Hoonah Bluffs Stabilization

Site Assessment

June 2017

Prepared for the City of Hoonah

300 Front Street

Hoonah, AK 99829

Prepared By PND Engineers Inc.

9360 Glacier Hwy Suite 100

Juneau, AK 99801

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1. INTRODUCTION

On April 26, 2017 three personnel from PND Engineers, Inc. (PND) visited the Hoonah Bluffs site. The Hoonah Bluff is a blast-excavated rock cut constructed for a new road alignment which intends to straighten several horizontal curves in Cannery Road. PND was requested by the City of Hoonah to observe and assess a large slope failure and potential future failures along the constructed rock face. Two engineers and a geologist spent a day assessing the cut slope for possible failure causes, the probability of future failures, the quality of construction methods implemented on the rock cut, and developing recommendations for future mitigation and stabilization of the slope. The limited time spent at the site precluded a detailed investigation that would include discontinuity/joint mapping or coring of rock. Limited access prevented a closer visual investigation of the upper benches of the rock slope. The assessment was visually based, with the majority of the access limited to the elevation of the adjacent roadway.

This report begins with a geological description, both regionally and locally of the Hoonah Bluffs site. Section 3 discusses potential and realized failure mechanisms observed at the site. Section 3 also provides a photographic narrative of the site with detailed descriptions of the rock cut characteristics. Section 4 presents recommendations for possible mitigation measures concerning future development of instabilities.

On May 8, 2017 another slope failure occurred and one PND engineer subsequently visited the site on May 11, 2017 to assess the failure and provide guidance on immediate mitigation and rock fall protection measures. Causes of the failure are discussed within the context of the observations made on the initial site visit.

2. REGIONAL STRUCTURAL GEOLOGY

The structural geology in the vicinity of Hoonah is controlled by a regional fold belt of rock that encompasses the northeast corner of Chichagof Island spanning from north of Tenakee Inlet northeastward to Icy Strait and from the mouth of Freshwater Bay in Chatham Strait northwestward into the entrance of Port Frederick. The United States Geological Survey (USGS) performed detailed, structural geologic mapping in the Freshwater Bay area directly southeast of Hoonah (Loney and others, 1963). The published results from the mapping program provided reference to the local bedrock geology of this report.

The belt of rock is a composite of Paleozoic-age sedimentary rocks approximated at more than 24,000 feet thick and folded into a regional southeast-ward plunging syncline known as the Freshwater Bay syncline. The U-shaped structure plunges gently southeastward with an approximate average 45° dip of the fold limbs. Multiple tectonic events post folding imposed further deformation to the rock strata that include: east striking faulting; northwestward-striking faulting; and southwestward verging thrust faulting. The folded rock strata are locally intruded by several small, igneous bodies of rock that range in size from less than one square mile to four square miles in extent. The igneous intrusions have abrupt contact borders with the intruded sedimentary rocks; steeply dipping minor marginal thrust faulting occurs, directed away from the igneous bodies.

The deformational events imposed fractures within the rock strata, termed discontinuities, of various orientations and characteristics that directly affect the stability of disturbed rock cut excavations such as at the Hoonah Bluffs.

SITE GEOLOGY

Rock Types

Bedrock types typically exposed in the vicinity of Hoonah can include a composite of marine sedimentary rocks, notably argillite, greywacke, slate, conglomerate and breccia, and limestone.

Bedrock exposed at the Hoonah Bluffs rock cut slope includes two basic units: a basal unit of interbedded argillite and greywacke; and an overlying unit of massive limestone. The limestone encompasses the majority of the cut slope face at the Hoonah Bluffs and forms the core of a synclinal fold nose with an approximate horizontal amplitude width of 300 feet and an approximate vertical face height of 225 feet. The limestone presents the primary concern to the rock cut slope stability.

The argillite and greywacke unit is composed of medium to thin, rhythmically interbedded strata that represent the original depositional surfaces of the rock. The argillite is typically dark gray-colored, fine-grained, massive and structureless and considered a deposit of marine mud; the greywacke is typically light gray to green colored, clastic textured, with fine-grained, angular sand-size particles suspended within a matrix of clay and interpreted to represent cyclical clastic deposition into the marine mud. The rock units are slightly metamorphosed with calcareous alteration. The interbedded rock when disturbed preferentially breaks along the bedding planes into planar-shaped sheets. The rock strength of the argillite alone is weak while the greywacke is the more competent of the two. The greywacke occurs locally in massive beds and then presents a very strongly competent rock.

The limestone rock is light gray-colored, medium grained, thick bedded to massive and brittle. Slight weathering, mainly oxidation of iron-bearing minerals within the limestone, stains the limestone a yellowish brown color. The rock when disturbed typically breaks into block-shaped fragments.

Rock Structure

Multiple discontinuity types are present in the cut slope rock mass that occur either isolated or in distinct, parallel geometric sets. The folded original bedding surfaces of the argillite and greywacke display one prominent discontinuity type. Partially exposed argillite-greywacke fold noses were observed possessing axial hinges with shallow and southeastward dipping orientations.

One prominent joint set was observed cross-cutting the interbedded argillite-greywacke and limestone units with variable spacing but locally occurring in 2 to 3 feet widths. The fracture surfaces appeared tight, moderately rough and devoid of gouge infillings; little relative movement was apparent. The strike of the joint set coincides with the structural lineaments of the region and indicates the joint set likely represents axial planar cleavage formed from regional folding of the bedrock. The persistence of the jointing was difficult to discern but potentially could be very high.

Additional discontinuities observed intersecting the axial plane cleavage include: joint sets of variable orientations; a low-angled fault with moderate weathering existing along the fault surface; and complex, often random jointing with less predictable orientations that may have resulted from the recent rock blasting.

The discontinuities together form zones of weakness within the rock mass that facilitated blocky, wedge-type slope failure of the limestone rock. Large blocks of partially intact rock mass remain in the slope face along with abundant coarse scree deposits that mantle the slope face. The rock debris will remain on the slope face over time awaiting an event that initiates transport of the debris down slope.

No groundwater movement or seeps were observed during the time of this investigation.

3. SITE VISIT OBSERVATIONS

Figure 1 shows a panoramic view of the Hoonah Bluffs, while Figure 2 shows the bluffs divided into sections for characterization and discussion purposes. Section 1A, Figure 2 shows the initial slope failure created by blasting operations that initiated the April 26 site visit. Detailed descriptions of the failure slope and other nuisance failures are provided in following sections.

Figure 1. Panoramic View of the Hoonah Bluffs with Rock Slope Failure (viewers left)

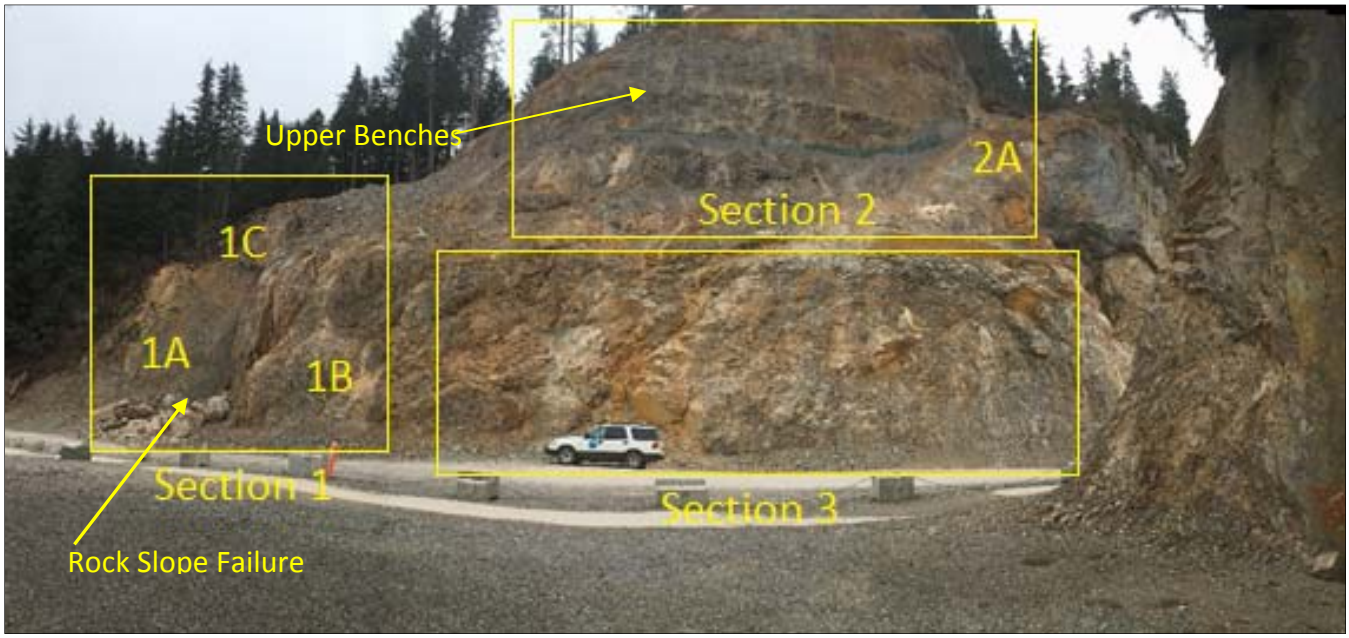

Figure 2. Hoonah Bluffs Separated into Sections

The primary causes for the current slope failures, nuisance failures (spalling, raveling, littering of small rock), and potential future failures are:

1. Non-implementation or incorrect implementation of controlled blasting methods with respect to the design neat line and poor blast design.
2. Alignment/cut design (along the north end, Section 1) that did not account for the rock geologic structure, in particular the natural strike and dip of discontinuities in the rock face
3. Inadequate construction control during construction.

CONTROLLED BLASTING AND POWDER FACTOR

Controlled blasting methods are used to greatly minimize overbreak/backbreak of the rock face beyond the design neat line of excavation. Overbreak is disturbed rock: rock that has been disturbed and fractured but remained in place after shot material has been excavated. Overbreak can be seen as tight fracture lines or fractures with considerable separation (heave) between rock blocks. Overbreak and/or excessive energy and high powder ratio (pounds of explosive per bank cubic yard), in addition to poor blast design, can also cause the heave/separation or disturbance of natural discontinuities and joints between rock blocks. The discontinuity may have been stable prior to blast disturbance, however uncontrolled blast energy moved or heaved the rock masses such that the discontinuity aperture had greatly increased after blasting. Separation of the rock masses at the discontinuity destabilizes the rock masses and reduces the friction or bonding infill that holds the rock masses in place. Figure 3 shows an example of overbreak, however evidence of overbreak can be seen over the entire surface of the project backwall/as-built neatline. Not evident in the photos presented, but observed in the field with binoculars, is the overbreak in the backwalls of the upper benches (Figure 2, Section 2).

The rock slope failure (Figure 2, Section 1A) was a wedge and toppling failure primarily due to uncontrolled blasting energy or a powder factor too large for the geological conditions. Excess energy separated the daylighted limestone rock mass wedge into approximately three-foot to five-foot thick vertical shives that rested on the interbedded argillite-greywacke layer beneath. The wedge trough dips at approximately 40° and 60°. The remaining rock mass that had not failed, but subsequently partially failed on May 8, 2017 (Figure 5, Figure 6) shows the development of vertical shives/slivers developed by overbreak and the heaved rock mass at the discontinuity interface (Figure 6).

Figure 3. Overbreak

Figure 6 also shows the smoothness of the moderately undulating failure surface (rock joint roughness profile/joint roughness coefficient [JRC]=14-16). The roughness of the discontinuity interface helps prevent the sliding of the rock block downward, as the roughness creates a mechanical interlock between the rock masses. However, as shown in Figure 6, if the rock mass is heaved and separated from the discontinuity, surface friction and mechanical interlock are greatly decreased.

Powder factor, hole spacing, explosive product, and blast timing, i.e. blast design appears to have been inappropriate for the geology at this site. Shallow or sliver cuts (slope failure location) are inherently difficult to control because of lack of confinement or mass between the free face and design neatline. Explosive energy distribution, energy confinement, and energy level for the geology may have been discovered with a pre-production test blast section as is common trade practice.

Figure 4 shows an example (Seward Highway) of controlled blasting using pre-split methods, good blast design, and best practice excavation methods. Pre-split methods are demonstrated on the backwall by parallel half boreholes that remain after the production blast. Low energy pre-split lines detonate just before the initiation of the production holes, "perforating" a line along the design backwall. The energy or compressive waves produced by the production blast are reflected and refracted at the cracked line of rock created during the pre-split initiation. The design backwall is exposed to minimal blast energy and the remaining rock is substantially undisturbed. This wall demonstrates best practice scaling of the backwall and crests and clean (of blast debris) benches. Stability of the backwall is demonstrated by tight (original) discontinuity apertures and no overbreak.

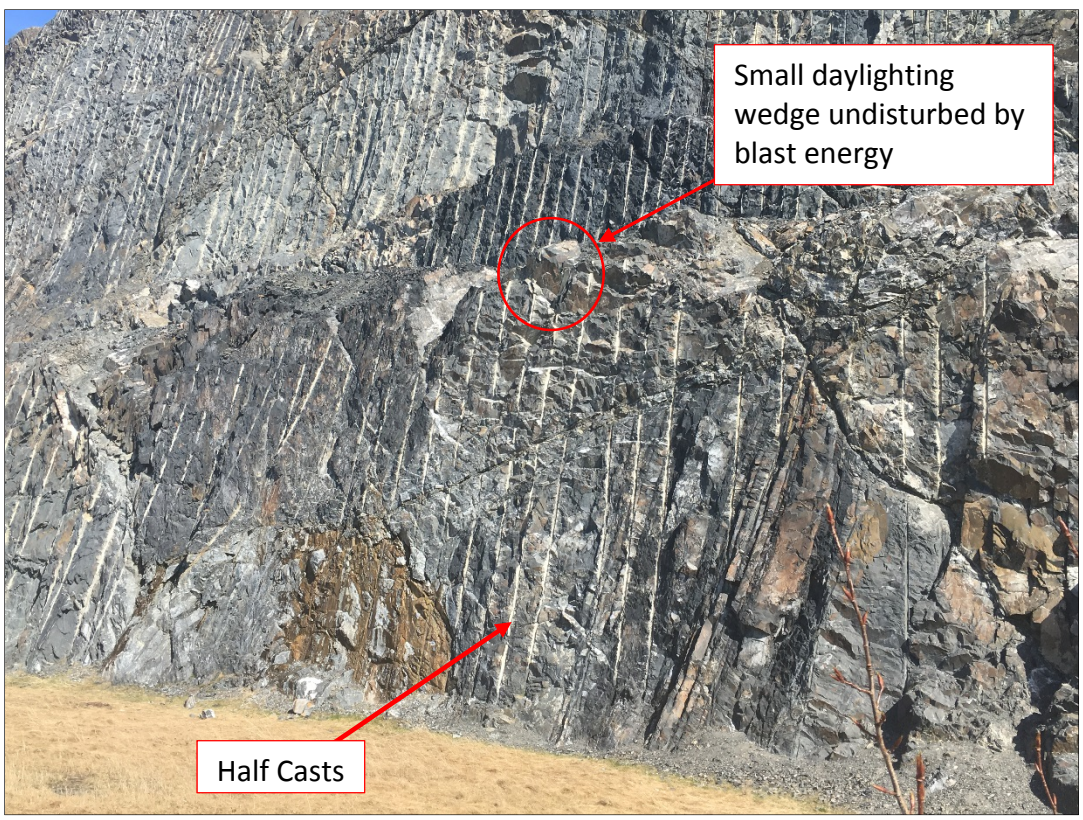

Figure 4. Example of Controlled Blasting (pre-split method) with Visible Half Casts

Heavy overbreak in the backwall, as seen on the Hoonah Bluffs backwall may result in nuisance failures in the form of continual raveling, spalling and littering of the rock face onto the road surface below. Disturbed rock due to overbreak can break free of the backwall by gravity forces and seismic loading, but the primary

mechanism will likely be frost jacking. Overbreak cracks fill with water and later freeze, and the force of expanding ice volume over years will break unstable rocks loose.

Figure 14 and Figure 15 show organic matter from clearing tasks stacked at the crests of Sections 1C and 2A. Below each crest are smooth, 40°-inclined (by visual observation) chutes which may contribute to a collapse or sloughing of the overburden above.

Figure 18 shows excessive blast debris and scree left on benches and at bench crests. Many of the bench sections appear to be of less than optimal width to provide an adequate rockfall catchment, and also dip downward toward the road further inhibiting rockfall protection effectiveness. Excessive debris may contribute to continued rock littering of the adjacent road below.

Rock Slope Condition Summary

Initial rock slope failure caused by:

- inadequate or no controlled blasting methods implemented
- improper blast design; insufficient control of explosive energy distribution, energy confinement, and energy level for the geology

Areas considered being at continued risk of significant failures:

- continued failure of remaining rock wedge at Section 1A and 1B is likely

Nuisance failure origins and considerations across the entire backwall:

- nuisance failures, resulting from inadequate or no controlled blasting methods, in the form of continual raveling, spalling and littering of the rock face onto the road surface below are likely
- no evidence of backwall scaling
- no scaling of crests
- excessive blast debris and/or scree left on benches
- benches of inadequate width and often sloping downwards towards the road creating an environment unable to catch falling rock from above
- excessive organic debris stacked or remaining at the crests of Sections 1C and 2A

Areas considered being at low risk for continued potential failures:

- the lower rock face in Figure 2, Section 3 appears undisturbed and will likely remain stable

ALIGNMENT AND ROCK FACE CUT DESIGN

The rock slope failure at Sections 1A and 1B may have been averted if more consideration had been given to the geologic structure along the proposed alignment. Had a discontinuity scan been conducted prior to design it would likely have shown the prominent dip and strike of the daylighting discontinuities of Section 1. A stereonet analysis would have shown that a cut in this region would have a high probability of failure – best practice blasting methods or not. Space for a less-invasive design alignment was available, but the failures and visible rock structure indicate that insufficient consideration was given to the site geology.

CONSTRUCTION CONTROL

Adequate design specifications, or the adherence to such, and construction control during construction may have averted many of the detrimental conditions demonstrated at the site. These conditions have created an environment of potential continual nuisance failures and instability over the long term. Construction control should have required backwall scaling, crest scaling and bench debris removal before the next bench was shot, while the backwall and bench were accessible. The quality of the backwall, including the amount of overbreak, should have been inspected by an experienced blaster or consultant. Excessive overbreak should have required a reassessment of the blast design and blast methods to prevent advancing with improper methods or unsuitable design. Design specifications should have required that a blast design be submitted to the project engineer or experienced blaster/consultant for review and acceptance before advancement of the next blast.

SECTION 1 DESCRIPTION

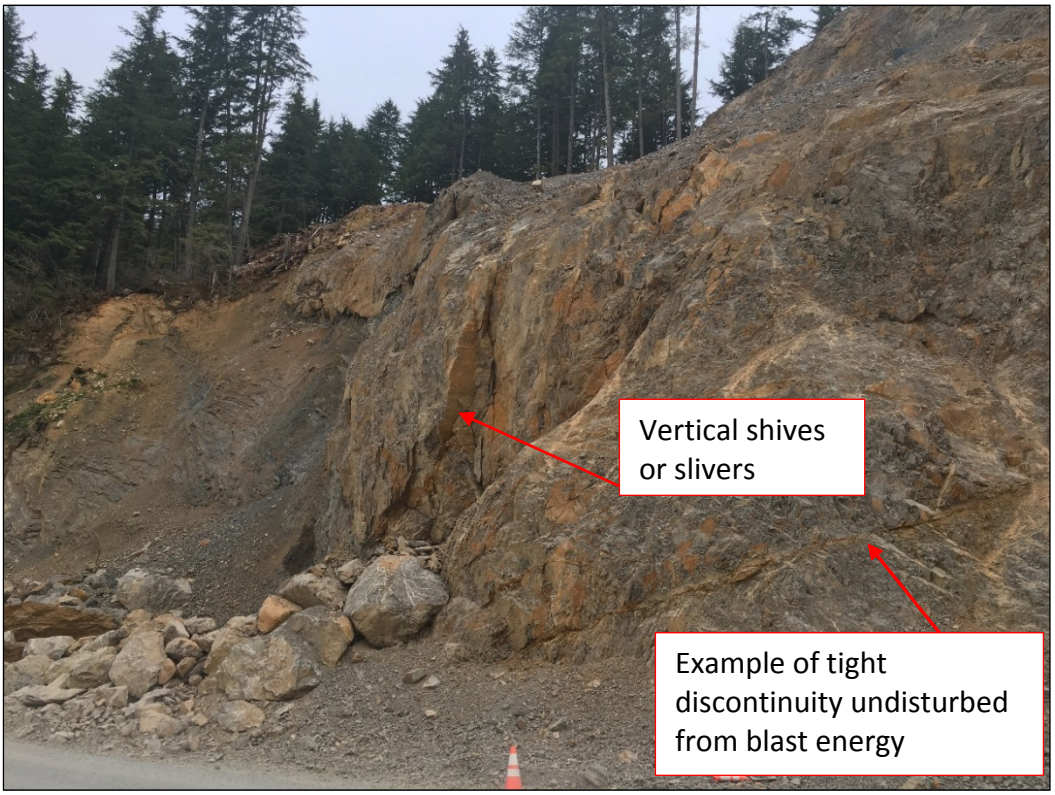

Figure 5. Overbreak

Figure 6. Failure Plane (featuring PND geologist Paul Dzwonowski)

Figure 7. Failure Plane Rock Joint Roughness

Figure 8. Dip of Failure Plane

Figure 9. Wedge Failure

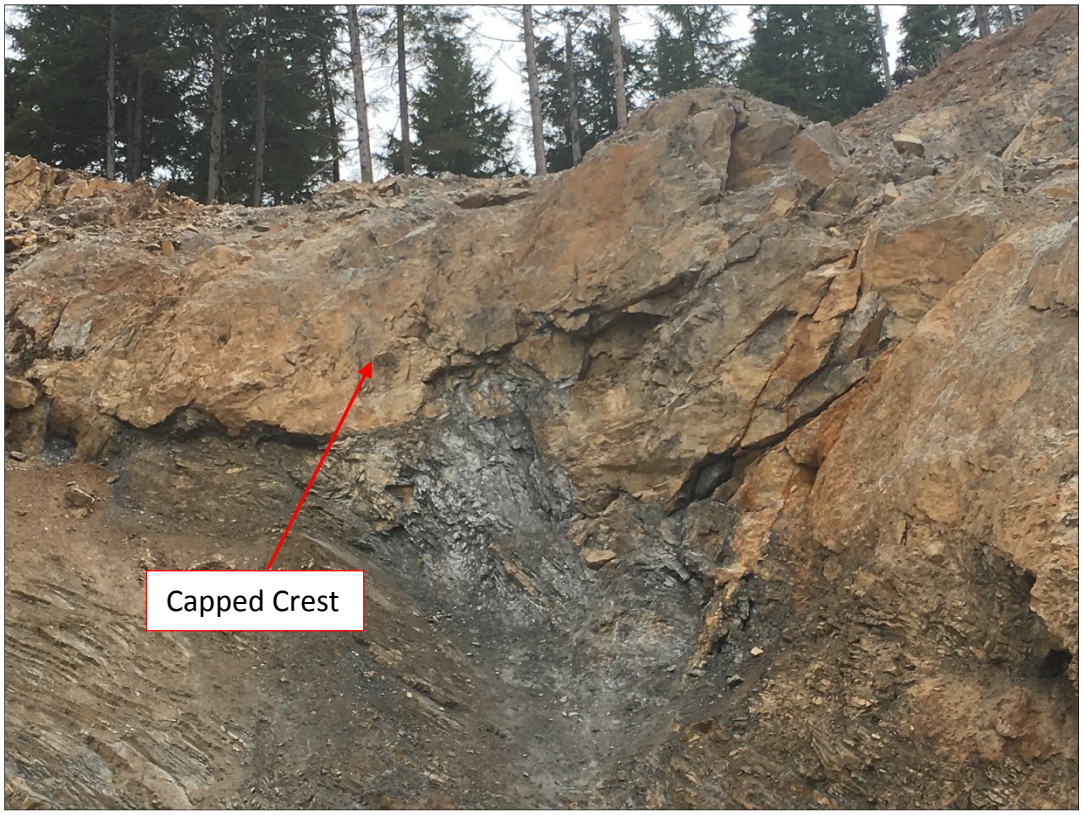

Figure 10. Limestone Cap-Crest

Figure 11. Slope Failure; May 8, 2017

Figure 12. Slope Failure with Potential Remaining Failure Wedge; May 8, 2017

Figure 13. Slope Failure with Potential Remaining Failure Wedge; May 8, 2017

Figure 14. Potential Debris Flow at Crest

SECTION 2 DESCRIPTION

Figure 15. Upper Benches and Undisturbed Lower Rock Face

Figure 16. Potential Debris Failure at Crest

SECTION 3 DESCRIPTION

Figure 17. Upper Benches and Blocky Mass Rock of Undisturbed (blast) Rock Face

Figure 18. Potential Nuisance Rock Littering from Blast Debris

4. RECOMMENDATIONS

Improper design and construction practices, and failed backwalls advanced to the completion of a rock cut project, can be expensive or not practically repaired or re-shot due to lack of access. However, the Hoonah Bluff backwall could be very responsive to fairly economical remediation.

It is assumed that the lower backwall (Figure 17) is stable and will require no additional remediation. However, excessive overbreak and bench debris on the upper benches (Figure 17) may contribute to future and continual raveling, spalling, and littering of rock onto the adjacent road. It is recommended that a rock catchment fence be installed along the length of Section 3. The location of the catchment fence, at the road elevation or along the top of the first bench (bench closest to road elevation) would require a rockfall analysis to determine rock trajectory. A catchment fence along the top of the first bench would have greater initial costs but reduce maintenance costs long term. It may also be more aesthetically desirable than a catchment fence at the road elevation.

The remaining potential rock slope wedge failure (Figure 19) could be mitigated by drilling and blasting from above (road access appears to be available to the first bench) or possibly using an excavator-mounted hydraulic rock breaker to bring down the remainder of the wedge from below. Rock bolting is not recommended as the rock mass is too disturbed for a high probability of success. Also, imparted energy from the rock anchor drilling operation may cause the wedge to fail and create a severe safety hazard for drilling personnel and equipment.

Figure 19. Remaining Wedge

A possible economical, albeit perhaps less aesthetic remediation would be to use the failed rock debris/boulders to construct a gravity revetment at the base of the wedge. A kinematic analysis would be required to assess this option to determine whether enough resistive force (weight) could be developed in the revetment to counter the driving force of the remaining wedge.

5. REFERENCES

Loney, R.A., Condon, W.H., and Dutro, Jr, J.T., 1963, *Geology of the Freshwater Bay Area Chichagof Island, Alaska, Geological Survey Bulletin*, 1108-C, 54p.