the spacing of large discontinuities.

Average frequency of large discontinuities per pillar	**Frequency factor (*FF*)**
0.0	0.00
0.1	0.10
0.2	0.18
0.3	0.26
0.5	0.39
1.0	0.63
2.0	0.86
3.0	0.95
>3.0	1.00

further adjustment is required to account for the fact that only a few of the pillars may be intersected if the large discontinuities are widely spaced relative to the pillar dimensions. Based on the observation that the spacing between large discontinuities appears to follow a negative exponential distribution, the likelihood that any individual pillar will be intersected by a large discontinuity can be calculated as follows:

$$P = 1 - e^{-f} \tag{2}$$

where f is the expected frequency of large discontinuities per pillar and the simplifying assumption is made that the pillar width is perpendicular to the strike of the discontinuities. The intersection probabilities for a likely range of discontinuity frequencies per pillar has been compiled in Table 6 and is called the frequency factor (*FF*). The *DDF* and *FF* can be combined as follows to estimate the average impact of large discontinuities on the strength of pillars in a layout and is called the large discontinuity factor (*LDF*):

$$LDF = 1 - DDF \times FF \tag{3}$$

If there are no large discontinuities present, *FF* is equal to zero and *LDF* will equal 1.0, having no effect on pillar strength. The expected frequency of large discontinuities per pillar can easily be estimated by dividing the pillar width by the average spacing of the large discontinuities.

The field observations did not reveal any cases where every pillar in a layout was intersected by one or more angular discontinuities. The validity of the *LDF* under such conditions could therefore not be verified. It is recommended that a detailed rock engineering investigation and pillar strength assessment should be carried out if more than about 30% of the pillars are expected to be intersected by large discontinuities that dip between 30° and 70°.

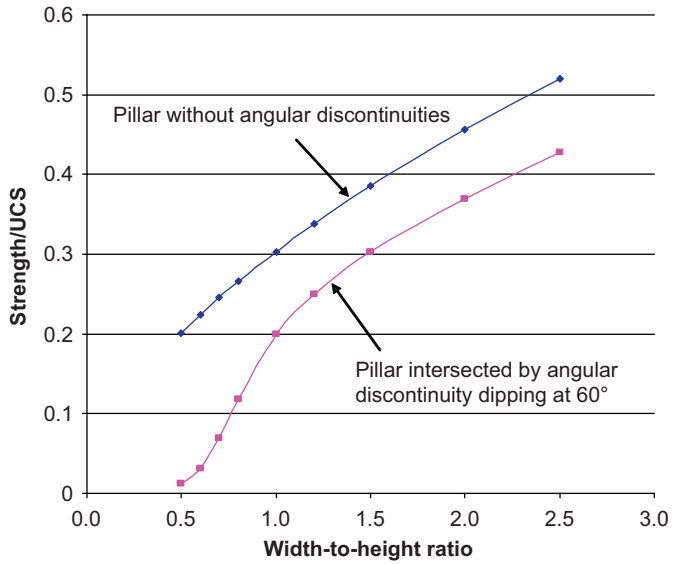

Fig. 9. Chart showing the predicted strength of a 14-m-wide pillar with varying width-to-height ratios, using Eq. (4). The effect of a large angular discontinuity dipping at 60° on the strength is also shown.

The final form of the pillar strength equation for stone mines is as follows:

$$S = 0.65 \times UCS \times LDF \times \frac{w^{0.30}}{h^{0.59}} \quad (4)$$

where *UCS* is the uniaxial compressive strength of the intact rock, *LDF* is the large discontinuity factor shown in Eq. (3), and *w* and *h* are the pillar width and height, respectively, in meters. Note that if the units of distance are in feet, the 0.65 factor becomes 0.96 because of the volume effect inherent in the equation.

Eq. (4) was used to calculate the strength of a 14-m-wide pillar with varying width-to-height ratios. The results are presented in Fig. 9, which shows the strength curve for a pillar that is unaffected by large angular discontinuities and the results for a pillar that is intersected by a large discontinuity dipping at 60°. It can be seen that the discontinuity has caused a much greater reduction in pillar strength as the width-to-height ratio drops below 1.0.

7. Adjustment for rectangular pillars

Rectangular pillars are used in stone mines to provide ventilation control and to assist with roof control. Rectangular pillars can be expected to be stronger than square pillars of the same width. Strength adjustments to account for the increased strength of rectangular pillars have been suggested by several researchers [29–31]. These methods all assume that the pillar strength will increase if the length is increased, regardless of the width-to-height ratio. However, the stone mines make use of very slender pillars which may not have been considered in the methods. A numerical model study that simulated brittle rock failure in stone mine pillars [32], indicated that the benefit of an increased length is likely to be zero when a pillar has a width-to-height ratio of 0.5 and it gradually increases as the width-to-height ratio approaches 1.4, upon which the length benefit becomes fully mobilized. The absence of a length benefit at the small width-to-height ratios is caused by the low confinement, which is insufficient to mobilize the frictional resistance of the brittle material [33].

Table 7
Values of the length benefit ratio (*LBR*) for rectangular pillars with various width-to-height ratios, used in Eq. (5).

Width-to-height ratio	Length benefit ratio (*LBR*)
0.5	0.00
0.6	0.06
0.7	0.22
0.8	0.50
0.9	0.76
1.0	0.89
1.1	0.96
1.2	0.98
1.3	0.99
1.4	1.00

The so called "equivalent-width method", proposed by Wagner [30] was selected as a basis for calculating the length benefit of rectangular pillars in limestone mines. According to Wagner's method, the length benefit is expressed as an equivalent increase in pillar width, which then replaces the true pillar width in the pillar strength equation. An additional parameter was added to account for the absence of a length-related strength increase in the very slender pillars used in stone mines. The parameter is called the length benefit ratio (*LBR*), which is a factor that increases from zero to 1.0 as the width-to-height ratio increases from 0.5 to 1.4. The modified form of Wagner's equivalent-width equation is proposed as follows:

$$w_e = w + \left(\frac{4A}{C} - w\right) \times LBR \quad (5)$$

where *w* is the minimum width of the pillar, *A* is the pillar plan area, *C* is the circumference of the pillar and *LBR* is the length benefit ratio. Table 7 shows the suggested values of *LBR* based on numerical model results. The calculated value of w_e is used in the pillar strength equation instead of the true width. When pillars are square, w_e will be equal to the pillar width *w*.

8. Factor of safety for design

An appropriate factor of safety for designing pillars can be selected from failure statistics if a sufficient number of failed and stable case histories have been observed [25,34]. In the case of stone mines, the few individual pillars that have failed are not sufficient to conduct this kind of study. A more pragmatic approach was followed in which the FOS of all the layouts and failed pillars was recalculated using Eq. (4) and the results were assessed by comparing the FOS values of the failed pillars and the successful pillar layouts, the FOS of the abandoned layouts, the numbers of pillars in the various layouts, the average pillar stress magnitude and the limits of past experience. The recalculated FOS values are presented in Fig. 10, which displays the FOS against the width-to-height ratio. Various symbols were used to indicate currently operating and abandoned layouts, failed pillars and the approximate number of pillars in the various layouts. Layouts might have been abandoned because of stability concerns or changes in operating procedures. The axis displaying the FOS is limited to 10.0, causing thirteen cases with FOS values greater than 10.0 not to be displayed.

The results show that the calculated average FOS of all the failed pillars is 2.0, which includes the cases that were intersected by large angular discontinuities. The average FOS of those pillars that are intersected by large discontinuities is 1.5. The minimum FOS for the stable layouts is 1.27, which is one of the disused layouts. It can also be seen that only one of the current pillar layouts has a FOS of between 1.0 and 1.8.

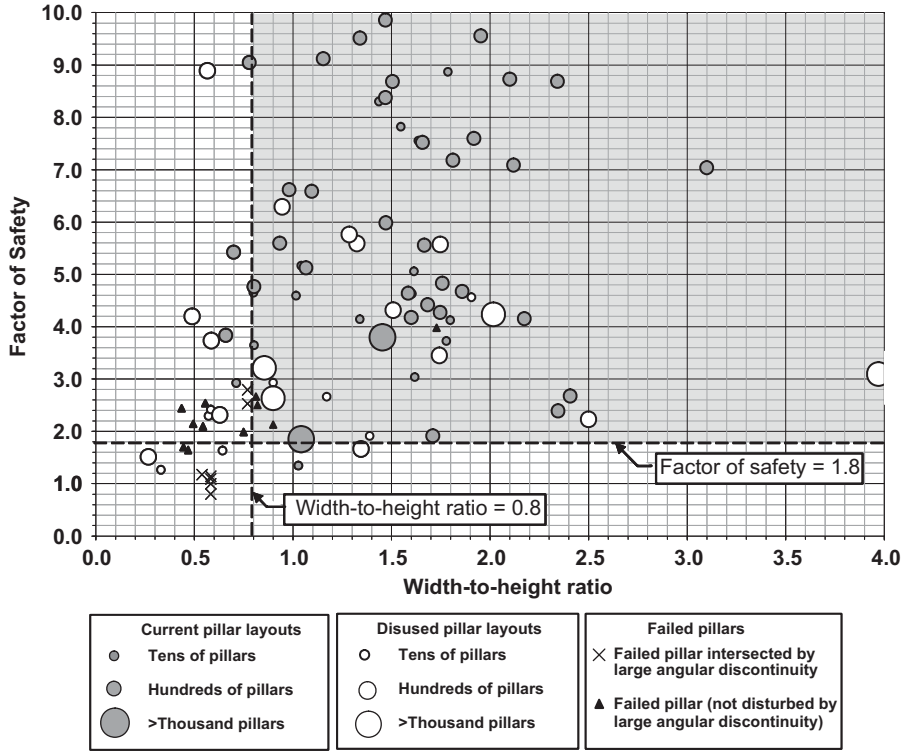

Fig. 10. Chart showing the factor of safety against width-to-height ratio using Eq. (4). Current and abandoned pillar layouts are shown as well as single failed pillars. The recommended area for pillar design is shaded.

Based on the available data, it would appear there is little experience with layouts having FOS values of less than 1.8. Layouts that approach the FOS=1.8 line in Fig. 10 are typically deep layouts (> 180 m) and are subject to relatively high average pillar stresses where rib spalling can become an issue. Therefore, it is concluded that it would be prudent to design pillars that have safety factors of at least 1.8 to remain within the range of known successful layouts.

9. Pillar width-to-height ratio considerations

Fig. 10 shows that there has been a natural tendency for mines to avoid slender pillars. Nine of the layouts that had width-to-height ratios of less than 0.8 are no longer in use for various reasons, while only four mines included in the study are currently operating with these slender pillars. In addition, this study has shown that slender pillars are more severely affected by the presence of discontinuities than wider pillars. Studies have also shown that as the width-to-height ratio decreases below 0.8, the confining stresses within a pillar approach zero and brittle fracturing can occur throughout the unconfined pillar core [5,6,18]. The confining stress can further be reduced if low friction contact surfaces exist between the pillar and the surrounding rock. The fracturing and spalling failure mechanism is poorly understood and it seems prudent to avoid designing pillars that might fail in this manner.

Inspection of Fig. 10 reveals that a number of stable layouts exist that have large safety factors (> 3.0) and the width-to-height ratios less than 0.8. These layouts are mostly located at very shallow depths of cover, typically less than 60 m. These pillars were found to be either very narrow, as little as 4.5 m wide, or very tall, up to 38 m high. The strength and loading of narrow pillars are both sensitive to small variations in the over-break, blast damage and pillar spacing. Large tall pillars have high ribs, which can represent a safety hazard, the high roof becomes difficult to inspect because it is poorly visible, and the severity of potential rock fall impacts increases. Therefore, it is not advisable to design layouts with such slender pillars, even if the calculated factors of safety are high.

10. Conclusions

The slender pillars used in stone mines in the United States present a unique challenge to mine designers. This study has shown that isolated pillar failures have occurred, which have resulted in safety and production concerns. An equation has been developed which can be used to estimate the pillar strength. The equation takes into account the intact rock strength and the potential impact of large angular discontinuities on pillar strength. An adjustment procedure is proposed to account for the increased strength of rectangular pillars.

The presence of weak, clay-filled or softening-bedding bands was seen to have a significant impact on pillar strength and was a contributing factor in the only collapsed pillar observed by the authors. Weak bedding bands are not accounted for in the design procedure and should be the subject of a detailed geotechnical investigation if they are encountered in a proposed mining area.

Pillars having a width-to-height ratio of less than 0.8 should be avoided because of their sensitivity to the presence of angular discontinuities, variations in dimensions and the absence of confinement in the core of the pillar.

A pillar factor of safety of at least 1.8 is recommended when using Eq. (4) for design. This will ensure that pillars remain within the limits of previously successful layouts.

The pillar strength equation and design recommendations are based on the empirical evidence from underground stone mines in the United States and should only be used to design pillar layouts in similar conditions.

Disclaimer

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