Large-Scale Slope Instability at the Gold Quarry Mine, Nevada

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Abstract

The upper East Wall of the Gold Quarry open pit mine is formed within the Carlin Formation – a fine-grained Tertiary volcaniclastic package deposited in a shallow lacustrine environment that is predominately comprised of tuffs, silts, sands and gravels with a typical thickness of 400 to 500 ft. Large-scale slope instability along a sector of the upper East Wall was routinely managed during various stages of pit development, culminating in a progressive failure of approximately 8 million tons in April 2009. A rapid catastrophic failure comprising approximately 1.5 million tons of material occurred with little warning on December 24, 2009.

Preliminary back-analyses performed shortly after the failure suggested that the most likely failure mechanism resulted from a deep seated circular-type displacement along a weak, brittle basal clay layer situated along the base of the Carlin Formation, and immediately above more competent bedrock. However, this preliminary hypothesis could not explain the deformation patterns of the multiple stages of slope instability that had been observed in the slope. Further back-analyses demonstrated that the base of the failure zone was more likely to be shallower and also suggested that weaker clayey materials were likely present at a higher elevation.

Additional geotechnical drilling and laboratory testing confirmed that the base of the failure zone had occurred in weak clay-rich materials situated well above the basal clay layer. Updated numerical stress modelling and slope stability analyses indicated that the large-scale slope failures were strongly influenced by unloading at the toe, which results in strain deformation and progressive weakening of the brittle clayey materials that are present above the basal clay layer. This improved understanding of the deformation patterns and failure mechanisms has resulted in the implementation of slope flattening and toe buttressing as a cost-effective remedial measure to enhance the stability of the upper East Wall.

1 Introduction

Newmont USA Limited (Newmont) is currently operating the Gold Quarry Mine, a large open pit operation located in the carbonate-hosted Carlin Trend in northern Nevada. The mine started production in the 1980s with an original projected mine life of six years. Ongoing discoveries and subsequent expansions have allowed the mine to continue operating to present. The current open pit is about 1.5 miles long and 1 mile wide. The slope height of the East Wall reached over 1300 ft in 2009.

The upper 400 to 500 ft of the East Wall (between 4900 and 5400 ft in elevation) exposes a weak fine-grained Tertiary volcaniclastic sedimentary package, namely the Carlin Formation. The upper East Wall was excavated at inter-ramp angles ranging from 30 to 35 degrees and the overall slope was typically around 26 to 27 degrees prior to the slope failure. The lower East Wall was developed in the competent siltstone bedrock.

Slope instability along a sector of the East Wall, namely the Nine-Points area, was first observed in mid-2008. Tension cracks were found along the 5260 bench and some bench ravelling occurred along some west-southwest trending micro-faults in the upper Carlin Formation. Seeps and wet bench faces were noted at the 5140 bench
prior to the failure. Noticeable toe heaving was observed along the 4970 bench immediately above the bedrock contact of the Carlin Formation in August 2008 (CNI, February 2010). The creeping type slope instability was managed by a partial unweighting cut, careful debris cleanout and a comprehensive monitoring program through late-2008 to early-2009.

The upper slope deformation accelerated in early-2009 due to the excavation of the debris material at the toe of the slope. The mining operation at the Nine-Points toe was forced to cease in February 2009 to control the rate of upper slope displacement. The slope movement rates gradually increased to 10 to 15 ft/day and a circular type progressive slope failure occurred on April 26, 2009. Approximate 3 million tons of runout material flowed over the existing ramp intersection. Newmont constructed a backfill ramp in the northern area of the pit immediately after the April 2009 slope failure to re-establish access to the lower pit in early May 2009. The existing ramp intersection was left for debris containment and the materials that spilled onto the lower bedrock benches were cleaned up when the failure slowed to reasonable displacement rates.

The April 2009 slope failure resulted in the formation of a near vertical scarp, 200 to 250 ft in height, along the crest of the East Wall. Newmont continued a rigorous slope deformation monitoring program in the slope instability area and attempted to clean up the debris material at the toe to reconnect the ramp to the south in the second half of 2009, to establish a platform for a potential buttress and then remediate the scarp. A rapid catastrophic slope failure comprising approximately 1.5 million tons of material occurred with little warning on December 24, 2009. The slope failure reactivated the April 2009 failure mass and resulted in a massive runout of several hundred feet of material into the pit bottom. The mining activity at the Gold Quarry Open Pit ceased after the slope failure.

A panoramic view of the post-failure East Wall of the Gold Quarry open pit is shown on Figure 1. The overall slope angle from the Nine-Points crest to the toe on the 4930 bench was about 22 degrees at the time of photo (March 2010).

![Gold Quarry open pit overview](image)

**Figure 1.** Gold Quarry open pit overview (looking east, March 2010).

This paper presents the geotechnical characterization and slope stability analysis process for the Gold Quarry Nine-Points slope instabilities. The findings of this study were utilized for slope remediation design and implementation.
2 Geological, hydrogeological and geotechnical characteristics

2.1 Geology and structures

The geology of the Gold Quarry East Wall area consists of competent siltstones of the Devonian Rodeo Creek Formation that are overlain by the weak sedimentary and volcaniclastic deposits of the Tertiary Carlin Formation. The Carlin Formation is crosscut by a series of north-northwest and north-northeast trending faults as shown on Figure 2.

The Carlin Formation is generally composed of volcaniclastic lacustrine sediments of inter-bedded gravel, sand, silt and clay. The beds are commonly discontinuous and are separated by erosional unconformities. These lake bed sediments are graded and are generally coarser grained in the upper section of the formation, where the sequence is dominated by silts and sands. The degree of compaction and induration is extremely variable in the inter-bedded lacustrine and tuffaceous sequences. The Carlin Formation stratigraphy is unpredictable and extremely variable at the mine scale (Golder, 2002).

Historic site investigation work and geotechnical studies defined five basic groups within the Carlin Formation occurring from bottom to top as follows:

- Basal gravel and clay: An angular basal gravel layer that commonly contains a discontinuous basal reduced clay layer immediately along the bedrock surface.
• Basal and lower tuff: A waxy yellow-green-grey basal tuff and lower laminated tuff with moderate to high clay content.

• Middle transition tuff: A transition sequence that separates the lower clay-rich tuffaceous materials from the upper sands/silts with minor clay or tuffaceous contents. The transition sequence contains gravelly sands, the middle tuff, and the middle massive sands and the upper tuff, but is highly variable depending on the location.

• Upper silts and sands along with a massive sand package.

• Debris flow.

A series of north-northwest trending, east dipping faults (Tuff Fault, Tails Fault and Nine-Points Fault) and north-northeast trending, northwest dipping faults (Challenger Fault and James Creek Fault) cut through the entire Carlin Formation along the East Wall. These steeply dipping fault structures progressively drop the Carlin Formation down to the east. A southeast-northwest trending normal fault cuts across the Nine-Points sector and numerous unnamed and largely un-mapped faults were observed along the upper East Wall.

2.2 Hydrogeological features

The Carlin Formation is a heterogeneous sedimentary sequence containing discontinuous strata with significantly different hydraulic properties. Clay-altered faults in the Carlin Formation act as groundwater barriers (aquitards) resulting in groundwater compartmentalization. Historic hydrogeological studies suggest that there are three separate perched groundwater zones in the Carlin Formation. Water is separated by three clay-rich aquitards; namely, the basal tuff, middle tuff, and upper tuff. However, continuous water-bearing aquifers are unlikely to occur in the Carlin Formation due to the complexity of stratigraphy and faulting structures (Golder, 2002).

A number of vibrating wire piezometers have been installed and maintained in the Carlin Formation since 2002. The piezometer monitoring results indicated that there were two perched water tables in the Nine-Points area. As of March 2010, the upper perched water table was at an elevation of approximately 5100 ft and the lower perched water table was at approximately 4800 ft. The bedrock immediately beneath the Carlin Formation was completely dry (i.e. the pore pressure at the top of the bedrock is less than or equal to zero) due to the ongoing pit dewatering operations since 1992.

The monitoring data suggested that a downward hydraulic gradient existed in the Carlin Formation. The pore water pressure applied on the base of the Carlin Formation appeared to be less than hydrostatic as a result of downward drainage to the bedrock (Itasca, March 2010).

2.3 Geotechnical characteristics

The Carlin Formation in the Nine-Points area consists of a very complex, inter-bedded, poorly consolidated sequence of sands, silts, tuffs, and altered tuffs. Much of the fine-grained material is tuffaceous, having been deposited as ash falls that subsequently underwent argillic alteration to clay-rich strata.

The combination of complex stratigraphy, faulting systems, and hydrogeological features in the Carlin Formation made it extremely difficult to differentiate the strata within the Nine-Points area, especially inside the failure zone. The geotechnical information around this area was not sufficient to delineate an accurate stratigraphic model for stability analysis immediately following the December 2009 slope failure.

A simplified geotechnical model was developed during the early-2010 preliminary slope stability assessment by other geotechnical specialists based on insufficient geologic information. This simplified geotechnical model separated the weak Basal Reduced Clay layer from the Carlin Formation and consolidated the rest of the upper sedimentary/volcaniclastic package into one geotechnical unit, namely the Carlin Formation. This was done because the preliminary back-analyses suggested that the Nine-Points slope stabilities were most likely deep-seated circular type failures along a continuous weak clay layer at the base of the Carlin Formation, and
immediately above more competent bedrock. A typical cross section (Section J) of the Nine-Points slope instability zone is illustrated in Figure 3.

The cross section showed that the December 2009 slope failure resulted in the formation of a new near vertical scarp further to the east along the crest of the Nine-Points Slope. Seeps were observed at various locations in the middle and lower East Wall and some of the locations produced a constant flow of water in March 2010. The debris flow covered a significant portion of lower East Wall below the Nine-Points slope and was largely comprised of sands and silts.

Geotechnical parameters for the Carlin Formation, Basal Reduced Clay and Bedrock units are summarized in Table 1. The strength parameters were based on the average values of historic laboratory testing data and back-analysis results at the Gold Quarry Mine. The average peak strength of the Carlin Formation was represented by a friction angle of 24 degrees along with 1700 pcf of cohesion. The residual friction angle of the Basal Reduced Clay unit was estimated to be 9.5 degrees.

Table 1. Geotechnical parameters for preliminary slope stability back-analyses.

<table>
<thead>
<tr>
<th>Simplified Geotechnical Unit</th>
<th>Unit Weight (pcf)</th>
<th>Peak Strength</th>
<th>Residual Strength</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(\phi_p) ((^\circ))</td>
<td>(c_p) (psf)</td>
<td>(\phi_r) ((^\circ))</td>
</tr>
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<td>Carlin Formation</td>
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<td>24</td>
<td>1700</td>
<td>18</td>
</tr>
<tr>
<td>Basal Reduced Clay</td>
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<td>0</td>
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<td>Bedrock</td>
<td>155</td>
<td>35</td>
<td>3000</td>
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</table>
3 Slope failure mechanism verification

3.1 Slope instability back-analyses

Preliminary studies suggested that the multiple-stage failures in the Nine-Points slope were likely deep-seated circular failures through weak rock mass in the Carlin Formation. The simplified geotechnical model was utilized for further slope stability back-analyses to verify the failure mechanism and to calibrate the strength parameters for the December 2009 Nine-Points slope failure.

Back-analyses were conducted using the SLOPE/W limit equilibrium stability analysis program (GEO-SLOPE International) for both the April and December 2009 Nine-Points slope instabilities. Pore pressure profiles were extrapolated based on limited piezometer monitoring data. Peak strength parameters were applied to the Carlin Formation while the residual strength parameters were used for the Basal Reduced Clay unit in the analyses. The Factor of Safety (FOS) for each case scenario was computed using the Spencer Method (1967).

The calculated FOS for the April 2009 failure along the weak basal clay was about 1.0. This showed that the slope instability was likely caused by removal of the toe and that a deep-seated failure mode was likely responsible for the April 2009 creeping type failure.

Limit equilibrium back-analysis results for the December 2009 slope failure are presented on Figure 4. A slightly higher FOS of 1.05 was computed for the deep-seated circular failure. However, the predicted critical slip surface was not consistent with the post-failure slope geometry. The post-failure slope geometry in the upper East Wall suggested that the December 2009 slip surface may not have been along the deep basal clay layer and that the simplified geotechnical profile used for the preliminary assessment may not be appropriate.

Additional slope stability sensitivity analyses were conducted and the results indicated that the stability of the Nine-Points slope in the Carlin Formation was very sensitive to groundwater pressure but less sensitive to the residual friction angle of the Basal Reduced Clay layer.

Alternative geotechnical models were developed for the December 2009 slope failure back-analyses in order to match the post-failure slope pattern in the upper East Wall. The results of the analysis suggested that the critical failure surface was likely along some upper weak layers in the Carlin Formation rather than in the deep Basal Reduced Clay.
3.2 Preliminary findings

The results of this back-analyses suggested the April 2009 slope failure was triggered by the removal of rigid bedrock and buttressing debris material at the toe of the Carlin Formation. The slope exhibited a creeping type failure that was manageable through careful monitoring and regular cleanup. The December 2009 slope failure was more brittle with less warning and the failure mechanism was different. The sudden catastrophic nature of the failure was likely caused by brittle failure in the upper Carlin Formation as opposed to sliding along the deep Basal Reduced Clay layer.

These preliminary findings led to the conclusion that the existing geotechnical model was incorrect. The critical slip surfaces were more likely through some weaker layer at a higher elevation and well above the deep-seated Basal Reduced Clay. However, there was not sufficient geotechnical data in the Nine-Points area to confirm this hypothesis at that time. Additional geotechnical drilling and instrumentation installation in the Nine-Points area were recommended to better define an appropriate geotechnical model for further slope stability analyses and remediation planning.

4 Further geotechnical characterization

4.1 Geotechnical drillhole data characterization

Over 10 geotechnical drillholes were completed around the Nine-Points area in early-2010 and five additional sonic drillholes were completed inside the Nine-Points failure zone in late-2010. Clayey soil samples were collected for laboratory strength tests. The 2010 geotechnical drillhole plan is illustrated on Figure 5.

Figure 5. Nine-Points 2010 geotechnical drillhole plan.

Further geotechnical characterization was conducted by revisiting all the available geotechnical drillhole logs in the Nine-Points area. A material type and strength versus drillhole depth and elevation plot was developed for each geotechnical drillhole. Each drillhole plot contains three curves representing the gravel, sand, and fines contents, respectively. The material strength was represented by the equivalent Unconfined Compressive Strength (UCS) values derived from the recorded hardness ratings. Typical material type vs. depth plots for three drillholes along the Nine-Points critical slope section are shown on Figure 6. Drillhole GTC-78 is located at the
front toe of failure, Drillhole GTC-75 is in the center of the failed mass, and Drillhole GTC-48 is out of the Nine-Points failure zone to the east.

The drillhole characterization results identified a significant clay-rich tuff layer at the front base of the Nine-Points Carlin Formation. The top surface of this clay-rich tuff layer appeared to be at El. 4920 to 4960 ft inside the failure zone, and is well above the Basal Reduced Clay layer (at El. 4600 to 4700 ft) in this area. This argillic altered clay-rich layer is relatively weak and typically weakens after exposure and/or under saturated conditions. The clay-rich tuff unit appeared to be truncated by the Nine-Points Fault to the east and by a normal fault to the east and north (see Figure 5). The fines content of this tuff layer diminished significantly outside of the Nine-Points failure zone.

The shear plane locations were identified in each sonic drillhole within the failure zone. These locations indicated that the critical failure surfaces were at approximately El. 4850 ft in the front slope near the toe, and at approximately El. 4760 ft in the center of the failure zone. It is evident that the critical shear surfaces of all slope instabilities went through the clay-rich tuff layer and were not along the Basal Reduced Clay layer.

![Drillhole material type vs. depth plots.](image)

Figure 6. Drillhole material type vs. depth plots.

### 4.2 Updated geotechnical model

An updated geotechnical design model for the Nine-Points slope area was developed based on the results of the geotechnical drillhole characterization. The revised geotechnical model contains four major groups:

- **Upper Silts and Sands**
- **Lower Laminated Tuff** (clay-rich tuff inside the Nine-Points zone and sand/clay inter-bedded tuff beyond the Nine-Points zone)
- Basal Tuff, Basal Reduced Clay and Basal Gravel, and
- Bedrock.

The geotechnical strength parameters were refined in accordance with the 2010 geotechnical drillhole logs and additional laboratory testing data. The peak strength of the clay-rich Lower Laminated Tuff and Basal Reduced Clay are represented by a friction angle of 15 degrees along with 2000 pcf of cohesion. The residual friction angle of the clay-rich units was estimated to be 11.5 degrees based on Newmont’s late-2010 laboratory testing results. The strength parameters for the upper silts and sands and bedrock remain unchanged. The updated geotechnical strength parameters are summarized in Table 2.

Table 2. Summary of geotechnical parameters for refined stability analysis

<table>
<thead>
<tr>
<th>Simplified Geotechnical Unit</th>
<th>Unit Weight (pcf)</th>
<th>Peak Strength</th>
<th>Residual Strength</th>
<th>Remark</th>
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<tbody>
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<td>Upper Sand and Silt</td>
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<td>1700</td>
<td>24</td>
</tr>
<tr>
<td>Clay-Rich Lower Laminated Tuff</td>
<td>110</td>
<td>15</td>
<td>2000</td>
<td>11.5</td>
</tr>
<tr>
<td>Sand/Clay Interbedded Tuff</td>
<td>110</td>
<td>15</td>
<td>2000</td>
<td>15</td>
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<tr>
<td>Basal Tuff, Basal Reduced Clay and Basal Gravel</td>
<td>110</td>
<td>15</td>
<td>2000</td>
<td>11.5</td>
</tr>
<tr>
<td>Bedrock</td>
<td>155</td>
<td>35</td>
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</table>

5 Revised slope stability assessment

5.1 Updated slope instability back-analyses

The stability back-analysis work was revisited using the updated geotechnical model in early 2011. Limit equilibrium stability back-analyses were conducted for the April and December 2009 slope instabilities.

For the April 2009 slope failure, peak strength parameters were applied for all geotechnical units. The calculated FOS was approximately 1.0. The critical slip surface was generally consistent with the shear plane observed from the geotechnical drillholes within the Nine-Points failure zone.

For the December 2009 slope failure, residual strength parameters were applied to the slope within the previous failure plane. Peak strength parameters were assigned to the material out of the previous failure plane. A FOS of approximately 1.2 was calculated for the December 2009 slope. This result indicated a stable slope, which contradicted the actual slope performance.

It should be noted that a rapid excavation into low hydraulic conductivity clayey materials would unload the soil and could cause a tendency for volumetric expansion. The volumetric expansion could be offset by a reduction in the pore water pressures, resulting in a nearly un-drained response. The clayey soil affected by excavation would swell over time, causing a reduction in the mean effective stresses. The reduction in mean effective stress in some areas of the soil could bring the stress states onto the failure surface. The overall collapse of the slope would be therefore delayed by the time required for pore pressure equilibration (Geo-Slope, 2010).
It was evident that the clay-rich, low permeability tuffaceous material in the Nine-Points area was susceptible to strain softening during excavation. The material near the front toe of the Carlin Formation likely reached its peak strength and then dropped to its residual strength after the April 2009 failure. Ongoing deformation could extend the disturbed zone further into the slope and would result in more clay material approaching its residual strength. At the same time, the already failed soil would experience a constantly increasing amount of shear strain that could cause a subsequent loss of strength. The FOS could suddenly drop below 1.0 once the slip surface mobilizes to a certain range. Figure 7 shows an extended disturbed zone in the clay-rich tuff; the resulting FOS was estimated to be below 1.0. This extended disturbed zone is consistent with findings from the 2010 sonic drilling inside the Nine-Points failure zone.

5.2 Numerical stress modelling

The PHASE2 finite element analysis program (Rocscience Inc.) was used to perform the numerical stress modeling to predict stress conditions and the potential failure surface for the Nine-Points slope instabilities throughout the various mine stages. Material strength parameters were consistent with those used in the revised limit equilibrium stability analyses. A perfect elastic-plastic strength model was applied for all the materials except for bedrock in order to simulate the strength mobilization during mine development. Appropriate deformability parameters were assigned to each geotechnical unit based on experience with similar soil and rock materials. A horizontal stress ratio (k) of 1.0 was assumed for this numerical modeling work.

Figure 8 illustrates the predicted maximum shear strain for the mid-2008, April 2009, and December 2009 slopes. Some shear strain was found at the toe of the mid-2008 slope, which is consistent with the observed toe heaving at that time. Significant increases in shear strain were predicted in the April 2009 slope after the removal
of the rigid toe at El. 4930 ft. A clear slip surface was identified in the upper slope. This slide pattern was exactly the same as that predicted in the limit equilibrium analysis. Another distinct slip surface can be found in the December 2009 slope due to ongoing strain softening of the clay-rich tuff and the presence of a steep scarp at the crest. This deformation pattern was consistent with the computed slip surfaces in the limit equilibrium model.

5.3 Slope failure mechanism

The Nine-Points slope instabilities were caused by a combination of the following key factors:

- **Removal of the rigid toe**: The East Wall Carlin Formation was contained by the east dipping siltstone bedrock of the Rodeo Creek Formation. Ongoing pit development gradually mined out the competent bedrock and increased the exposure of the weak Carlin Formation in the upper East Wall. Slope deformation started to develop when the rigid bedrock limb at the toe of the Carlin Formation was removed in 2008.

- **Toe unloading**: Newmont tried to re-establish the ramp intersection access by cleaning up the debris materials at the toe after the mid-2008 and April 2009 slope failures. The debris materials at the toe were typically acting as a buttress and provided additional resistance force against potential sliding. The removal of this buttress could have reactivated the slope instability in this already weakened area.

- **High pore water pressure**: The presence of low permeability lacustrine and tuffaceous deposits and clay-altered faults results in groundwater entrapment within the Carlin Formation. Pit dewatering was not
successful in the Carlin Formation. High pore water pressure likely existed behind the Nine-Points slope, which could have reduced the effective strength of the materials in the Carlin Formation.

- **Strength reduction of clay-rich tuff**: A geology anomaly - a clay-rich, low hydraulic conductivity tuff layer was found in the middle-lower Carlin Formation immediately beneath the Nine-Points failure zone. Ongoing deformation in the clay-rich low permeability tuffaceous material brought stress states onto the failure envelope. The materials near the toe, at the base of the excavation, reached their peak shear strength early, and then approached their residual strength when further deformation occurred. The strength reduction zone propagated further into the slope while progressive deformation continued. The critical slip surfaces of all the slope instabilities to date went through this clay-rich tuff layer and were not along the basal reduced clay layer, as was originally assumed. The December 2009 brittle style slope failure was likely caused by weakening of the clay-rich tuff toe material.

6  **Slope remediation**

6.1  **Remedial slope stability**

The December 2009 slope failure formed a 300 to 350 ft high, near-vertical scarp along the Nine-Points crest. The post-failure slope was marginally stable except for the upper scarp, which appeared to be too high and too steep. The slope stability analysis results suggested that the upper scarp be removed and the upper slope be laid back to an overall slope angle of 12 degrees.

Further slope stability analyses were conducted for the proposed ultimate pit slope in the Nine-Points area. The results of these analyses indicated that slope instability would likely occur along the weak disturbed clay-rich tuff layer due to the removal of the current buttress (debris material) in accordance with the proposed final pit plan. Slope stabilization measures including toe buttressing and/or slope depressurization should be incorporated into the ultimate pit mining plan.

6.2  **Slope stabilization measures**

Slope stability analyses were performed to assess the Factors of Safety against various buttressing options and groundwater pressures for the ultimate pit configuration in the Nine-Points area. The base case groundwater conditions were updated based on the January 2011 piezometer monitoring results. The upper perched water table in the Carlin Formation gradually dropped to an elevation of approximately 5050 ft in early 2011.

The modelling results indicated that the buttress crest should not be lower than El. 4980 ft as shown on Figure 9. Given the current groundwater conditions, the upper slope of the ultimate pit will only be marginally stable with a computed FOS of 1.1. The FOS for the remedial slope could increase to 1.3 if the groundwater table in the Carlin Formation drops from the current elevation of 5050 ft to 5000 ft.

6.3  **Implementation of slope remediation**

Newmont started upper slope remediation work in February 2010 following preliminary recommendations provided by various geotechnical consultants. The over-steepened scarp was removed and the upper slope was flattened to an overall slope angle of 12 degrees.

Newmont also constructed a three-bench rockfill buttress along the toe of the Nine-Points Carlin Formation in early 2011. The base of the buttress was constructed on the siltstone bedrock along the entire Nine-Points failure zone. The buttress was 120 ft high (between El. 4860 and 4980 ft) and approximately 1500 ft long. The buttress was constructed with well-drained clean waste rock in order to provide sufficient weight and allow free drainage at the front of the slope. An overview of the January 2011 Nine-Points slope is shown on Figure 10. It shows that the upper slope has performed well and the buttress design is appropriate. It should be noted that the final lift of the buttress was being placed at the time the photograph in Figure 10 was taken.
The slope remediation work was completed in early 2011 and the Gold Quarry pit operation fully resumed in April 2011.
7 Conclusions

Large-scale slope instabilities at the Gold Quarry Mine were triggered by the unloading of rigid toe, which resulted in strain deformation and progressive weakening of the brittle clayey materials in a geologically anomalous zone. A better understanding of the slope deformation patterns and failure mechanisms allowed for an appropriate remediation plan to be implemented. The major findings from this geotechnical assessment are summarized below:

- The Carlin Formation in the Nine-Points area consists of a very complex, inter-bedded, poorly consolidated sequence of sands, silts, tuffs, and altered tuffs. Much of the fine-grained material was tuffaceous, having been deposited as ash falls that subsequently underwent argillic alteration to clay-rich strata. These materials were weak and typically weaken after exposure and/or under saturated conditions.

- Geotechnical characterization unveiled a localized clay-rich, low hydraulic conductivity tuff layer in the middle lower Carlin Formation immediately beneath the Nine-Points failure zone. The clay content of this tuff layer decreases towards the north, east, and south outside of the failure zone.

- The mid-2008 and April 2009 slope failures were triggered by removal of rigid bedrock and buttressing debris material at the toe of the Carlin Formation. The December 2009 brittle style slope failure was caused by a combination of weakening of the toe material (clay-rich tuff) and an over-steepened upper scarp. Critical slip surfaces of all the slope instabilities extended through the clay-rich tuff layer and did not extend deeper to the Basal Reduced Clay layer as initially postulated.

- Hydrologic drilling conducted as part of this investigation has identified much higher conductivity within the lower portion of the Carlin Formation in the Nine-Points area. One well constructed in 2010 has been producing over 100 gallons per minute for over six months. No previous wells constructed within the Carlin Formation near the pit produced more than 50 gallons per minute, and most were in the 10 to 20 gallons per minute range. It is probable that this high conductivity zone contributed to the April and December 2009 instabilities.

- The 12-degree remedial upper slope is currently stable, but potential slope instability is expected if the existing toe buttress is removed for the final pit development. The construction of a 120 ft high, 1500 ft long rockfill buttress along the bedrock contact appears to be an effective stabilization measure to prevent further instability of the Nine-Points slope. An extensive slope depressurization and dewatering program in the Carlin Formation will be beneficial for slope stabilization.

Geotechnical information for large-scale open pit mines is often insufficient. Ongoing geotechnical/hydrogeological data collection and geological model refinement should be routinely implemented during pit development, especially in the areas where slope instability has occurred. It is prudent to identify the failure mechanisms of the existing slopes. Slope stability back-analyses using conventional limit equilibrium and finite element modelling techniques provide reasonable solutions for large-scale slope instabilities in weak rock mass. A rigorous slope deformation monitoring program should be implemented throughout the life of pit operations.

8 Acknowledgements

The Gold Quarry open pit slope instability and remediation study was a collective work completed by a group of geotechnical consultants and Newmont’s technical team over a period of over 15 months. The authors would like to thank Mr. Jack Henris of Newmont, Mr. Ross Barkley of Call & Nicholas Inc., Mr. Richard Dawson of Norwest Corp., and Itasca for providing background information, historic data, and valuable insights to this study. This Paper only presents a small portion of the entire case study and it is expected that further detailed aspects of this project will be presented by other specialists in the future. The permission of Newmont to publish the details of the Gold Quarry open pit mine analysis is gratefully acknowledged.
9 References