Foundations for the Bridge at Pitkins Curve

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ABSTRACT

A case history is presented on the design, construction, testing, and performance of drilled shafts for a bridge located at a difficult site on Highway 1 along California's Big Sur Coast. The purpose of the bridge is to traverse a landslide that has been a long-term maintenance challenge for Caltrans. The site is underlain by rock of the Franciscan Formation and consists of metamorphosed siltstone and sandstone with inclusions of metabasalt. The rock is highly folded and fractured, difficult to sample in some locations, and exhibits wide variations in strength and quality. Construction challenges included the need to place bridge piers on a steep slope just outside the limits of active sliding and installation of drilled shafts into highly fractured rock prone to caving. The presence of perched water tables caused highly variable and unpredictable inflow of water to drilled shaft excavations. This paper describes how these constructability issues were addressed and how they influenced the selection and design of the foundations. This paper demonstrates a rational approach to a difficult design and construction problem, including: (1) the use of careful engineering geologic studies to design a structure with difficult access while traversing a major landslide (2) the need for careful attention to constructability for drilled shafts in highly fractured rock with variable groundwater, and (3) the interaction between load testing and site investigation and its application to LRFD design of rock-socketed drilled shafts.

SITE CONDITIONS AND GEOLOGICAL SETTING

Pitkins Curve is located on Highway 1, approximately 1.5 miles south of Lucia, on the Big Sur Coast in Monterey County, CA. This area lies in the Santa Lucia Mountain range in the Coast Range Geomorphic Province. The area is characterized by rugged, steep terrain with steeply incised drainages and narrow crested ridges.

Figure 1 is a photograph of Pitkins Curve and adjacent sections of Highway 1. Immediately south of Pitkins Curve is major rockfall area referred to as Rain Rocks.

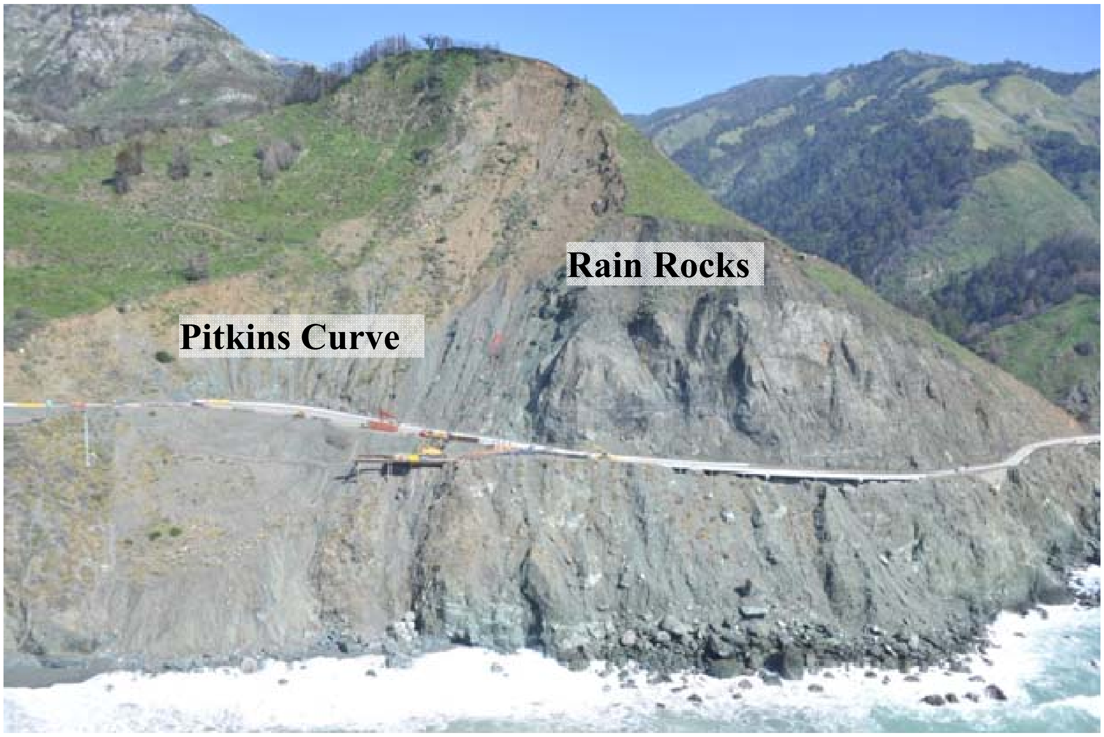


Figure 1. Pitkins Curve and Adjacent Sections of Highway 1, Big Sur Coast.

The area is underlain by the Franciscan complex, consisting of metamorphosed sedimentary and volcanic rocks. A matrix of dark gray, highly sheared siltstone and shale, metamorphosed to argillite or phyllite, contains blocks of medium-grained fractured meta-sandstone and greenstone (metabasalt). The metabasalt (greenstone) is relatively hard and erosion resistant.

Quaternary colluvial and debris flow deposits overly the Franciscan rocks, forming a thick sequence of crudely bedded silty sand with numerous angular cobbles and boulders. Below the roadway, fill is intermixed with this material where it was used to construct and maintain the roadway embankment. This material is derived from the upper adjacent slopes and is therefore similar to the colluvium and in most cases indistinguishable.

Pitkins Curve Landslide. In the Pitkins Curve section (Figure 1) the roadway embankment has been creeping since its original construction in the late 1930's. Largely the sliding was restricted to shallow failures receding up into the traveled way. Repairs were made by end dumping slide debris over the side. The slopes below the roadway are subject to constant erosion by high surf at beach level and full exposure to oncoming Pacific storms. In 1998 a landslide below the roadway caused the loss of one lane. In February of 2000, 130,000 cubic yards of material translated down slope, undermining and destroying one hundred meters of roadway. This slide is a combination translational/rotational movement, slipping above competent bedrock at depth. Based upon surface features and subsurface borings, the depth to sliding is approximately 65 ft below grade. The slide material consists of pre-existing landslide deposits, side cast materials, and roadway embankment. Triggering mechanisms included high groundwater levels, surface water infiltration, and toe erosion by high surf leading to undermining.

A variety of repair strategies were considered to remediate effects of the 2000 slide and later that year the roadway was relocated inland, away from the landslide into the hillside. Approximately 100,000 cubic yards of material was excavated to cut-slope angles varying between 1:0.75 and 1: 1 (V:H). This strategy reduced driving forces from the head of the slide and diverted a portion of infiltrating groundwater and surface runoff away from the slide area. This was supplemented by the installation of horizontal drains above and behind the slide. To control post-construction rockfall a wire mesh drapery system was placed on the slopes above the roadway.

Pitkins Curve Rockslide. In February 2001, heavy winter storms again hit the site accompanied by development of rockfall and small rock slides in the new cut slopes above the roadway. Sliding rapidly progressed upslope to the ridgeline. Daily rockfalls, 1 foot to 10 foot in dimension, and rockslides of 50 to 100 cubic yards destroyed the wire mesh drapery system. To maintain traffic safety the roadway was shifted seaward away from the hillside and a rockfall catchment ditch was constructed. This new roadway alignment is close to the pre-2000 alignment. The ditch is approximately 33 feet wide and 13 feet high. Slope movement was so regular and consistent that ditch cleaning was required daily. The total accumulation of these small events totaled more than 20,000 cubic yards in three months.

Immediately south and adjacent to Pitkins Curve are the high cliffs known as Rain Rocks (Caltrans, 1995). Since the road was constructed in the 1930's Rain Rocks has been a known rockfall location. Rockfall has occurred regularly during heavy rains and high winds. Mitigation historically consisted of warning signs, rock patrols, and rock scaling. In January 1993 a rockfall of approximately 20 ft in dimension damaged a concrete crib wall supporting the roadway. The crib wall failed and was replaced in 1995. In 1997/1998 a sidehill viaduct was constructed. During construction of the viaduct falling rocks traveled beyond the catchment area and into the construction zone. In response, additional rockfall mitigation measures were implemented by covering the slope with a wire mesh drapery system.

Following the 2000 repair the slopes above Pitkins Curve receded, eventually reaching the top of Rain Rocks where Pitkins Curve and Rain Rocks transition at the ridge crest. This triggered an increase in rockfall in the most northern chute of Rain Rocks, which at the time was covered by wire mesh drapery. Finally a 350 cubic yard rockslide destroyed the wire mesh drapery in the chute. Slopes above the chute continued to destabilize and fall onto the roadway. Caltrans maintenance crews were continually interrupting traffic to clear the roadway of rockfall and scale the slopes of loose rock. In spite of these efforts several vehicles were struck by falling rock. A temporary rockfall barrier was installed, which required narrowing the roadway to two 10-foot lanes. In 2001 the temporary barrier was replaced with a cable net drapery system, restoring the roadway to its original width.

The slope instability issues described above have made the Pitkins Curve/Rain Rocks section of Highway 1 one of the most costly and dangerous sections of roadway in the United States (Wills et al., 2001). As a result of frequent landsliding, Caltrans maintenance costs have become excessive, roadway closures are frequent, and emergency work poses a risk to highway workers and the traveling public. To address these challenges, an interagency task force recommended a long-term engineered solution consisting of: (1) a bridge across the Pitkins Curve landslide and (2) a rockshed at Rain Rocks. A rendition of the roadway with the two proposed structures is shown in Figure 2. An important feature of the bridge at Pitkins Curve is the location of the two bridge piers. The piers are located just outside the lateral limits of movement of the Pitkins Curve landslide. These limits were established on the basis of long-term monitoring of air photos and satellite imagery by the U.S. Geological Survey and detailed site characterization involving borings, geological mapping, and instrumentation. The bridge is designed to span the landslide while allowing the slide material to continue moving down slope in response to natural geologic processes. This approach solves a civil engineering problem, *i.e.*, providing reliable transportation, while accommodating the challenging geology of the Pitkins Curve site. That is to say, match the structure to the ground conditions.

Figure 2. Proposed Bridge and Rock Shed at Pitkins Curve/Rain Rocks.

GEOTECHNICAL FEATURES

Site characterization included extensive core sampling at the sites of the bridge piers. Detailed subsurface profiles were developed from core logs. Rock mass characteristics, including core recovery, RQD, and uniaxial compressive strengths, varied significantly with depth. The detailed profiles were used to assess both constructability issues and design parameters for potential foundation schemes. Based on the degree of fracturing and observations of perched water tables with locally high water inflow, the potential for caving and seepage were identified as construction challenges for foundations requiring excavation. From a design perspective, rock uniaxial strengths were considered favorable for developing side and base resistance of rock socketed foundations.

FOUNDATION SELECTION

Foundation types considered for the bridge piers ranged from a single large-diameter drilled shaft (14-ft diameter) to drilled shaft groups of (2 to 4 shafts) supporting a footing at each pier. The most critical load combination resulted from analysis of the bridge for Extreme Event I (earthquake) loading and resulted in significant lateral and overturning forces at the foundation level. One of the site factors that favored a larger number of smaller diameter shafts is the steepness of the slope on which the piers are located. Larger diameter shafts require larger and heavier equipment (drilling rigs and cranes). Positioning of the required equipment on the steep slopes required either (a) construction of temporary trestles out over the slope, or (b) construction of stabilized benches cut into the slope. While both means were considered feasible, both could be achieved more cost-effectively if the loads to be supported were minimized. Accounting for the steep (and potentially unstable) slopes, the lateral and overturning loads, and the characteristics of the rock mass, a four-shaft group supporting a 8-ft thick concrete footing at the top-of-rock elevation was selected as the most efficient and cost-effective foundation system.

DRILLED SHAFT LOAD TEST

Several factors created a high degree of uncertainty with regard to the application of predicted values of side and base resistance to the design of drilled shafts deriving their support from the Franciscan rocks underlying the Pitkins Curve bridge site. First is the large degree of variability in rock mass characteristics observed in core logs. Features such as degree of weathering, degree and orientation of fracturing, characteristics of the fracture surfaces, and the strength of intact rock specimens exhibit wide ranges both vertically and horizontally. Second, it had been Caltrans policy to neglect base resistance of drilled shaft in rock due to uncertainties about the quality of rock mass beneath the tip and uncertainties about the effectiveness of contractors' cleanout procedures, particularly for shafts placed under water or slurry. Water, in the form of perched groundwater with potential for large inflow to drilled shaft excavations, created further uncertainty due to its potentially adverse impacts on both side and base resistance due to disturbance and caving.

To address these issues and to obtain valuable information on constructability, Caltrans conducted a pre-design load test on a prototype drilled shaft constructed just on the inland side of the roadway and upslope from Pier 2. A boring was made at the exact location of the test shaft and carefully logged to provide a detailed profile of the rock. Core samples that were sufficiently intact were tested for uniaxial compressive strength. At a depth corresponding to the tip of the test shaft (35.5 ft) the rock is described as very hard metabasalt.

The test shaft was 42-inches in diameter, 35.5 ft deep. The O-cell was placed at a distance of approximately 3 ft above the base of the shaft. The test shaft was drilled using a rock auger without casing or other support. Seepage of water into the excavation was observed during drilling. Water at the base was mixed with cement and re-excavated just prior to the final concrete pour in order to minimize potential adverse base conditions. Strain gages placed at a depth of 20 ft were used to determine the magnitude of load transfer over the depth intervals from 0 – 20 ft, 20-ft to the O-cell, and O-cell to the base of the shaft.

Full results of the O-cell test are presented in a report prepared by Loadtest Inc. (2007). The maximum load applied to the test shaft was 5,200 kips. At this maximum load, axial displacements above and below the O-cell were 1.02 and 0.76 inches, respectively. Table 1 presents values of unit side resistance measured at the maximum test load. In addition, base resistance was mobilized under very small displacement, clearly showing that base resistance could be relied upon for design, provided that proper cleanout and inspection procedures were deployed as part of the construction process. Mobilized base resistance was calculated to be 396 ksf.

Table 1. SUMMARY OF UNIT SIDE RESISTANCES FROM O-CELL TEST

Depth Interval	**General Material Description**	**Unit side resistance (ksf)**
1.6 – 20.0	top 2-3 ft pavement base course underlain by interbedded metasediments/metabasalt rock	5.51
20.0 – 32.3	metasediment and metabasalt	29.43
32.3 – 37.0	metabasalt	27.0

ANALYSIS AND DESIGN

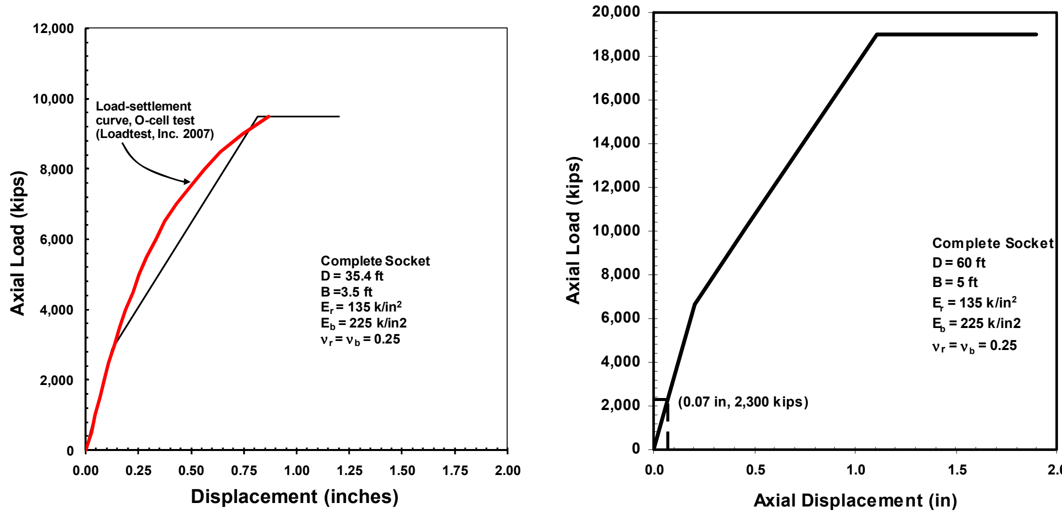
Drilled shafts were designed to satisfy AASHTO LRFD criteria for all applicable limit states. For design under lateral loading, the *p-y* method of analysis was used to evaluate the drilled shaft response to combined lateral, moment, and axial force effects predicted by structural modeling of the bridge. User-specified *p-y* curves were input to the program LPILE (Ensoft, 2007). The user-specified curves were developed using the hyperbolic model proposed by Liang et al. (2009). This model requires the following parameters to define each hyperbolic curve: (1) the initial slope, which is the subgrade modulus K_h and (2) the asymptote, which is the ultimate lateral resistance p_{ult}. Both of these are approximated on the basis of empirical correlations given by Liang et al. (2009) to rock mass characteristics, in particular the Geological Strength Index (GSI) and uniaxial compressive strength of intact rock, q_u. GSI was evaluated from examination of rock core and photos of rock core. Lateral loading considerations and the resulting moment demand, governed the final shaft diameters (5 ft).

Design for axial loading incorporated both side and base resistances for evaluation of strength, extreme event, and service limit states. This approach was validated by results of the O-cell load test. Values of side resistance (f_s) measured in the load test were fit to the following expression relating f_s to uniaxial compressive strength of intact rock (q_u):

$$\frac{f_s}{p_a} = C\sqrt{\frac{q_u}{p_a}} \tag{1}$$

in which: p_a = atmospheric pressure in same units as f_s and C = fitting coefficient. For the inerlayered meta-sedimentary rock and basalt observed in the test location, this yields an average value of $C = 0.62$. For highly fractured rock this agrees well with the lower-bound value of 0.63 reported by Kulhawy et al. (2005). The back-calculated value of $C = 0.62$ was then used to evaluate design values of side resistance based on uniaxial compressive strengths measured for cores samples taken from the borings at each of the pier locations. For tip resistance, two factors were considered: (1) results of the O-cell test, in which the tip was bearing on a layer of metabasalt, and (2) careful evaluation of the boring logs so that tip elevations of the production shafts corresponded to high quality rock, i.e., high RQD material. The final design, considering LRFD criteria and the objective of high quality rock at the tip elevations, resulted in 50 ft long rock sockets at Pier 3 (south) and 60-ft long rock sockets at Pier 2 (north).

Service limit state design was based on a tolerable settlement for individual shafts of ½ inch under a nominal axial force of 2,300 kips, as established by the bridge structural engineer. A load-displacement model was developed from the O-cell load test results, as described by the authors in an earlier paper (Turner et al., 2009). A simplified model of rock socket load-settlement behavior given by Kulhawy and Carter (1992) is fit to the measured axial load displacement curve from the load test through trial values of the rock mass elastic and strength properties. Where borings verify that the rock mass has similar lithology, strength, and discontinuity characteristics, the analysis can then be used to evaluate load-deformation behavior of trial designs (Turner, 2006). Figure 3a shows the O-cell curve and the resulting modeled curve, while Figure 3b shows the curve extrapolated to the conditions at Pier 2. For an axial compression load of 2,300 kips, the predicted displacement is approximately 0.07 inches and the shaft response is in the linear elastic range. By this analysis, the proposed trial design easily satisfies the service limit state criterion that limits settlement to ½ inch. The final shaft dimensions are governed by strength and extreme event load considerations and not by the axial settlement criterion.



(a) O-cell Load Test Results (b) predicted for shafts at Pier 2

Figure 3. O-Cell Equivalent Top-Load Settlement Curve and Modeled Curve.

CONSTRUCTION

Construction began in January of 2010 with the erection of trestles to support construction equipment for drilled shaft construction. Two trestles were erected, one from the north abutment and one from the south, as shown in Figure 4. Trestles are supported on 24-inch diameter piles driven into rock using a downwhole hammer.

Figure 4. Trestles extending from north and south abutments.

At each pier location, a temporary shored excavation was extended to the top of rock. A group of four 5-ft diameter drilled shafts was constructed inside each excavation by drilling from the trestle, as shown in Figure 5.

Figure 5. Drilled shaft excavation from south trestle.

Difficulties encountered during drilled shaft construction were those anticipated during the planning phases, namely caving of fractured rock and high inflow of water upon encountering pockets of perched water. The contractor addressed these difficult conditions by first drilling into the caving material with a slightly oversized auger and removing as much material as possible. A low strength concrete mix was then added to the bottom of the excavation and mixed with the caved materials. When the mix was sufficiently hardened, the socket was re-drilled to the design diameter, essentially into the cement-stabilized material. This technique made it possible to extend the shafts through the caving zones to the target tip elevations. Figure 6a shows conditions at the bottom of the shored excavation for construction of the foundations at Pier 2, while Figure 6b shows the excavation after cement stabilization of one of the caving zones.

(a) (b)

Figure 6. (a) Drilled shaft construction inside shored excavation, (b) cement-stabilized excavation in caving rock.

SUMMARY AND CONCLUSIONS

The bridge at Pitkins Curve provides an example of foundation design and construction that required the unique and challenging aspects of site geology to be taken into account properly. The primary purpose of the bridge is to avoid a landslide that has impacted Highway 1 since its construction in the 1930's. Location of the bridge foundations was dictated by the slide geometry and characteristics, requiring long-term monitoring, detailed site characterization, and understanding of the geologic processes. Design of the foundations to meet LRFD criteria required very detailed characterization of rock mass characteristics and was made feasible by conducting a pre-design axial load test. Construction difficulties were anticipated by conducting the site investigation for constructability as well as for establishing geotechnical design parameters.

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