Rock Failure Analysis and Stability

RMR Aggregates, Inc.

Mid Continent Limestone Quarry

August 29th, 2023

KUE Project P-23018SS

KILDUFF UNDERGROUND ENGINEERING, INC.

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ROCKY MOUNTAIN INDUSTRIALS

August 29, 2023

RMR Aggregates, Inc.
6200 S. Syracuse Way, Ste. 450
Greenwood Village, CO 80111

Attention: Mr. Robert Wagner

Re: Rock Failure Analyses and Stabilization
Mid Continent Limestone Quarry
Glenwood Springs, CO
KUE Project No. P-23018SS

Dear Mr. Wagner,

Kilduff Underground Engineering, Inc. (KUE) is pleased to submit this report on the 2023 ground event analyses and headwall stabilization at the Mid Continent Limestone Quarry in Glenwood Springs, Colorado. KUE's services were performed in accordance with our contract between KUE and Rocky Mountain Industrials dated March 24, 2023.

This report consists of a summary of findings from three site reconnaissance, kinematic and steady state stability analyses of both the West and East face, and rockfall modeling. The scope of work was performed to evaluate the failure mode of the January 2023 ground event to assess the long-term stability of the overall headwall and determine the best path forward for long-term slope stability. Data and recommendations are subject to the provisions and requirements outlined in the Limitations section of this report.

We trust that our findings and recommendations outlined in this report will be responsive to your needs at this time. We thank you for this opportunity to be of service to you and your team on this exciting and interesting project. Should you have any questions or require additional information, please do not hesitate to contact the undersigned.

Sincerely,

KILDUFF UNDERGROUND ENGINEERING, INC.

Sean Sundermann, PG, CEG
Principal Engineering Geologist

Todd Kilduff, PE
Principal Engineer

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APPENDICIES

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1. PROJECT BACKGROUND

1.1. GENERAL DESCRIPTION

A headwall ground event occurred on the West Wall of the Mid Continent Limestone Quarry (Figure 1) on January 18, 2023. Mr. Robert Wagner of RMR Aggregates, Inc. (RMRA) reached out to Kilduff Underground Engineering, Inc. (KUE) the following day to discuss the potential causes of failure. KUE Principal Geologist Sean Sundermann, PG, CEG and Senior Construction Specialist Jim Johnson performed a site walk of the site on January 26, 2023 with RMRA staff to evaluate the current condition the slope and initial assessment of the root cause of the event. At that time a general scope of work was developed to evaluate the geotechnical factors that led to the West highwall ground event and to assess the long-term stability of the West and East sides of the highwall in its current state. This report is the program deliverable that delivers on the following scope of work items:

- Description of findings from multiple site reconnaissances;
- Kinematic analyses of failure modes;
- Long Term Steady-State stability analysis;
- Rockfall modeling;
- Rockfall mitigation recommendations for current configuration;
- Geotechnical input to long-term stabilization plan; and
- Mechanical stabilization scheme.

Results and findings of the above scope of work are summarized in the report sections below. Photos, field measurements, and model runs from multiple analyses are provided in the attached appendices:

Appendix A – 2023 Field Photographs
Appendix B – Discontinuity and Structure Measurements
Appendix C – Kinematic Analyses
Appendix D – Planar Failure
Appendix E – Rockfall Modeling
Appendix F – July 2023 West Face Field Photographs
Appendix G – Active Stabilization Design

1.2. REGIONAL AND QUARRY GEOLOGY

The quarry lies primarily within the Mississippian-age Leadville Limestone, a very fossiliferous, massive, coarse to finely crystalline limestone and dolomite formation, as mapped by the Colorado Geological Survey (Kirkham et al., 2008[1]). The unit is described by Kirkham et al. as 200 feet thick in the site area. The Leadville Limestone formation consists of gray to bluish-gray, coarse to finely crystalline limestone underlain by Dolomitic limestone with 20 feet to 30 feet of varying amounts of sand expected in the basal unit. Underlying the Leadville Limestone is the Upper Devonian-age

[1] Kirkham, R., Streufert, R., Cappa, J., Shaw, C., Allen, J., and J. Jones, 2008, Geologic map of the Glenwood Springs quadrangle, Garfield County, Colorado; Colorado Geological Survey, Map Series 38, scale 1:24,000.

Chaffee Group. Near the southeast flank of the White River Uplift, the Gilman Sandstone, the upper unit of the Chaffee Group, is predominantly a 16-foot thick calcareous sandstone (Kirkham et al., 2008), pinching out towards Glenwood Springs. The site area is bound to the north by a mapped bedrock graben, just south of the Glenwood monocline axis, exposing the younger fossiliferous limestone unit of the Lower Pennsylvanian-age Belden formation. Outcrops of the Belden appear below the existing quarry as well, unconformably overlying the Leadville Limestone.

Mid Continent Limestone Quarry Geology

Leadville Limestone in the location of active quarry operations is mapped by Kirkham et al. as dipping between 24 and 38 degrees to the south-southwest, which forms dip slopes and tends to control hillside slope topography. A series of roughly east-west trending normal faults crosscut the area but are not mapped as continuous across the proposed expansion area. These structures are likely a westward extension of the normal-oblique Grizzly Creek Shear Zone, and the secondary influence on the site's rock mass, outside bedding.

Previous analyses of the Leadville Limestone performed for RMRA indicates the coarse crystalline rock is composed of 90% to 98% calcium carbonate and low in both magnesium chloride and silica. Boreholes completed by Colorado Fuel & Iron Corporation (CF&I) in the 1950's and 1960s were provided by RMRA to KUE as scanned hardcopy with approximate locations. Borehole Lynx 05-001 in the area of the ground event describes the upper 36 feet of the rock as follows:

> *Upper Leadville Limestone, med to dark gray, hard, fine grained, some recrystallized; numerous fractures in all directions re-cemented with w/ white, yellow, brown, & pink calcite. Some limonite stain, Good acid reaction. Porous zones at 28-0 to 28-5 and 29-9 to 30-6.*

Lower in section, thin mud seams are identified at 51-9 and 58-3, but no description of soft interbeds are included.

Also of significant note to the modeling of the recent ground event, 1966 borehole Lynx 05-099 drilled vertically in the area of the event identifies dip in the upper 40 feet of the section as 40 degrees.

Site Topography

RMR Aggregates, Inc. has developed a very detailed site topographic map, which shows a moderately-steep, south-facing slope. Slope topography decreases upslope, from south to north, along the top of the fold. The topography lays back from 1.4:1 at the southern extent at the existing quarry highwall to roughly 2.5:1 moving uphill 220 yards towards the potential area of proposed long-term stabilization.

2. SITE RECONNAISSACE

2.1. JANUARY 26, 2023 – SITE RECONNAISSANCE

The aforementioned site reconnaissance performed eight days following the ground failure was primarily an overview of the event, documenting existing conditions and formulating a plan across represented disciplines to evaluate and stabilize the ground event. Photos with detailed captions from the reconnaissance are included in Appendix A. Photo 1 shows an overall view of the ground event on the west face that released along a northeast dipping joint with high to very high persistence. No other obvious cracking above the recent ground event release was observed on foot or by drone. Photos 2 and 3 highlight the upper two beds of limestone that sit on the more massive limestone below. The slide plane for the ground event occurred along bedding at these two upper beds.

Two thin interbeds of laminar bedded, shaley mudstone bound the upper two limestone beds. The observed thin interbed of laminar bedded, shaley mudstone creates a potential failure plane of lesser cohesion and fiction angle than the limestone. In addition, significant icicles had formed primarily at the basal contact of the upper limestone to the mudstone interbed. However, it is somewhat unclear if the water was draining out along this basal contact, or if seepage down the face of the limestone was dripping and causing the icicles, or most likely both.

2.2. APRIL 14, 2023 – SITE RECONNAISSANCE

A site mapping program was performed to collect structure data on the East Face, evaluate the strength parameters of the interbed and overall geologic/rock mass conditions and stability of the south-facing slope. Mapping on the East face was used as proxy for the currently unstable West face, which had been mapped previously to have a very similar stratigraphy and structure. During the field reconnaissance, the bedrock conditions were evaluated and classified by visual examination of surficial deposits and outcrops. Bedrock joints, structure, fractures and weathering were assessed and classified, and the geometry of discontinuities (dip and dip direction) were measured with a Brunton compass. Structure measurements made during the April reconnaissance are provided in Appendix B and were supplemented with previous mapping for modeling. Measurements were made of rock mass discontinuities along the entirety of the slope to evaluate the range and variability of discontinuity geometry and character. The collected datasets are believed to be representative of the exposed rock mass. Exposed outcrops were characterized using the Hoek-Brown rock mass classification system to assess in-situ strength properties (Hoek, 2000[2]). Joint surface conditions, such as continuity, spacing, aperture, infilling, roughness, seepage, and a rating of significance were characterized, and collated on data tables. The degree of roughness and larger-scale waviness of joint surfaces was evaluated using the Joint Roughness Coefficient (JRC) methodology of Barton (1977[3]).

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[2] Hoek, E., 2000, Practical rock engineering: on-line document, rocscience.com

[3] Barton, N.R. and Choubey, V., 1977, The shear strength of rock joints in theory and practice: Rock Mechanics, Vol. 10 (1-2), pp. 1-54.

Digital photos were taken to document rock identification, typical and atypical rock conditions, locations of measurements, zones of localized weakness, and/or locations of geologic interest. Field measurements, mapping control, and feature location were recorded using a hand-held Global Positioning System (GPS) unit (Garmin™60 Cx), with typical degree of positional uncertainty of +/- 9 feet (as calculated by the GPS device).

On the East Face, planar, moderately rough to rough, south-dipping bedding planes with very high persistence defined the structure between the two upper limestone beds. Photo 5 in Appendix A shows this structure. The bedding plane dips 30 degrees in a 189-degree azimuth direction. Appendix A photos demonstrate the plane becomes more undulatory and rough with large crystals and second-order asperities traversing from east to west. The mudstone interbed has eroded back from the face of the outcrop and was difficult to evaluate for strength parameters. The mudstone appears to be well cemented with clasts of limestone entrained. In areas reachable with a geologic pick, with difficulty, the mudstone was evaluated as weak (R2). $CaCO_3$ stalactites are forming across the 4 to 7-inch aperture between the upper and lower limestone forming a connection between the two beds. The larger stalactites are forming on the face of the limestone indicating deposition from $CaCO_3$-rich surface runoff. Smaller stalactites were observed within the asperity. The stalactite connections indicate the East Face has not slid on the bedding plane over an extensive period of geologic time.

The structure dominating the limestone bed face appears to be comparable to the apparent release plane for the West Face ground event. The primary structure bedding plane can be visually carried across the East and West faces and further west across the drainage. The secondary joint set mapped on the East face is visually comparable to the release plane for the ground event on the West face. The secondary joint set dips 45 degrees in a 055-degree azimuth direction. The joint is generally planar, slightly rough, low to medium persistence with moderately close to wide joint spacing. The secondary joint set was observed consistently across the East Face.

3. KINEMATIC ANALYSES OF FAILURE MODES

Kinematic analyses incorporate the discontinuity data collected from the Mid-Continent Limestone Quarry and slope above to help identify potential rock slope failure conditions. Discontinuity data from the field mapping were compiled on stereographic projections (lower hemisphere, equal angle) and analyzed with the computer program DIPS v. 8.021 (RocScience, 2022) to evaluate trends and discontinuity sets. The resulting stereographic plots are included in Appendix C. The purpose of these analyses is to evaluate the potential for shallow failures in the cut slope walls rather than circular failure. The results are used in analyzing the stability and factor of safety for failure modes.

Characteristics of individual discontinuities identified on the East Face slope and above are provided in Appendix B. Global mean planes and rosette plots illustrate the East Face rock mass is controlled primarily by bedding, dipping moderately to the south-southwest, creating dip slopes that dictate slope topography. Nine bedding structure measurements from the CGS throughout the quarry expansion area are presented on Kirkham et al., 2008, ranging from 24 to 44 degrees, all dipping to the south- southwest. The CGS measurements are generally consistent with data collected during the

KUE April 2023 field reconnaissance on the East face that indicate a tighter cluster of dip ranging from 29 to 32 degrees, all dipping to the south-southwest (192° +/-10). The steeper CGS measurement of 44 degrees is assumed to be lower on the face where the fold is steeper. The primary discontinuities controlling rock mass stability in the slope are generally persistent and control rock mass response.

After defining the discontinuity sets, analyses for each mode of potential failure were performed. The number of the discontinuity stereonet poles that meet the kinematic criteria of lying within the critical zone for failure are represented on Table 1 as a percentage of the total number of discontinuities.

Table 1. Summary Results of Kinematic Stability Analyses for East Face – Critical Failure Poles

Failure Mode		Critical Poles	Percentage of Poles
Wedge	All Intersections	4	1.33%
	Sets Only	0	0.00%
Planar Slide (No Limits)	Limestone (Bedding Only)	0	0.00%
	Mudstone (Bedding Only)	10	100.00%
Planar Slide (Lateral Limits)	Limestone (Bedding Only)	0	0.00%
	Mudstone (Bedding Only)	7	70.00%

Note: Failure mode numbers in table represent the percentage of total discontinuity poles that kinematically lie within the critical zone for failure.

Based on the kinematic analyses, there is a low probability of wedge failure. The results from the wedge stability analyses indicate a very low probability of failure.

The kinematic analyses corroborate field observations from the field reconnaissance that indicate the primary failure mode is planar sliding along the limestone bedding planes consisting of mudstone dipping adversely along the south-facing highwall. Wedge sliding of rock blocks occurs when the intersection line between two discontinuities plunges in the direction of the cut face at an angle steeper than the rock friction angle but less steep than the angle of the cut slopes (Wyllie and Mah, 2004[4]), as seen in Photo 16. Critical intersections represent wedge geometries that satisfy frictional

[4] Wiley, D.C. and C.W. Mah, 2004, Rock Slope Engineering, 4th Edition, Spoon Press, New York, NY.

and kinematic conditions for sliding. This point must fall outside the cut slope's great circle but within the rock friction kinematic boundary cone to be considered to have the potential for wedge sliding (red-shaded area in Appendix C figures). The thin interbed of shaley mudstone observed along some of the limestone bedding planes creates a potential failure plane of lesser cohesion and fiction angle than the limestone. Stability modeling was completed to evaluate this geometry for potential failure.

4. STABILITY MODELING ANALYSES OF FAILURE MODES

Long Term Steady-State stability analyses along the cut slopes was performed to evaluate the potential bedrock failures along discontinuities in the rock mass. Results from these analyses were used to evaluate the cause of failure on the West wall and informed the conceptual design and mitigation support for the East and West faces. General limit equilibrium method slope stability analyses for the East and West face were performed using the software program RocPlane from RocScience (v.4.011). A factor of safety is calculated by modeling the effects of joint shear strength (in this case, primarily the weak interbed), water pressure within the joint, joint orientation and slope geometry intersections within a Monte Carlo sampling method. The models were checked by the limit equilibrium method of slices (Morgenstern-Price) using the software program Slope/W from Geostudio 2023.1. Using this methodology, the factor of safety for a given geometry is determined by calculating the ratio of resisting forces to driving forces on trial failure surfaces. Slip surface scenarios analyzed for this report were block specified. The slip surface with the lowest factor of safety against sliding is described as the minimum factor of safety for the defined conditions. The Long Term Steady State was analyzed to consider the extended term stability of the highwall, and the rock strength is characterized by effective stress parameters.

To determine the geologic input parameters for the Mid-Continent Limestone Quarry stability modeling, characteristic values of the Leadville limestone were initially taken from empirical data in peer-reviewed publications and verified by publicly available typical values for the units encountered on the slope. Based on tests performed by the United States Bureau of Reclamation[5] on the Leadville Limestone in the Paradox Valley, the friction angle of the limestone is approximately 40 degrees, and the cohesion is approximately 3,050 psi. Caltrans[6] estimates for hard rock masses, like limestone, the friction angle of the rock mass varies from 35 degrees to 45 degrees and the friction angle of the joint areas can vary from 35 degrees to 40 degrees. No site-specific strength testing has been completed. Mohr-Coulomb strength criterion framework was utilized to define bedrock and joint material strengths. Mohr-Coulomb assumes an inherent cohesion in over-consolidated fine-grained or cemented soils and bedrock. And finally, a back analysis of the West face ground event was used to corroborate these empirical values. The West face stability analyses parameters were manipulated to achieve a Factor of Safety (FOS) of less than 1.0, in both RocPlane (FOS 0.99) and checked in Slope/W (FOS 0.92), indicating probable failure (Appendix D). Plane water pressure was modeled at 30% filled.

[5] Ake, J., Mahrer, K., O'Connell, D., Block, L., 2005, *Deep Injection and Closely Monitored Induced Seismicity at Paradox Valley, Colorado.*, United States Bureau of Reclamation.
[6] California Department of Transportation., 2013, *Rock Strength and Its Measurements.*

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The initial empirical and final properties reevaluated following the back analysis are summarized in the table below.

Table 2. Leadville Limestone and Interbed Strength Parameters				
Material	**Parameter**	**Cohesion (psf)**	**Friction Angle (deg)**	**Unit Weight (pcf)**
Leadville Limestone	Empirical	5,000	35	150
	Post-Backanalysis	10,000	35	150
Interbed Material	Empirical	40	25	150
	Post-Backanalysis	550	25	130

East Face Stability

Slope stability results of the East Face based on modeling of the above conditions indicate a factor of safety of 1.2 for the south facing highwall. This factor of safety is along a failure plane angle of 30 degrees which correlates to bedding dip of the soft interbed material. A tension crack was inserted as a release plane for the planar slide that correlates to the secondary joint set (mean set plane 45°; 055) mapped in the field on the East face. This joint set is perceived as the release plane for the West face 2023 ground event that can be seen in Photo 2 (Appendix A). Critically, water pressure was deterministically modeled as 30% filled with peak pressure at the tension crack base. Sensitivity analysis shows the factor of safety is particularly sensitive to water level assumptions.

For any rock mass there is the possibility of large-scale, random joints with a low strength such as from weathering, historic sliding, or clay infilling. If such a joint or several joints exist and if these joints have a disadvantageous orientation and location, then there could be a large-scale slope instability. However, field observations by KUE did not reveal any such joints beyond those previously identified.

5. ROCKFALL

5.1. ROCKFALL MODELING

Rockfall modeling was performed on three transects along the East face that are representative of the varying geologic and topographic conditions (Figure 2a). The three slope geometries were created from LiDAR data provided by RMRA. Modeling was performed using the computer program Rockfall v.8.004 by RocScience that simulates the bounce paths of rock blocks down a slope, and calculates block velocities, end points and kinetic energies at user specified points along the slope. The rockfall simulation uses coefficient of restitution (both normal and tangential) parameters to model the loss of kinetic energy between the rockfall block and ground surface at the point of impact. Based on the site reconnaissance, two slope materials were identified: limestone headwall and Limestone Scree / Blast pile. A mean value was assigned for each property with a normal distribution of standard deviation. Similar to the slope stability analyses, input values for normal restitution, tangential restitution, dynamic friction and rolling friction were initially derived from desktop literature review. The values were verified under a back analysis on the west wall along trend of the January 2023 ground event. Input values were revised until the rockfall runout and energy resembled that of the 2023 ground event, correlated to topographic data of the rockfall debris field. Summary of slope input parameters is provided in Table 3.

Table 3. Rockfall Simulation Input Parameters

Material		Normal Restitution (Rn)	Tangential Restitution (Rt)	Dynamic Friction	Rolling Friction
Leadville Limestone	Mean	0.32	0.71	0.55	0.15
	Standard Deviation	0.04	0.04	0.04	0.02
Interbed Material	Mean	0.32	0.71	0.55	0.30
	Standard Deviation	0.04	0.04	0.04	0.04

Damping was disabled for viscoplastic and forest & vegetation. Slope roughness parameters were set to 0 degrees because roughness is already accounted for by the detailed slope geometry used in the model. Three rock types were used with increasing size and mass to mimic the January ground event. The rigid body method was used to allow definition of rock size, mass and shape. The 1) Small (2022 lbm), 2) Medium (20,227 lbm), and 3) Large (93,642 lbm) blocks were assigned square, pentagon and rhombus shapes to simulate the ground event blocks observed in the debris pile.

Computational modeling was completed with a linear seeder point at the top of the upper limestone bed with a minimum of 3,000 rocks simulated. A crest loss of the overhanging limestone bed was induced to remove that geometry at point of rockfall initiation to maximize the translational velocity. Detailed results on the distribution of bounce height, velocity, and impact forces for each run were obtained by locating data collectors along the slopes. Those results were used to evaluate appropriate berm height, setback from the slope toe, and determined total energy impacting the berm.

5.2. ROCKFALL MITIGATION RECOMMENDATIONS

Based on the results of the rockfall modeling along the West face and multiple East face transects, the following recommendations and descriptions of rockfall treatments are provided below.

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Rockfall Runout Setback

A prescriptive setback was defined from the base of the highwall to the maximum extent of rock block endpoints across the three East face transects. The 2D sections illustrating the steps, bounce height and endpoints for the 3,000-block run are provided in Appendix E. The maximum endpoint block with the longest runout is highlighted. In all three transects, the maximum runout block was an outlier and considered a conservative estimate for probable rockfall. Figure 2b represents the setback zone from the base of the highwall that is defined by this conservative estimate for maximum rockfall runout. No man work shall be performed within the setback without additional stabilization or barriers. Figure 2b illustrates the rockfall maximum endpoints and the boundaries of the rockfall setback zone from the toe of the highwall. Coordinates of the setback and a Google Earth kmz file have been provided to RMRA to designate the setback.

Rockfall Berm

A rockfall berm was modeled on the three East face transects as a remedial measure to reduce the size of the setback zone (Figure 2b), defined above. The berm size and location were defined through an iterative modeling process to minimize the size of the berm and decrease the setback from the highwall toe. Based on computational rockfall modeling, we support using the equivalent of a berm composed of limestone scree with a height of 15 feet, crest width of 5 feet and maximum slope angle of 32 degrees. Maximum kinetic energies modeled along the ten transects are all within that tolerance of maximum allowable impact energy. Rockfall analyses provided in Appendix E indicates that 100% of simulated rockfall blocks were contained by the rockfall barrier, in tandem with the catchment basin. Where the rockfall berm is impacted by larger blocks, the barrier should be repaired. The berm is considered in tandem with a setback from the highwall toe that will act as a catchment basin. A Rockfall Catchment Area Ditch (RCAD) is recommended along the entire length of the East face. Parameters contributing to RCAD effectiveness include 1) slope height and angle, 2) ditch width, depth and shape, 3) anticipated block size and quantity of rockfall, and 4) effect on rock fall trajectories of slope irregularities (Wyllie and Mah, 2004). The RCAD will also act as a retention basin for fallen rock to be cleaned over time. Rockfall modeling of the RCAD and berm design is effective at reducing the southern extent of the rockfall setback zone.

Long Term Inspection Program

An effective proactive approach to slope stabilization will require a consistent, long-term program of inspections and periodic maintenance of the berm and catchment area. Rockfall blocks should not be permitted to accumulate. Damaged portions of the berm should be repaired immediately. Periodic inspections of the slope and outcrops by an engineering geologist or geotechnical engineer will be required over time to investigate natural deterioration of the stability conditions due to 1) weathering/erosion of the surface rock, 2) increases in fracture aperture by water causing loosening of surficial blocks, 3) loss of block interlock or support following minor block failure, and 4) growth of vegetation roots. Inspections after seasons of significant precipitation should be a high priority, particularly with freeze-thaw potential.

6. LONG-TERM STABILIZATION AND CONFIGURATION

Long-term steady state stability analysis of the west face highwall within the massive limestone was performed to evaluate the potential bedrock failures along simulated discontinuities in the rock mass. No weak interbeds or adverse bedding planes daylight in the massive limestone in the active quarry wall. The January 2023 ground event failed along the lower weak bed above the massive limestone. The massive limestone was modeled at various slopes angles to determine the stability of the lower limestone layer. The slope geometry was analyzed using limit equilibrium method slope stability analyses using the software program Slope/W from Geostudio. Geologic input parameters defined above were used for stability modeling.

Using this methodology, the factor of safety for a given geometry is determined by calculating the ratio of resisting forces to driving forces on trial failure surfaces. Slip surface scenarios analyzed for this report were block specified. The slip surface with the lowest factor of safety against sliding is described as the minimum factor of safety for the defined conditions. The Long Term Steady State was analyzed to consider the extended term stability of the highwall, and the rock strength is characterized by effective stress parameters. A factor of safety is calculated by modeling the effects of joint shear strength, friction angle, and water pressure within tension cracks.

Based on information provided by RMRA mining staff and KUE site reconnaissance, no known tension cracks or discontinuities are visible or known to exist within the massive limestone layer. For the analysis, tension cracks were placed within the upper slope of the highwall to simulate long-term weathering and the release plane comparable to the ground event of the west slope. The tension cracks were modeled as 50% water-filled plane. Strength properties of the massive limestone utilized empirical values similar to those used within the west slope back analysis to provide a conservative factor of safety due to no site-specific strength testing having been completed.

The Colorado Department of Reclamation Mining and Safety (DRMS) recognizes that a suitable minimum factor of safety is dependent on the engineering analyses performed, accuracy of model input parameters and level of impact to life and facility safety. The FOS values recommended by the Colorado DRMS[7] are scaled based on the robustness of input parameters and analyses weighted by the consequence of failure. The Colorado DRMS recommended FOS values are reproduced in Table 4 below.

[7] Colorado Division of Reclamation, Mining and Safety, 2016, Design Standard of Care for Slope Stability/Geotechnical Analyses.

Table 4. Slope Stability Design Standard		
Consequence of Failure / Analyses	**Minimum Factor of Safety**	
	Generalized, Assumed, or Single Test Strength Measurements	**Strength Measurements Resulting from Multiple Tests**
Non-Critical Structures - Static	1.3	1.25
Non-Critical Structures – Pseudostatic	1.15	1.1
Critical Structures - Static	**1.5**	1.3
Critical Structures – Pseudostatic	1.3	1.15

For the RMR Aggregate site, the failure path of the slope could impact quarry facilities on the bench, and potentially impact life safety, therefore the structures were considered Critical. Static analyses were performed, together classified in Table 4 as "Critical structures – static". The slope stability analyses were performed using some historic data provided by the client, but the model input parameters were largely informed by empirical values that were confirmed through our backanalysis, classified as "Generalized, assumed, or single test strength measurements" in Table 4. Therefore, a FOS of 1.5 was considered as the minimum acceptable FOS for the project.

As a cross check, other mine-industry references commonly used for recommendation of an acceptable minimum value for FOS include a table of minimum FOS for slope scale vs. consequence of failure from the Acceptance Criteria chapter in the Guidelines for Open Pit Slope Design manual, reproduced here as Table 5.

Table 5. Typical Mine Design Criteria for Slopes (Wesseloo & Read, 2009[8])			
Slope Scale	**Consequence of Failure**		
	Low	**Medium**	**High**
Bench		FOS ≥ 1.1	
Inter-Ramp	FOS ≥ 1.15-1.2	FOS ≥ 1.2	FOS ≥ 1.2-1.3
Overall	FOS ≥ 1.2-1.3	FOS ≥ 1.3	FOS ≥ 1.3-1.5

[8] Wesseloo, J. and Read, J. (2009). Chapter 9 - Acceptance Criteria. In: Guidelines for Open Pit Slope Design, J. Read and P. Stacey (eds), pp 221-236. CIRSO publishing. 496p.

For the RMR Aggregate site, the failure path of the slope could impact quarry facilities on the bench, so the consequence of failure from moderate to high was evaluated. This correlates with the recommendation from the DRMS recommendations reproduced in Table 4, corroborating a FOS of 1.5 was considered as the minimum acceptable FOS.

Based on the slope stability results for the west wall, the massive limestone is stable for a variety of bench slope geometries. Slope stability results for the west face based on the above conditions are summarized in Table 4.

Table 4. Summary of Factor of safety for varying bench slope geometry	
Bench Slope Geometry (Horizontal:Vertical)1	**Factor of Safety (long-term)**
1:1	1.40
1.4:1	1.54
1.67:1	1.63

1. Max bench height of 30 feet

Using the accepted minimum FOS of 1.5, a H:V bench slope geometry of 1.4:1 or larger is deemed acceptable.

7. JULY 6 SITE RECONNAISSANCE

A site reconnaissance on the West wall was performed on July 6, 2023 with RMRA staff and Ben Langenfeld of Lewicki & Associates to evaluate the north limit of the upper limestone beds and formulate a plan across represented disciplines to determine the optimal quarry headwall extent for long-term slope stability using the modeled long-term slope geometries discussed above. Photos with detailed captions from the reconnaissance are included in Appendix F.

A site mapping program was performed to collect structure data on the West wall and to determine the north limit of the upper limestone beds as they pinch out upslope. The bed thickness and bedding dip of the two upper limestone beds decreases to the north from the quarry headwall as the forelimb of the monocline rolls over toward the fold axis. Kirkham et al. map the upper limestone beds as dipping between 38 degrees near the quarry to 24 degrees toward the top of the hill to the north, which forms dip slopes and tends to control hillside slope topography. Borehole Lynx 05-099 (1966) drilled vertically in the area of the event identifies dip in the upper 40 feet of the section as 40 degrees. The July 2023 site reconnaissance mapped the upper limestone beds from near the ground event and northward as the beds pinch out and bedding decreases to approximately 28 degrees. Appendix F photos document the beds decreasing in thickness at points along the slope. Locations of bedding dip and dip direction on the top of the massive limestone (contact with the upper limestone units) and measurement of the presence/thickness of the upper limestone beds are designated on

Figure 3 (provided by Lewicki & Associates) with corresponding site numbers, detailed below from south to north:

- Site 2: 39.57167; -107.322920, approx. elev. 7,170 ft.
  - Structure measurement (dip/dip direction): 34°;185.
  - 15 ft thick upper/lower limestone beds.
  - Laterally continuous bed surface. No significant aperture or adverse interbed.
  - Pairs with comparable surface looking west across drainage.
- Site 3: 39.57190; -107.32288, approx. elev. 7,200ft.
  - Upper limestone layer measured 10 ft thick.
  - Laterally continuous bed surface without significant aperture or interbed.
- Site 4: 39.57211; -107.32257, approx. elev 7,290 ft.
  - Daylight of laterally continuous bedding plane. Upper limestone beds have pinched out at this line of latitude.
- Site 1: 39.57250; -107.32250, approx. elev. 7,370 ft.
  - Structure measurement (dip/dip direc): 28°;183.
  - Laterally continuous joint surface (>10 meters), wavy, covered in thin talus.
  - Pairs with comparable surface looking west across drainage.

Photos of the site locations are provided in Appendix F. Talus cover and the waviness of the bed surface limited the ability for high confidence structural measurements at some sites.

The spatial intent of the slope configuration models discussed in section 6 should be to extend north to where the upper limestone layers pinch out above the current mining activities. The bedrock bedding slope, which is the boundary between the massive limestone below and the adverse limestone units above, was observed as very laterally continuous but could undulate locally creating a slight change in dip direction. To meet the established minimum FOS for the slope, using a bench slope geometry of 1.4H:1V or greater that leaves varying thicknesses of the upper limestone layers in place will likely require the use of mechanical stabilization. This is considered a lesser option for long-term stability and safety, but is considered below on an as needed basis.

8. MINE STABILIZATION

A summary of the mine operation plan recommendations, from a geotechnical perspective, is provided here to inform a full mine operation plan provided by RMRA under separate cover. The overall intent is to achieve longterm slope stability by eliminating the potential for headwall failure along the upper limestone beds. The mine plan works under the assumption the upper unit planar slide failure mode , dipping to the south along the laterally continuous bedding plane, will only release in that direction. The mine plan is a phased and stepped approach working from the southwest corner of the headwall to the northeast with the intent to eliminate the possibility of permanent or temporary condition of the upper limestone in the cutslope wall.

It is the opinion of KUE that the upper limestone layer should be removed completely from the highwall to minimize the risk of another release. This can be performed via multiple mining methods.

RMRA will develop and discuss the process under their full mining plan. RMRA will perform this work from a safe position outside of the release plane. This approach stabilizes the slope for long term stability within the massive limestone. As stated above, there is a possibility of local wedges of the upper limestone unit remaining where the bedding slope has locally dipped differently or a unknown joint in the top of the massive limestone has created a wedge. In these circumstances, if the wedge cannot be scaled or blasted safely, mechanical stabilization would be warranted for life safety.

8.1. ACTIVE MECHANICAL STABILIZATION

Mechanical stabilization shall be utilized on the Mid Continent Limestone Quarry if the upper limestone layer is encountered within a highwall or bench. Mechanical stabilization will be utilized to pin the upper limestone layer to the lower massive limestone with the use of tiebacks to increase the resisting force of the upper limestone layer.

A preliminary design of the anchorage system was performed. For this analysis, a general limit equilibrium method slope stability analyses for the East and West face were performed using the software program RocPlane from RocScience (v.4.011). A factor of safety is calculated by modeling the effects of joint shear strength (in this case, primarily the weak interbed), water pressure within the joint, joint orientation and slope geometry intersections within a Monte Carlo sampling method. Potential upper limestone slope heights ranging from 5 feet to 15 feet were modeled to determine the resisting force required to reach a factor of safety of 1.5.

Several mechanical stabilization methods were considered, ultimately a 7-strand anchorage was selected for both logistical purposes and the stand lengths can be changed to accommodate longer lengths for this difficult to reach location. In some modeled instances, the upper limestone could exceed greater than 10 ft thickness which would require a long total length of 45 feet. Given the load and lengths necessary, a traditional bar would be exceptionally long requiring coupled bars and likely a crane, becoming logistically cumbersome. A concrete bollard with tie backs was also considered and has been effectively used locally. However, the concrete bollard would require either pumping concrete from the base of gravity feed from above. Neither are logistically realistic for the as-needed local stabilization approach. The 6-inch hole could only be reduced in diameter if the number of strands was reduced, requiring much longer bond lengths per strand, much longer strands and therefore a much longer drill hole. The longer drill hole would require a much larger drill rig.

Due to the 6-inch hole diameter and depths required, a berm is needed to resist the upper layer from failing and to be used as a work bench to install the required anchors. Table 5 below provides details of the necessary anchorage, hole diameter and lengths.

Table 5a. Summary of Required Anchor Stabilization [1,3]				
Upper Limestone Height (ft)	Anchorage	Vertical Spacing (ft)	Horizontal Spacing (ft)	Hole Diameter (in)
10-15	7 strand	10	10	6
5-10	7 strand	-	15	6
0-5	#8 Gr. 75	-	30	4

Table 5b. Summary of Required Anchor Stabilization [1,3]					
Upper Limestone Height (ft)	Bond Length (ft)	Free Length (ft) [2]	Tail Length (ft)	Total length (ft)	Lock off Load (kips)
10-15	28	15	2	45	235
5-10	13	15	2	30	225
0-5	3	15	2	20	30

Notes:
1.Lengths shown are the minimum lengths required.
2. Minimum free length per PTI is 15 feet.
3. Data presented is a preliminary design. Final design of the anchorage system will need to be performed if required on site.

In order to achieve a higher factor of safety, KUE recommends horizontal drains to be installed through the joint of the upper and lower limestone if active support is required onsite.

9. LIMITATIONS

This report has been prepared for RMRA for specific application to the Mid Continent Limestone Quarry project as understood at this time, in accordance with generally accepted geotechnical engineering practices common to the local area. No other warranty, express or implied, is made. In the event that changes in the nature, design, or location of the planned construction are made, the conclusions and recommendations contained in this report should not be considered valid, unless the changes are reviewed by KUE and the conclusions of this report are modified or verified in writing.

Nothing contained in this report shall be construed to create, impose, or give rise to any duty owed by KUE to any individual or entity other than RMRA. This report is for the sole use and benefit of RMRA and may not be used or relied upon by any other individual or entity without the express written approval of KUE.