Pillar and Roof Span Design in Stone Mines

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ABSTRACT: Underground stone mines in the United States use the room and pillar method of mining in relatively flat laying deposits. Roof and rib falls account for 15% of all lost time incidents. This paper presents the result of a study to improve pillar and roof span design methods so that unexpected ground instabilities will be reduced. A pillar design equation and related design guidelines are presented based on the observation of pillar performance at 34 different underground operations. A roof span design procedure is also proposed that systematically addresses the main issues related to roof instability. The guidelines are based on the observation of actual pillar and roof span performance in stone mines in the Eastern and Midwestern United States and therefore only apply to stone mine design within these regions.

INTRODUCTION

Underground limestone mines in the United States use the room-and-pillar method to extract limestone formations that are generally flat lying. Pillar stability is one of the prerequisites for safe working conditions in a room-and-pillar mine. Unstable pillars can result in rock sloughing from the pillar ribs and can lead to the collapse of the roof if one or more pillars should fail. In addition, the roof span between pillars should be stable to provide safe access to the working face. Falls of ground from unstable roof and pillar ribs account for about 15% of all lost working days in underground limestone mines (Mine Safety and Health Administration 2009). In the past, pillar and roof span dimensions were largely based on experience at nearby mines, developed through trial and error or designed by rock engineering specialists. This paper presents the results of research carried out by the National Institute for Occupational Safety and Health (NIOSH) in cooperation with participating underground stone mines to develop guidelines for designing stable pillars and roof spans in stone mines. The research is based on the observation of failed and stable pillars and roof spans at 34 operating stone mines supplemented by laboratory testing and monitoring of rock mass response.

STONE PILLAR DESIGN

In a room-and-pillar mine, the pillars are required to provide global stability which can be defined as providing support to the overlying strata up to the ground surface. In addition, local stability, which is defined as stable ribs and roof spans between the pillars, is required to provide safe working conditions. Pillar design typically estimates the pillar strength and the pillar stress, and then sizes the pillars so that an adequate margin exists between the expected pillar strength and stress. The factor of safety (FOS) relates the average pillar strength (*S*) to the average pillar stress (σp), as follows:

$$FOS = \frac{S}{\sigma_p} \tag{1}$$

When designing a system of pillars, the FOS is critical to stability, because it must compensate for the variability and uncertainty related to pillar strength and stress and varying dimensions of rooms and pillars. The selection of an appropriate safety factor can be based on a subjective assessment of pillar performance or statistical analysis of failed and stable cases (Salamon and Munro, 1967; Mark, 1999; Salamon et al., 2006). As the FOS decreases, the probability of failure of the pillars can be expected to increase. In practical terms, if one or more pillars are observed to be failed in a layout, it is an indication that the pillar stress is approaching the average pillar strength. The relationship between FOS and failure probability, however, depends on the uncertainty and variability of the system under consideration (Harr, 1987).

Table 1. Uniaxial compressive strength of limestone rocks collected at mine sites

Group	Average MPa (psi)	Range MPa (psi)	Samples tested	Representative limestone formations
Lower Strength	88 (12,800)	44–144 (6,400–20,800)	50	Burlington, Salem, Galena-Plattesville
Medium Strength	135 (19,600)	82–207 (11,900–30,000)	100	Camp Nelson, Monteagle, Plattin, Vanport, Upper Newman, Chickamauga
High Strength	220 (31,800)	152–301 (22,000–43,700)	32	Loyalhanna, Tyrone

Table 2. Summary of mining dimensions and cover depth of mines included in study

Dimension	Average	Minimum	Maximum
Pillar width, m (ft)	13.1 (43.0)	4.6 (15.0)	21.5 (70.5)
Pillar height, m (ft)	11.1 (36.5)	4.8 (15.8)	40.0 (124.6)
Width-to-height ratio	1.41	0.29	3.52
Cover depth, m (ft)	117 (385)	22.9 (75)	670 (2,200)

PILLAR STABILITY ISSUES IN STONE MINES

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 (Esterhuizen et al., 2006). 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 design. 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 software package (Heasley and Agioutantis, 2001) was used to estimate of 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 (Bauer and Lee, 2004).

At each mine rock samples were collected to determine the uniaxial compressive strength (UCS) and the rock mass was classified using the Rock Mass Rating (RMR) system (Bieniawski, 1989; Hoek et al., 1995). The UCS results were grouped into three categories based on the average strength obtained at the individual mine sites and are shown in Table 1. The data shows that there is a considerable variation in the intact rock strength of the formations mined. The RMR varied between 60 and 85 out of a possible 100, which indicates the relatively strong rock mass conditions found in these mines.

All the pillar layouts surveyed could be considered to be successful in providing global stability by supporting the overburden weight up to the ground surface, (Esterhuizen et al., 2008). 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 single unstable pillars among otherwise stable pillars were observed. These failed pillars are a small percentage of the more than one thousand pillars that were directly evaluated. Roof instability was observed at almost every mine, but was not necessarily widespread at the individual mines. Table 2 summarizes the dimensions and cover depth of the pillar layouts that were investigated.

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

- Large angular discontinuities that typically extend from roof-to-floor in a pillar. Sliding can occur along these discontinuities which can significantly weaken these slender pillars (Esterhuizen, 2006). Of the 18 unstable pillars observed, seven were affected by these large angular discontinuities. Figure 1 shows a pillar that is weakened by two angular discontinuities that contributed to failure of the pillar at a relatively low pillar stress.
- Weak bands within pillars that can extrude resulting in progressive spalling of the pillar ribs (Esterhuizen and Ellenberger, 2007). Figure 2 shows a pillar that has been severely compromised by this mechanism of failure. It

Figure 1. 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.

Figure 2. 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.

appears that moisture on the weak beds was a contributing factor in this failure. Other pillars in the immediate surrounding area were unaffected, while a number of other pillars at this mine appeared to have been affected in the same manner.

High pillar stress caused by deep cover or high extraction ratios can cause spalling of the pillar ribs (Lane et al., 1999; Krauland and Soder, 1987; Lunder, 1994; Pritchard and Hedley, 1993). 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. Figure 3 shows a pillar that has failed and taken an hourglass shape due to rib spalling. Figure 4 is a series of pillars at one of the deeper stone mines showing the effects of minor rib spalling.

The observed unstable pillars were typically surrounded by pillars that appeared to be stable, showing minimal signs of disturbance. Therefore, the failed pillars represented a very small percentage (typically less than 1%) of the total number of pillars at any particular mine. The observations lead to the conclusion that the failed pillars represent the low end of the distribution of possible pillar strengths, and not the average pillar 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 failed pillars.

PILLAR STRESS AND STRENGTH ESTIMATION

The stability of a pillar can be evaluated by comparing the average stress in the pillar to its strength. The average stress in pillars that are of similar size and are located in a regular pattern can be estimated with relative ease using the tributary area method. The overburden weight is simply assumed to be evenly distributed among all the pillars. The average pillar stress (σp) is calculated as follows:

$$\sigma_p = \gamma \times h \times \frac{(C_1 \times C_2)}{(w \times l)} \qquad (2)$$

Where γ is the specific weight of the overlying rocks, h is the depth of cover, w is the pillar width, l is the pillar length and C_1 and C_2 are the heading and cross-cut center distances respectively. 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. In conditions where the tributary area method is not valid such as irregular pillars, limited extent of mining or variable depth of cover, numerical models such as Lamodel (Heasley and Agioutantis 2001) can be used to estimate the average pillar stress.

Figure 3. Partially benched pillar failing under elevated stresses at the edge of bench mining. Typical hourglass formation indicating overloaded pillar.

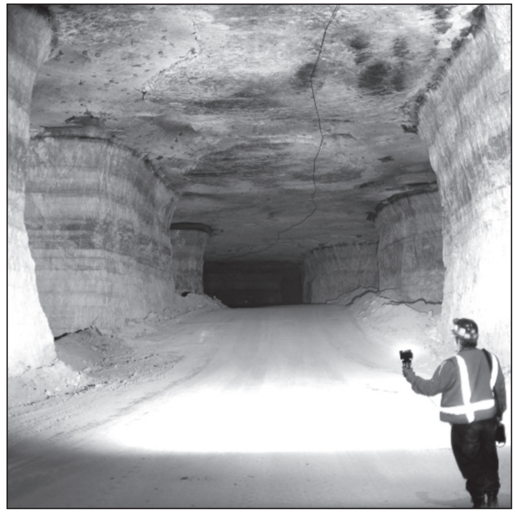

Figure 4. Stable pillars in a limestone mine at a depth of cover of 275m (900 ft). Slightly concave pillar ribs formed as a result of minor spalling of the hard, brittle rock.

Estimating pillar strength is more difficult and has been the subject of much research in the mining industry (Hustrulid, 1976). Owing to the complexity of pillar mechanics, empirically based pillar strength equations, that are based on the observation of failed and stable pillar systems, have found wide acceptance (Mark, 1999). A similar approach was followed to develop a pillar strength equation for underground stone mines. Very few pillar failures have occurred in stone mines, therefore the observations alone were inadequate to develop a stone pillar strength equation from the field data. A pillar strength relationship that was originally developed by Roberts et al. (2007) was used as a starting point. Their pillar strength relationship was based on the observation of a large number of collapsed and stable pillars in Missouri Lead Belt mines where the lead mineralization is hosted in dolomitic limestone rocks (Lane et al., 1999). The rock conditions and mining dimensions are similar to those found in stone mines. This relationship was expressed as a power equation incorporating the UCS of the rock, the pillar width and pillar height and was modified to account for the potential impact of large angular discontinuities (Esterhuizen et al., 2008). The final equation is expressed in the following form:

$$S = 0.92 \times UCS \times LDF \times \frac{w^{0.30}}{h^{0.59}} \qquad (3)$$

where *UCS* is the uniaxial compressive strength of the intact rock, *LDF* is a factor to account for large angular discontinuities and *w* and *h* are the pillar width and height in feet (when using dimensions in meters the 0.92 factor becomes 0.65). If no large discontinuities are present the *LDF* will equal 1.0, in other cases, the value of the *LDF* can be calculated using the following equation:

$$LDF = 1 - DDF \times FF \qquad (4)$$

where *DDF* is the discontinuity dip factor shown in Table 3, and *FF* is a frequency factor related to the frequency of large discontinuities per pillar as shown in Table 4.

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 (Galvin et al., 1999; Wagner, 1992; Mark and Chase, 1997). A numerical model study that simulated brittle rock failure in limestone pillars (Dolinar and Esterhuizen 2007), indicated that slender pillars are not expected to benefit as much from a length increase as wider pillars because of a lack of confinement. The numerical model results

Table 3. Discontinuity dip factor (DDF) representing the strength reduction caused by a single discontinuity intersecting a pillar at or near its center, used in equation 4

Discontinuity dip	Pillar width-to-height ratio								
(deg)	≤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 4. Frequency factor (FF) used in equation 4 to account for large discontinuities

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

Table 5. Values of the length benefit ratio (LBR) for rectangular pillars with various width-to-height ratios

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

indicate 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 to a maximum as the width-to-height ratio approaches 1.4.

The so called "equivalent-width method," proposed by Wagner (1992) was selected as a basis for calculating the length benefit of rectangular pillars in limestone mines. According to this 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. A modification is made, 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:

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

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 5 shows the suggested relationship between width-to-height ratio and the value of LBR. The calculated value of we is used in the pillar strength equation instead of the true width. When pillars are square, we will equal the pillar width w.

PILLAR FACTOR OF SAFETY CONSIDERATIONS

Equations 1 to 5 were used to calculate the strength and FOS of all the pillars that were recorded in the field studies. The results are presented in Figure 5, 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. Abandoned layouts may have been because of stability concerns or changes in operating procedures. The axis displaying the FOS was cut-off at 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.

Based on current experience it would appear that almost all the stable pillar layouts have FOS values of greater than 1.8. Layouts that approach the FOS=1.8 line, in Figure 5 are typically deep layouts (> 180m (600 ft)) and are subject to relatively high

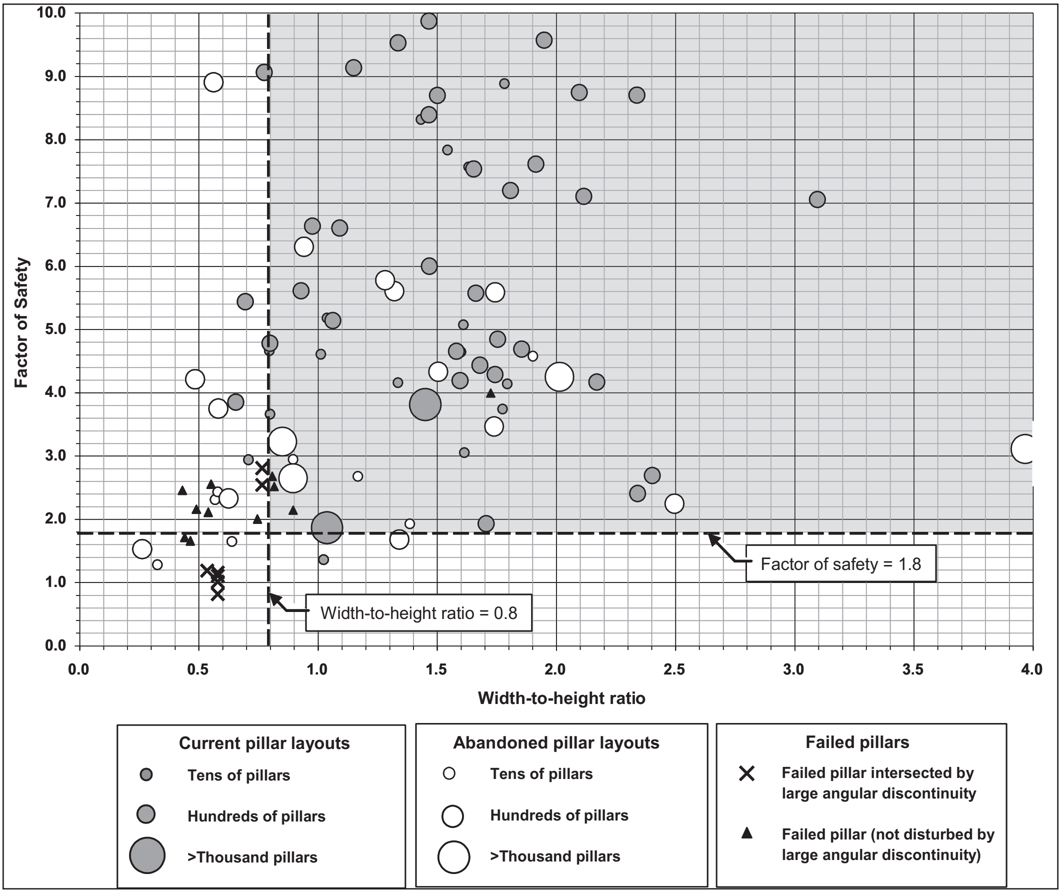

Figure 5. Chart showing the factor of safety against width-to-height ratio using Equation 3. Current abandoned pillar layouts are shown as well as single failed pillars. The recommended area for pillar design is shaded.

average pillar stresses where rib spalling can become an issue. Therefore, 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.

PILLAR WIDTH-TO-HEIGHT RATIO CONSIDERATIONS

Figure 5 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 are currently operating with these slender pillars. In addition, this study and other investigations have shown that slender pillars are more severely affected by the presence of discontinuities than wider pillars (Esterhuizen, 2000 and 2006). 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 (Lunder, 1994; Martin and Maybee, 2000; Esterhuizen, 2006). 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 Figure 5 reveals that a number of stable layouts exist that have large safety factors (>3.0) and the width-to-height ratios are less than 0.8. These layouts are mostly at very shallow depths of cover, typically less than 60 m (200 ft). These pillars were found to be either very narrow, as little as 4.5 m (15 ft) wide, or very tall, up to 38 m (125 ft) 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, on the other hand, have high ribs which can represent a safety hazard and the roof becomes inaccessible and poorly visible with increased severity of potential rock fall impacts. The strength of these slender pillars is also more adversely impacted by the presence of unfavorable discontinuities than wider pillars. Therefore, it is not advisable to design layouts with such slender pillars, even if the calculated factors of safety are high.

PILLAR DESIGN GUIDELINES

The results shown in Figure 5 formed the basis for developing the design guidelines that follow. The guidelines are empirically based; therefore, their validity is restricted to rock conditions, mining dimensions and pillar stresses that are similar to those included in this study. Therefore, they should be applicable to the greater majority of limestone mines in the Eastern and Midwestern United States. The guidelines for designing stable pillar layouts are as follows:

1. Confirm that the rock conditions and the rock strength are similar to those observed in the stone mines that were part of this study. A geotechnical investigation including rock mass classification, joint set analysis and rock strength testing is recommended.
2. The pillar strength, loading and safety factor can be estimated using equations 1 to 5. The recommended factor of safety against pillar failure is 1.8 and can be seen in Figure 5 to represent the lower bound of current experience.
3. Pillars having a width-to-height ratio of less than 0.8 should be avoided. Slender pillars are susceptible to the weakening effect of angular discontinuities and are inherently weaker than wider pillars because the pillar core is unconfined.
4. The effect of large discontinuities is accounted for in the pillar strength equation. Further investigation will be required if more than about 40% of the pillars in a layout are likely to be intersected by one or more large discontinuities that dip between dip 30 and 70 deg.
5. Pillars should be designed so that the average pillar stress does not exceed 25% of the UCS to remain within the limits of past experience. The design approach is entirely based on past experience; therefore, no comment can be made about pillar stability when pillars are loaded beyond 25% of the UCS.
6. Pillar design cannot be carried out without considering roof stability. Roof spans directly impact the pillar stresses, because wider roof spans imply higher stresses in the pillars. As part of the pillar design, an evaluation of roof span stability and likely maximum stable spans should be conducted, as described in the next section of this paper.

The shaded zone in Figure 5 indicates the area in which Equation 3 is likely to produce stable pillar layouts and coincides with the current experience in limestone mines and observations of failed pillars. The design recommendations are based entirely on the observed performance of stone pillars. Therefore, pillars that plot to the left or below the shaded area in Figure 5 are beyond the validity of these design guidelines and specialist rock engineering advice should be sought.

ROOF SPAN DESIGN

In room-and-pillar mines, the roof between the pillars is required to remain stable during mining operations for haulage as well as access to the working areas. In underground stone mines, the size of the rooms is largely dictated by the size of the mining equipment. Underground stone mines use large mining equipment to operate economically and require openings that are on average 13.5-m (44-ft) wide by approximately 7.5 m (25 ft) high to operate effectively. The dimensions of the desired roof spans are largely pre-determined and design is focused on optimizing stability under the prevailing rock conditions. If the rock mass conditions are such that the desired stable spans cannot be achieved cost effectively, it is unlikely that underground mining will proceed. NIOSH research into stone mine roof stability has focused on identifying the causes of instability and techniques to optimize stability through design.

ROOF STABILITY ISSUES IN STONE MINES

Observations were carried out at 34 operating stone mines to identify the factors that contribute to roof instability. Data were collected on rock strength, jointing and other geological structures, room and pillar dimensions, roof stability and pillar performance. Two to five data sets were collected at various locations at each mine site. A data set describes the stability of the roof and pillars in an area of approximately 100 × 100 m (300 × 300 ft). The range

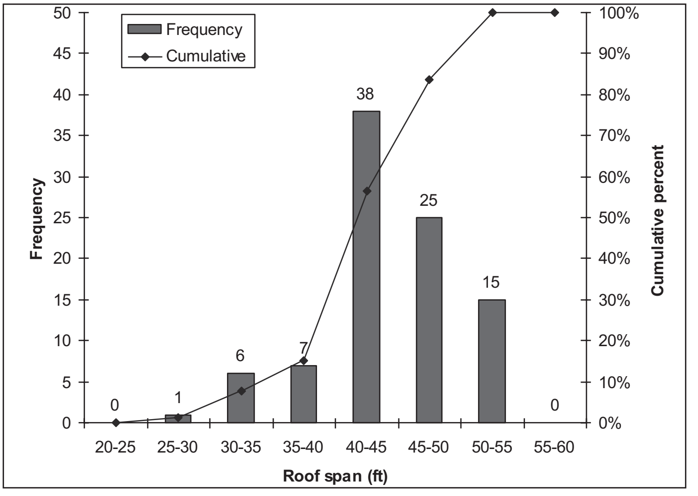

Figure 6. Distribution of roof span dimensions measured at 34 different underground stone mines

of roof spans observed is shown in Figure 6, which shows that the 92% of the mine openings are more than 10 m (35 ft) wide. Of the mines visited, 41% were installing support in a regular pattern while the remaining 59% did not install any roof support, or only rarely installed spot bolting if required.

All but four of the 34 mines visited had experienced some form of small-scale rock falls or larger roof falls. The observed areal extent of smaller scale rock falls, less than 1m in length, was as follows:

- Thin slabs/slivers: 11% of the total roof area
- Joint bounded blocks: 6% of the total roof area
- Bedding defined beams/slabs: 11% of the total roof area

In the remaining 72% of roof area observed, the roof was stable with no sign of current or past instability. Figure 7 shows an example of a 13-m (43-ft) wide, naturally stable excavation with excellent roof conditions. Most of the smaller scale instabilities are addressed by scaling, rockbolting or screen installation as part of the normal support and rehabilitation activities.

In addition to small scale rock falls, large *roof falls*, which typically extend over the full width of the opening, were observed at 19 of the 30 mines that experienced small scale roof instability. Roof falls were categorized by identifying the most significant factor contributing to each fall. A summary of these factors and the relative frequency of occurrence of each are presented below:

- Stress: Horizontal stress was assessed to be the main contributing factor in 36% of all roof falls observed. These falls are equally likely to occur in shallow or deep cover. A roof fall related to stress-induced damage was observed at a depth of as little as 50 m (150 ft) in one case. The characteristics of stress-related roof falls are described in Iannacchione et al. (2003) and Esterhuizen et al. (2007). Figure 8 shows a typical ellipsoidal stress related fall and Figure 9 shows an example of how such a fall progressed through the mine workings in a direction perpendicular to the major horizontal stress.
- Beams: The beam of limestone between the roof line and some overlying weak band or parting plane failed in 28% of all observed large roof falls.
- Blocks: Large discontinuities extending across the full width of a room contributed to 21% of the roof falls.
- Caving: The remaining 15% of the roof falls was attributed to the collapse of weak shale inadvertently exposed in the roof or progressive failure of weak roof rocks.

Although the large roof falls only make up a small percentage of the total roof exposure, their potential impact on safety and mine operations can be very significant. Most cases of large roof falls required barricading-off or abandonment of the affected entry. When large roof falls occur in critical excavation areas, the repair can be very costly.

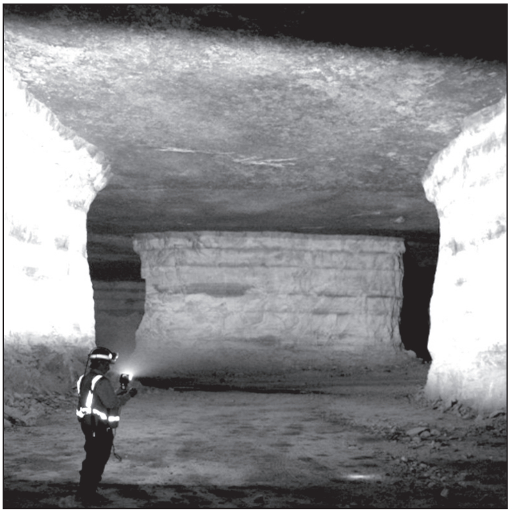

Figure 7. Naturally stable 13-m (44-ft) wide roof span in a stone mine

MAXIMUM STABLE ROOF SPAN

The majority of roof spans in operating mines falls within a narrow range of 10 m to 17 m (35 ft to 55 ft), and is related to the space needed to effectively operate large loaders and haul trucks. Very few of the mines used roof spans wider than 15 m (50 ft), so it is not clear whether the stability limit is approached at 17 m (55 ft) or whether it simply satisfies the practical requirements for equipment operation. Given that a large proportion of the mines are able to mine without installed support, it seems to indicate that wider spans can be achieved if additional supports are used. Whether these larger spans would be cost effective will of course depend on the support costs.

One way of assessing the potential maximum span is to compare the stone mine data to experience in other mine openings around the world. The Stability Chart originally developed by Matthews et al. (1980), modified after Potvin (1988), Nickson (1992) and Hutchinson and Diederichs (1996) was used as a basis for comparison. The Stability Chart plots a modified Stability Number N' which represents the rock mass quality normalized by a stress factor, an orientation factor and a gravity adjustment. Stability zones have been indicated on the chart based on 176 case histories from hard rock mines around the world. The following stability zones are indicated:

- Stable—support generally not required
- Stable with support—support required for stability, the support type is cable bolting

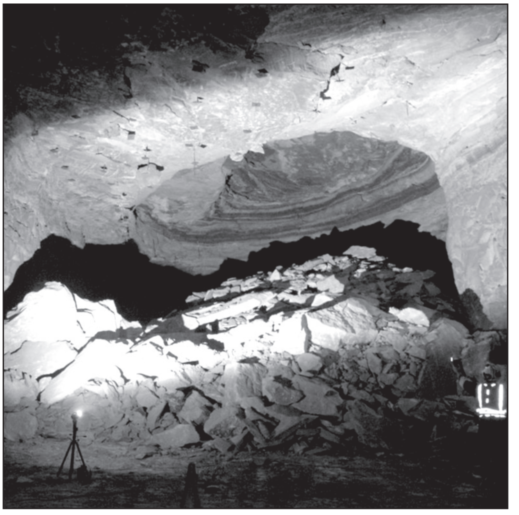

Figure 8. Example of large elliptical shaped roof fall that was related to high horizontal stress in the roof

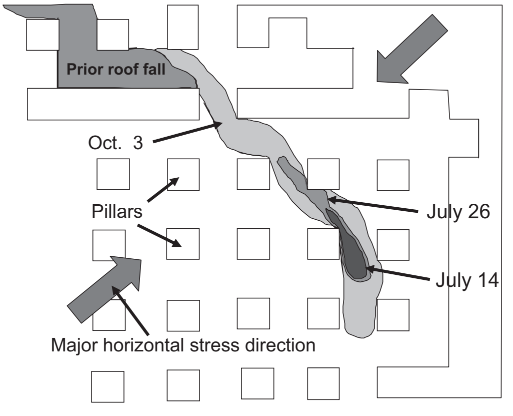

Figure 9. Plan view showing the development of a stress related roof fall in the direction perpendicular to the direction of the major horizontal stress, after Iannacchione et al. (2003)

- Transition—stability not guaranteed, even with cable bolt support
- Unsupportable—caving occurs, cannot be supported with cable bolts

Figure 10 shows the stability chart with the stone mine case histories and stability categories. In this chart the actual heading width is shown instead of the “hydraulic radius” which is customarily used. The conversion from hydraulic radius to heading

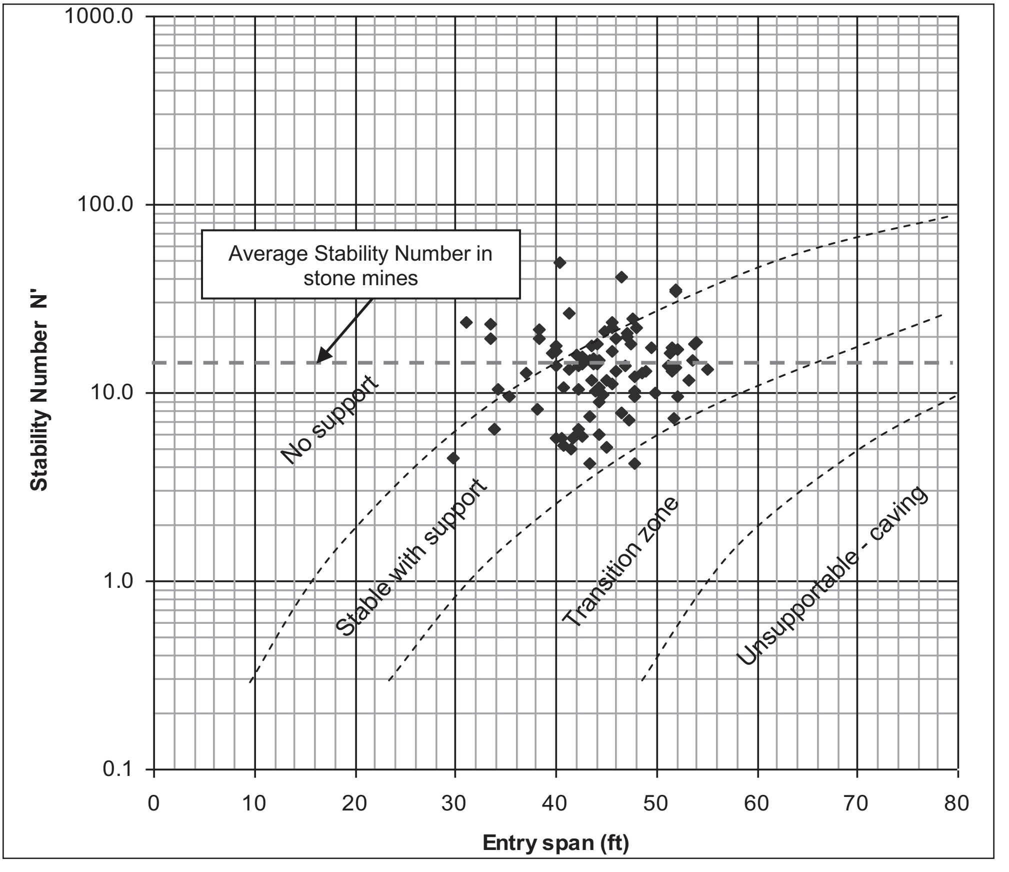

Figure 10. Stability chart showing stone mine case histories and stability zones, modified after Matthews et al. (1980), Potvin (1988), Nickson (1992) and Hutchinson and Diederichs (1996)

width assumes the heading is a parallel-sided excavation. The chart also indicates the average Stability Number for stone mines as a horizontal dashed line.

It can be seen that the majority of stone mine case histories plot in the region of "stable" to "stable with support" and only one is located in the transition zone. This agrees reasonably well with the observed stability and support used in stone mines, although stone mines have been able to achieve stability with light support compared to cable bolting used in the hard rock mine case histories. Based on the average Stability Number for stone mines, it would appear that stable supported excavations can reliably be achieved with spans of up to about 20 m (65 ft) using cable bolt supports. Cable bolt lengths in the hard rock mines are typically greater than half the excavation span, which would imply 10-m (33-ft)-long or longer cable bolts to achieve stability in a 20-m (65-ft) wide stone mine entry. Unsupportable and caving conditions are indicated when the span increases to about 27 m (90 ft). These results are in line with current experience. It appears that stone mines are working near the span limit that can reliably be achieved using rock bolts as the support system. Increasing the spans beyond the 15–17 m range (50–55 ft) is likely to incur considerable cost and productivity implications as cable bolting would become necessary.

STABILITY OF THE IMMEDIATE ROOF BEAM

The stability of excavations in bedded deposits is closely tied to the composition of the first beam of rock in the roof. An assessment of the data collected showed that 25 of 34 mines were attempting to maintain a specific thickness of limestone beam in the immediate roof. In some cases the upper surface of the beam was a pronounced parting plane while in others it was a change in lithology, typically when the limestone beam is overlain by weaker rocks. A constant thickness of roof beam is achieved either by probe drilling to determine the thickness of the roof beam or by following a known parting plane or marker horizon.

The average roof beam thickness in mines that were able to mine without regular support was 2.25 m (7.4 ft), while the average beam thickness in the mines that were using regular roof support was 1.3 m (4.3 ft). Several of the mines that used regular support do so to alleviate the effects of horizontal stress, which is not related to beam thickness. If these mines are removed from the data, the average beam thickness in mines that use regular support drops to 0.8 m (2.6 ft). These results seem to indicate that mines with a relatively thin beam of limestone in the immediate roof are more likely to encounter unstable roof and regular roof bolting becomes necessary. There was no correlation between roof beam thickness and excavation span.

The beam thickness is obviously not the only factor to consider when deciding on roof reinforcement. Other aspects such as roof jointing, bedding breaks, blast damage, groundwater and horizontal stress can contribute to roof instability resulting in the need for rock bolt support. However, experience seems to indicate that a roof beam thickness of less than about 1.2 m (4 ft) is highly likely to be unstable and a regular pattern of rock bolt supports will be required to maintain the roof stability.

HORIZONTAL STRESS CONSIDERATIONS

This study showed that horizontal-stress-related roof instability can occur at any depth of cover. This is not unexpected, given that the horizontal stresses are caused by tectonic compression of the limestone layers, which is not related to the depth of typical limestone mines (Dolinar, 2003; Iannacchione et al., 2003). Observations show that the tectonic stresses in limestone formations that outcrop may have been released over geologic time by relaxation towards the outcrop (Iannacchione and Coyle, 2002). Consequently, outcropping mines can have highly variable horizontal stress magnitudes which depends on the amount of relaxation that occurred over geologic time and the distance from the outcrop.

A review of horizontal stress measurements in limestone and dolomite formations in the Eastern and Midwestern US and Eastern Canada, (Dolinar 2003; Iannacchione et al. 2002) has shown that the maximum horizontal stress can be expected to vary between 7.6 MPa (1,100 psi) and 26 MPa (3,800 psi) up to depths of 300 m (1,000 ft). Limited information is available at greater depths. The orientation of the maximum horizontal stress is between N60°E and N90E in 80% of the sites. This agrees with the regional tectonic stress orientation as indicated by the World Stress Map Project (2009). The minimum horizontal stress is approximately equal to the overburden stress.

An analysis of the impact of horizontal stress on beams of rock that may exist in the roof of stone mine workings showed that horizontal stress can be expected to cause buckling of thinly bedded roof strata (Iannacchione et al., 1998). Further analyses using numerical models showed that brittle spalling (Kaiser et al., 2000) of the roof rocks under near-uniaxial loading conditions can explain roof failure at the stress levels encountered in stone mines (Esterhuizen et al., 2008).

Once a stress-induced roof fall has occurred, it can be costly and difficult to arrest the extension of the fall into adjacent areas. Avoidance of these falls through layout modifications has proved to be very successful in several operating mines (Iannacchione et al., 2003). It is first necessary to establish the direction of the major horizontal stress, which can be determined by various stress measurement techniques or can be inferred from stress-related roof failures (Mark and Mucho, 1994). The layout is then modified so that the main development direction is parallel to the maximum horizontal stress and the amount of unfavorably oriented cross-cut development is minimized (Parker, 1973). A further modification that has proved to be successful is offsetting the cross-cuts and increasing the length of the pillars, so that a continuous path does not exist along which a roof fall can progress across the layout. Modifying a layout in this manner will not necessarily eradicate all stress-related problems, but has been shown to considerably reduce these problems (Kuhnhein and Ramer, 2004).

Figure 11 shows a mine layout that has been optimized for horizontal stress. The main heading direction is parallel to the maximum horizontal stress, pillars are elongated so that unfavorably oriented cross-cuts are minimized, the cross-cuts are narrower than the headings and the cross-cuts are off-set so that potential stress related roof falls will abut against solid pillar ribs, rather than snake through the layout.

ROOF REINFORCEMENT

The survey of roof support practices showed that grouted rock bolts are the most widely used form of support. Bolt lengths are typically between 1.8 m (6 ft) and 2.4 m (8 ft), and where the bolts

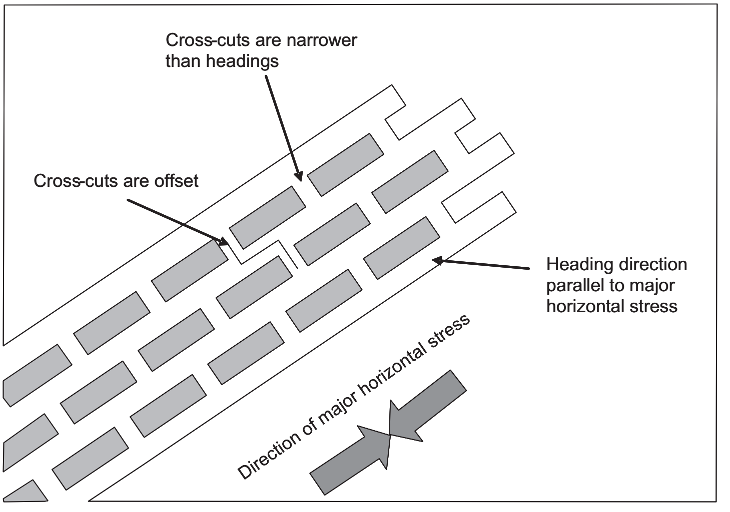

Figure 11. Diagram showing room and pillar layout modified to minimize the potential impact of horizontal stress related damage

are installed in a regular pattern, the most commonly used bolt spacings are either 1.7 m (5 ft) or 1.8 m (6 ft). As with most other roof bolting designs in strong rocks, high strength and stiff bolts are more likely to provide the desired rock reinforcement than low strength and low stiffness systems (Iannachione et al., 1998). Roof screen or other supplemental supports such as cable bolts are rarely used.

Roof reinforcement in the relatively strong bedded rock encountered in stone mine can have one or more objectives. Depending on the geological conditions the support system can be expected to:

- Provide suspension support for a potentially unstable roof beam
- Provide local support to potentially unstable blocks in the roof
- Combine thinly laminated roof into a thicker, stronger unit
- Provide surface control when progressive spalling and small rock falls occur

The above support functions can usually be achieved by the 1.8 m (6 ft) and 2.4 m (8 ft) bolts used in the stone mines. When poor ground is encountered locally or when horizontal stress related roof failures occur, intense bolting, steel straps and cable bolts have been used with mixed success to halt the lateral extension of these large roof falls.

From a design point of view, a stone mine is unlikely to be economically feasible if heavy support such as cable bolts and screen would be required on a daily basis. Such rock conditions would probably require reduced excavation spans and the support costs would be prohibitive. Therefore, the first objective in designing an underground stone mine should be to confirm that the rock mass quality is adequate for creating the typical 10-m to 17-m (35-ft to 55-ft) roof spans without resorting to elaborate support systems.

ROOF SPAN DESIGN GUIDELINES

Designing stable roof spans for underground stone mines should be conducted using basic engineering principles. Good geotechnical information, combined with a pragmatic assessment of the likely modes of instability and providing the required support is likely to produce a stable initial design. Once an initial design has been developed, monitoring of its performance can be carried out to optimize the design.

Useful information can be obtained from neighboring mines that are operating under similar conditions. A particularly useful piece of information would be to identify whether horizontal-stress-related roof problems exist and the orientation of the stress related damage. This information can go a long way in selecting the orientation of the main headings in the proposed mine. The following steps should be followed to carry out a mine layout and roof span design:

1. Geotechnical characterization: Designing stable roof spans for stone mines can be successfully carried out if adequate geotechnical

investigations are conducted ahead of the design. Such investigations are best conducted by experienced ground control specialists and are likely to include rock strength testing, core logging, bedding layering assessment, joint orientation assessment and rock mass classification. If horizontal-stress-related issues are expected, stress measurements can assist in providing an indication of the orientation and magnitude of the maximum horizontal stress.

2. Confirm rock mass quality: Using the results of a rock mass classification or direct inspection of workings, confirm that the rock strength and rock mass quality is similar to that found in Eastern and Midwestern stone mines. The RMR should exceed a value of 60.0 and the rock strength should exceed 45 MPa (6,400 psi).
3. Selection of mining direction: The direction of the headings in the production areas should be favorably oriented to any expected horizontal stress and the prevalent jointing. As with any underground excavation layout, it is preferable to intersect the main joint strike direction by at least 45 degrees. Since room and pillar mines have two orthogonal directions of mining, the heading direction should be favored over the cross-cut direction when selecting the orientation of the layout. If the orientation of the maximum horizontal field stress is known, and stress related problems are anticipated, the heading direction should be oriented parallel to the direction of major horizontal stress, with due consideration of joint orientations and cross-cut stability. Often it will be a compromise to select the final heading orientation. Other modifications to the pillar layout should be considered, as discussed below.
4. Modification of pillar layout: A simple square pillar layout with headings and cross-cuts of equal width is sufficient in most cases. However, if horizontal-stress-related instability is expected, the pillar layout can be modified in improve the likelihood of success. Possible layout modifications were shown in Figure 11.
5. Selection of mining horizon: The location of the roof-line relative to pronounced bedding planes or lithology changes should be identified next. Experience has shown that if the immediate roof beam is less than 1.2-m (4-ft) thick, it is very likely to be unstable. Thicker roof beams may be required if excessive horizontal stresses are encountered. Persistent parting planes can be selected to form the roof-line if they are present at a convenient location in the formation being mined. Using a pre-existing parting plane as the roof line helps to act as a marker and usually provides a clean breaking surface for blasting operations. Many of the mines that do not use roof supports have a natural parting as the roof line.
6. Selection of roof span: Past experience has shown that stable roof spans in the range of 10 m to 15 m (35 ft to 50 ft) have been regularly achieved in underground stone mines. NIOSH studies have shown little correlation between mining roof spans and rock quality, mainly because there is such a small range of rock qualities in operating mines. For an initial design, it might be prudent to design for no more than 12 m (40 ft) spans. The spans can be increased incrementally, if warranted by monitoring of actual roof performance. There is little experience with spans that are greater than 15 m (50 ft).
7. Support considerations: Depending on the characteristics of the immediate roof, basic support in the form of patterned rockbolts may be required. The importance of the thickness of the first beam in the roof, the orientation of excavations relative to the maximum horizontal stress and characteristics of rock joints will determine whether and how much support is required. Mines that do not use bolting are located in formations with a favorable combination of geological conditions, and they conduct blasting practices that maintain an unbroken roof horizon.
8. Monitoring and confirmation: Once a roof design has been finalized and mining is underway, monitoring should be implemented to verify the stability of the roof. Monitoring results can be used to identify potential stability problems before they occur and may indicate that a change in the design is required. Monitoring technologies that are available include borehole-video logging (Ellenberger, 2009), roof deflection monitoring (Marshall et al., 2000), roof stability mapping using the Roof Fall Risk Index (RFRI), (Iannacchione, et al. 2006) and micoseismic monitoring of rock fracture, (Iannacchione and Marshall, 2004; Ellenberger and Bajpayee, 2007).

DISCUSSION AND CONCLUSIONS

A study of pillar and roof span performance in stone mines that are located in the Eastern and Midwestern United States showed that various stability issues can be addressed by appropriate design. Pillars can be impacted by rock joints, large angular discontinuities and can exhibit rib spalling at elevated stresses. Thin weak beds in the pillars, although rare, can have a significant impact by reducing pillar strength. If the roof strata are bedded, beam deflection and buckling can result in roof failure. The roof can also be impacted by large discontinuities and the effects of horizontal stress.

A pillar design procedure is proposed that takes into consideration the rock strength, pillar dimensions and the potential impact of large angular discontinuities. Based on current performance of pillars in stone mines, a safety factor of 1.8 is suggested for pillar design with a lower limit, width-to-height ratio of 0.8.

A roof span design procedure is also proposed that systematically addresses each of the main stability issues. The procedure focuses on selecting an appropriate mining horizon and mining direction. The importance of the thickness of the first bed in the roof and the likelihood for added rock bolting is described. Layout modifications are described that can be made to reduce the incidence of horizontal-stress-related instability.

Both the pillar design and roof span guidelines require that a good understanding be obtained of the geotechnical characteristics of the formation being mined. The essential data are the uniaxial compressive strength of the rock, characteristics of the discontinuities and the rock mass classification. Knowledge of the magnitude and orientation of the stress field can assist in orienting the layout appropriately.

The design procedures are based on observation of the actual performance of pillars and roof spans in stone mines within the Eastern and Midwestern United States. The guidelines should only be used for design under similar geotechnical conditions.

Disclaimer

The findings and conclusions in this paper have not been formally disseminated by the National Institute for Occupational Safety and Health and should not be construed to represent any agency determination or policy.

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