ABSTRACT: Design and construction of roads and foundations on steeply dipping slopes and weak bedrock creates challenging rockslope engineering problems because of the kinematic potential for failure into the rock excavations. Nevada Power Company (NPC) is constructing the 83km (50mile) Harry Allen Mead (HAMD) 500 kV power transmission line, their largest. Because ideal terrain was occupied by towers owned by another utility, tower pad HAMD 24/1 and access road had to be constructed on the dip slopes of the adjoining ridge. The slopes created design problems because they dip unfavorably into the excavation creating potentially large planar failures and rockfall. During construction NPC encountered engineering and geological challenges, which necessitated design modifications: adversely dipping rock structure, very weak and fractured unstable rock, rockfall, backbreak from preshear blasting, and poor bonding of resin grout. To stabilize the backwall of the tower pad and road, over 1311m (4300 ft) of rockbolts and dowels were installed.

1 INTRODUCTION

1.1 Project

The Nevada Power Company (NPC) is constructing the Harry Allen Mead (HAMD) 500 kV power transmission line, NPC’s largest. Los Angeles Department of Water and Power (LADWP) owns a high power transmission line that traverses prime terrain in a pass formed from the shoulder of Lava Butte and the steeply dipping limestone slope of the Horse Spring Formation (Figs. 1-2). The new power line parallels the line operated by LADWP. However, because the present power line occupies prime terrain in the pass tower pad HAMD 24/1 and access road had to be constructed on the dip slope of the adjoining ridge. To achieve the proper tower pad and access road size, vertical cut slopes were proposed. These dip slopes created rockslope design problems in that the slopes dip unfavorably into the excavation of the proposed structures creating potentially large planar failures and rockfall. This section of the project was unique because no other significant cut slopes were present along the alignment.

1.2 Location

The HAMD 24/1 tower pad and access road are located approximately 33km (20 miles) northeast of downtown Las Vegas, NV. The transmission line traverses desert terrain southeast from the HAMD power station adjacent to Interstate 15 for approximately 83 km (50 miles) to the Las Vegas area. Most of the land over which the power lines traverse is owned and managed by the Bureau of Land Management (BLM).
1.3 Project Challenges

During construction of the HAMD 24/1 tower pad and access road the following engineering and geological challenges were encountered that necessitated design modifications: adversely dipping rock structure, very weak and fractured unstable rock, rockfall, backbreak from preshear blasting, and poor bonding of resin grout.

Construction of the tower pad and access road dictated a 9.8m (32ft) vertical backwall, which exacerbated the stability of the rock cut. Because of the proximity of the present power transmission line, and the concern for rockfall damage to the lattice structures, NPC’s engineers required their consultant design the backwall with a factor of safety of 3.0. This increased the cost of the project at least three-fold. In addition, because exploratory drilling access was difficult and expensive, NPC engineers elected to forgo initial exploratory drilling at the pad site and go with one exploration borehole drilled near the LADWP lattice tower.

To control the stability of the backwall, about 1024m (3360ft) of sub-vertical dowels and rockbolts were installed behind the proposed cutlines followed by preshear blasting. Initially NPC’s consultant recommended installation of about 692m (2270ft) of rock bolts in the face of the rockslope above the access road and pad. However, during excavation to the first bench the contractor encountered very weak and fractured rock and a 0.6m (2ft) stratum of gypsum below the cut line. Once the access road and first tier of the pad was excavated, three exploratory boreholes were completed to further quantify the rock mass characteristics. In addition, preshear blasting to control backbreak beyond the cutline performed poorly.

The change in geologic conditions necessitated design modification of the backwall. Based on the changed geologic conditions the backwall design was modified to a reinforced shotcrete wall with 457m (1500ft) dowels and 84m (275ft) of drains and weep holes.

2 ENGINEERING GEOLOGIC CONDITIONS

2.1 Rainbow Gardens Geologic Preserve

The Rainbow Gardens Geologic Preserve is a unique geologic feature just northeast of downtown Las Vegas. Tertiary rock members of the Horse Spring Formation form a mixture of very weak to strong limestones, sandstones, shales, gypsum deposits and conglomerates (Castor, et. El, 2000). Colors range from bright white in the gypsum beds to ocher yellows and deep reds in the sandstones and shales. Structures of these units strike en echelon to the northeast and dip steeply to the southeast (Figs. 1-3). Saddles, valleys and steps form in the weak rock. Before the Rainbow Gardens Geologic Preserve was designated as a Preserve, gypsum mining was popular in the valleys because of the easy access to the deposits. Presently, tourists pay top dollars for ATV excursions along the 4x4 trails in the Preserve.

Lava Butte is a young Tertiary volcanic cone, which forms the northeast shoulder of the saddle through which the power transmission lines traverse (Fig. 1).

2.2 Regional Geology

Castor and others (2000) describe the regional geology and structure in the project area. Bedrock units in general strike to the northeast following the topographic ridges and dip to the southeast. Rock units, which crop out in the site vicinity, are primarily Tertiary members of the Horse Spring Formation and include the Rainbow Gardens Member, the Thumb Member and a gypsum-rich sequence of very weak rock. There are no apparent faults on site.

2.3 Engineering Geology & Geomechanics

The tower pads and access road rest primarily within the limestones of the Rainbow Gardens Member. Table 1 displays geometric mean values of the rock mass properties exhibited by the limestones.

<table>
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<th>Depth (m)</th>
<th>Strength (MPa)</th>
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<th>RQD</th>
<th>RMR</th>
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<td>42</td>
<td>6</td>
<td>12-14</td>
<td>57</td>
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1 JRC: Joint Roughness Coefficient (Barton, 1976)
2 RQD: Rock Quality Designation (Deere, 1973; Planstrom, 1975)
3 RMR: Rock Mass Rating (Bieniawski, 1989)
At the surface, the limestones appeared strong (R4) and competent. However, just below the surface, core logs indicated the limestones were highly to moderately fractured and interspersed with some mud zones. In addition, in the upper slope beneath the competent limestone we encountered a very weak gypsum zone. The fractured rock and weak gypsum zones presented major stability and construction problems.

2.4 Structure

Beds within the onsite rock units strike to the northeast and dip southeast at approximately 45 degrees towards the saddle between Lava Butte and the project site (Figs. 2-3).

To facilitate assessment of the kinematic stability of the slope and to aid in our design, we plotted the orientations of the discontinuities on stereonets using the computer programs Dips Version 5.0 by Rocscience and ROCKPACK III by C. F. Watts (2001). Stereonets provide a two-dimensional representation of the three-dimensional discontinuity data. We plotted both poles and dip vectors. The poles tend to accentuate the orientation of steeply dipping discontinuities while the dip vectors lend themselves to performing Markland analyses. Where the stability of a rock cut is controlled by the structure of the rock mass, a Markland analysis is a well documented and widely accepted design tool even though the analysis does not provide a safety factor relating shear stresses to shear strength (Hoek and Bray, 1981; FHWA, 1998). The information required to perform the analysis are the design slope dip and dip direction, the orientation of the discontinuities within the rock mass, and the friction angle of the lithologies represented in the rock cut.

The Markland analysis does not consider a cohesion intercept when modeling the strength of discontinuities. This method also assumes that the discontinuities are continuous and through going with no “bridging” within the discontinuity. The effect of “bridging” would allow a tensional component (or cohesion intercept) of discontinuity strength. The factor of safety of the slope is estimated by dividing the tangent of the friction angle by the tangent of the dip of the discontinuity or the plunge of the line of intersection of two discontinuities. Therefore, when the factor of safety is less than 1.0, the factor of safety is greater than 1.0. In either case, the factor of safety is estimated by dividing the tangent of the friction angle by the tangent of the dip of the discontinuity or the plunge of the line of intersection of two discontinuities. Therefore, when the dip of a discontinuity or the plunge of the line of intersection is greater than friction angle, the factor of safety is less than 1.0. When the dip of a discontinuity or the plunge of the line of intersection is less than the friction angle, the factor of safety is greater than 1.0. In either case, the dip or plunge has to be less than the dip of the slope face, or the structure will not daylight the slope.

We assumed a friction angle of 34 degrees based on the detailed geomechanical information that we collected in the field, experience with similar rock types, and guidance from the Rock Slopes Reference Manual (FHWA, 1998). Direct shear tests were completed following ASTM guidelines on a saw cut rock sample. The direct shear test provided a basic friction angle for the rock mass. Because the sample was saw cut, it does not account for asperities or surface roughness of the discontinuities. The results of the direct shear test suggested a friction angle for the limestone of approximately 35 degrees.

Figure 3 displays the kinematic relationship between the discontinuities. Note the large population of dip vectors that crop out in the gray crescent moon of the stereonet. This suggests that the major population of discontinuities daylight the slope at angles greater than the friction angle and less than the slope angle creating kinematically unstable conditions. In addition, there is a clear potential for toppling as displayed by the population of dip vectors grouped in the gray triangular area. A major release joint is also evident in the northwest quadrant of the stereonet. Furthermore, it is clear that sidehill cuts excavated into the dipslope that exhibit a backwall dipping greater than the dipslope are unstable in that the blocks will slide out of the cut face. The adverse structure and unstable blocks created both construction and design problems. During excavation of the access road and tower pad, we observed numerous small translational block slides into the excavation.
3 STABILIZATION & ROCKFALL CONTROL

3.1 General
At the conclusion of the geologic mapping and the kinematic analysis of the rock slopes, it was apparent that the slope would require some type of reinforcement prior to blasting and for the final slopes. The proximity of the present power transmission line, and the concern for rockfall damage to the lattice structures required a design factor of safety of 3.0 on the backwall of the tower pad and 1.5 along the access road. In addition, rockfall developed during construction had to be controlled with some type of control structure.

3.2 Slope Stability Block Analysis
We followed up our kinematic evaluation using limit equilibrium analyses to estimate safety factors for the block geometries that may fail out of slope. We employed the computer programs RocPlane by Rocscience® to complete a planar-type geometry analysis. Peak ground acceleration for this region is 0.10g based on United States Geologic Survey mapping [http://eqhazmaps.usgs.gov](http://eqhazmaps.usgs.gov). We assumed one half of the peak ground acceleration for our analyses. We assumed no groundwater was present within the fractures.

The existing slope is approximately 40-50 degrees. During our field reconnaissance, we observed translational failures in the bedding that occurred in slopes ranging from 40-45 degrees. To achieve the proper sized pad, a 9.8m (32ft) vertical cut backslope was required (Fig. 4). Initially we assumed the cut slope of Tower 24/1 pad would be composed of strong competent limestone. Moreover, the backslope would mirror the existing slope (dipslope) at about 45 degrees (1H: 1V) to the top of the ridge (Fig. 4). We assumed tension fractures would be present in the limestone (release joints as observed in the field and displayed on the stereonet Fig. 3) and that bedding would daylight at the base of the constructed slope. Based on these assumptions the results of our analyses for the proposed vertical cut slope design without reinforcement suggested a safety factor of 0.85 or unstable.

To increase the factors of safety, we applied active support anchors, rock bolts, to the backslope limestone beds and into the cut slope face of the tower pad. The results of our modeling analysis suggested we could achieve a safety factor greater than 3.0 with one row of anchors designed to 142kN/m (32 kips/ft) in the backslope and two rows of anchors designed to 80kN/m (18 kips/ft) and 71kN/m (16 kips/ft) in the cut slope face. In addition, the back row anchors would reduce the potential for block sliding during blasting and ripping.

3.3 Rockfall Hazard Mitigation & Catchment Design
Protecting the existing down slope lattice towers from rockfall was a major concern. Therefore, as part of our design, we simulated rockfall that may occur on the slope below Tower 24/1 during construction using the computer program Colorado Rockfall Simulation Program (CRSP III) (Jones et al, 2000). Rocks on the slope averaged about 0.3m (1ft) in diameter and ranged upwards to about 0.6m (2ft) in diameter. We simulated rolling 500 of each diameter down the slope.

Based on the CRSP models for existing slope, less than 10 percent of the 0.3m (1ft) diameter rock blocks reached the base of the existing transmission tower. However, greater than 10 percent of the 0.6m (2ft) diameter rock blocks reached the base of the existing transmission tower.

To verify our model, in the field we rolled example tabular rock blocks of similar dimensions from above the proposed tower pad. Our observations demonstrated that the rocks stopped at least 15m (50 ft) from the tower base. Moreover, most of the rock blocks broke into smaller clasts before reaching the talus slope below the proposed tower pad.

Based on the CRSP simulation, it was clear that rockfall reaching the tower during construction of the access road and tower pad was a probability. To capture the rockfall, we designed a 1.5m (5ft) high soil/colluvium berm 15m (50ft) upslope and parallel between the proposed tower pad and the existing transmission tower. Based on the design and simulation, the berm would reduce the potential for rockfall reaching the existing transmission tower during construction. Again, the existing slope and the berm was modeled using CRSP and the results demonstrated that the berm would indeed stop greater than 99 percent of the design rock blocks.

Figure 4. Proposed excavation cut for 24/1 tower pad.
4 CONSTRUCTION PROBLEMS

4.1 General

Once the conceptual designs were approved by NPC and the BLM, a specific Request for Proposals (RFP) was sent out to prospective contractors. Prior to submittal of bids from the contractors, a pre-bidding conference was conducted to describe the geologic conditions and construction problems associated with the specific location. The contract was awarded in the summer of 2005 based on the lowest bid from a firm that typically constructs high power transmission lines. Construction of the rockfall barrier, access roads and tower pads began in January 2006.

4.2 Rockfall Protection

A 1.5m (5ft) chevron berm comprised of native soil and rock was constructed 30m (100ft) upslope of the tower. The apex of the chevron shaped berm pointed up hill to deflect the rock as it rolled from construction of the access road and pad. Figure 5 displays a photo of the rockfall protection berm. Throughout construction the berm worked as designed and rockfall that originated from the construction site was contained.

4.3 Pioneered Construction Road

During excavation and construction of the access road and tower pad 24/1, the contractor elected to pioneer a construction road above the access road and tower pad. The reasoning behind this was to facilitate excavation and installation of the first row of stabilization rockbolts. The rock was excavated with a hoe-ram. Because of the geologic structure, the rock strata resembled a deck of cards leaning against a wall. With the hoe-ram, the contractor was able to split and rip the rock from the face. However, because of the structure, this also created large slab failures and unstable overhanging rock. To reduce the overhanging and toppling problems, the rock was ripped to where it daylighted the upper natural bench. This created an unplanned overage of excavated rock. Figure 6 is a photo taken during construction of the upper road.

4.4 Rockbolts

After review of the kinematic characteristics and stability of the rock slope, it was established that tensioned anchors such as rockbolts would be required to reinforce the slope. Because of the blocks that dipped out of slope, the design called for installation of a row of closely spaced rock bolts behind the cut line of the backwall of the access road and tower pad 24/1 prior to excavation. The purpose was to stabilize the backwall during blasting and ripping. In addition, it was part of the design to achieve a factor of safety of 1.5 for the access road and 3.0 for the tower pad. The rockbolts spacing ranged from 0.5m to 1.5m (1.5ft to 5ft) and 2m (7ft) behind the cutline. In total, approximately 1024m (3360ft) of rock bolts were installed behind the backwall (Fig 7).

Wyllie and Mah (2004) provide an excellent description of tensioned anchors to include rockbolts. Rockbolts comprise two parts, a bond length and an unbonded length. In the bond length, the bolt is bonded by resin, grout or a mechanical device to the surrounding rock. In the unbonded zone, the bolt has no bond and is free to strain as tension is applied. The bond zone of the bolt is located below the failure plane in good rock so that when tension is applied to the bolt it increases the factor of safety.

Techniques of securing the distal end of the bolt include resin, cement grout and mechanical devices. On this project the contractor elected to use resin anchors. To install the cartridges, the contractor simply slides a sufficient number of cartridges into the drill hole to fill the annular space around the bolt. Tolerances between the bolt size and the bore-
hole are critical to the size of cartridge selected for a proper bond between the rock and bolt when the rockbolt is spun into the hole to mix the resin. During installation the rock bolt is spun through the cartridges to mix the resin and achieve a bond between the rock and bolt.

On the project, the first two rock bolts failed during proof testing by the contractor (Fig. 7). Failure of the rockbolt bond may be attributed the following potential problems:

1. Resin not setting properly
2. Resin cartridges too small for the annular space
3. Not installing the rockbolt properly per resin specifications
4. Drill hole widening in the weaker rock material because of drill bit “wobble”

To find a solution, the contractor first consulted with the manufacture to insure that the right cartridges of resin were being employed. Next tests were completed on the resin to establish mixing and setup times. Two test anchors were installed using different methods. After trial and error it was established that the proper cartridges were being used in the hole. The problem became clearly evident that the annular space between the rockbolt and the rock was irregular and wider in some areas. The irregular hole was created from chatter and wobble of the drill stem in the weak fractured limestone. The increase in annular space around the steel did not allow for good contact between the resin, rock and rockbolt.

To alleviate the problem, the contractor placed additional cartridges in the hole and packed the cartridges in place. When the steel was spun, they achieved a good bond between the resin, steel and rock. Once this issue was solved, the rockbolts passed the performance and proof tests at design loads.

4.5 Blasting Overbreak

Wyllie and Mah (2004) have shown that slope instability is often related to blast damage of the rock-slope behind the face. The damage is often related to overbreak.

Overbreak is excessive rock breakage beyond the excavation limits. Overbreak includes backbreak, which is breakage behind the last row of shot holes and endbreak that occurs at the end of the shot line.

To achieve a clean stable backwall of the access road and tower pad, presplit blasting was done 2m (7ft) below the first bolt line. During drilling of the presplit holes, the contractor observed drill activity (drill speed, drops, etc) that represented varying geologic conditions at depth. Presplit blasting worked as designed along the backwall of the access road because the limestone was competent and of good quality. However, presplit blasting did very poorly in the backwall of the tower pad (Fig. 8). In this case the backbreak originated because of the poor quality rock exacerbated by unfavorable structural geology. The backbreak may have been reduced if the bolt line was moved downslope closer to the blast cutline.
4.6 Backwall Tower Pad 24/1 Redesign

In mid-April 2006, the first row of rock bolts was completed above the cutline for tower pad 24/1 and the access road.

Excavation of the pad was stopped at a depth of approximately 3.5m (12 ft), approximately 6m (20 ft) short of the planned depth because of unstable conditions, rockfall and potential hazards to the workers. The lithology uncovered during excavation was composed of approximately 1.5m (5.5 ft) of moderately fractured, moderately strong limestone underlain by approximately 2m (7 ft) of highly to extremely fractured, moderately strong limestone with some weak gypsum and soil layers.

Some of the loose blocks were stabilized with spot rock dowels. However, it was concluded that the original rockslope design had to be modified to account for the changing conditions. In addition, NPC elected to reduce the design factor of safety to 1.5 because of the cost to construct the wall.

Before modifying the design, three boreholes were completed to further characterize the subsurface geology below the pad. Based on the weak fractured nature of the limestone and associated rock in the face, the following mitigation designs were selected.

1. Cut the back slope at the tower pad as a vertical slope in the limestone.
2. Install three rows of rock dowels (in three lifts) in the sidewall and back wall cut slope on a 1.8m x 1.8m (6 x 6 ft) pattern for a Factor of Safety greater than 1.5.
3. Drape the cut slope face and rock dowels in lifts with 16mm x 16mm (4 x 4 in) wire mesh.
4. Install 3m (10 ft) horizontal drains on 6m (20 ft) spacing just below the third row of rock dowels to reduce the potential for hydrostatic pressure to build behind the shotcrete face.
5. Apply shotcrete to the cut slope face at a thickness of 76mm (3 in).
6. Install 111kN (25 kip) spot rock bolts as needed in cut slope face along the lower section of the pad and access road.

Figure 9 displays the contractor applying shotcrete on the backwall of Tower Pad 24/1. 263 linear meters (864 ft) of dowels were installed in the backwall of tower pad 24/1 and access road. Horizontal drains were reported to work as designed after a severe cloud burst. Figures 10 and 11 display the completed Tower 24/1 pad with foundations and the monopole structures installed.

5 LESSONS LEARNED

The issues that occurred during the design and construction provided a number of lessons learned.

Exploratory drilling prior to design and construction is an important aspect to providing the most appropriate and cost-effective design options. As discussed, exploratory drilling was not completed as part of the investigation because of difficult access and costs. A boring completed at the 24/1 pad location would have provided detailed information regarding the fractured nature of the subsurface lithology.

The poor quality rock mass and the adverse dipping structure exacerbated the back break along the 24/1 pad and access road. In addition, the rock with alternating zones of competent and incompetent rock had a negative impact on the blast performance.

There are good and bad points to using resin anchors and cement anchors. Some of the good points for resin anchors are:

- Quick setting time
• One visit to install anchor and fill hole
• Cleaner operation
• Requires less bond zone

As observed during the construction, resin anchors have some bad points associated with resin:

• Critical hole size for cartridges

Using a cement-type anchor may have alleviated some of the issues encountered with resin anchors. The drill bit “wobble” and hole widening in the weaker zones would have filled in the bond zone with the use of cement grout. Some of the good points for grout anchors are:

• Works well in poor quality rock
• Easier to prestress rock anchors

However, cement-type anchors also have some bad points associated with the grout:

• Potentially messy operation
• Slow setting
• Two grouting periods required for rock bolts (bond zone and unbonded zone)

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