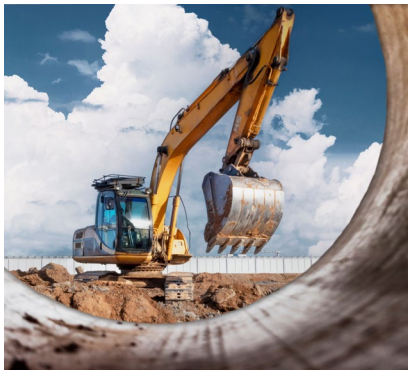
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INNOVATIVE SECANT PILE EXCAVATION SUPPORT: WEAPONS STORAGE AND MAINTAINANCE FACILITY – MALMSTROM AIR FORCE BASE

(Published at DFI50)

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ABSTRACT

The United States Air Force is constructing a new facility to consolidate weapon maintenance, storage, and training to support intercontinental ballistic missile operations at Malmstrom Air Force Base in Montana. Part of the new facility requires an approximately 30 ft by 75 ft by 40 ft depth basement to house significant mechanical equipment, with a thick ground-level structural concrete slab above. Concurrent construction of other portions of the structure on all sides precluded open cut or offset temporary support. A single pass system of reinforced concrete secant walls was selected to overcome these constraints. In the completed condition, these walls span between the bottom floor slab and ground-level structural slab, however the upper slab could not be constructed until after the major mechanical equipment was installed inside the basement, requiring alternative means of lateral support during construction. The design team considered several alternatives, including temporary internal steel wale frames and top-down construction with block-outs for equipment installation, but none provided adequate access for equipment installation. Grouted tiebacks were also evaluated, but deemed infeasible due to numerous adjacent drilled shaft foundations. Ultimately, the team devised a hybrid system utilizing three different methods of lateral support: cantilever secants, shallow tie rods to temporary deadman shafts, and tie rods anchored to the adjacent permanent structure foundations. This facilitated successful construction and equipment installation with no obstructions inside the excavation, while meeting all performance objectives.

Keywords: deep excavation, permanent secant piling, complex sequence, large-diameter drilled shafts, dense construction

INTRODUCTION

The United States Air Force is constructing a new facility to consolidate weapon maintenance, storage, and training to support intercontinental ballistic missile operations at Malmstrom Air Force Base in Montana, which includes heavy reinforced concrete building structures. One of the structures includes a 30 ft by 75 ft by 40 ft deep basement to house significant mechanical equipment. The basement area will have a thick ground-level structural concrete slab above that is contiguous with the surrounding building structure. The general configuration of this basement area is illustrated in Fig. 1 and 2.

The construction schedule and sequencing required concurrent construction of other portions of the structure on all sides of the basement, which precluded open cut excavation or conventional support of excavation (SOE) and construction of conventional cast-in-place formed concrete basement walls. The prime contractor, Archer-Western Federal JV subcontracted Condon-Johson & Associates to design and construct a single-pass SOE and permanent basement wall system and the required temporary supports until the permanent supporting structure was completed. Condon-Johnson & Associates engaged Brierley Associates to provide the required design and construction phase services to successfully complete the work.

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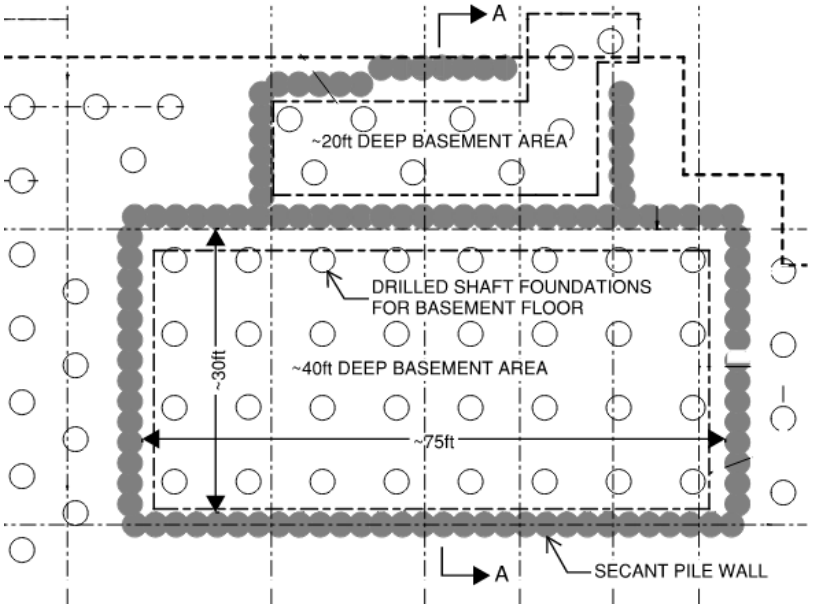


Fig. 1. Basement Structure Plan View

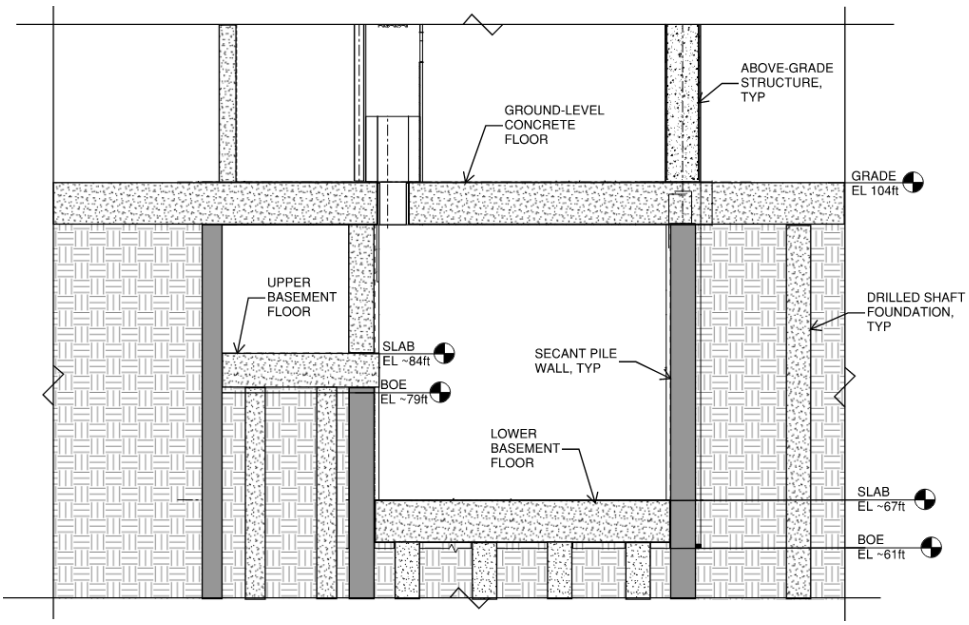


Fig. 2. Basement Structure Section A-A

This paper focuses on the concept development, alternatives analysis, and selection of the preferred SOE/permanent wall system and its successful construction. Due to the proximity to adjacent drilled shaft foundations and concurrent construction, the contract documents specified secant pile basement walls with size, reinforcement, connection to the permanent top slab, and temporary support delegated to the contractor. Condon-Johnson and Brierley considered a wide variety of alternatives, ultimately devising a hybrid system utilizing three different methods of lateral support: cantilever secants, shallow tie rods to temporary deadman shafts, and tie rods anchored to the adjacent permanent structure foundations. This facilitated successful construction and equipment installation with no obstructions inside the excavation, while meeting all performance objectives.

Schedule was a significant challenge and required preparation for construction prior to completing design. Furthermore, Brierley and Condon-Johnson were not initially aware of all design constraints, particularly those associated with installation of mechanical equipment within the basement, as well as embeds and block-outs. These constraints were discovered during the design review process as well

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as coordination meetings with the project design team and required creative solutions to adapt the designs after construction was underway. Condon-Johnson worked closely with Brierley to ensure that design solutions employed readily available equipment and materials to minimize delays and disruptions.

SITE CONDITIONS

The construction site is located on the Malmstrom Air Force Base near Great Falls, Montana. The site geology consists of surficial sand and glacial lakebed clays overlying glacial till clays. Sandstone, clay shale, and shale underly the glacial till clays, generally at depths on the order of 75 ft.

Based on recommendations from the project geotechnical report, a uniform cohesive design soil profile was used, with a total unit weight of 125 pcf and undrained shear strength of 1,875 psf, as shown in Fig. 3. Groundwater was not encountered in the subsurface exploration, so design for hydrostatic loads or dewatering was not recommended.

Layer No.	Depth	USCS Classification	Parameter	Recommended Value
1	0.0 to 73.0	CL, CH	Effective Unit Weight, γ	125 pcf
			Undrained Shear Strength, Su	1,875 psf
			K-value	1000 pci (static) 400 pci (cyclic)
			Strain, ε_{50}	0.005

Fig. 3. Design Soil Parameters

SUPPORT OF EXCAVATION CONCEPTS AND SELECTION

As previously introduced, the contract drawings envisioned the SOE as a single-pass secant-pile system, with the final configuration of the structure utilizing the walls spanning vertically between the ground-level and basement slabs. The secant piles would be installed prior to excavation, but since the top and bottom slabs could not be completed until after excavation, means to laterally support the secants in their interim conditions would be required. Since Brierley and Condon-Johnson were responsible for designing the secants, we had considerable freedom in sequence and methods to accomplish the work, subject to the site, construction sequences, and other constraints. However, each approach had differing tradeoffs, such as cost, schedule, and complexity, and particularly coordination with other trades and related construction. Thus, to thoroughly evaluate each option, we prepared a total of six initial concepts for consideration, which were:

1. Grouted Tiebacks
2. Tie Rods and Deadman Anchors
3. External Wale and Deadman Anchor Shafts
4. Internal Temporary Wale Frame – Below the Top Slab
5. Internal Temporary Wale Frame – Above the Top Slab
6. Top-Down Construction

All these options would utilize similar reinforced concrete secant piles, but varying means to support the tops of the secant piles until the top and bottom floor slabs could be completed to provide the permanent load path. Since the permanent condition required the secant pile walls to span the full height from the bottom to the top slab, and Brierley/Condon-Johnson was responsible for both the permanent and temporary design cases, all options were based on a single level of temporary supports at or near the top of the secants. The following sections summarize each of the options considered with their respective advantages and disadvantages.

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Grouted Tiebacks

The grouted tiebacks option had the advantage of allowing a fully open excavation plan with no interior obstructions. However, due to the number and pattern of surrounding drilled shafts (which had to be installed prior to excavation) the permissible paths for tiebacks were impractical, requiring precise alignment at many varying angles, and would likely have struck drilled shafts during installation.

Tie Rods and Deadman Anchors

To address the complexity and risk of drilling inclined grouted tiebacks through the surrounding drilled shafts, this option utilized shallow buried horizontal (or slightly inclined) tie rods, anchored to buried deadman walls. However, the permissible paths for the tierods were still challenging, and would require precise and varying angular orientation of anchor trumpets in the secant piles. Furthermore, preliminary calculations indicated that developing adequate capacity in the deadman walls would require significant depth, and would conflict with planned surrounding construction activities.

External Wale and Deadman Anchor Shafts

The previous tieback and tie rod options utilized one rod in each (reinforced) secondary secant pile. To reduce the complexity of threading so many rods between the surrounding drilled shafts, this option introduced external wale beams that allowed reducing the number of tie rods by approximately half, which provided easier positioning around the surrounding drilled shafts. Instead of conventional wall-type deadmen, this option also introduced large-diameter drilled shafts as deadman anchors. These were required at greater depth, but installation would be easier and less disruptive.

Internal Temporary Wale Frame – Below the Top Slab

To eliminate the interference with surrounding construction activities and additional external elements of the previous three options, an alternative that utilized a conventional internal wale frame was developed. The wale frame would be located several feet below the top slab, allowing completion of the excavation, base slab, and top slab, after which the frame would be cut up and removed through openings in the top slab. The secant piles would be designed to have adequate capacity to act in the cantilever condition during excavation to the relatively shallow depth before the wale frame would be installed.

Internal Temporary Wale Frame – Above the Top Slab

To avoid the complexity of constructing the top slab falsework around the wale frame, a modified version of the previous option was devised that used a similar internal steel wale frame but located it at an elevation above the top slab. This would utilize heavy steel W-shapes embedded vertically into the reinforced secondary secants, extended to carry the lateral load to the steel frame above. The construction sequence would be similar to that for the previous internal frame below the top slab, except the top slab would be poured around the W-shape pile extensions, after which they would be cut off flush with the top slab, or in a shallow pocket that could be patched with grout to conceal them.

Top-Down Construction

Lastly, a concept using top-down construction was developed. This would involve installing the secants, then pouring the final top slab on prepared grade with a smooth formwork surface (such as plyform on grade) to provide the final top support load path before commencing any excavation. Block-outs in the top slab sufficient to gain access with small excavation equipment (e.g., mini excavators and skid loaders) would be left, but small enough to maintain adequate strength in the top

slab to provide the required support to the secant pile walls. The excavation would then be performed below the top slab, after which the base slab would be poured and internal mechanical equipment installed, followed by infilling the top slab block-outs. The primary disadvantage of this method was the slower and more costly excavation process, necessitated by using small equipment and working below the top slab.

Selected Design Alternative

Due to the simplicity and minimal impact on surrounding construction activities, the top-down approach was the favored method by most of the construction team, including the general contractor, and was progressed to an initial design submittal. However, after submittal, the specialty mechanical equipment installer indicated that this method would not be compatible with installing its equipment, requiring nearly the entire top slab to remain open until after the equipment was installed. This requirement also eliminated the feasibility of both internally braced options, as the required wale frame members could not be designed around the space required for equipment installation.

This required the design team to redesign and use an approach with an external means of support. The selected option for final design was a version of the third option, utilizing external tie rods to deadman shafts, but with further refinement and optimization.

SUPPORT OF EXCAVATION DESIGN

Secant Pile Diameter Selection and Tolerances

Condon-Johnson's original bid utilized 3 ft diameter shafts for the secant piles, installed using 900 mm sectional casing. This was to match the drilled shaft foundations Condon-Johnson was also constructing on the project and allowed for shared drill tooling. This original approach also utilized a mid-height internal wale frame to reduce demands until the base slab was installed. When it became clear that internal bracing was not possible, and the concepts for a single level of support at the top of the secants were being developed (as described in the previous section), it became evident that the higher moments resulted in dense rebar cages and inefficient use of reinforcement. To increase the capacity of the shafts and reduce congestion to meet best practices for concrete placement, the secant piles were upsized to 1m diameter.

Secant pile placement tolerances were selected to be 1in for plan location at ground surface and 1% maximum out-of-plumbness. The spacing between adjacent reinforced secondary secants in the shallower portion of the basement was set at 5'-0" (to provide approximately 9" of overlap between each primary and secondary), and at a closer 4'-6" (to provide approximately 12" of overlap) with piles in the deeper section, to ensure adequate overlap at the bottom of the excavation after placement tolerances.

Secant Pile Design

After the secant pile size and layout was established, design groups were identified, which included four basic pile types and corresponding design cross sections. All walls were designed with reinforced secondary secant piles carrying the entire tributary load based on their spacing, with unreinforced primary piles acting only as lagging. All walls were required to carry some permanent vertical loading from the structure above, with some portions subjected to higher loads.

Since the secant piles acted as both temporary SOE and permanent basement structural walls, load cases were prepared that captured all conditions that would be encountered, from the beginning of construction to the final service life of the structure. As boundary conditions varied throughout these periods, the applicable earth pressure theories varied as well. During construction, when piles were in a cantilever condition, active lateral earth pressure was used, and when temporary construction conditions utilized a support at the top of the wall, apparent earth pressure profiles were used. In the

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permanent condition, lateral earth pressures were taken as at-rest values, along with lateral seismic pressure and soil swell pressure. Lateral pressures from relevant surcharge loads were included as well, including construction equipment.

Each secant pile design case was analyzed for all load and boundary conditions throughout its lifecycle, and the resulting demand envelopes (shear, moment, and axial) were used to design reinforcement that provided adequate capacity for all conditions. See Fig. 5 for an example.

Longitudinal (vertical) reinforcement was designed with conventional deformed Grade 60 reinforcement. Due to uncertainty in availability of reinforcement for the project, a variety of bar options were analyzed, including #11, #14, and #18 bars. The final design provided options for both #11 and #18 bars, which required bundling of bars in both cases to provide adequate clearance for concrete placement due to the ratio of reinforcement required in the highest-demand zones. See Fig 4. Some of the bundled longitudinal bars were terminated in lower-demand zones to economize the use of steel and reduce congestion.

Transverse reinforcement was designed using Grade 80 spiral bars to minimize congestion. Typical pitch varied between 6 in and 12 in, depending on location and corresponding shear demand.

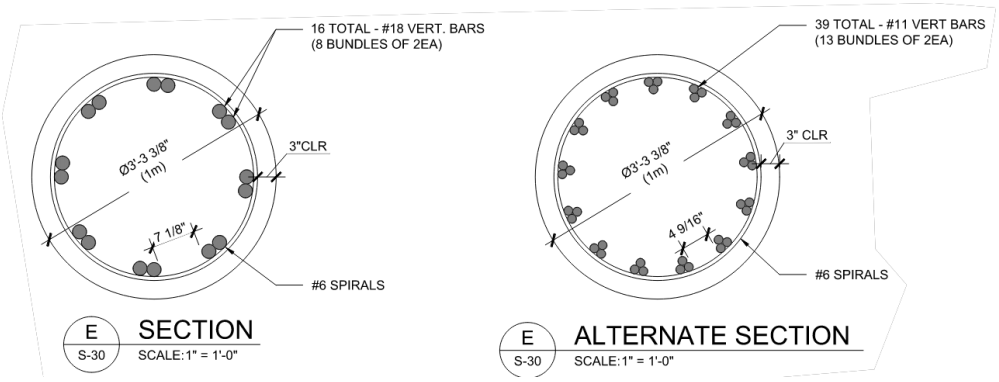


Fig. 4. Example of Secant Pile Reinforcement Cross Sections

Reinforced Concrete Wale Design

The design used external tie rods at approximately every other reinforced secondary secant pile, requiring a wale beam. This was provided as a reinforced concrete beam integral with the top of the secant piles, just below the permanent top slab. The beam was designed to share the top 4'-0" height of the secant piles, with reinforcement detailed to pass through each other uninterrupted, as shown in Fig. 6.

The wale was designed for continuity around corners, which reduced the number of tie rods required near corners. It was also extended around the entire perimeter of the basement at the top level, including the secants that were designed as cantilevers to provide continuity.

Tie Rods, Deadman Shafts and Anchor Points

On the east and west sides, the tie rods supporting the wale beam were anchored by reinforced concrete deadman piles. These were 6 ft diameter drilled shafts, 50 ft deep. They were designed as laterally loaded shafts and analyzed using LPile to determine their capacity at an acceptable deformation, and structural demands for which their reinforcement was designed. They were offset from the secant pile walls (80 ft) to ensure the full capacity could be mobilized. The tie rods and tops of the deadman shafts were located at an elevation that avoided conflicts with future construction in the area and permitted abandonment in place after they were no longer required.

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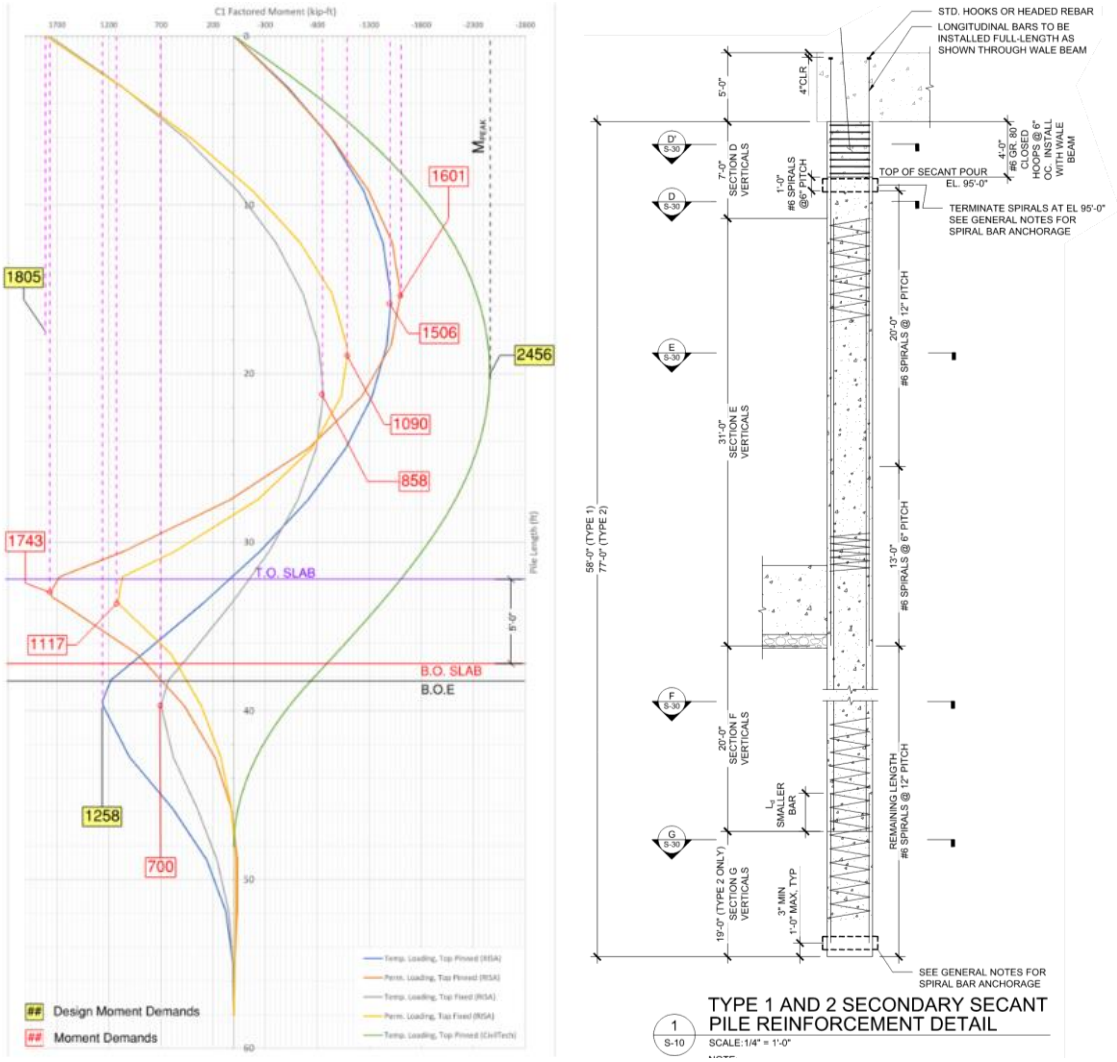


Fig. 5. Example of Secant Pile Bending Moment Envelope and Reinforcement

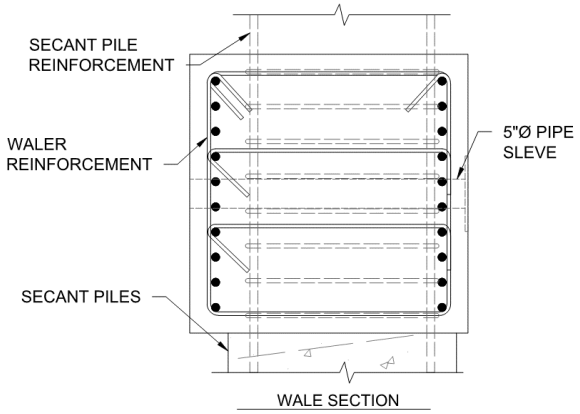


Fig. 6. Example Wale Beam Cross-Section

On the south side of the secant pile basement structure, a large permanent reinforced concrete slab was located with a dense array of permanent drilled shafts. With approval from the permanent structure designer, this portion of the structure was utilized as an anchor, avoiding the use of temporary deadman shafts. Since the secant pile excavation was a schedule priority, the minimum required number of drilled shafts required was calculated, which permitted only a portion of the overall slab to be poured and reach strength before it could be utilized as an anchor. The tie-rods were located below the permanent slab and connected to the slab with a cast-in-place corbel secured to the

bottom of the slab with reinforcement dowels. All tie rods were proof-loaded and pretensioned to 250 kips. See Fig. 7.

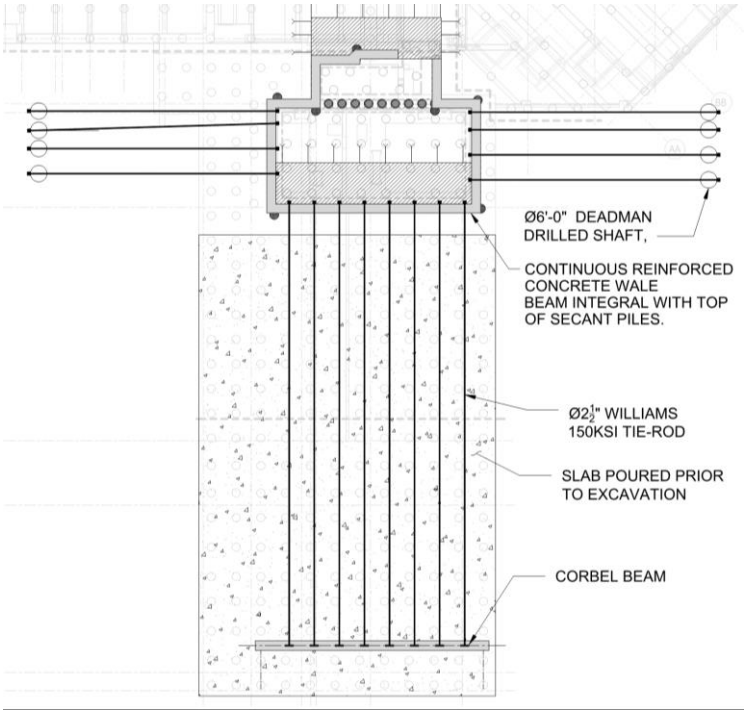


Fig. 7. Tie Rod, Deadman Pile, and Anchor Beam Plan

CONSTRUCTION

Secant Pile Installation

A reinforced concrete guidewall was constructed prior to drilling the secant piles to ensure they were all located with the proper overlap. The guidewall also aided the vertical alignment of the shafts, particularly the secondaries when the casing was coring through the adjacent piles.

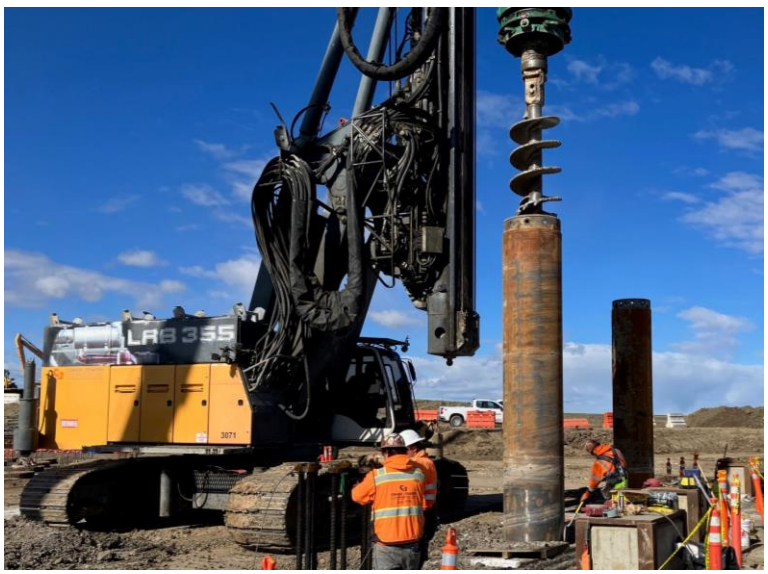


Fig. 8. Secant Pile Drilling

The consistent and uniform stiff clays on this site were ideal for open hole drilling. Condon-Johnson was able to install all primary shafts in this manner, including in depths near 80 ft, with negligible

groundwater. The secondary shafts were installed using 1M sectional casing. This casing cored through the adjacent piles creating the interlock between the primary and secondary piles. The casing also aided in maintaining vertical alignment. The stiff clays on this project allowed the casing to be removed prior to reinforcement and concrete placement, which greatly simplified the installation process and increased production. All concrete was placed using free fall methods.

Both primary and secondary secant piles were poured to the bottom of the wale beam to allow its construction after secants were complete. The transverse reinforcement in the top 4 ft of the secants was designed with Grade 80 closed hoops that were installed with the wale beam rebar, rather than the continuous spirals used in the rest of the secant length below.

Deadman Drilled Shaft and Tie Rod Installation

The deadman shafts were drilled using open hole drilling methods. For ease of tie-rod installation, a steel trumpet was installed in the reinforcement cage prior to shaft construction. Once these shafts were installed, and prior to all the wale beam reinforcement being placed, the tie rods were installed. Shallow trenches were dug for each tie rod between the waler and the corbel beam and deadman shafts. The tierods were connected at the corbel beam and deadman shafts and laid in the trench to within 4 ft of the back face of the wale. Leaving the tie rods a few feet behind the waler ensured that the trumpets were placed in the correct location within the waler and also prevented the need to drill holes in all the waler form work. The tie rods were encased in a PVC sleeve within the trench to allow free elongation during stressing and minimize disturbance to the surrounding soils during tensioning and detensioning.



Fig. 9. Tie Rod Installation to Deadman Shafts

The tie rods and corbel beam below the 5 ft thick on grade mat slab to the south were placed and constructed prior to the slab. After the corbel and slab had reached strength, tie rods were proof-loaded and locked off at 250 kips. A week after locking off the tie rods, a lift off test was performed on each tie rod to determine if deadman shaft creep deflection was causing any decline in lock-off load. The results showed negligible decrease in lock-off loads.

Once the top slab over the secant excavation is placed and reaches strength, it will provide the permanent support at the top of the secant pile walls, so the tie rods will no longer be necessary, at which time they will be detensioned.

Construction Challenges and Solutions

The site was graded to the elevation of the bottom of the 5 ft thick mat slab. The secant reinforcement extended up into this mat slab, and secant reinforcement cages were no more than 5 ft on center. This prevented Condon-Johnson from utilizing an oscillator to extract the sectional casing during concrete placement without a substantial amount of cribbing. The favorable soil conditions allowed for Condon-Johnson to overcome this issue by extracting the casing with the drill rig prior to reinforcement and concrete placement.

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Fig. 10. Excavation at Final BOE

SUMMARY AND CONCLUSIONS

To date, the facility construction is ongoing, with permanent top slabs under construction, and scheduled to be completed and reach design strength by mid-May 2025, after which the tie rods will be detensioned and the structure will function in its permanent configuration.

This paper focuses on the innovative approaches used to design and construct the basement structure on a new weapons storage and maintenance facility on Malmstrom Air Force Base, and how these approaches enabled efficient construction, despite a challenging schedule and physical constraints. By considering a wide variety of concepts at the beginning of the project, the design and construction team was able to identify the best methods and ultimately combine portions of multiple original concepts to arrive at the best solution, even after unexpected new constraints were introduced. The selected design used a superimposed concrete wale beam to support the top of the secant piles, along with buried tie rods and deadman drilled shafts, which were able to avoid all conflicts and impediments to the permanent construction, particularly the unique and challenging installation of the mechanical equipment within the basement area. Use of a portion of the permanent building and foundations provided cost and schedule savings in lieu of installing additional temporary deadman anchor shafts. The careful planning and design allowed for a smooth and successful construction process, and a system that performed as intended and met all project goals.

ACKNOWLEDGMENTS

Brierley and Condon-Johnson would like to acknowledge the contributions of the Walsh/Archer-Western Federal team, U.S. Army Corps of Engineers and Black & Veatch. The success of the project is due in large part to the strong communication channels between all parties on the project, and the results-oriented and collaborative attitude of the project team.

REFERENCES

Sketches and figures by Brierley and Condon-Johnson
Photos courtesy of Condon-Johnson and Archer-Western Federal/Walsh

GROUND IMPROVEMENT AT US 101/BROADWAY INTERCHANGE RECONSTRUCTION PROJECT

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ABSTRACT

Constructed in 1949, the overcrossing ramp at Broadway provided access to the four-lane US Highway 101 (US 101) for the City of Burlingame, California. Later, US 101 was widened to an eight-lane divided freeway, becoming a principal artery serving local and inter-regional traffic along the San Mateo Peninsula and for the greater Bay Area, and since that time, the 1949 structure has reached its capacity despite improvements in the late 1960s, 1970s and 1990s. In 2005, the San Mateo County Transportation Authority (SMCTA), formed in 1988 when voters approved a half-cent sales tax measure to fund transportation projects within San Mateo County, proposed reconstruction of the interchange at US 101 and Broadway to improve traffic movement and to accommodate future increases in traffic volume within and adjacent to the interchange. A buttonhook/diamond interchange alternative was selected and the center piece of the selected interchange alternative was the overcrossing ramp structure at Broadway. To accommodate Caltrans' freeway access ramp standards, approach embankment fills as high as 32 feet were required. Since the site is underlain by highly compressible, soft and low permeability clay, known locally as Bay Mud, long-term consolidation settlement was identified as the primary geotechnical concern at the site as this could result in downdrag loads on pile foundations, and excessive settlement of approach fills, retaining walls and existing transmission tower structure foundations. Instability of embankment slopes underlain by soft clays was of further concern, as well as potential liquefaction settlement and lateral spreading associated with a shallow groundwater table, artificial fills and loose native sands in this seismically active region. Because of limited right-of-way and the need to maintain traffic flow during construction, ground improvement methods to accelerate or eliminate consolidation settlement were evaluated, including surcharge loading and use of lightweight fill. Cement deep soil mixing (CDSM), a process whereby soil is improved by injecting cement through augers that mix the soil, forming in-place soil-cement columns, was ultimately selected since this ground improvement method would not only decrease compressibility and increase strength of the soft clays, but would also mitigate adverse effects of liquefaction and lateral spreading at the overcrossing structure. Unconfined compressive strength tests performed on CDSM column core samples collected during construction resulted in average strengths well above the specified 150 pounds per square inch. Field surcharge load tests and monitoring for areas treated to depths of 25 and 50 feet indicated essentially no post-construction consolidation settlement where the ground had been improved. This paper will review the project background and subsurface conditions encountered at site and present information pertaining to the design, load testing and construction of the CDSM ground improvement for this interchange reconstruction project.

Keywords: liequefaction, cement deep soil mixing, ground improvement

INTRODUCTION

US 101 is one of the busiest highway corridors in the San Francisco Bay Area. The original interchange overcrossing structure at Broadway in the City of Burlingame (see Fig. 1) was constructed as a 12-span cast-in-place slab bridge configured with circuitous traffic movements, substandard weaving distances, and inadequate capacity to accommodate projected traffic growth. SMCTA in cooperation with the California Department of Transportation (Caltrans) and the City of Burlingame proposed reconstruction of the US 101 Broadway Interchange in 2011. The project presented many challenges. First, because of the heavy traffic along this key corridor, all lanes were required to remain open during construction of the replacement overcrossing bridge and associated interchange improvements. Second, construction impacts to existing underground and overhead utilities in this urban metropolitan setting needed to be considered. Lastly, the presence of

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compressible and soft Bay Mud and potentially liquefiable soil underlying the bridge site presented geotechnical concerns in terms of excessive settlement and embankment instability affecting the long-term performance of the bridge structure, associated improvements and existing above and below grade utilities. Based on a cost-benefit analysis during the planning stage, the option to improve the subsurface conditions with CDSM was selected.

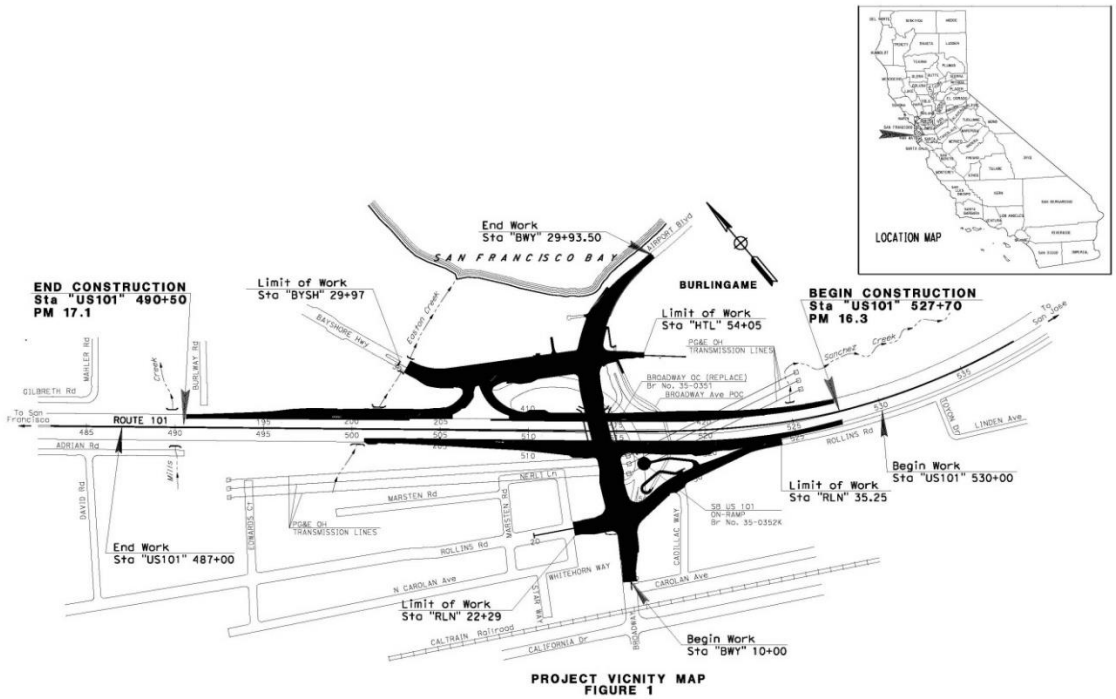


Fig. 1. Project vicinity map

SITE AND SUBSURFACE CONDITIONS

The US101/Broadway Interchange is located on the western margin of the San Francisco Bay block within the central portion of the Coast Ranges geomorphic province of California. Northwest to southeast trending valleys and ridges characterize the regional morphology of the Coast Ranges province. These topographic features are controlled by folds and faults that resulted from the collision of the Farallon and North American plates and subsequent predominantly strike-slip faulting along the San Andreas fault system between the Pacific and North American plates. The San Francisco Bay block is a relatively stable, seismic block bounded by the San Andreas and Hayward faults to the west and east, respectively.

The project site is located just east and northeast of the historic San Francisco Bay margin. The historic tidal flats have been covered with artificial fill to create the present day topography. The artificial fill covers the majority of the project area and is mapped from about Rollins Road extending north to the edge of the bay (Witter, et al, 2006; Brabb, et al, 1998). Witter (2006) shows the artificial fill as being underlain by estuarine deposits (Bay Mud). South and southwest of Rollins Road, Quaternary deposits exposed at the surface are mapped by Witter (2006) as Holocene (less than 10,000 years old) alluvial fan deposits consisting of sand, silt and clayey silt, underlain by older Pleistocene alluvial fan and fluvial deposits consisting of gravel, sand and silt. Mapping by Brabb, (1998) classifies the deposits south of Rollins Road as the Pleistocene age Colma Formation consisting of fine to medium-grained sand with silt and clay.

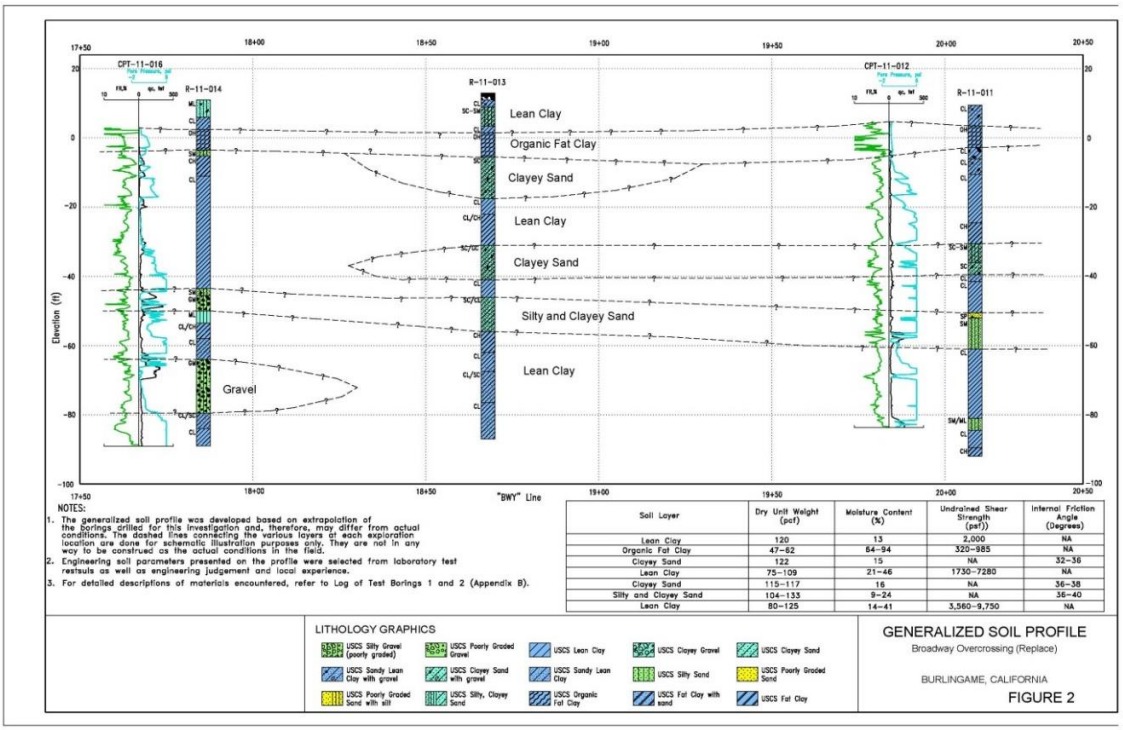


Fig 2. Generalized Soil Profile

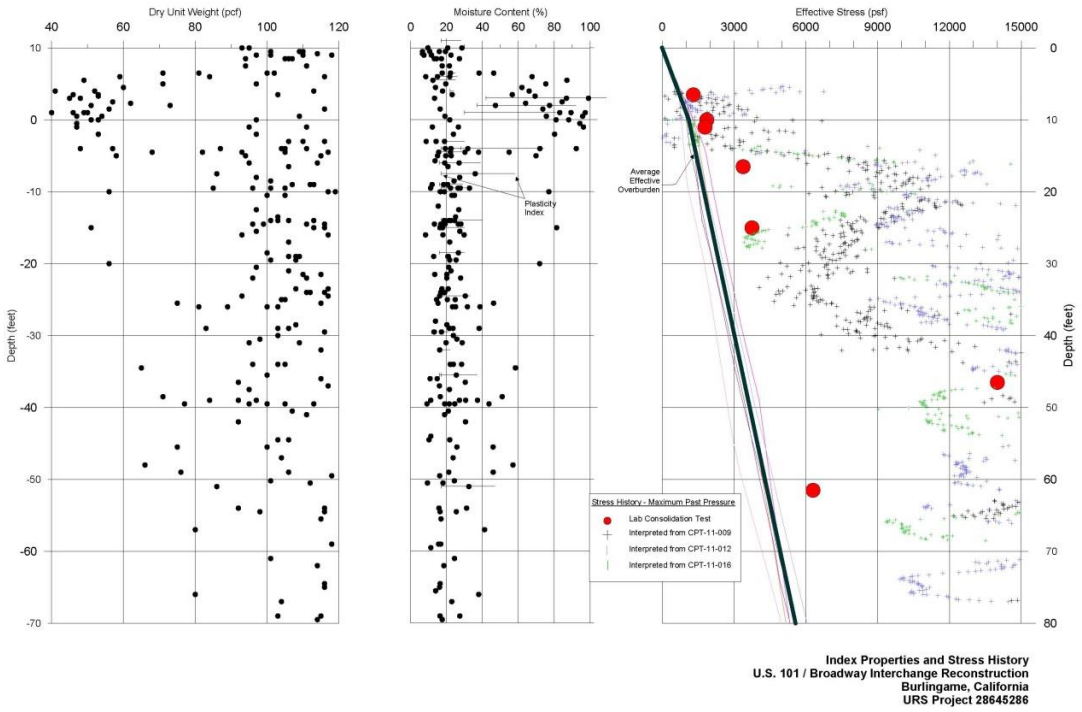


Fig. 3. Index Property Plots

Field explorations including rotary wash borings and cone penetration tests (CPTs), revealed artificial fill consisting of predominantly medium to stiff sandy lean clay with gravel and medium dense to dense silty and clayey sand with gravel to a depth of about 9 feet below ground surface (bgs), corresponding to about Elevation 3.5 feet, underlain by soft sandy lean clay with gravel to a depth of

about 11 feet bgs (Elevation 1.5 feet). Below the fill material, soft to medium stiff organic fat clay, locally known as Bay Mud, was encountered. The Bay Mud is generally 6 to 9 feet thick in the vicinity of the interchange and extends to depths of approximately 12 to 19 feet bgs (Elevation -2.5 to -5.5 feet). Loose silty sand and medium dense clayey sand were encountered directly below the Bay Mud, underlain by medium stiff to very stiff lean clay to depths of about 40 to 55 feet bgs. Alternating layers of stiff to hard lean and sandy lean clay as well as medium dense to dense silty sand and clayey sand were encountered below the stiff clay deposits to the terminal depths of the explorations, at about 100 feet. A generalized soil profile at the interchange is shown on Fig. 2. The variation of index properties with depth for the soils encountered in the explorations is presented graphically on Fig. 3.

DESIGN CONSIDERATIONS

The proposed overcrossing bridge and approaches are blanketed by about 6 to 12 feet of fill, underlain by up to 9 feet of very soft to medium stiff and highly compressible Bay Mud and occasional discontinuous layers of moderately compressible lean to fat clays. In addition, potentially liquefiable sands were encountered in many explorations near the surface at the proposed abutment and bent locations.

The principal geotechnical issues at the site were identified as:

- Excessive total and differential settlements due to consolidation of foundation soils under the weight of new approach fills ranging to 32 feet high,
- Risk of embankment slope instability if fill placed rapidly due to weak underlying Bay Mud,
- Selection of the type and depth of foundation that would be compatible with the underlying soils, and
- Construction issues associated with the proximity of new fills to existing structures and utilities.

For the bridge foundation, 16-inch square precast, prestressed (PCPS) concrete piles, vertical and batter, were selected to support the abutments, and 24-inch diameter cast-in-steel shell (CISS) piles, vertical only, were selected to support the center bent.

Elastic and long-term consolidation settlement of the Bay Mud and other clays under the weight of the proposed fills was estimated to be as much as 17 inches beneath the thickest approach fills, and on the order of about 7 to 8 inches at the abutments. Settlement of this magnitude would induce downdrag loads on the piles and potentially cause distress of approach fills and the overlying structural pavement sections. Due to the very low permeability of the Bay Mud and other native clays, the settlement period to reach 90 to 95% consolidation was estimated to be approximately 3 to 4 years.

Three alternatives for approach fill construction to mitigate consolidation settlement and soil shear strength concerns at the site were evaluated and included:

- Surcharge proposed structure backfill and embankment fill areas using wick drains to facilitate drainage and to accelerate settlement and strength gain of the underlying Bay Mud and other native clays.
- Use of very lightweight expanded polystyrene (EPS) blocks as an alternative to Caltrans standard embankment fill and structure backfill so that new loads imposed on the underlying highly compressible clays are reduced significantly, resulting in tolerable post-construction total and differential settlements at foundations.
- CDSM ground treatment of the underlying Bay Mud, potentially liquefiable granular deposits and fat clays at the abutment locations to reduce post-liquefaction and long-term consolidation settlements to tolerable levels and increase shear strength to improve embankment stability.

Based on the findings from the value engineering analysis, CDSM was selected as the most cost-effective method to improve subsurface conditions in the vicinity of the abutments and adjacent to overhead utility towers. Since no new fills were planned in close proximity to the structure bent, CDSM treatment was not required in the highway median. CDSM treatment was also necessary to

improve approach embankment stability at Abutment 3 to meet Caltrans requirements for static and seismic conditions.

CDSM ground improvement typically consists of a regular grid of in-place columns that are placed in rows (treatment zones) aligned parallel to the longitudinal axis of an embankment. The horizontal extent and pattern of the CDSM columns used near the bridge abutments are shown on the CDSM Plans, Figs. 4–5.

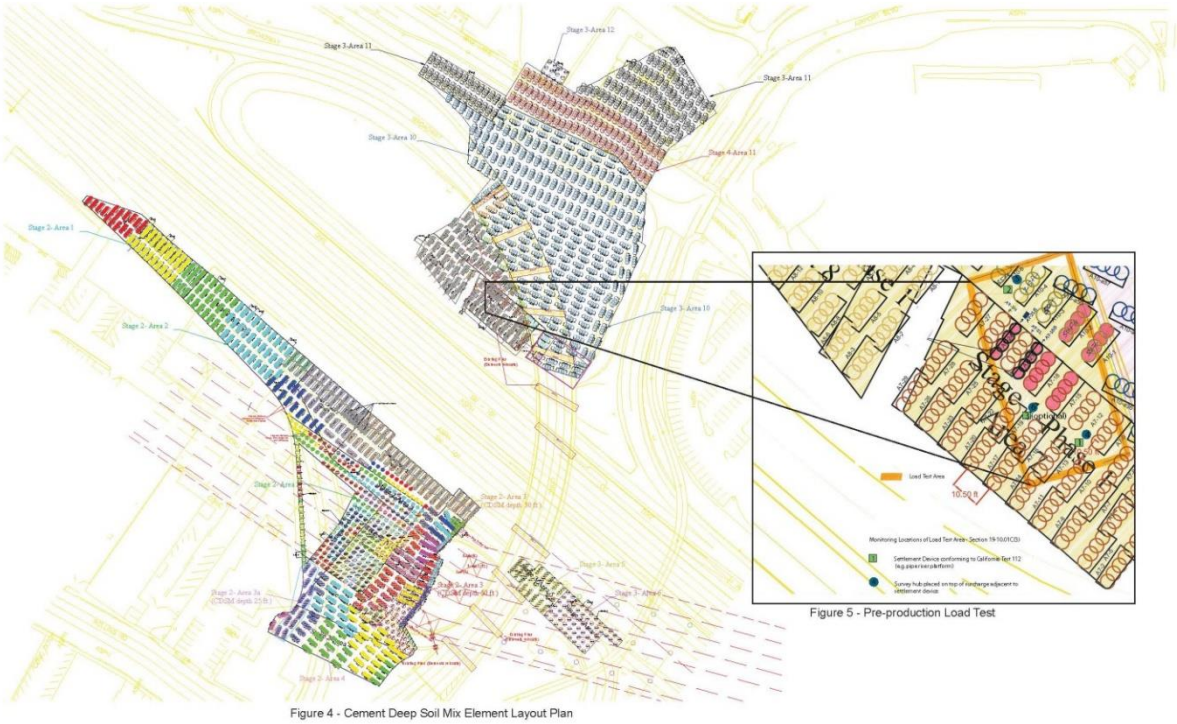


Figure 5 - Pre-production Load Test

Figure 4 - Cement Deep Soil Mix Element Layout Plan

Fig. 4. Cement deep soil mix element layout plan and Fig 5. Pre-production load test

CDSM DESIGN PARAMETERS

The analysis and design of the CDSM elements included determination of the required configuration and treatment ratio for the structure footprint, determination of a project specific field trial program to evaluate strength and settlement of the treated areas, and preparation of performance specifications for construction of CDSM elements. Based on the soil profile (Fig. 3), the primary ground improvement objective focused on minimizing the increase in effective stresses in the compressible Bay Mud and the underlying medium stiff lean clays by spreading the embankment load to a broader footprint and to a deeper, less compressible soil layer below Elevation -25 feet.

A secondary objective of CDSM improved subgrade was to minimize the potential for induced downdrag on pile foundations for the overcrossing structure at the abutments. By forming an underground CDSM column wall curtain around the abutment footings, the likelihood of negative skin friction developing around the pile foundation was greatly reduced. Therefore, pile installation could proceed without the need to implement a settlement period with surcharge.

Design of Treatment Grid

Figure 4 presents the specified configuration of the CDSM treatment grid with the corresponding area replacement ratio shown. Area replacement ratio is defined as the ratio of the CDSM column area to the tributary soil area surrounding the column.

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$$a_{s,center} = \frac{\pi d^2}{4s_{center}^2}$$

The coverage configuration for typical single auger and quadruple augers ranges from 25 percent to 33 percent. To improve the shear resistance along the edge of the approach embankment and along the edges of abutment footings, overlapping CDSM columns were oriented perpendicular to the footing or embankment edge; the column-panels act as shear keys against deep-seated slip surfaces.

Settlement Evaluation

Compression of the deep mixed zone was calculated on the basis of equal strains occurring in the deep mixed ground and the adjacent untreated soil within the deep mixed zone as follows:

$$M_{comp} = a_{s,center}E_{dm} + \left(1 - a_{s,center}\right)M_{soil}$$

The compression of the treated zone, ΔH_{dm}, was subsequently calculated using the following equation:

$$\Delta H_{dm} = H_{dm}\frac{q}{M_{comp}}$$

Using a load-spread method for driven pile groups, the compression of strata beneath the CDSM columns was computed to be less than ¼ inch. The effective, $M_{soil,}$ of the Bay Mud, and of the upper and lower lean clay units were estimated to be 0.5 and 5.6 psi, respectively. For a target total settlement of the embankment to be less than ½ inch, the E_{dm} needed to be a minimum of 1.6 x 10^6 psi, corresponding to an unconfined compressive strength of 160 psi for the CDSM column.

Pre-Production (PPC) Program

The project was advertised following Caltrans' special project guidelines and specification format. One section of the special provisions for deep mixing methods included a pre-production program by the contractor demonstrating the effectiveness of the CDSM to meet project design requirements. The pre-production program included the establishment of the following components:

Water to cement ratio

Based on supplemental explorations of subsurface clays, the Contractor established that a water to cement ratio of 100% and cement dose of 200 kg/m^3 would result in batch-mixed specimens with a minimum unconfined compressive strength of 160 psi. The production water to cement ratio was fine-tuned based on field performance of mixing efficiency and effectiveness. A ratio of 150% with the cement dose of 200 kg/m^3 was determined to be optimal for the target 28-day strength and production rate of CDSM.

Core testing

Apart from batch-mixed cylinder testing, the 28-day strength was further confirmed by testing of cores retrieved from finished CDSM elements. The thin-wall coring technique developed by the Contractor enabled the retrieval of CDSM cores with over 90 percent recovery. Strength testing results of the cores indicated over 95 percent exceedance of the specified strength of 160 psi.

Load testing

An area improved with CDSM columns (dimensions of 65 by 100 feet) was loaded with a surcharge of 3,000 psf and monitored with settlement devices (California Test Method 112). Figure 5 presents

the location of the load test area that was selected which included several types of single- and quadruple-auger CDSM elements. The measured settlement was less than one-tenth of an inch for a settlement period of 60 days and the rate of settlement became essentially negligible after two weeks.

CONCLUSIONS

Based on settlement monitoring results with the surcharge load, we estimated the overcrossing approach embankment and ramps would settle less than ¼ inch, meeting project objectives. In addition, the core test results of over 95 percent confidence level exceeding the specified minimum strength of 160 psi provided the strength necessary for embankment slopes over soft Bay Mud to meet the project minimum factors of safety. Finally, CDSM ground improvement facilitated the construction by eliminating the need for surcharge and/or the traditionally long settlement period required for embankment construction on soft and compressible subgrade. Therefore, the choice of using CDSM to improve the subsurface soils at the Broadway interchange reconstruction project was successful and appropriate.

SUMMARY

Constructing embankments on soft ground traditionally requires preloading of the subgrade to mitigate the impact of long-term consolidation settlement on deep foundation elements, pavements, and embankments, as well as potential relocation of existing underground utilities that cannot tolerate the anticipated total and differential settlements. Because of the heavy use of the traffic corridor at the US 101/Broadway interchange, this traditional approach would have been unacceptable. As presented in this paper, implementation of CDSM ground improvement eliminated the need for surcharging and a settlement period, facilitating construction and meeting project objectives.

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DYNAMIC PILE TESTING AND ANALYSIS: UTILIZING TEST RESULTS WITH A FOCUS ON CAPACITY

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ABSTRACT

Driven piles have long been one of the primary options when soil conditions require a deep foundation for structure support. Due to the unique characteristics of their installation techniques revealing soil resistance information, they have also been the topic of many studies and research investigations. Dynamic pile testing has been one of those topics for many decades and is now generally implemented on projects routinely utilizing specialized data collection equipment. Depending on the details of a particular project, the results of dynamic pile testing may be used differently by different engineers, depending on the focus of the quality control and the level of detail desired. This paper will present technical discussions on how the dynamic pile test results, in conjunction with signal matching analysis, may be utilized for pile ultimate capacity considerations including the discretized shaft resistance and end bearing components. One case history with dynamic pile test results and signal matching analysis output and is presented for discussion.

Keywords: pile testing, dynamic, capacity, shaft resistance, end bearing, signal matching

INTRODUCTION

Dynamic data from driven piles is collected with proprietary software as impact hammers advance the piles through substrata. Although this sounds simplistic, it should be noted that the piles may be of many different types (e.g., steel, concrete, timber, etc.), shapes (e.g., pipes, H-piles, cylinders, square, octagonal, solid round sections, etc.), sizes (from less than a foot to many feet in diameter and with lengths from the tens of feet to hundreds of feet), and similarly have a wide range of load carrying requirements. The piles may even be composite in nature but should have enough known details to later model the entire driving system as a series of masses, springs, and dashpots.

Note that the "dynamic data" are collected near the pile top typically from reusable accelerometers and strain gages. It may also be noted that the hammers similarly come in several different types (e.g., hydraulic, diesel, air/steam powered, all which may be single-acting or double-acting, as well as drop hammers) and sizes vary somewhat proportionally to the pile sizes and their load carrying requirements.

The substrata may consist of rock, sand, silt, clay, or some combination of these geo-materials. Despite all these potential variations, the dynamic data collected and their documented output results are often the crucial pieces of information that allow for a rational assessment of the actual, in-place conditions. Perhaps the most important information will be the resistance these geo-materials provide as a reaction to the hammer impacts.

DESIGN OF A TEST PILE PROGRAM

Typically, test piles are the first piles to be driven for the construction of a foundation unit (i.e., a Pier, Bent, Abutment, etc.). The pile designer allows for several feet of additional pile length on test piles both to explore the potential bearing layer(s) to deeper than anticipated depths and also to account for strain gages and accelerometers mounted on the pile (typically 2 diameters below the pile top) not being driven below soil or other obstructions.

It should be noted that the dynamic testing results will be presented as ultimate values of nominal resistance (i.e., safety factors of 1.0) so that it is clear the results are not design values and the appropriate safety factors or phi factors will need to be applied at the discretion of the design engineer. In fact, it is the design engineer's responsibility to make sure the appropriate and up-to-date

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code and specifications are utilized. Recommended factors of safety in the US may range from 1.9 to 6.0 for ASD design. For LRFD design, the sum of the factored loads must be less than the nominal resistance, R_n, multiplied by a resistance factor, ϕ.

$$\Sigma\eta_i\gamma_i Q_i \leq \phi R_n$$

The 2020 AASHTO LRFD design specifications recommend the following resistance factors, ϕ_{dyn}, be applied to the nominal resistance based on the selected construction control procedures.

- ϕ_{dyn} = 0.80 for driving criteria established by static load test of 1 pile per site condition and dynamic testing with signal matching of at least 2 piles per site condition but no less than 2% of production piles.
- ϕ_{dyn} = 0.75 for driving criteria established by successful static load test of 1 pile per site condition without dynamic testing.
- ϕ_{dyn} = 0.75 for driving criteria established by dynamic testing with signal matching conducted on 100% of production piles.
- ϕ_{dyn} = 0.65 for driving criteria developed by dynamic testing with signal matching, quality control by dynamic testing on 2 piles per site condition, but no less than 2% of production piles.
- ϕ_{dyn} = 0.50 for wave equation analysis without dynamic measurements or load test but with field confirmation of hammer performance.
- ϕ_{dyn} = 0.40 for FHWA modified Gates dynamic formula (end of drive condition only).
- ϕ_{dyn} = 0.10 for Engineering News dynamic formula as defined in AASHTO 10.7.3.8.5 (end of drive conditions only).

As seen in the above example for LRFD design, the governing Code for a project may dictate or give guidance to how many dynamically tested test piles are required, and there may be advantages (i.e., higher phi factors or lower required ultimate pile capacities) to testing more than the mandatory requirements, such as testing 100% of the piles for some projects. This may be advantageous when high design loads are difficult to obtain. Conversely, codes may allow for only wave equation analysis with a penalty of lower phi factors (or equivalently higher ultimate required capacities per pile), which is understandable since the soil properties are not obtained through testing. Although there may be cases for this approach, such as when soils are expected to provide pile capacities substantially greater than needed, it has the potential to risk insufficient foundation support and should generally be avoided. In any case, project documents should clearly indicate what phi factors (or factors of safety) and levels of testing are required for a given project.

The state-of-the-art method is typically that for a given test pile, a set number of production piles or foundation elements will rely on those results for their installation criteria.

OVERVIEW OF TEST PILE RESULTS

Hammer efficiency is often assumed to be in a particular range when deciding what size of hammer is appropriate for a project. However, once a hammer is ready to be operated, the dynamic pile testing results will be telling, and the need to assume is over. The dynamic pile testing results indicate the actual hammer efficiency during the test pile program. This important benchmark is set, although the hammer efficiency may change prior to finishing the production piles, and changes in hammer operation and performance should continue to be observed and documented.

For some projects, overstressing the piles during driving may be the greatest concern. The dynamic pile test results indicate the driving stresses throughout testing. In addition to the compression stress at the gage location on the pile, other stresses of concern may be the compression stress calculated at the pile toe, and the calculated maximum tension stress in concrete piles. The good news is that there are usually ways to control these stresses, either by reducing or limiting the hammer energy, or by increasing the pile cushion thickness for concrete piles. It should however be noted that maximum stress levels from stress concentrations are beyond the capability of dynamic pile testing, and results are presented as averages over the entire cross-sectional pile area.

For most projects, the pile ultimate axial compressive capacity is the quantity of most interest. This quantity is best obtained from signal matching analysis performed at the depth of interest to reveal the proper soil quake, damping and the separation of the capacity between shaft resistance and end bearing. Another item of high interest is the distribution of soil resistance. In order to observe how these resistances may change with time, restrike testing is performed, and signal matching analysis performed on data at the beginning of restrike can be compared to signal matching analysis performed on data at the end of initial driving.

CASE HISTORY

As an example, we will consider a steel H-pile (HP14X117 section, ASTM A572 Grade 50) approximately 125 feet in length, driven and tested, using an APE D30-52 single-acting diesel hammer. At a pile penetration depth of approximately 76 feet, a thick bearing layer of hard elastic silt with sand is encountered, and pile driving continues through this layer to the end of driving at depth of approximately 110 feet (the soil above this is a sand and gravel mix).

Below are the dynamic pile testing results from the initial driving, including the calculated closed form Case Method Capacity, RMX. Note that 504 kips is the target capacity.

GRL Engineers, Inc. Page 1
PDIPLOT2 2021.1.61.0 Printed 09-September-2025

Case Method & iCAP® Results

Bridge Y - Abutment 4 Pile A_S2 APE D30-52 Hammer
OP: mqb-grl Date: 08-November-2023

AR: 34.40 in² SP: 0.492 k/ft³
LE: 121.00 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.90

CSX: Compression Stress Maximum
TSX: Tension Stress Max-Full Rec Search
CSB: Compression Stress at Bottom of Pile
STK: Hammer Stroke
EMX: Maximum Energy
FMX: Maximum Force
VMX: Maximum Velocity
DMX: Maximum Displacement
RMX: Maximum Case Method Capacity (JC)

BL#	Depth ft	TYPE	CSX ksi	TSX ksi	CSB ksi	STK ft	EMX k-ft	FMX kips	VMX f/s	DMX in	RMX kips
2	61.00	AV2	23.0	13.1	8.7	9.64	38.7	791	18.0	6.00	234
4	62.00	AV2	24.8	15.0	8.4	7.56	39.0	852	20.3	6.00	231
10	63.00	AV6	25.7	16.0	8.4	7.81	38.1	883	21.2	2.09	274
17	64.00	AV7	25.7	16.4	7.9	7.82	35.6	885	21.2	1.76	266
24	65.00	AV7	25.3	16.6	6.6	7.57	34.9	869	21.4	1.93	223
32	66.00	AV8	24.9	16.8	6.1	7.41	35.3	857	21.4	2.10	209
39	67.00	AV7	25.8	16.4	8.1	7.75	36.2	887	21.2	1.96	249
46	68.00	AV7	26.3	17.2	7.8	7.92	38.3	903	22.4	2.03	239
54	69.00	AV8	26.3	17.2	8.3	7.93	38.5	904	22.3	1.87	245
61	70.00	AV7	26.7	17.3	8.5	8.09	38.7	919	22.5	1.85	263
69	71.00	AV8	26.4	17.2	7.3	7.96	35.4	908	22.0	1.58	257
79	72.00	AV10	26.3	17.0	7.7	7.91	35.6	906	21.4	1.53	269
93	73.00	AV14	27.3	15.7	8.5	8.32	36.1	939	19.3	1.27	338
110	74.00	AV17	27.6	16.2	9.3	8.47	36.9	951	19.8	1.25	341
123	75.00	AV13	27.7	16.7	8.8	8.45	36.1	952	20.0	1.20	331
134	76.00	AV11	27.9	17.4	8.8	8.55	36.8	961	20.5	1.27	331
145	77.00	AV11	28.0	17.4	9.0	8.61	36.0	964	20.5	1.24	340
155	78.00	AV10	28.1	18.3	7.6	8.65	38.9	968	21.5	1.41	308
167	79.00	AV12	28.3	16.9	8.7	8.75	37.6	973	19.6	1.20	315
181	80.00	AV14	28.7	14.7	10.7	8.93	37.0	988	17.0	1.01	347
197	81.00	AV16	28.7	13.5	11.2	8.98	37.1	987	16.3	1.01	345
209	82.00	AV12	28.5	12.8	11.3	8.92	37.4	980	15.9	1.04	324
223	83.00	AV14	28.5	12.1	12.0	8.92	38.0	979	16.1	1.07	324

Case Method & iCAP® Results

Bridge Y - Abutment 4 Pile A_S2
OP: mqb-grl

APE D30-52 Hammer
Date: 08-November-2023

BL#	Depth ft	TYPE	CSX ksi	TSX ksi	CSB ksi	STK ft	EMX k-ft	FMX kips	VMX f/s	DMX in	RMX kips
238	84.00	AV15	28.6	10.9	12.5	8.97	38.3	983	16.0	1.04	322
254	85.00	AV16	28.4	9.6	13.2	8.91	38.0	977	15.9	1.03	316
271	86.00	AV17	28.5	8.8	13.5	8.97	38.0	981	16.0	1.01	317
288	87.00	AV17	28.4	8.0	14.0	8.97	37.4	978	15.9	0.99	324
307	88.00	AV19	28.7	7.1	14.4	9.14	38.1	988	16.1	0.98	339
327	89.00	AV20	28.7	6.5	14.7	9.17	37.7	989	16.2	0.97	341
350	90.00	AV23	28.7	5.6	15.0	9.26	37.6	987	16.1	0.93	360
373	91.00	AV23	28.7	4.9	15.2	9.29	38.0	988	16.1	0.93	373
397	92.00	AV24	29.0	4.1	15.9	9.43	38.7	999	16.3	0.91	390
423	93.00	AV26	29.3	3.9	16.6	9.52	39.1	1,006	16.4	0.88	406
453	94.00	AV30	29.3	4.1	17.0	9.50	39.2	1,007	16.4	0.87	426
485	95.00	AV32	29.4	4.4	17.8	9.56	39.7	1,013	16.5	0.85	443
517	96.00	AV32	29.7	4.3	18.7	9.71	40.4	1,020	16.7	0.85	463
552	97.00	AV35	29.8	4.2	19.3	9.75	40.9	1,027	16.8	0.85	483
588	98.00	AV36	29.9	4.4	19.4	9.74	40.9	1,027	16.7	0.85	493
628	99.00	AV40	30.0	4.7	20.1	9.74	41.2	1,030	16.9	0.85	506
668	100.00	AV40	30.0	5.1	20.4	9.72	41.4	1,030	16.9	0.86	516
707	101.00	AV39	30.2	5.4	20.6	9.85	42.3	1,039	17.0	0.87	528
750	102.00	AV43	30.3	5.7	20.8	9.93	42.7	1,044	17.1	0.87	541
797	103.00	AV47	30.3	5.9	21.2	9.92	42.7	1,043	17.1	0.87	549
847	104.00	AV50	30.4	6.2	21.8	9.99	43.1	1,047	17.2	0.87	567
902	105.00	AV55	30.4	6.4	22.3	10.03	42.7	1,044	17.1	0.86	578
963	106.00	AV61	30.4	6.6	23.0	10.13	42.6	1,046	17.1	0.86	599
1033	107.00	AV70	30.5	7.0	23.8	10.30	43.2	1,050	17.2	0.86	628
1110	108.00	AV77	30.7	7.3	24.5	10.46	43.5	1,056	17.4	0.86	655
1197	109.00	AV87	30.6	7.9	25.1	10.48	43.0	1,054	17.3	0.84	680

Case Method & iCAP® Results

Bridge Y - Abutment 4 Pile A_S2
OP: mqb-grl

APE D30-52 Hammer
Date: 08-November-2023

BL#	Depth ft	TYPE	CSX ksi	TSX ksi	CSB ksi	STK ft	EMX k-ft	FMX kips	VMX f/s	DMX in	RMX kips
1303	110.00	AV106	30.6	8.5	25.9	10.56	42.9	1,054	17.3	0.84	712

BL# Sensors

1-1303 F5: [S364] 91.2 (1.00); F6: [K441] 94.0 (1.00); A7: [K11438] 370.3 (1.00); A8: [K11432] 388.9 (1.00)

Time Summary

Drive 34 minutes 7 seconds 11:13 AM - 11:47 AM BN 1 - 1303

The calculated capacity results above show that over the final foot of driving; the average capacity was 712 kips. The Case Method Capacity is a closed-form solution allowing capacity calculation for

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each hammer blow. However, it requires an input value for the Case damping factor, which was 0.9 in this case. Data from the end of driving was analyzed with the signal matching program to obtain this value. Below are the signal matching analysis results from the end of driving.

Bridge Y; Pile: Abutment 4 Pile A_S2 — Test: 08-Nov-2023 11:47
APE D30-52 Hammer ; Blow: 1302 — CAPWAP(R) 2014-3
GRL Engineers, Inc. — OP: mgb-grl

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 710.0; along Shaft 505.0; at Toe 205.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				710.0				
1	20.2	9.2	0.0	710.0	0.0	0.00	0.00	0.00
2	26.9	15.9	1.0	709.0	1.0	0.15	0.03	0.29
3	33.6	22.6	1.0	708.0	2.0	0.15	0.03	0.29
4	40.3	29.3	1.0	707.0	3.0	0.15	0.03	0.29
5	47.1	36.0	1.0	706.0	4.0	0.15	0.03	0.29
6	53.8	42.8	4.0	702.0	8.0	0.60	0.12	0.29
7	60.5	49.5	4.0	698.0	12.0	0.60	0.12	0.29
8	67.2	56.2	4.0	694.0	16.0	0.60	0.12	0.29
9	73.9	62.9	4.0	690.0	20.0	0.60	0.12	0.29
10	80.7	69.7	4.0	686.0	24.0	0.60	0.12	0.29
11	87.4	76.4	35.0	651.0	59.0	5.21	1.07	0.29
12	94.1	83.1	46.0	605.0	105.0	6.84	1.41	0.29
13	100.8	89.8	100.0	505.0	205.0	14.88	3.07	0.29
14	107.6	96.5	100.0	405.0	305.0	14.88	3.07	0.29
15	114.3	103.3	100.0	305.0	405.0	14.88	3.07	0.29
16	121.0	110.0	100.0	205.0	505.0	14.88	3.07	0.29
Avg. Shaft			31.6			4.59	0.95	0.29
Toe			205.0				139.56	0.19

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.08	0.12
Case Damping Factor		2.38	0.62
Damping Type		Viscous	Sm+Visc
Unloading Quake	(% of loading quake)	30	101
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	79	
Resistance Gap (included in Toe Quake)	(in)		0.00
Soil Plug Weight	(kips)		0.063

CAPWAP match quality = 1.56 (Wave Up Match) ; RSA = 0
Observed: Final Set = 0.11 in; Blow Count = 106 b/ft
Computed: Final Set = 0.11 in; Blow Count = 106 b/ft
Transducer F5 (S364) CAL: 91.2; RF: 1.00; F6 (K441) CAL: 94.0; RF: 1.00
A7 (K11438) CAL: 370; RF: 1.00; A8 (K11432) CAL: 389; RF: 1.00

max. Top Comp. Stress = 29.5 ksi (T= 36.2 ms, max= 1.095 x Top)
max. Comp. Stress = 32.2 ksi (Z= 87.4 ft, T= 41.4 ms)
max. Tens. Stress = -6.45 ksi (Z= 87.4 ft, T= 61.6 ms)
max. Energy (EMX) = 40.1 kip-ft; max. Measured Top Displ. (DMX)= 0.82 in

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1317.0	1247.1	1177.2	1107.3	1037.4	967.5	897.7	827.8	757.9	688.0
RX	1317.0	1247.1	1177.2	1107.3	1037.4	967.5	897.7	827.8	757.9	696.7
RU	1358.5	1292.8	1227.1	1161.3	1095.6	1029.9	964.1	898.4	832.7	766.9

RAU = 0.0 (kips); RA2 = 485.9 (kips)

Current CAPWAP Ru = 710.0 (kips); Corresponding J(RP)= 0.87; J(RX) = 0.88

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS	KEB
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips	kips/in
16.4	36.00	1007.5	1008.4	1008.4	0.82	0.12	0.11	40.2	1033.8	1696

PILE PROFILE AND PILE MODEL

Depth ft	Area in²	E-Modulus ksi	Spec. Weight lb/ft³	Perim. ft
0.0	34.4	30000.0	492.000	4.85
121.0	34.4	30000.0	492.000	4.85

Toe Area 211.5 in²

Top Segment Length 3.36 ft, Top Impedance 61 kips/ft/s

Wave Speed: Pile Top 16807.9, Elastic 16807.9, Overall 16807.9 ft/s
Pile Damping 1.00 %, Time Incr 0.200 ms, 2L/c 14.4 ms
Total volume: 28.906 ft³; Volume ratio considering added impedance: 1.000

Note that the end of driving signal matching calculated capacity is 710 kips, with 505 kips along the pile shaft and 205 kips at the pile toe. The bottom approximately 20 feet of the pile has a unit shaft resistance of 3.07 kips per square foot.

The results for dynamic pile testing are detailed; however, these are only valid for the time of testing. Restrike testing after some waiting period provides a better indication of what the long-term values may be. For this pile, a rather short-term restrike test was performed, which indicated significant set-up at this location. Note that the end of drive blow count was 106 blows/ft at an average hammer ram stroke height of 10.6 feet. For the 1.25-hour restrike test the blow count was 30 blows for 0.75-inches at an average hammer ram stroke height of 11.1 feet. The results for this restrike test are presented below.

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Bridge Y - Abutment 4 Pile A_S2_1st restrike — APE D30-52 Hammer
OP: mqb-qrl — Date: 08-November-2023

AR: 34.40 in² — SP: 0.492 k/ft³
LE: 121.00 ft — EM: 30,000 ksi
WS: 16,807.9 f/s — JC: 0.90

CSX: Compression Stress Maximum
TSX: Tension Stress Max-Full Rec Search
CSB: Compression Stress at Bottom of Pile
STK: Hammer Stroke
EMX: Maximum Energy
FMX: Maximum Force
VMX: Maximum Velocity
DMX: Maximum Displacement
RMX: Maximum Case Method Capacity (JC)

BL#	Depth ft	TYPE	CSX ksi	TSX ksi	CSB ksi	STK ft	EMX k-ft	FMX kips	VMX f/s	DMX in	RMX kips
1	110.00	AV1	20.5	7.7	19.3	**	22.1	705	11.1	0.66	655
2	110.00	AV1	33.2	13.6	31.6	11.78	49.1	1,143	18.8	0.89	965
3	110.01	AV1	31.7	13.4	30.3	10.92	45.0	1,089	18.3	0.87	939
4	110.01	AV1	32.4	13.7	30.8	11.16	46.7	1,116	18.5	0.87	955
5	110.01	AV1	33.2	14.0	31.6	11.75	50.3	1,143	18.9	0.92	997
6	110.01	AV1	33.4	14.5	31.8	11.74	51.0	1,149	19.1	0.93	1,000
7	110.01	AV1	33.1	13.6	31.5	11.55	49.2	1,137	18.9	0.91	982
8	110.02	AV1	32.4	13.8	30.8	11.21	47.5	1,114	18.6	0.91	958
9	110.02	AV1	32.4	13.9	30.7	11.03	47.3	1,115	18.6	0.90	958
10	110.02	AV1	32.1	13.8	30.5	10.95	47.0	1,103	18.5	0.91	955
11	110.02	AV1	32.8	13.9	31.0	11.18	48.4	1,128	18.8	0.91	963
12	110.03	AV1	31.9	13.6	30.3	10.86	46.6	1,098	18.4	0.90	939
13	110.03	AV1	32.1	13.7	30.5	10.88	46.5	1,105	18.5	0.88	948
14	110.03	AV1	32.2	13.8	30.4	10.98	47.8	1,109	18.5	0.92	953
15	110.03	AV1	32.8	13.9	30.9	11.28	48.2	1,129	18.9	0.90	967
16	110.03	AV1	32.3	13.6	30.5	10.92	47.4	1,110	18.6	0.90	947
17	110.04	AV1	32.7	13.8	30.8	11.18	48.9	1,125	18.9	0.93	953
18	110.04	AV1	32.3	13.7	30.5	10.94	47.0	1,110	18.7	0.91	944
19	110.04	AV1	31.4	13.3	29.4	10.52	44.9	1,081	17.9	0.86	928
20	110.04	AV1	32.5	13.6	30.5	11.14	47.7	1,118	18.7	0.89	946
21	110.04	AV1	31.6	13.2	29.4	10.59	45.3	1,088	18.0	0.88	919
22	110.05	AV1	31.7	13.2	29.5	10.68	45.0	1,090	18.0	0.85	923
23	110.05	AV1	32.3	13.4	30.0	10.91	47.1	1,110	18.5	0.90	934
24	110.05	AV1	32.1	13.5	30.1	10.85	47.0	1,104	18.4	0.89	931
25	110.05	AV1	32.0	13.4	29.3	10.86	46.8	1,101	18.4	0.90	929
26	110.05	AV1	32.5	13.6	29.7	11.08	48.1	1,119	18.6	0.90	941
27	110.06	AV1	32.3	13.5	29.5	10.92	47.9	1,110	18.5	0.93	928
28	110.06	AV1	32.5	13.7	29.9	11.04	48.3	1,118	18.5	0.91	944
29	110.06	AV1	32.4	13.5	30.0	10.98	47.5	1,115	18.5	0.88	937
30	110.06	AV1	32.2	13.5	30.0	10.89	47.9	1,107	18.5	0.93	927
		Average	32.0	13.4	30.0	11.06	46.7	1,100	18.3	0.89	939

Total number of blows analyzed: 30

BL# Sensors

1-30 F5: [S364] 91.2 (1.00); F6: [K441] 94.0 (1.00); A7: [K11438] 370.3 (1.00); A8: [K11432] 388.9 (1.00)

BL# Comments

30 30 bl/0.75 inches

The signal matching analysis was performed on data from the 6th hammer blow and indicated a mobilized capacity of 1,000 kips with a significant increase in the shaft friction from the end of initial drive. The restrike signal matching analysis is presented below.

Bridge Y; Pile: Abutment 4 Pile A_S2_1st restrike — Test: 08-Nov-2023 13:03
APE D30-52 Hammer ; Blow: 6 — CAPWAP(R) 2014-3
GRL Engineers, Inc. — OP: mgb-grl

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 1000.0; along Shaft 795.0; at Toe 205.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				1000.0				
1	20.2	9.2	0.0	1000.0	0.0	0.00	0.00	0.00
2	26.9	15.9	1.0	999.0	1.0	0.15	0.03	0.21
3	33.6	22.6	2.0	997.0	3.0	0.30	0.06	0.21
4	40.3	29.3	3.0	994.0	6.0	0.45	0.09	0.21
5	47.1	36.1	4.0	990.0	10.0	0.60	0.12	0.21
6	53.8	42.8	4.0	986.0	14.0	0.60	0.12	0.21
7	60.5	49.5	4.0	982.0	18.0	0.60	0.12	0.21
8	67.2	56.2	4.0	978.0	22.0	0.60	0.12	0.21
9	73.9	63.0	4.0	974.0	26.0	0.60	0.12	0.21
10	80.7	69.7	4.0	970.0	30.0	0.60	0.12	0.21
11	87.4	76.4	40.0	930.0	70.0	5.95	1.23	0.21
12	94.1	83.1	145.0	785.0	215.0	21.57	4.45	0.21
13	100.8	89.8	145.0	640.0	360.0	21.57	4.45	0.21
14	107.6	96.6	145.0	495.0	505.0	21.57	4.45	0.21
15	114.3	103.3	145.0	350.0	650.0	21.57	4.45	0.21
16	121.0	110.0	145.0	205.0	795.0	21.57	4.45	0.21
Avg. Shaft			49.7			7.23	1.49	0.21
Toe			205.0				139.56	0.40

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.08	0.06
Case Damping Factor		2.71	1.34
Damping Type		Viscous	Viscous
Unloading Quake	(% of loading quake)	74	44
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	56	
Soil Plug Weight	(kips)		1.185

CAPWAP match quality = 3.06 (Wave Up Match) ; RSA = 0
Observed: Final Set = 0.03 in; Blow Count = 480 b/ft
Computed: Final Set = 0.03 in; Blow Count = 477 b/ft
Transducer F5 (S364) CAL: 91.2; RF: 1.00; F6 (K441) CAL: 94.0; RF: 1.00
A7 (K11438) CAL: 370; RF: 1.00; A8 (K11432) CAL: 389; RF: 1.00

max. Top Comp. Stress = 33.9 ksi (T= 36.2 ms, max= 1.158 x Top)
max. Comp. Stress = 39.3 ksi (Z= 94.1 ft, T= 41.8 ms)
max. Tens. Stress = -10.60 ksi (Z= 87.4 ft, T= 60.4 ms)
max. Energy (EMX) = 50.5 kip-ft; max. Measured Top Displ. (DMX)= 0.93 in

Bridge Y; Pile: Abutment 4 Pile A_S2_1st restrike — Test: 08-Nov-2023 13:03
APE D30-52 Hammer ; Blow: 6 — CAPWAP(R) 2014-3
GRL Engineers, Inc. — OP: mgb-grl

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1627.1	1557.4	1487.7	1417.9	1348.2	1278.4	1208.7	1139.0	1069.2	999.5
RX	1627.1	1557.4	1487.7	1417.9	1348.2	1278.4	1208.7	1139.0	1069.2	999.5
RU	1668.8	1603.2	1537.6	1472.1	1406.5	1340.9	1275.4	1209.8	1144.2	1078.6

RAU = 414.7 (kips); RA2 = 533.2 (kips)

Current CAPWAP Ru = 1000.0 (kips); Corresponding J(RP)= 0.90; J(RX) = 0.90

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS	KEB
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips	kips/in
19.1	36.00	1175.0	1149.5	1149.5	0.93	0.03	0.03	51.0	1282.7	3417

PILE PROFILE AND PILE MODEL

Depth ft	Area in²	E-Modulus ksi	Spec. Weight lb/ft³	Perim. ft
0.0	34.4	30000.0	492.000	4.85
121.0	34.4	30000.0	492.000	4.85

Toe Area 211.5 in²

Top Segment Length 3.36 ft, Top Impedance 61 kips/ft/s

Wave Speed: Pile Top 16807.9, Elastic 16807.9, Overall 16807.9 ft/s
Pile Damping 1.00 %, Time Incr 0.200 ms, 2L/c 14.4 ms
Total volume: 28.906 ft³; Volume ratio considering added impedance: 1.000

Note that the unit shaft resistance over the bottom portion of the pile has increased to 4.45 kips per square foot. The shaft resistance overall has increased by approximately 290 kips.

An important point here is that the total mobilized resistance minus the dynamic portion of the resistance equals the calculated static resistance. For this restrike, the total pile set was less than 0.03 inches per hammer blow. In general, any testing event with a pile set less than 0.1 has likely not fully mobilized the soil's available resistance. This does not negate the test, but rather it indicates that a lower-bound of the available resistance has been observed. This may be analogous to a static test performed to the hydraulic jack limit, but the pile does not fail (i.e., it is clear the pile is good for the jack's limit, but it is not clear how much additional capacity is available). For this restrike test, the pile indeed has at least 1,000 kips capacity, but even though it is likely to have somewhat more, the additional value is not known.

CONCLUSION

Although many topics were covered in this short presentation, the main takeaway should be the value of having the documented dynamic pile test results, as well as the complementary signal matching analysis results. The signal matching analysis is performed on dynamic testing data from a selected hammer blow. The results provide a total static capacity of the pile, separates the portion from the end bearing and from the shaft resistance, and gives a distribution of the shaft resistance. Other soil characteristics such as the damping values and quakes are also obtained.

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ABOVE AND BELOW: NAVIGATING SUBSURFACE SURPRISES IN REMOTE, HELICOPTER ACCESS BUILDS

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ABSTRACT

This paper highlights a construction project which encountered multiple schedule delays and foundation construction sites where actual geotechnical conditions encountered during construction differed from those anticipated during the bid phase and detailed design. These foundations were constructed using helicopter access to support high-voltage transmission line structures across extremely steep terrain in Northern California.

During the bid phase, site-specific geotechnical data was unavailable. Instead, a desktop study was used to develop assumed subsurface conditions, with shallow bedrock presumed for design purposes. However, during construction, bedrock was not encountered at multiple construction sites. The schedule delays and unexpected change of conditions required the contractor to adjust resources and develop alternative foundation designs and construction methods to adapt to the actual conditions and maintain the project schedule.

Keywords: micropiles, remote construction, transmission line construction, helicopter access

PROJECT INTRODUCTION

This project included design-build micropile foundations supporting eleven (11) H-Frame and four (4) 3-Pole transmission structures along a portion of a 115 kV rebuild in the Feather River Canyon in Northern California. The existing transmission line and wood support pole structures were damaged in the 2021 Dixie Fire and required replacement. Steel transmission structures were selected to replace the existing wood support structures due to their inherent longevity and fire hardening characteristics.



Fig. 1. Steep and rugged construction sites along the alignment

The structures were located on an approximate 1 mile segment of the transmission line with more than 2,800 foot elevation gain and no access to conventional construction equipment, requiring helicopter supported foundation construction and structure erection. A Black Hawk helicopter with an approximate 6,000 lb lift capacity and a Bell 407 helicopter with an approximate 1,500 lb lift capacity,

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both with a 45 minute fuel window when accounting for elevation and ambient temperature, were used to transport all construction equipment and materials to the individual construction sites. The steep and rugged terrain proved difficult for personnel to hike in and out of construction sites and Human External Cargo methods were used to safely transport personnel on and off the hillside daily.

Challenges for construction personnel included steep and rugged work sites (see Fig. 1), prolonged exposure to sun and hot temperatures, rattlesnakes, an abundance of poison oak, and constant fire danger.

COMPETETIVE DESIGN-BUILD PROCESS

The competitive design-build process is a procurement method used in construction and engineering projects where both design and construction services are contracted through a single entity. This process is especially common in public sector projects and large-scale private developments where it can provide the owner with a single point of responsibility for design and construction and cost and schedule certainty.

The process typically begins when the owner issues a Request for Proposal (RFP) asking contractors for detailed submissions that include design concepts, technical approaches, schedules, and cost estimates. These proposals are then evaluated based on criteria such as innovation, value, and team experience. After thorough review, the owner typically selects the winning team based on best overall value, not just lowest cost, and awards the contract. The selected team then proceeds with finalizing the design and executing construction, often in a fast-tracked manner to save time and reduce costs.

In the design-build process, the contractor typically owns geotechnical design and construction risk, therefore proposal pricing may vary proportional to the level of perceived geotechnical risk. In other words, the contractor may carry less risk and may be able to offer lower pricing where detailed geotechnical data is available and where site observations can be made at or near each individual structure location. Conversely, the contractor may carry more risk, and pricing may be higher where there is uncertainty in geotechnical conditions or observations cannot be made at individual structure locations. Nonetheless, contractors that can offer competitive pricing often increase their chances of winning a seat at the contracting table.

GEOTECHNICAL CONDITIONS

Included with the RFP data for this project was a preliminary alignment .kmz file and a geotechnical desktop study focused primarily on the anticipated depth to bedrock at the planned structure locations. Because of the steepness and ruggedness of this section of the transmission alignment, detailed geotechnical and geophysical data, such as borings or seismic refraction readings, were not obtained by the owner. Bidding contractors were invited to an in-person site visit as part of the RFP phase, however, none of the structure locations were safely accessible by foot, therefore site-specific observations could not be made by the contractors.

In order to develop bid level foundation designs, the contractor performed an independent geotechnical desktop study to estimate the conditions likely to be encountered at the structure locations. As part of this study, the contractor referenced the owner-provided geotechnical desktop study, publicly available geologic mapping, and Google Earth satellite imagery. Notable indications of the expected geotechnical conditions from the references were:

1. The owner-provided desktop study indicated silty sand underlain by Monzotonalite bedrock was expected at all structure locations. In the desktop study, a depth to bedrock table (see Table 1) was provided which identified variable bedrock depth across the alignment, but generally expected to be encountered shallow (2.9 feet below ground surface or shallower where noted in the table). Where depth to bedrock was not specifically listed at a structure location, field observation notes included in the table generally indicated intact hard rock exposed at or near surface, large boulders at surface, rock outcrop with jointed boulders, etc.

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2. Geologic mapping indicated the primary bedrock at the structure locations is Monzotonalite, a bedrock similar to granite consisting of coarse-grained hornblende-biotite-quartz-plagioclase rock with potassium feldspar. This is a good bedrock for micropile foundations which generally exhibits high grout-to-ground bond strength.
3. Google Earth satellite imagery generally confirmed the field observations made in the owner-provided desktop study with observable significant presence of surface rock and large boulders.

Table 1. Sampling of owner-provided depth to bedrock table with problematic structure sites highlighted

Design Structure Number	Exposed Rock	Geologic Unit	USCS Classification from Soil Survey	Depth To Rock (ft) from soil survey	Field Observations
4	No	Monzotonalite (Unit KJmt)		-	Intact rock outcrop. Shallow embed okay.
5	No	Monzotonalite (Unit KJmt)		2.0	Loose boulders with shallow rock. Shallow embed likely okay.
7	No	Monzotonalite (Unit KJmt)	SM	2.9	Soil. No intact rock. Shallow embedment not recommended.
8	No	Monzotonalite (Unit KJmt)	SM	2.9	Soil. No intact rock. Shallow embedment not recommended.
9	No	Monzotonalite (Unit KJmt)	SM	2.9	Soil. No intact rock. Shallow embedment not recommended.
10	Yes	Monzotonalite (Unit KJmt)	SM	-	Exposed bedrock with slope parallel foliation. Shallow embedment okay.

The result of the contractor's independent desktop study, highlighting the reliance on the depth to rock information in the owner-provided geotechnical desktop study, was that the contractor moved forward assuming bedrock would be encountered for the foundations at all structure locations. This assumption would provide the most efficient foundation designs and would give the contractor the best opportunity to provide competitive pricing.

FOUNDATION SOLUTION

Micropile foundations (see Fig. 2) were identified as the preferred solution by the owner in the RFP and proposed by the contractor for this project. These foundations consist of small-diameter, high-capacity drilled and grouted piles that can be installed using lightweight, portable equipment, making them particularly well-suited for remote or difficult-to-access sites where conventional heavy machinery cannot be deployed. This advantage is especially critical in helicopter-accessible locations.

Micropiles can be installed at varying angles and depths, allowing engineers to adapt the foundation system to steep or uneven terrain without the need for extensive grading or excavation. Their ability to resist both axial and lateral loads makes them highly effective for supporting structures on slopes.

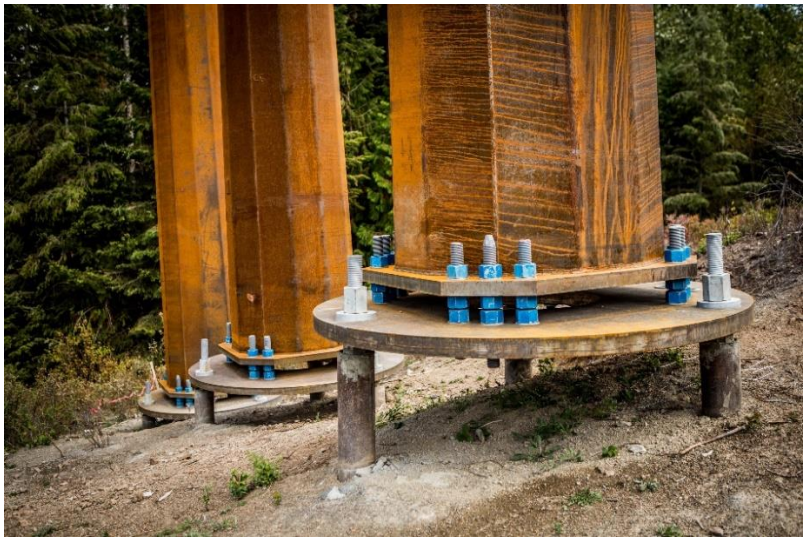


Fig. 2. Micropile foundation

Axial compressive and uplift forces on the piles are transferred to the surrounding subsurface materials through a "bond zone," which is simply the zone of the pile where cement grout encompasses the high strength reinforcing bar of the pile and is in continuous contact with the surrounding subsurface material. Pile axial forces are resisted in the subsurface material by frictional forces between the grout and ground. In regions with shallow bedrock, micropiles can be installed by drilling directly into the rock and filling the bond zone with grout without the use of pressurizing equipment. This further makes them a practical and cost-effective solution in mountainous or rocky environments where traditional deep foundation systems are often unfeasible.

CONSTRUCTION

Foundation construction was originally planned to occur between May 6 and July 17 under a scheduled outage, however, due to factors beyond the contractor's control, the line outage and construction start was delayed nearly two months. A requirement of the project was to energize the line before the winter season and with the high elevation and tendency for early snowfall, the contractor added personnel and equipment resources to ensure the foundations could be constructed in time to accommodate structure setting and line energization. Foundation construction commenced on July 31. However, just days after mobilization a nearby wildfire started (see Fig. 3) which necessitated reassignment of the project helicopters for approximately 1 month to support firefighting efforts. Now, with nearly 3 months' delay, work resumed August 26 with an estimated completion date of October 16, accelerating the baseline construction schedule by 3 weeks.



Fig. 3. Nearby wildfire at start of constuction

With construction finally underway, the contractor strategized execution of the work with crews simultaneously working from the top and bottom of the canyon and planning to meet near the middle of the segment to wrap up construction. Construction progressed on schedule with installed foundations encountering bedrock as planned. Structures near the top of the canyon encountered highly weathered bedrock near ground surface transitioning to moderately weathered strong bedrock within 5 feet of ground surface, whereas structures near the bottom of the canyon encountered as much as 15 feet of silty sand underlain by highly to very weathered weak bedrock. As September came to an end, foundation construction began on a structure near the middle of the segment, and the encountered geotechnical conditions there were nothing like the previously constructed foundations.

The equipment exhibited rock drilling characteristics (i.e., good hammer firing and slower advancement rates) at shallow depth during drilling of the first pile at this particular location, however, downhole sampling painted an entirely different picture with highly variable characterization values that were more indicative of a weak soil matrix. So far, all foundations at other structure sites had encountered decent bedrock within 15 feet of surface, and a similarly loaded foundation at an adjacent site was constructed with piles drilled to 20 feet below ground surface and a 10 foot bond zone. At this location competent bedrock still was not encountered at 20 feet below ground surface, so crews continued to drill deeper in the hopes that bedrock would be discovered. After nearly a week of advancing multiple piles on each leg of the structure to a depth of 50 feet below ground surface, the encountered conditions were characterized as a bouldery-soil matrix with variable strength, and the contractor had to adjust the construction approach from the bedrock installations that were previously planned.

At this time 5 structure locations were left to occupy and the risk of additional sites encountering a change of soil conditions became a real concern. To accommodate conditions at the structure location with no bedrock, and to mitigate potential risk at the remaining structures, the contractor shifted the construction approach to allow for a pressure grouted bond zone installation method. This required an immediate redesign for the change in conditions and rapid mobilization of additional construction equipment and materials including grout, casing and reinforcing bar, and a schedule impact of approximately double the originally planned construction time at each remaining foundation.

Although the contractor had a path forward to completion, seasonal weather changes led to a cumulative 3 week delay, pushing the end date into the second week of November. With structure erection on the heels of foundation construction and winter weather looming, the contractor shifted to a 7 day work week to finish foundation construction. Of the remaining 5 structure locations, only one additional adjacent location encountered the same bouldery-soil matrix and required pressure grouted bond zones. With the compounding schedule delays and change of geotechnical conditions, foundation construction was finally completed on November 14, approximately 3.5 months beyond the original date but within the contractor’s planned total construction duration.

IN HINDSIGHT

The change in site conditions was ultimately traced to a historic, large-scale rockslide that had deposited debris at the affected structure locations. Given the urgency of the situation, it's understandable that attention shifted toward reactive measures, with a focus on minimizing delays and financial impacts. However, in retrospect, there were observable indicators that could have signaled the underlying conditions. These included surface rock samples, vegetation patterns, rock mass characteristics, and even a closer review of Google Earth imagery.

Multiple rock types were observed in characterization samples (see Fig. 4) at the structure location, ranging from fine-grained to coarse-grained materials and some with discernible weak foliation. This range of rock types were also observed in smaller surface samples while hiking around the structure location. The presence of such varied material within a single characterization sample at a foundation location is a strong indicator of a colluvial rockslide debris deposit. Typically, foundation samples

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show more consistency in material composition; the heterogeneity seen here suggested a chaotic depositional environment consistent with past mass-wasting events.

A noticeable transition from exposed surface rock to areas with no apparent rock, or a distinct change in vegetation patterns, can serve as a valuable indicator of underlying geological variation (see Fig. 5). Such shifts may suggest a contact between different geologic units or the presence of a buried colluvial deposit or other subsurface features. Vegetation often responds to soil composition, moisture retention, and mineral content; therefore, careful observation of both surface materials and ecological changes can provide early clues to subsurface conditions.

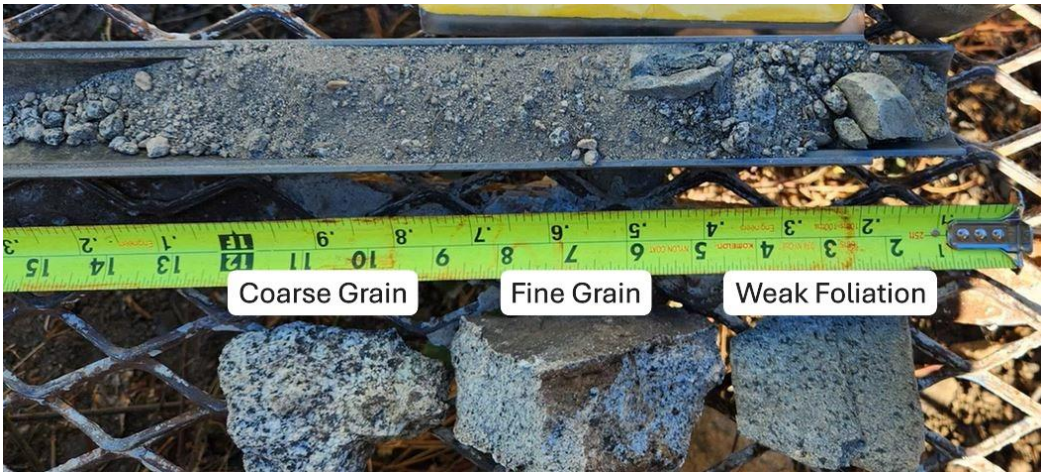


Fig. 4. Multiple rock types at single location

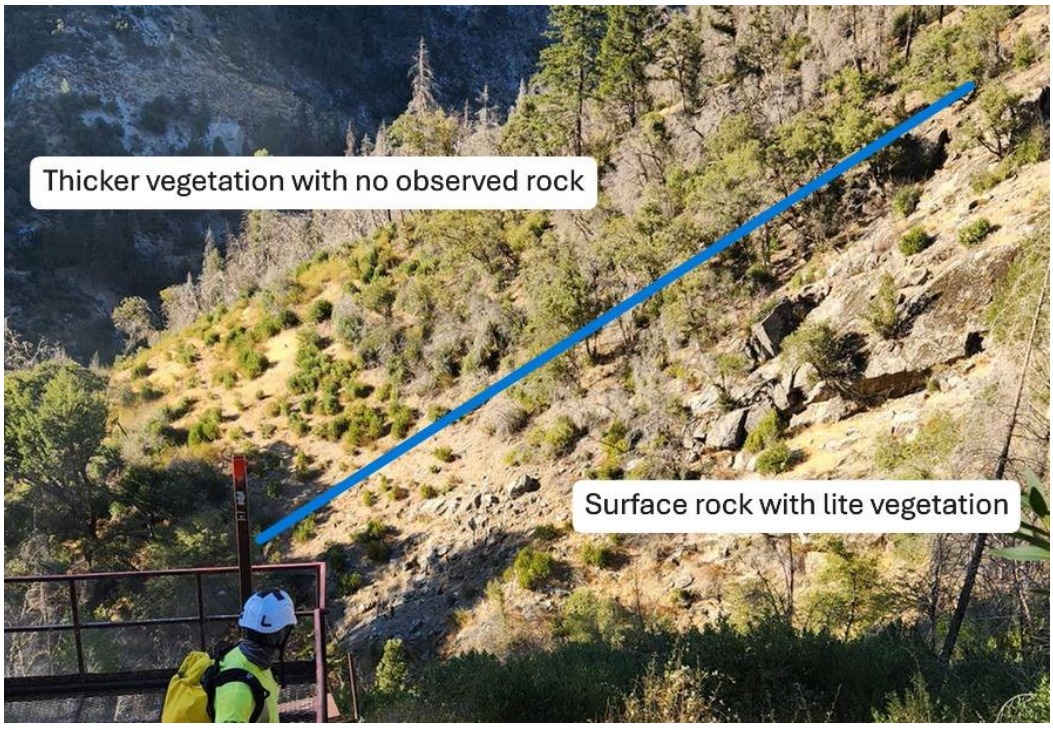


Fig. 5. Change in vegetation with possible contact line

Rock formations typically exhibit three dominant joint sets. By spending time examining exposed prominent rock faces within the surrounding canyon the orientation angles of these major joint sets can be identified (see Fig. 6). Comparing the orientation of these joint sets to the large surface samples at the structure locations can help determine whether the surface rock is part of the intact bedrock or has been displaced. The joint orientations of the large surface rock at the structure sites differed

significantly from those in nearby bedrock exposures, thereby suggesting the presence of a rockslide debris deposit.



Fig. 6. Primary rock joints (horizontal joints creating slab thickness)

A closer examination of Google Earth imagery, particularly when comparing different time periods, including immediately after the previous wildfire, revealed subtle but telling signs of geological instability, such as scarps and slide deposits. These features strongly suggest the presence of a historic mass-wasting event. In fact, this may have been one of the clearest indicators that the two structure locations were unlikely to encounter competent bedrock.

Ultimately, downhole imaging during construction was instrumental in distinguishing between a bouldery-soil matrix and intact bedrock (see Fig. 7). The owner's desktop study had indicated an approximate rock joint spacing of 6 feet, implying that boulders of similar size could be expected in near-surface weathered bedrock zones. However, visual inspection of the borehole revealed randomly oriented rock fragments significantly smaller than the expected joint spacing. The lack of consistent joint orientation and the fragment size confirmed that the affected structures were located within a rockslide debris deposit.



Fig. 7. Bond zones – bedrock vs bouldery-soil matrix

CLOSING

This project presented challenges that extend beyond the scope of traditional classroom instruction. Real-world conditions often introduce unpredictable variables, such as weather events or natural disasters, that can significantly impact project schedules and profitability. In subsurface work, particularly drilling, uncertainty is not a possibility but a certainty. Even with thorough geotechnical investigations, ground conditions can vary dramatically between borings, especially on long linear projects like transmission lines which may traverse diverse geologic formations. Success in the field requires more than technical knowledge; it demands situational awareness and adaptability. Observing the environment during bidding and execution of design and construction can reveal subtle clues about what lies ahead, helping teams anticipate and mitigate potential issues.

Importantly, setbacks and failures should not be viewed as defining moments, but rather as opportunities for growth. Each challenge faced and overcome adds to our collective experience, strengthening our expertise and resilience. Though stressful in the moment, these lessons are invaluable and contribute to our development as professionals. Embracing them with a mindset of continuous learning ensures that we not only improve but lead with confidence and insight in future endeavors.

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