Utilization of Displacement Monitoring to Modify Rock Slope Designs during Construction

Interstate 90, Washington State

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Disclaimer

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ABSTRACT

Construction of a 5-mile-long, multiphase, interstate-widening project on Snoqualmie Pass along Lake Keechelus will require 800,000 cy of steep side-hill excavation incorporating 0.5H:1V to 0.25H:1V rock cuts up to 130 feet in height. The highly variable condition of the volcanic rock mass includes deeply weathered/altered and very closely fractured basalt; adversely oriented, clay-infilled flow boundaries; and very strong tuffs with extremely persistent, adversely oriented, planar discontinuities. The latter condition resulted in a catastrophic rock slide in 1957, when the original construction for the interstate unsuspectingly undercut such structure.

The rock slope design includes untensioned pattern reinforcement concurrent with excavation and real-time monitoring for slope deformation, utilizing 1) automated motorized total stations (AMTS) and survey prisms and 2) strain gages on 100-foot-long sacrificial anchors. Data is telemetered to web-based platforms, allowing real-time monitoring with remote access as well as automated messaging in the event displacement or anchor load thresholds are exceeded. Periodic ground-based laser scanning supplements the optical survey and strain gage data.

Experience from the first four construction seasons has demonstrated the need for refinements to the rock slope designs to account for geologic conditions encountered and to mitigate deformation behaviour of the slopes. These design changes have taken advantage of the complementary monitoring systems to integrate surface and subsurface point displacement measurements with areal displacements reported by laser scanning. Discontinuity orientations, spacing and persistence measurements have been obtained from the laser scans. These activities have confirmed structural controls for the rock slope displacements and provided greater confidence in the modified reinforcement designs.
INTRODUCTION

Interstate 90 (I-90) within Washington State is the most heavily used east-west crossing of the rugged Cascade Mountains. Currently, average daily traffic is about 27,000 vehicles a day, with weekend and holiday traffic climbing to as high as 58,000 vehicles a day. By 2030, traffic volumes on this facility are expected to grow to an average of 41,000 vehicles per day. I-90 across Snoqualmie Pass is a strategic freight corridor serving local, national, and international markets and shippers. It is estimated that on any given weekday twenty five percent of the traffic consists of trucks, and on annual basis they carry an estimated 35 million tons of freight with an estimated value of $500 billion (US).

The 5-mile-long highway corridor between Hyak, located just east of Snoqualmie Pass, and the Lake Keechelus Dam is currently a narrow four-lane highway facility located immediately adjacent to Lake Keechelus and bounded upslope by steep mountainous terrain (Figure 1). Construction of a $551 million (US) infrastructure improvement project to upgrade this highway facility, administered by the Washington State Department of Transportation (WSDOT), was started in spring 2010. The project will improve this section of I-90 by adding one additional lane in each direction accommodated by about 800,000 cy of large rock excavations along the uphill side, constructing new bridges, reducing sharp substandard curves, repairing deteriorated concrete pavement, replacement of the snow avalanche shed, installation of snow avalanche fences, and stabilizing unstable slope conditions along the project alignment. When completed in 2018, the highway capacity through this section of Snoqualmie Pass will be increased by 50 percent in each direction of travel, resulting in reduced congestion. In addition, winter closures due to snow avalanches and needed control work will be reduced, and the risk associated with slope instabilities will be lessened. The project has been divided into three construction
phases, 1A, 1B, and 1C, ordered from west to east. The major rock excavations occur within 1B and 1C, which are the sections located to the west and east sides of the existing snow shed, respectively (Figure 1).

The geologic challenges associated with the hill-slope excavations are considerable. During the initial construction of the interstate in the late 1950s, a catastrophic slope failure occurred within a large rock excavation at a location now referred to as Slide Curve (Figure 1). The cause of the rock slide was the undercutting of extremely persistent, adversely-dipping structure, which is also present within the existing cuts and natural slopes throughout the project limits.

Site geology

Bedrock in the western project area consists primarily of basalt flows, local pillow structure; silty sandstones and siltstones are locally interbedded. These rocks also exhibit low-grade metamorphism. The overall rock mass can be characterized as weak to moderately strong.

Bedrock in the eastern project area consists of volcanic tuffs, primarily pyroclastic flows with variable welding and extreme heterogeneity in their chemical and depositional characteristics. Frequency of flows, flow thickness and proximity of sequential eruptive centers affected the cooling rates of both individual flow units and successive deposits. The variability in the original composition of the tuff, degree of welding and in-situ porosity and permeability throughout the flow deposits is further complicated in the project area by subsequent low-grade metamorphic alteration. The tuffs are characterized as strong with unconfined compressive strengths typically between 5,000 and 15,000 psi, with a few test results exceeding 20,000 psi.

The range of discontinuities within the rock mass include faults, shear zones, joints, and lava and pyroclastic flow boundaries. Many of the encountered discontinuities have adverse orientations, high persistence, and/or weak infilling material. They pose significant design concerns due to their potential for day-lighting in the excavations and the fact that they have proven to be sufficiently unpredictable or ubiquitous in nature that they can exist anywhere within the cuts. Joints in the vicinity of Slide Curve exhibit persistence values in excess of 300 feet with adverse inclinations directed out of the proposed cut slopes.

Slope design

Rock slopes as high as 130 feet are required for the new alignment. Where the basalt rock mass is of lower quality and the natural slopes above the cut are favorable, the cut slopes were designed with inclinations ranging from 1H:1V(45°) to ½ H:1V (63°). However, throughout most of the project, traditional steep ¼ H:1V (76°) slopes were employed with the integration of slope reinforcement and mechanical methods for rockfall control. Slope stabilization design measures include untensioned rock dowels, tensioned rock bolts, PVC-cased drain holes, cable net drapery, reinforced shotcrete, and scaling.

Rock conditions in this volcanic terrain are highly variable and defied accurate characterization, irrespective of drilling and mapping intensity. Provisions were made to make design changes to rock slopes during excavation as actual rock conditions were encountered. Site geotechnical engineering during construction coupled with predictive slope displacement monitoring were included in the project to recognize and mitigate slope instability in a timely manner, minimize traffic interruptions and to provide for worker and public safety.
SLOPE MONITORING

An integrated set of displacement monitoring methods were employed:

- Point measurement of surface displacements utilized conventional surveying methods consisting of targets and a robotic total station.
- Point measurement of subsurface displacements utilized vibrating wire strain gages attached to passive, fully-grouted steel bars.
- Terrestrial LiDAR to interpolate between surface and subsurface point measurements.

The overall objectives of these systems were to retrieve accurate displacement measurements in near real-time and to be able to collect, transmit and receive the data via internet connection. A secondary consideration was the desire to set threshold levels of displacement that would initiate warning messages to designated monitoring personnel responsible for slope performance.

Automated Motorized Total Station (AMTS) / Prism System

Permanently mounted, optical glass prisms (generic term for targets) were deployed at an approximate horizontal spacing of 75 to 100 feet and an approximate vertical spacing of 20 to 40 feet dependent on local structural geology. The initial row was placed on bedrock outcrops above the top-of-cut with subsequent rows on alternate excavation lifts. Prisms were offset between rows.

Instrument towers were situated along the eastbound shoulder of I-90 such that the maximum sight distance to any prism was approximately 300 feet. Multiple AMTS instruments were required to operate at any given time to provide full-face displacement monitoring. It was necessary to mount the total station instruments on towers some 15 feet above roadway elevation to provide line-of-sight over truck traffic and to minimize interference from road spray. Monitoring continued on each rock cut until several months after excavation to final grade but was not required when the area was under snow cover during winter construction shutdown.

The x, y and z coordinates for each prism were measured to a contractually specified accuracy of +/- 0.2 inches in the x, y and z directions. Polling frequency ranged from 15 minutes for cuts undergoing active blasting and excavation to as much as 60 minutes after the cuts were completed.

A Leica AMTS with IRIS control program was configured to run on the Campbell Scientific Inc, CR800/1000 data logger as a gateway platform to control and store data from the AMTS. It used PakBus protocol to send data via spread-spectrum radio so that a common ADAS Base Station (also a CR1000) may be used. The geodetic data was to be retrieved by CSI LoggerNet automatic polling of the Base Station and FTP forwarded along with other data tables to an ARGUS or ATLAS web-based database.

Strain gage system

Instrumented rock dowels were installed along the crest of the highest rock cuts for both the Phase 1B and 1C projects. Typical spacing was on the order of 150 to 250 feet. Each installation consisted of a fully-grouted, 100-foot long steel bar (#9 or #14 bar) inclined at -15° and equipped with vibrating wire strain gages (VWSG) at five predetermined depths. The strain gages were factory-assembled and consisted of ½ inch diameter, 3 to 4–foot long “sister bars” that were clamped to the dowels with cable clips (Figure 2). This methodology obviated the requirement to remove epoxy coating from the
production bars and greatly facilitated field fabrication. In addition, the installation procedure exactly mimicked that of the production reinforcement thereby providing crew familiarity for drilling and grouting.

![Image of sister bar with vibrating wire strain gages (VWSGs) attached to an untensioned rock dowel and schematic deployment of VWSG along bar]

Figure 2. A. Sister Bar with Vibrating Wire Strain Gages (VWSGs) Attached to an Untensioned Rock Dowel B. Schematic Deployment of VWSG Along Bar

The specified data logger specified was a model CR1000 provided by Campbell Scientific Inc. (CSI). It utilized CRBasic programming language and PakBus communications protocol compatible with the ATLAS web-based monitoring system. A vibrating wire interface (Model AVW200 provided by CSI) was required to read the strain gages, which employs FFT spectral analysis to measure the frequency output of the strain gages. This was connected to a 16-channel multiplexer (Model AM16/32). A modular autonomous photovoltaic power supply system was to be provided at the ADAS site. The solar power supply system was based on a CSI Model SP65 Solar Panel with a 12-volt photovoltaic power module (solar panel) capable of producing minimum 65 Watts of power; a deep cycle type battery with a nominal capacity of 100 AH (amp-hours), and a charge/load controller. The data logger, multiplexers, and other related and necessary components were housed in a lockable rainproof enclosure. The ADAS site included necessary components to allow remote communication with the data logger using a model RAVEN XTA digital-cellular telephone package. The data was transmitted to an ARGUS or ATLAS web-based database (Figure 3).
Figure 3. Screen Capture in ARGUS Showing Locations of Instrumented Dowels (SGD), Prisms (e.g. P132) and Instrument Towers (T100A and T100B).

Terrestrial LiDAR

The specified surface and subsurface monitoring above was supplemented with terrestrial light distance and ranging (LiDAR) technology to monitor for surface deformation, using WSDOT's short-range, Leica laser scanner (Figure 4A). The scanner has a resolution of about 6 mm at 50 m (0.02 ft. at 150 ft.). The scan data was processed with the software PolyWorks V11 by InnovMetric. A triangular irregular network (TIN) was first created with the initial scan data. The point clouds from subsequent scans were then comparatively analyzed against the initial TIN mesh to identify areas of surficial slope movement (Figure 4B). Several hundred meters of slope face could be scanned and data transmitted within a few hours, and data processing could usually be completed within an hour. Scans were not typically scheduled in advance but made upon request, generally after significant exposure of new cut faces or when the strain gages and/or AMTS detected slope distress.
ROCK SLOPE DESIGN MODIFICATION EXAMPLE

Phase 1C Sectors IX & X

This 2000-foot long interval is comprised of continuous 0.25H:1V rock cuts up to 130 feet in height. The image analysis software “PolyWorks” enabled the determination of mean orientation, persistence, and geo-referenced location of targeted discontinuities present on the existing natural and cut slopes (Figure 5). Stereonet analyses of the entire structural database derived from televiewer logging, Sirovision mapping and conventional mapping in comparison to the structural data derived from the PolyWorks data showed good agreement for wedge-forming joints defined as sets J1 and J3 (Figure 6).
At issue was the potential size of wedges formed by the intersection of these two joint sets which in turn was governed by the persistence of the wedge-forming joints. From the PolyWorks analysis of the laser scan data, the distribution of the measured persistence values was compiled (Figure 7) which showed that at a 95 percent probability level, the measured persistence values for J1 and J3 joints were 78 and 45 feet, respectively. It is noted that remote sensing has more opportunity to underestimate persistence than to overestimate it due to the limited exposure slope height and to vegetative or soil cover.
Figure 7. Persistence Distributions for Wedge-Forming Joints from “PolyWorks”

The “design” persistence values, in combination with the planned slope cut height and orientation, and the natural slope inclination above the planned cut, defined the potential wedge size. This analysis is summarized in Figure 8 in which the J1 and J3 trace lengths on the planned slope face were conservatively adjusted to be slightly more than the persistence values quoted above. Above the cut it was assumed that the wedge would be truncated by a tension crack coincident with the orientation of J2 joints previously mapped during the design investigations. Since the location of the tension crack (J2 joint) was unknown, an arbitrary and conservative assumption of 80 feet measured along J1 was assumed. Figure 8 shows the resultant generic wedge in cross section. The important feature was the line of intersection (LOI) which has an orientation of 40°/248° (plunge/trend) and a length of 124 feet. The trend was nearly orthogonal to the planned slope face (255° dip direction) and the plunge was greater than the anticipated frictional shear strength on the joint surfaces. This meant that in the absence of cohesion on the joints or in the presence of groundwater pressures, an unreinforced wedge was probably unstable. As shown, the wedge height, measured vertically at the slope face, was 50 feet or about 40 to 50 percent of the planned cut slope heights.

It should also be noted that although the persistence values for both J1 and J3 were selected at the 95% level, there was no way of knowing whether these outlier members would be located in space so as to actually intersect and form a wedge. This conservatism was offset by the potential for blasting or other processes to cause less persistent features to coalesce and lead to a larger scale wedge. Accordingly,
engineering judgment was used to accept the generic wedge geometry shown in Figure 8 as the basis for design.

Figure 8. Generic Wedge Geometry

Wedge Stability

Having defined the design wedge size, the follow-on issue was to determine the required passive dowel reinforcement to provide the minimum WSDOT margin of stability (static Factor of Safety, FS = 1.25). The project-standard reinforcement consisted of 40-foot Grade 75 #9 bars typically spaced at 12 ½ feet horizontal by 12 feet vertical to accommodate drilling, blasting and excavation sequencing. Figure 9 upper shows the superposition of a generic “design” wedge on the reinforced slope face. Depending on the exact location, the number of dowels perforating the wedge face ranged from 12 to 14. For the lower value, the total force available to the wedge was approximately 700 kips (12 dowels @ 60 kips per dowel).

To analyse for stability, two cases for shear strength (friction angle and cohesion) on joints J1 and J3 were considered:

Case 1: $\phi = 38^\circ$, $c = 250$ psf  
Case 2: $\phi = 38^\circ$, $c = 0$ psf

The former corresponds to the values routinely used on the Phase 1B and 1C projects while the latter represents an extreme condition potentially resultant from extremely deficient blasting practices. The analyses indicated the planned reinforcement pattern should be adequate for reasonable groundwater conditions. For the most probable shear strength combination (Case 1), even at the extreme transient groundwater condition corresponding to total wedge saturation, nominal stability was predicted. These analyses underscored the importance of the horizontal drains specified in the contract design.
The remaining issue related to the dowel length in comparison to the “design” wedge size. This evaluation showed that the dowels should be lengthened to a minimum of 50 feet at the crest of the wedge to provide adequate bond zone beneath the line of intersection (see Figure 8 Cross Section).

The intent of the reinforcement layout design was to graduate the dowel length from the top down utilizing project-standard horizontal and vertical spacing. The upper rows were lengthened (60 feet at 2630 elevation or higher) followed by two 50-foot rows. 40-foot dowels were specified for the next three rows followed by the lowermost row (one lift above final grade) that was shortened to 30 feet. The rationale for this layout was that the structural geology would not be recognized until the first few lifts were excavated and therefore the initial reinforcement should assume adverse conditions. The mid to lower rows could be tailored to predictable structural geology that would be mappable on the exposed cut faces. An interval of greater J1 persistence was identified in the vicinity of 1369+00 to 1373+00 LW. Accordingly, it was recommended that all bars above nominal elevation 2630 feet (to nearest half lift) consist of Grade 75 #14 bars in place of the #9 bars. This was an insurance policy.
against the possibility of a larger size wedge and in recognition of the steep natural topography above the cut line.

Construction Experience

Figure 10 illustrates the partially completed cut slopes as the excavation proceeded from project east to project west. The final rock cut slopes were in excess of 100 feet in vertical height requiring at least four 24-foot benches (excavation lifts for drilling, blasting and excavation). Pattern dowel reinforcement was installed after each half-lift of excavation, supplemented with spot dowels and/or tensioned bolts as required. It was observed that J1 joints occurred in the rock mass as both short persistence features defining a structural fabric as well as highly persistent “master joints” that transected the entire slope height. Consistent with the design assumptions for generic wedges, the stability of the majority of the cut slope was derived from the limited persistence of the complementary J3 joints. The exception to this generalization was a more closely jointed zone located beneath a swale in the natural mountainside (highlighted zone above man-lifts in Figure 10). This 300-foot long interval also exhibited J2 joints steeply-dipping into the slope, many of which were characterized by clay development up to several inches in thickness. Groundwater seepage was more prevalent than for adjacent slopes. During excavation, slope monitoring detected anomalous deformation trends that eventually necessitated localized redesign of the reinforcement system.

Figure 10. Interim Slopes Showing Persistent J1 Joints.

Highlighted central interval indicates rock cut below swale in which rock mass is more closely jointed and exhibited greater groundwater seepage. Location of SGD03 indicated.
Slope Monitoring

Fortuitously, instrumented dowel SGD03 was located at the crest of the slope in the central area of the swale. Figure 11 summarizes the 2013 dowel load and blast history for the slope interval proximal to SGD03. The dates for all blasts within 100 feet of SGD03 are shown along with the bench number that was being shot. On Figure 11 the blasts are differentiated as presplit only, bench only, or presplit & bench. The noteworthy features of the 2013 and previous year’s load and blast histories were:

- The access ramp shots on 9/5 and 9/6, 2012 produced an 8 kip load at depth of 50 feet on SGD03.
- Strain gages at all depths reported increasing load over a one week period at the end of October 2012. At a depth of 50 feet the load increased by approximately 7 kips during this period. No construction activity was associated with this load increase.
- The 2013 incremental load reactions at 50 and 70-foot depths became progressively larger as the slope height increased (lower bench numbers).
- All load changes in 2013 appeared to be related to blast events. Following blast events, loads asymptotically approached a higher plateau value.
- The gages indicated the load on SGD03 was located between 50 and 70-foot depth. All other gages (<50 feet and >70 feet) showed nominal loads less than a few kips.

![Figure 11. SGD03 Dowel Load History Related to Blasting Events.](image)

Each Bench Represents 24 feet Above Final Ditch Grade.
Based upon the load sensitivity to blasting and the occurrence of a 7-kip instantaneous load increase at 50-foot depth on August 28th, 2013, apparently unrelated to blasting or excavation, the area of concern was designated “yellow alert”. This constrained the contractor’s activities in the area until a more detailed site specific stability evaluation could be undertaken. Blasting was suspended on September 16, 2013. Subsequent PolyWorks analyses identified and quantified J1 joints #1321 and #1322 as well as other major discontinuities.

Figure 12 shows in cross section view the sequential load reaction of SGD03 to the excavation of the adjacent slope. For the purposes of this schematic, the precise timing and duration of the blast muck removal activity was not known. Therefore, the dates shown for the benches are those at which the bench (i.e. production) blasting was completed. Noteworthy observations from Figure 12:

- The initial loads on SGD03 – 50 feet occurred during access construction and Bench 4 excavation. At this stage, the slope cut was at a higher elevation than the strain gage reporting the load increase.
- The progressively greater load increases as the bench elevation was lowered occurred in spite of the pattern reinforcement installed.
- As the bench elevation was lowered from Bench 3 to Bench 2 the load increase migrated down the gage bar to 70-foot depth.
- Plane 1322 (J1 joint with PolyWorks orientation 36°/211°) intersected SGD03 at a depth of 23 feet. This depth was too shallow to account for the observed loads.
- Plane 1321 (J1 joint with PolyWorks orientation 40°/209°) intersected SGD03 at a depth of 72 feet. This feature was probably associated with observed load increases.

![Figure 12. Load History SGD03 Related to Blast and Excavation Sequence.](image)
Figure 13 summarizes the vector displacements for prisms situated proximal to SGD03 superimposed on a change analysis from successive LiDAR scans for a similar one month period. The vector values include the color-coded 3D displacement (inches) and the plunge and trend of the movement vector. The change analysis is plotted in decimal feet. Salient observations:

- Three prisms (shown in red) reported 3D survey displacements greater than 0.3 inches. The corresponding LiDAR displacements for the same prisms were 0.72 to 0.48 inches indicating reasonable correlation.
- The vector orientations of movement for the three prisms were -03°/272°, -37°/241° and -14°/242°, also reasonably consistent with the J1/J3 wedge LOI (-35°/237°) suggesting a wedge mechanism for movement.
- The interval of slope deformation appeared to be limited to the west by Plane 1321 (1367+00 LW) and to the east between P127 and P125 (+1368+75 LW). Coincidentally, this was the only interval in Sector X that did not include pattern #14 dowels for the upper rows.

Figure 13. Comparison of Prism Displacements with Laser Scan Deformation Analysis.
Based on this ongoing slope deformation, the magnitude and depth of the measured loads, the consistency between the three monitoring techniques and the compatibility of the measured displacements with the structural geology, a program of supplemental slope reinforcement was recommended for immediate installation. Site specific analyses for the inferred wedge at 1368+00 LW indicate that 4000 kips of passive reinforcement should be installed along with slope drains to adequately stabilize the overall slope. In addition, a second instrumented dowel (SGD30) was installed at approximately mid-height below SGD03 to provide information on subsurface load accumulation for the two dowels on a common cross section. Figure 14 shows the supplemental reinforcement as well as the secondary instrumented dowel. It was interesting to observe that the lower dowel (SGD30) started to accumulate load within days of installation and at the depth corresponding to the intersection with PolyWorks plane 1321 (Figure 14).

![Figure 14. Supplemental Reinforcement Design for “Yellow Alert” Area.](Image)

Note insufficient lengths of original lower capacity #9 Type “L” bars relative to projection of plane 1321. Mitigation also included multiple PVC-cased horizontal drains up to 100 feet long.

**Post Reinforcement Slope Performance**

During the month of September 2013, the project received essentially zero precipitation. Thus all the measured slope deformation was related to blasting and excavation activities leading up to the suspension of work on September 16, 2013. Between September 28 and October 1 a major storm event resulted in greater than 9 inches of precipitation as reported for a weather station a few miles to the
The supplemental reinforcement and drainage program had just been initiated and the work had to be suspended due to dangerous working conditions. This storm was followed by six weeks of very dry weather followed by a second slightly less intense storm on November 17th. Fortunately all the supplemental reinforcement and drainage was installed between these two storm events thereby providing an opportunity to compare the load reactions for the two storms.

On Figure 15, the time scale for the two storm events was matched by aligning the lag time between the start of precipitation and the initiation of load increase on dowel SGD03 50-foot depth for each storm. The lag time was approximately 85 hours and was clearly the result of hydrostatic loading within discontinuities penetrated by the instrumented dowel. The Oct 1 storm produced a steep increase in load of greater than 7 kips within a 24 hour time period. Conversely, the Nov 17 event produced an increase of 1.5 kips and reached a plateau value within 24 hours. This was attributed to the combined effects of more reinforcement to absorb the hydrostatic loading and to the beneficial effects of horizontal drains.

Figure 16. Comparison of Load Reactions to Storms Before and After Supplemental Reinforcement and Drainage (SGD03-50 feet)
The ultimate test for the supplemental work will be experienced mid-year 2014 after the slopes have had the opportunity to drain following the winter snow melt. It is anticipated that the hydrostatic loading component of the total loads being reported by the instrumented dowels will decrease. The blasting and excavation of the final lowermost bench will be the final confirmation of the adequacy of the supplemental reinforcement program.

CLOSURE

The monitoring program for the I-90 rock excavations has been largely successful in identifying areas of slope deformation, informing necessary modifications for excavation sequencing and for additional slope stabilization, and for “real-time” safeguarding of construction personnel and the traveling public. The integration of surface and subsurface displacement measurements with terrestrial LiDAR has greatly expanded the monitoring capabilities of the AMTS/prism and strain gage systems by providing critically important monitoring redundancy and slope deformation data between non-instrumented portions of the cuts. The strain gage system has proven to be both highly reliable and sensitive to loading induced by blasting, excavation, and groundwater infiltration. Accurate slope displacement measurements coupled with geo-referenced discontinuity mapping from LiDAR facilitated interpretation of structural controls and design (or redesign) of supplemental reinforcement strategies.