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GEOTECHNICAL REPORT
AUXILIARY WATER SUPPLY SYSTEM (AWSS)
PUMP STATION NO. 1
698 SECOND STREET
SAN FRANCISCO, CALIFORNIA
PROJECT NO. CUWAWSAW05
CONTRACT NO. CS-998.B

January 2012

Prepared for:

San Francisco Public Utilities Commission
1155 Market Street
San Francisco, California 94103

Owner:

San Francisco Public Utilities Commission

GTC Project No. SF11024

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Mr. David Myerson, P.E.
San Francisco Public Utilities Commission
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San Francisco, California 94103

January 30, 2012
GTC Project No. SF11024

Subject: Geotechnical Report
AWSS – Pump Station No. 1
698 Second Street, San Francisco, California
Project No. CUWAWSAW05, Contract No. CS-998.B

Dear Mr. Myerson:

We are pleased to present this geotechnical report for the design of seismic retrofits to Pump Station No. 1. This geotechnical report presents the results of our evaluation of potential seismic geo-hazards and geotechnical parameters for structural retrofit design of the existing structure. This report was developed in accordance with *Task Order No. 10 – Pump Station 1 Geotechnical Services* of Project No. CUWAWSAW05 for Contract No. CS-998.B, and our proposal dated August 10 (revised August 16), 2011.

Geotechnical data, including historic data from previous studies in the vicinity of the site, is included under separate cover in a Geotechnical Data Report (GTC, 2012). The GDR provides the primary geotechnical data from which the interpretations and recommendations in this report are based upon.

Sincerely,
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1.0 INTRODUCTION AND PURPOSE

The San Francisco Public Utilities Commission (SFPUC) is assessing the seismic conditions of Pump Station No. 1 (PS1) located at the San Francisco Fire Department (SFFD) Headquarters at 698 Second Street in San Francisco, California. Pump Station No. 1 is a key component of the Auxiliary Water Supply System (AWSS), which is an independent high-pressure water supply dedicated to fire protection that is used by SFFD. PS1 was built circa 1911 in response to the 1906 San Francisco Earthquake and Fire. Geotechnical Consultants, Inc. (GTC) has been retained by SFPUC to provide geotechnical design services for the project. This geotechnical report presents the geologic and geotechnical discussions, interpretations, and recommendations for the design of seismic improvements to PS1. In particular, this geotechnical report presents the results of our evaluation of settlement beneath a distressed portion of the basement slab located in the vicinity of an existing pump, potential seismic hazards, and geotechnical parameters for performing structural design and analyzing seismic forces including evaluation of site-specific response spectra. This report was developed in accordance with *Task Order No. 10 – Pump Station 1 Geotechnical Services* of Project No. CUWAWSAW05 for Contract No. CS-998.B, and our proposal dated August 10 (revised August 16), 2011.

Geotechnical data, including exploratory drilling for this study as well as historic data from previous studies in the vicinity of the site, is included under separate cover in a Geotechnical Data Report (GTC, 2012). The GDR provides the primary geotechnical data from which the interpretations and recommendations in this report are based upon.

2.0 PROJECT DESCRIPTION

2.1 General

The AWSS is a high-pressure water distribution system designed solely for use by SFFD. The system includes one water storage reservoir (Twin Peaks), two storage tanks (Ashbury and Jones Street), two salt-water pump stations (Pump Stations Nos. 1 and 2) with intake tunnels, and approximately 135 miles of piping, 3,800 valves, 1,600 hydrants, and 177 underground storage cisterns. There are also 52 suction connections along the northeastern waterfront that allow fire engines to pump water from the San Francisco Bay and five manifolds that can be connected to fire boats to pump water from the bay. The AWSS was originally built between 1908 and 1913. Ownership and maintenance of this system was transferred from SFFD to SFPUC in May 2010.

AWSS facilities show deterioration, seismic safety issues, corrosion due to rust, poor foundations, leakages, and outdated technologies. The repair and upgrade of the AWSS is part of planning and development of a comprehensive long-term capital improvement strategy for repair, retrofitting, improvement, and expansion of the

firefighting pipelines, cisterns, and seawater intake tunnels. A critical goal of the AWSS upgrade project is to maximize the likelihood that the AWSS will effectively provide required fire-fighting capabilities after a major seismic event.

2.2 Pump Station No. 1

Pump Station No. 1 (PS1) is housed in the basement of the SFFD Headquarters at 698 Second Street, which is located in the South Beach / China Basin neighborhood of San Francisco. The site is bound by Second Street to the northeast, Townsend Street to the southeast, Stanford Street to the southwest, and a three-story masonry office / retail building to the northwest. The project location is shown on **Figure 1 – Project Location Map**.

PS1 is an emergency saltwater pump station for delivering seawater from the San Francisco Bay to the AWSS. PS1 sits at El. 0 feet (San Francisco City Datum [SFCD]) and supplies water to the AWSS Lower Zone located between sea level and El. 150 feet. It is also capable of providing seawater to the AWSS Upper Zone above El. 150 feet. The pumps within the facility can deliver a pumping capacity of 2,700 gallons per minute (gpm) at a pressure of 300 pounds per square inch (psi). A concrete intake tunnel of approximately 1,100 feet long traverses beneath Townsend Street from PS1 to the Bay.

During the original construction circa 1911, PS1 was housed in the basement of an industrial-type building measuring approximately 100 feet by 155 feet in plan dimensions. Boiler tanks for the steam-driven pumps were located in the southwestern half of the building (along Stanford Street), and were supported on a reinforced concrete floor at approximately El. 0 feet (about 10 to 13 feet below the grade of surrounding streets). The pumps, intake tunnel and piping were located in the central portion of the building. Underground water storage reservoirs were located below the boilers and a portion of the pumps in the southwestern two-thirds of the building (**Appendix A**: Longitudinal Section of Building, Sheet No. 2). The slab of the water reservoir was at approximately El. -19 feet (about 32 feet below the grade of surrounding streets) and the slab of the intake tunnel at approximately El. -21 feet. Offices, kitchen facilities, a reception area and restrooms occupied the northeastern portion of the building. The foundation for this portion of the building was at a higher grade ranging from El. 0 feet along the northwestern half of the Second Street frontage to El. 7 feet along the southeastern half.

PS1 was remodeled around 1973 and again around 1998. The 1973 renovation consisted of converting the pumps to a diesel driven system. Seismic strengthening of the pump station room was also performed. As part of this conversion or possibly during

FIGURE 1 – PROJECT LOCATION MAP



Source: ArcGIS by ESRI.

the 1998 renovation, the underground water storage reservoirs were filled with compacted sand backfill, and the pipe trenches in the pump station room were filled with concrete. In about 1998, a major renovation was completed to convert the structure to the headquarters for SFFD. PS1 remains functional in the basement of the building. The rest of the building was converted into three stories of offices with underground parking partially replacing the old boiler room. The remodeled building was constructed with steel frames and reinforced concrete floors.

3.0 WORK PERFORMED

The scope of work completed for this geotechnical evaluation comprises:

- **Review of Existing Data and Information.** Reviewed available resources including previous soil investigation reports in the site vicinity, and maps and reports published by the United States Geological Survey (USGS) and California Geological Survey. A complete list of reports reviewed is included in Section 4 and the References sections of this report.
- **Site Visits.** Conducted site visits for the purpose of general reconnaissance.
- **Subsurface Exploration Program.** Attempted to perform an additional subsurface exploration in the area of the distressed slab near the pumps. Due to the thickness of the concrete slab, the proposed boring was abandoned. Logs and photos of the coring operations are included in the GDR (GTC, 2012).
- **Engineering Analysis.** Analyzed findings to develop conclusions regarding seismic and geologic hazards, and to develop geotechnical recommendations for the improvements to Pump Station No. 1. As requested by SFPUC, a site-specific seismic hazard analysis was performed for the project.
- **Report.** Prepared this report presenting our geotechnical findings, conclusions, and recommendations for the design of the proposed project.

4.0 EXISTING DATA

4.1 Previous Reports

We reviewed geotechnical information from previous reports for Pump Station No. 1 and for sites located in the vicinity of the project to evaluate past findings which may be pertinent to the project. The following pertinent reports were reviewed:

- Arup, 2009, Draft Geotechnical Data Report, Downtown Extension Project, Transbay Joint Powers Authority, May 7.

- EQE/AGS, 1989, San Francisco Department Facilities, Seismic Evaluation Study, Summary Report and Appendix A, Geotechnical Report, Prepared for the Department of Public Works, City and County of San Francisco for San Francisco Fire Commission, May.
- Harding Lawson & Associates (HLA), 1992, Soil Investigation, Remodeling of Pump Station No. 1, Second and Townsend Streets, San Francisco, California, Prepared for Urban West Associates, HLA Job No. 17952.053.04, April 8.
- HLA, 1997, Geotechnical Investigation, One Embarcadero South Development, San Francisco, California, Prepared for Urban West Associates, HLA Project No. 37293.1, March 3.
- Metcalf & Eddy/AECOM, 2009, Final Report, Auxiliary Water Supply System (AWSS) Study, Prepared for Capital Planning Committee, City and County of San Francisco, January 23.
- Transbay Joint Powers Authority (TJPA), 2011, Transbay Transit Center Program, Downtown Rail Extension Project – Second and Townsend Streets Soil Borings, Letter Accompanied with Boring Location Plan and Logs, Prepared by Mr. Bryan Dykes (TJPA) and Addressed to Mr. David Myerson (SF Water, Power and Sewer), CIN No. 1109528, August 3.
- Treadwell & Rollo (T&R), 1995, Geotechnical Investigation, 650 Second Street, San Francisco, California, Prepared for The Lansing Group, Project No. 1784.01, October 5.
- T&R, 2000a, Geotechnical Investigation, 639 Second Street, San Francisco, California, Prepared for Rosenberg Hood Ventures, Project No. 2688.03, October 4.
- T&R, 2000b, Geotechnical Investigation, Seismic Upgrade, 123 Townsend Street, San Francisco, California, Prepared for JMA Properties, Project No. 2598.02, January 7.

4.2 Historic Project Drawings

We also reviewed historic project drawings which were provided to us by SFPUC. Available drawings included those from the original construction of Pump Station No. 1 in 1911, and the drawings from the renovations in 1973 and 1998. Selected drawings which show pertinent plans and sections (profiles) are compiled in **Appendix A – Historic Project Drawings**, and they include the following:

- Culley Associates, Inc. (CAI), Undated, SFFD Headquarters at Pump Station No. 1, 698 Second Street, San Francisco, California, Job No. 5228A.
    - Exterior Elevations, Sheet Nos. S3.01 and S3.02
    - Longitudinal Section, Sheet No. S3.03
    - Concrete Wall Details, Sheet No. S5.2

- San Francisco Department of Public Works (SFDPW), 1909, Plans for an Auxiliary Water Supply System for Fire Protection, Contract No. 27 – Sec. H for the Construction and Installation of Mechanical Equipment for Pumping Station No. 1
  - Plan, Sheet No. 1
  - Longitudinal Section of Building, Sheet No. 2
  - Transverse Section of Building, Sheet No. 3

- SFDPW Bureau of Engineering, 1973, Fire Protection High Pressure Pump Station #1, Convert to Diesel Drive, October 24.
  - Vault Plan, Sheet 2 of 21, File B-38, 507
  - Building Cross Sections, Sheet 3 of 21, File B-38, 508
  - Pit and Wall Details, Sheet 6 of 21, File B-38, 511

- SFDPW Bureau of Architecture, 1993, SFFD Headquarters at Pump Station No. 1, 698 Second Street, Job No. 5228A, November 30.
  - Basement Plan, Sheet A2.1
  - Transverse Building Sections, Sheet A3.3
  - Longitudinal Building Section, Sheet A3.6

6.0 GEOLOGY

This section summarizes the geologic conditions at the project site. The regional geologic setting is described in Section 6.1. The site geologic conditions are discussed in Section 6.2. The seismic setting and local faulting are described in Section 6.3.

6.1 Geologic Setting

The San Francisco Bay area lies within the central portion of the Coast Ranges physiographic province of California. This province generally consists of rugged and moderately high mountain ranges separated by intervening valleys. The Coast Ranges province extends northwestward some 600 miles through western California between the Transverse Ranges in the south to beyond the Oregon border in the north. The ridges and valleys within the Coast Ranges province, including the San Francisco Bay area, are primarily controlled by geologic structure and generally reflect the northwest trend of the regional faults and folds.

Faults belonging to the San Andreas fault system have broken the basement rock underlying the San Francisco Bay Area into several major structural blocks. The City of San Francisco occupies the San Francisco Bay block; bounded on the west by the San Andreas fault and on the east by the Hayward fault. During the past two million years, the San Francisco Bay block has tilted eastward forming the topographic

depression now occupied by San Francisco Bay. During this same geologic interval, the Santa Cruz Mountains, Diablo Range, and Berkeley Hills have been uplifted. The waters of San Francisco Bay now separate the mountains of the San Francisco Peninsula from the parallel range of the East Bay Hills.

San Francisco lies along the boundary between the Pacific and North American tectonic plates. In Central California, the relative movement between these two tectonic plates generally occurs over a number of faults across a 50-mile wide zone extending from the San Gregorio fault in the southwest to the Great Valley Thrust Belt in the northeast. Since the Miocene Epoch (approximately 23 million years ago), about 200 miles of right-lateral slip has occurred along the San Andreas fault zone to accommodate the differential offset between these two plates. This tectonic movement has juxtaposed the granitic rocks lying southwest of the San Andreas fault with rocks of the Franciscan Complex and Great Valley Sequence present on the northeast. The basement rock underlying San Francisco, situated just east of the San Andreas fault, consists of tectonically mixed rock of Jurassic-Cretaceous age known as the Franciscan Complex. These diverse rocks include shale, greywacke sandstone, conglomerate, chert, serpentinite, and altered volcanics.

In general, the sedimentary deposits overlying the bedrock across much of the flatlands of the San Francisco Bay area are geologically recent; typically less than 2 million years in age. Deposition of these sediments coincided with the global rising and falling of sea level that accompanied the glacial periods ("ice ages") and the intervening interglacial periods. As Earth's surface water became stored as glaciers and ice-sheets, sea levels fell worldwide. San Francisco Bay drained, and terrestrial alluvium (silt, clay, sand, and gravel) and aeolian deposits (wind-blown sand dunes) accumulated on the former bay floor. During the intervening high stands of sea level between glacial episodes, the Bay was once again filled and shallow marine and estuarine sediments of variably organic clays, silts, and minor sand accumulated within the Bay and across the surrounding tidal marshlands. In response to the last Pleistocene glacial advance beginning about 70,000 years ago, falling sea levels emptied San Francisco Bay and exposed the sediments deposited within the Bay. These soft sediments dried and hardened, and the exposed terrain was locally eroded. Sandy alluvial fans, channel deposits, and sand dunes developed on the eroded surface. Sea level began to rise again about 20,000 years ago at the end of the last glacial period, and San Francisco Bay was filled once again. Soft sediments again accumulated within the Bay. The most geologically recent sediments deposited within the Bay are soft, predominantly clayey deposits referred to as Bay Mud. Beginning about the mid-1800s, man-made fill was placed along the Bay shoreline to raise tidal flatlands and elevate the shallow Bay margins.

6.2 Site Geology

The site is located at what historically was known as Steamboat Point, which was a small hill at the shoreline of San Francisco Bay and Mission Bay prior to subsequent grading activities. The hill was comprised of a sand dune rising to about 100 feet above sea level (Arup, 2009). By the 1870s, the topography in this area began to change to reclaim Mission Bay. The sand from Steamboat Point was reportedly mined to use as fill for this reclamation project, thus bringing the grade near Second and Townsend Streets down to its present elevation, approximately 80 feet below the original grade. The excavation exposed the underlying sandstone bedrock of the Franciscan Complex, which now underlies PS1 at shallow depths. PS1 was constructed at this site after the 1906 San Francisco Earthquake and Fire presumably because it was a bedrock knob at the water's edge. This site likely did not experience the ground deformations due to liquefaction and lateral spread that affected large areas of man-made land in the area.

Published maps of surficial deposits indicate that the site is underlain by artificial fill (af) at the edge of undivided surficial soil deposits (Qu) (Schlocker, 1974; Knudsen et al., 1997; Brabb et al., 2000). Geotechnical boring logs (Arup, 2009; HLA, 1992), however, indicate that sandstone bedrock of the Franciscan Complex is at shallow depth, and that PS1 footings are predominantly founded on bedrock. The geologic units mapped in the vicinity of the project site are shown on published maps by Blake et al. (2000) and Knudsen et al. (1997), and are described as:

- **Franciscan Complex, Sandstone and Shale (Kfs) and Massive Sandstone (Kfss):** Fine- to coarse-grained sandstone (greywacke), with interbedded siltstone and shale. Siltstone and shale interbeds generally constitute less than 20 percent, but in places form sequences as much as several tens of meters thick. In many places, shearing has obscured bedding relations; rock in which shale has been sheared to gouge generally constitutes about 10 percent of unit. Total thickness of unit is unknown but is probably at least many hundreds of meters.
- **Artificial Fill (af):** Loose to very well consolidated gravel, sand, silt, clay, and rock fragments, organic matter, and man-made debris in various combinations. Thickness is variable and may exceed 30 m (100 feet) in certain places. Some fill is compacted and quite firm, but fill placed before 1965 is generally not as well compacted and consists simply of dumped materials. Much of the mapped fill overlies estuarine sediment and forms of new land, levees, or dikes near sea level.
- **Dune Sand (Qd):** Predominantly loose to medium- to coarse-grained, well sorted sand but also includes pebbles, cobbles, and silt. Thickness less than 6 m (20 feet) in most places, but in other places may exceed 30 m (100 feet).
- **Undifferentiated Deposits (Qu):** Alluvium deposited in fan, terrace, or basin environments. This unit is mapped where separate types of alluvial deposits

could not be delineated due to complex inter-fingering of depositional environments or the small size of the area. Typically, undifferentiated alluvium is mapped in relatively flat, smooth valley bottoms of small- to medium-sized drainages. Deposits are probably sand, silt, and gravel that are poorly to moderately sorted.

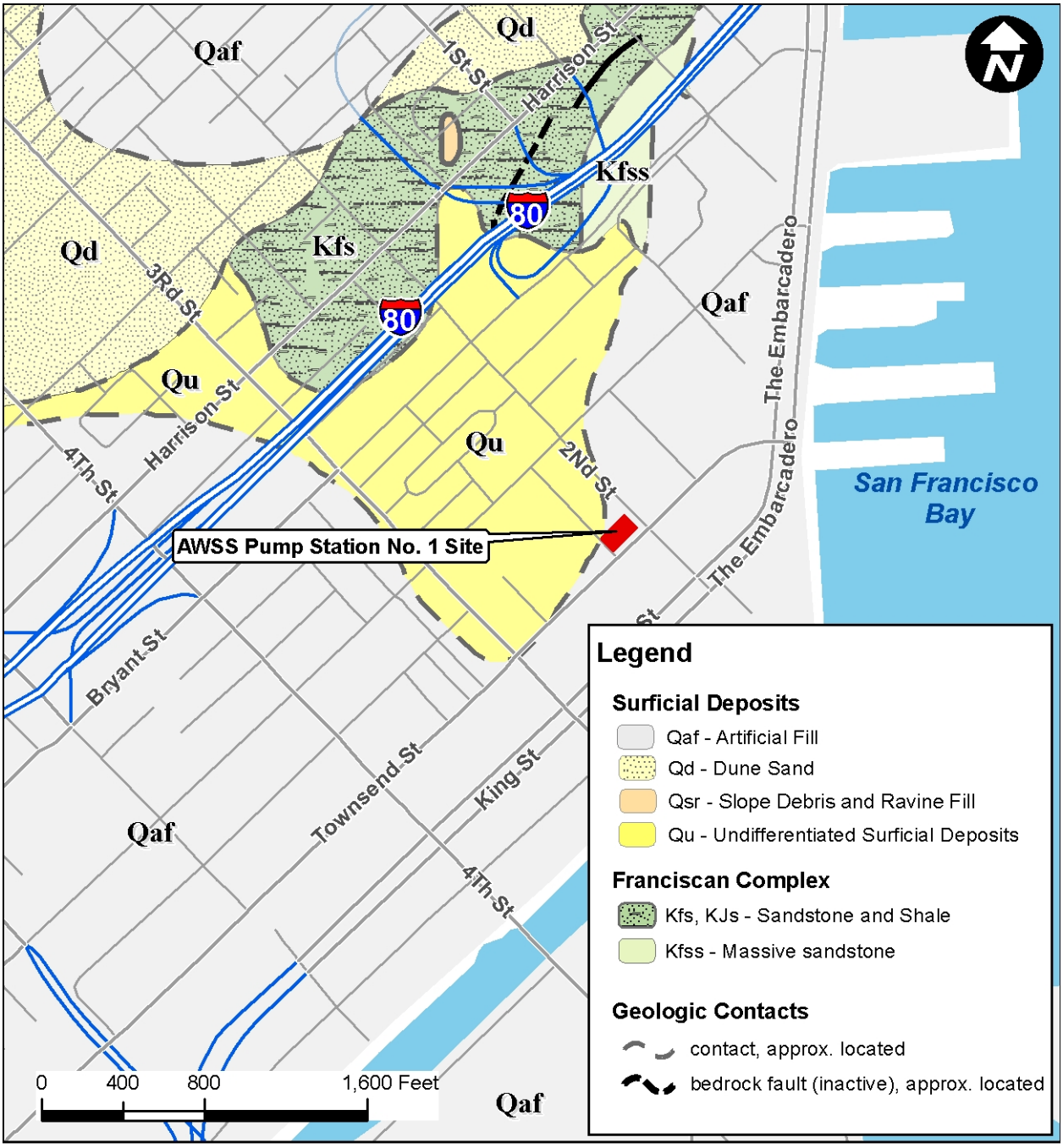
Regional geologic mapping of the project site is shown on **Figure 2 – Regional Geologic Map**. As noted above, the regional geologic mapping does not indicate the presence of shallow bedrock at the site as indicated by past geotechnical borings.

6.3 Seismic Setting and Local Faulting

The project site is located within a seismically active area near the boundary between two major tectonic plates, the Pacific Plate to the southwest and the North American Plate to the northeast. Movement of these plates is primarily translated in the Bay Area as right lateral slip along the San Andreas Fault Zone, with some compressional component which is accommodated by vertical reverse-slip displacement on the Great Valley and other reverse/thrust faults in the central California area. The relative movement between the Pacific Plate and the North American Plate generally occurs across a 50-mile wide zone extending from the San Gregorio Fault in the southwest to the Great Valley Thrust Belt to the northeast. The United States Geological Survey (USGS) Working Group on California Earthquake Probabilities concluded that there is a 62 percent probability of a strong earthquake (M≥6.7) occurring in the San Francisco Bay Region in a thirty year period between 2003 and 2032 (WGCEP, 2003). Additionally the 2007 Working Group on California Earthquake Probabilities (WGCEP, 2008) has concluded that within the next 30 years the probability of a strong earthquake (M≥6.7) occurring on regional faults is as follows: 21% for the N. San Andreas Fault Zone, 31% for the Hayward-Rodgers Creek Fault Zone, and 7% for the Calaveras Fault Zone.

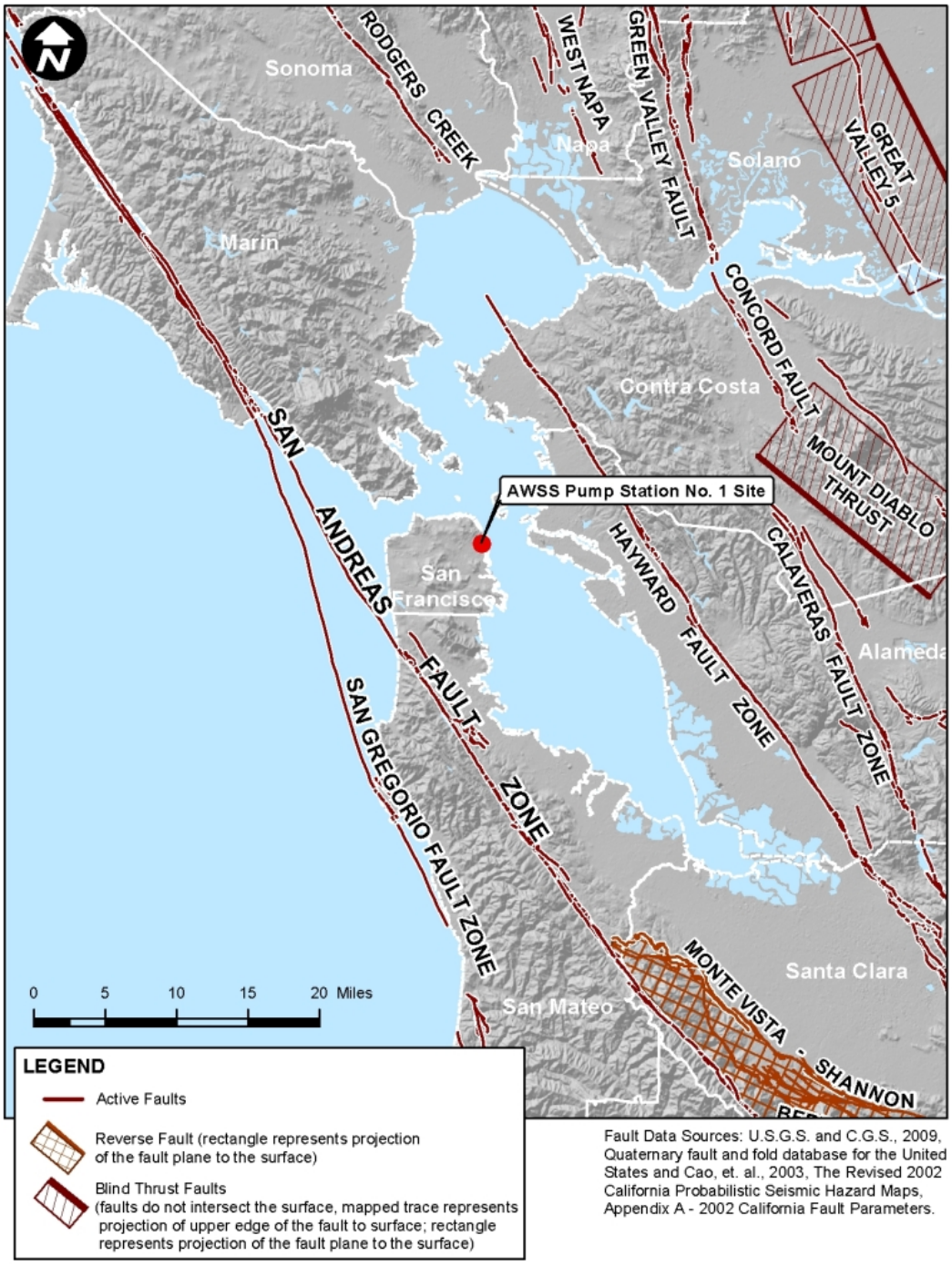
Strong ground shaking at the project site could occur as a result of an earthquake on any one of the active regional faults shown in **Figure 3 – Regional Active Fault Map**. Active faults in California have been divided into activity categories by the California Geological Survey based on their predicted activity and ability to generate strong earthquakes; "Type A" faults which generally have higher and more well defined slip rates and well defined recurrence intervals and "Type B" faults with well defined slip rates but poorly constrained recurrence intervals. "Type A" faults are commonly considered more active (generally with higher slip rates) and/or capable of generating larger earthquakes than "Type B" faults. Both "Type A" and "Type B" faults that are mapped in the vicinity of the project site are summarized in **Table 1 – Significant Active Faults**. The nearest fault to the project site is the Northern San Andreas Fault, passing about 8.2 miles (13 kilometers) to the southwest. The distance to significant active faults,

FIGURE 2 – REGIONAL GEOLOGIC MAP



Source: USGS MF-2337, Geologic map and map database of parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties, California: Digital database Version 1.0

FIGURE 3 – REGIONAL ACTIVE FAULT MAP



Fault Data Sources: U.S.G.S. and C.G.S., 2009, Quaternary fault and fold database for the United States and Cao, et. al., 2003, The Revised 2002 California Probabilistic Seismic Hazard Maps, Appendix A - 2002 California Fault Parameters.

Table 1 – Significant Active and Potentially Active Faults

Fault	Distance to Fault [1]		Estimated Earthquake Magnitude [2]	30-Year Probability of M>6.7 Earthquake [3] (%)
	(km)	(miles)		
Active Type A Faults				
N. San Andreas Segments: Offshore, N. Coast, Peninsula, Santa Cruz (SAO+SAN+SAP+SAS)	13.2	8.2	7.9	21
N. San Andreas Segments: Peninsula and Santa Cruz Mtn (SAP+SAS)	13.2	8.2	7.5	
N. San Andreas Segment: Peninsula (SAP)	13.2	8.2	7.1	
Hayward – Rodgers Creek: Rodgers Creek, N. & S. Sections (RC+HN+HS)	15.8	9.8	6.9	31
Hayward – Rodgers Creek: N. and S. Sections (HN+HS)	15.8	9.8	6.9	
Hayward – Rodgers Creek: S. Section (HS)	16.4	10.2	6.7	
Calaveras Segment: N. Section (CN)	33.6	20.9	6.8	7
Calaveras Segments: N.& Central (CN+CC)	33.6	20.9	6.8	
Calaveras Segments: N., Central, & S. (CN+CC+CS)	33.6	20.9	6.9	
Potentially Active Type B Faults				
San Gregorio Connected	19.1	11.9	7.5	n.a
Mount Diablo Thrust	32.7	20.3	6.6	n.a.
Monte Vista-Shannon	40.1	24.9	6.7	n.a.

Notes:

1. Fault-to-site distances based on the 2008 updated National Seismic Hazard Mapping program (Petersen et al, 2008) and companion interactive seismic de-aggregation site at URL https://geohazards.usgs.gov/deaggint/2008.
2. Maximum Moment Magnitude based on California Geological Survey (CGS) fault parameters as updated in 2002 (Cao, et al., 2003), and/or as suggested by the SFPUC's General Seismic Requirements – Appendix A (SFPUC, 2009).
3. 30-year probability of M>6.7 earthquake based on 2007 Working Group on California Earthquake Probabilities (WGCEP, 2008).

CGS assigned fault type ("A" or "B"), estimated maximum magnitude earthquake, and 30-year probability of a M≥6.7 earthquake on the fault are summarized in the table.

7.0 RECOMMENDATIONS

7.1 General

The following sections provide our conclusions and recommendations for evaluation and design of the seismic retrofit and upgrade of Pump Station No. 1 (PS1). Based on our findings from our review of past geotechnical borings, the site is underlain by Franciscan Formation sandstone. The existing building is supported on shallow foundations founded predominantly on the bedrock. Minor distress has occurred to the slab in the vicinity of the northern pump. Mitigation of soil settlement, which is considered to be the cause of the distress, is provided in Section 7.6.2.

We consider the proposed improvements to be geotechnically feasible, provided that our geotechnical recommendations are incorporated into design and construction documents.

7.2 Seismic Design Considerations

7.2.1 General

The site is on stable bedrock at the historic location of Steamboat Point. Excavation of the overlying dune sand in the late 1800s exposed Franciscan Complex sandstone upon which PS1 was constructed circa 1911. As site excavation inevitably involves some overexcavation, some artificial fill is expected to underlie the structure as a result of backfilling basement walls and utility trenches. As evidenced by settlement of a portion of the slab in the pump room, some of this fill may not have been well compacted. There is a potential for some localized liquefaction and/or dynamic settlement of this loose fill. Generally, however, the potential for liquefaction, dynamic settlement, and seismically-induced landslides to affect PS1 is very low for this flat-lying bedrock site. Also, as no active or potentially active faults are known to cross the site, the potential for fault rupture is very low. The main seismic hazard at the site is therefore strong ground shaking. The following sections provide seismic design parameters provided by USGS and our site-specific ground motion study performed for PS1.

7.2.2 Code-Based Seismic Design Parameters

The seismic design parameters, calculated using the USGS web-based Java Ground Motion Parameter Calculator (http://earthquake.usgs.gov/research/hazmaps/design), are presented in **Table 2 – Seismic Design Parameters (ASCE 7-05)**. Since a site-specific seismic hazard analysis was performed for the project, the seismic design parameters in **Table 2** are provided primarily for comparative purposes. The site-specific acceleration response spectra are provided in Section 7.2.3 and **Appendix B**. As can be seen in **Figure B1** in **Appendix B**, the response spectrum for a Maximum Considered Earthquake (MCE) event is higher than the corresponding site-specific response spectrum for Basic Safety Earthquake 2 (BSE-2), and the response spectrum for a Design Earthquake is higher than that for the corresponding BSE-1. Hence, a design using code-based seismic design parameters would be more conservative for the identified seismic hazard levels.

Table 2 - Seismic Design Parameters (ASCE7-05)

Mapped Spectral Acceleration	
PGA	0.600
S_s at 0.2-second	1.500
S_1 at 1-second	0.610
Site Adjustment Factor	
Site Class	B
Site Coefficient F_a	1.0
Site Coefficient F_v	1.0
Site Adjusted Spectral Acceleration	
SMs = F_a x S_s	1.500
SM1 = F_v x S_1	0.610
Design Spectral Acceleration	
SDs = 2/3 x SMs	1.000
SD1 = 2/3 x SM1	0.407

7.2.3 Site-Specific Acceleration Response Spectra

In accordance with SFPUC's request, we developed site-specific acceleration response spectra for the site. Ground shaking at any site is primarily influenced by the underlying geologic materials. PS1 is located on a bedrock site with an estimated shear wave velocity in the upper 100 feet of approximately 3,600 feet per second (1,100 meters per second) which was developed based on review of available in-situ shear wave velocity measurements from geophysical suspension logging performed at borings in the

vicinity (Arup, 2009). The site can be classified as Site Class B according to IBC2009/CBC 2010 and ASCE 7-05. We performed a seismic ground motion study, which is attached as **Appendix B – Site-Specific Acceleration Response Spectra**. The summary in **Appendix B** explains methodology and presents the results of a site-specific seismic hazard analysis. Since the components at Pump Station 1 have not been assigned to a specific Seismic Performance Class (SPC), we developed design response spectra for each of the Seismic Performance Classes, SPC I, II and III, in accordance with the SFPUC General Seismic Requirements (GSR) (2009). We also developed design response spectra for Basic Safety Earthquake 1 (BSE-1) and BSE-2 as defined in the SFPUC GSR and ASCE 41-06. The site-specific design response spectra for horizontal ground motion at 5% damping are presented on **Figure B1**.

7.3 Groundwater

HLA (1992) noted that a stabilized groundwater level was measured at the west corner of the site (Boring B-4) at 14.5 feet below ground surface. Plans from the original construction (SFDPW, 1909) indicate that the groundwater level varies from El. -3.5 feet to El. -14.25 feet SFCD, which corresponds to approximately 15.5 to 26 feet below ground surface. Excavations greater than 15 feet below ground surface, if required for seismic retrofit, may encounter some water within the fractures in the bedrock and in backfill soils. We recommend that a groundwater elevation of zero (SFCD) be used for design.

7.4 Earthwork

It is not known at this time during the preliminary evaluation of the seismic retrofit if significant earthwork activities will be required for the project. We are not aware of any requirements to excavate below grade. The most likely earthwork activities on this project are anticipated to be associated with grouting or underpinning of the slab in the vicinity of the pumps where some past soil settlement is evident. The soils in this confined area are anticipated to be backfill overlying hard sandstone bedrock. The backfill soils are not known but may include sand, broken rock fragments, and/or construction debris. The bedrock, which may need to be excavated in the event of retrofitting with micropiles, should be able to be excavated using heavy limited-access drilling equipment due to the fracturing in the rock, though some of the bedrock may be very difficult to excavate locally. Smaller limited-access equipment will likely need to be used due to space constraints. Construction considerations for the grouting or underpinning activities are discussed further in Section 7.6.2. We should be given the opportunity to review the proposed retrofit, and to provide additional earthwork requirements, if necessary.

7.5 Lateral Earth Pressures

7.5.1 Summary

A summary of the recommended lateral earth pressures for design are provided in **Table 3 – Lateral Earth Pressures**. The lateral earth pressures are further described in Sections 7.5.2 through 7.5.6.

Table 3 - Lateral Earth Pressures

Equivalent Earth Pressure		
	Above El. 0	Below El. 0
Active including Hydrostatic [1]	25 pcf	75 pcf
At-Rest including Hydrostatic [1]	40 pcf	85 pcf
Passive including Hydrostatic [1]		
Allowable Displacement 0.06 H [2]	600 pcf	350 pcf
Allowable Displacement 0.03 H [2]	510 pcf	310 pcf
Allowable Displacement 0.01 H [2]	300 pcf	180 pcf
Seismic Active Pressure Increment [3]	25H psf	
Surcharge [4]	0.35 q	
Base Friction for Resisting Lateral Sliding		
Base Friction Factor [5]	0.4	

Notes:
1. Design groundwater elevation at El. 0 SFCD.
2. H is the embedment height below adjacent finished exterior grade.
3. The seismically induced active earth pressure increment should be applied to the wall as an inverted triangular distribution that decreases linearly from the stated maximum active earth pressure increment at the top to zero at the bottom. H is embedment height below the adjacent exterior grade.
4. For surface pressure loads distributed uniformly over a large surface area, we recommend that a uniform lateral pressure increment of **0.35*q** be applied to the wall/retaining structure, where **q** is the applied surface pressure load. This load should be applied in a rectangular pressure distribution for the upper portion of the wall/retaining structure over a height equivalent to the width of the loaded area.
5. Base friction factor should only be applied to dead load for evaluating frictional resistance.

7.5.2 Active Earth Pressure

Active earth pressures are imposed by the soil on walls that are unrestrained so that the top of the wall is free to translate or rotate at least 0.004H, where H is the height of the wall. The active earth pressure may be calculated using a design equivalent fluid pressure (EFP) of 25 pcf above the groundwater level and 75 pcf below the groundwater level.

7.5.3 At-Rest Earth Pressure

At-rest pressures should be used for design of walls that are restrained such that the deflections required to develop active earth pressures cannot occur or are undesirable. The at-rest earth pressures may be calculated using a design EFP of 40 pcf above the groundwater level and 85 pcf below the groundwater level.

7.5.4 Seismic Earth Pressure

In addition to the static earth pressures, retaining walls should be designed to consider additional earth pressures due to earthquake loading. For restrained walls, the seismic earth pressure should be added to the active earth pressure rather than the at-rest earth pressure. The increment in earth pressure due to seismic loading, for both restrained and unrestrained below-grade walls, may be calculated using an inverted triangular distribution with the pressure at the top of the wall equal to 25H psf and diminishing to zero at the base of the wall. "H" refers to the height of the wall in feet.

7.5.5 Passive Earth Pressure

Lateral loads on structures can be resisted by passive pressures that develop against the sides of below-grade structures such as walls or footings. The passive pressure depends on the lateral displacement of the wall or footing. In accordance with FEMA 356 (FEMA, 2000), the ultimate passive pressure is mobilized at a displacement of approximately 6 percent of the wall height or footing depth. The ultimate passive earth pressure may be calculated using a design EFP of 600 pcf and 350 pcf above and below the groundwater level, respectively. Oftentimes, the displacement to achieve ultimate passive earth pressures exceeds the allowable displacement of the structure. Eighty five percent of the ultimate passive resistance may be used for a displacement of approximately 3 percent of the wall height/footing depth, and 50 percent of the ultimate passive resistance may be used for a displacement of approximately 0.8 percent of the wall height/footing depth. Passive pressures may be combined with the base friction mobilized at the concrete-soil interface to resist lateral loading.

7.5.6 Base Friction

A coefficient of friction of 0.4 may be used for estimating the resistance due to base friction. The coefficient should be multiplied by the dead load only. The passive earth pressure and base friction mobilized at the concrete-subgrade interface may be combined to resist lateral loading.

7.6 Foundations

7.6.1 Existing Shallow Foundations

The existing building housing PS1 is supported on a combination of reinforced concrete mat foundations, slabs-on-grade and shallow spread footings. The slabs range from approximately 6 inches to 24 inches thick (SFDPW, 1909). Based on the shallow bedrock at the site, the foundations are predominantly supported by relatively strong, sandstone bedrock. However, due to varying levels of below-grade structures, portions of slabs and footings close to the deeper structures are likely underlain by backfill soils. The bearing capacities and modulus of subgrade reaction of existing foundations supported by bedrock and by backfill are provided in **Table 4 – Bearing Capacities and Modulus of Subgrade Reaction.** These values can be used in the analysis of the structures at PS1.

Table 4 – Bearing Capacities and Modulus of Subgrade Reaction

Bearing Material	Allowable Bearing[(1)] Capacity (psf)	Ultimate Bearing[(2)] Capacity (psf)	Modulus of Subgrade Reaction (pci)
Bedrock	12,000	36,000	360
Backfill Soils	3,000	9,000	60

Notes:
1. Allowable bearing capacities may be increased by 1/3 to account for short term seismic loading.
2. Ultimate bearing capacities are estimated based on a factor of safety of 3 against bearing failure.

7.6.2 Mitigation of Soil Settlement

Minor distress has occurred to the slab in the pump room since the pump conversion and seismic strengthening in 1973. A crack with some vertical offset has developed in the slab to the north of the northern pump. Also, the concrete pedestal for the northern pump appears to have settled. The cause of the distress and settlement is likely due to settlement of backfill behind the wall of the pump pit that is aligned southeast to northwest. The pump pit wall is approximately 18.5 feet below the foundation for the pump, extending down to approximately El. -23 feet. The wall backfill, placed circa 1911, may not have been well compacted and has likely settled over time. Due to the heterogeneous nature of undocumented historic backfill placed over 100 years ago, it is difficult to predict if there is potential for further settlement.

In order to limit future settlement potential, mitigation techniques include grouting the backfill or underpinning the affected portion of the slab with jet grout columns or micropiles. In our opinion, grouting techniques would be preferable due to

cost considerations, access limitations, and the unknown areal extent of loose backfill. For limited access sites such as in basements of existing structures, typically a small drill rig with compact dimensions and low overhead clearance are used for both grouting and micropile retrofits. Because drilling for grouting requires less downward force than micropile installation, very small hand-held equipment is sometimes used to create the grout injection holes. The grout pump can be located away from the work area, and pumped under pressure through a grout injection hose. Therefore, grouting may be more suitable for the PS1 project with its very tight access around the existing pumps. A series of brochures and project case histories with pictures of typical drill rigs are provided in **Appendix C**.

We recommend that some consideration be made to treat the backfill behind the southeastern half of the pump pit wall (between the stairway and Townsend Street), and not only the area with surface manifestations of settlement. Generally, the subsurface conditions should be similar along this segment of the wall, and partial remediation may lead to moving the differential settlement to an adjacent, untreated area.

7.6.2.1 Grouting

Several grouting methods are feasible. The most suitable for the site are:

a. Permeation grouting with ultra fine cement;
b. Compaction grouting; or
c. Jet grouting.

Permeation grouting involves the injection of a highly fluid grout under pressure through a grout pipe inserted into a pre-drilled hole. The use of ultra fine cement rather than standard Portland cement allows the grout to permeate more readily into the surrounding soil. This method of grouting would be the least expensive and least disruptive to existing operations, but is only suitable for sandy soils with less than approximately 15 percent fines. For fine-grained soils, the grout cannot permeate into the pore spaces between soil particles and would not be effective.

Compaction grouting can be used in a wide range of soil types and involves the pumping of a low viscosity grout bulb into the soil at high pressures. The grout bulb displaces and compacts the surrounding soil. When the grout take diminishes, the grout tube is raised or lowered to the next elevation and another grout bulb is pumped. Compaction grouting is more expensive than permeation grouting. Also, because the compaction grout is injected under higher pressures, care must be taken to not heave slabs or damage underground pipes and other structures.

Jet grouting involves the cutting of existing soil with a highly-pressurized stream of grout sometimes aided with air and/or water. The grout is mixed with the soil to form soilcrete columns. This method of grouting is suitable for a wide range of soil types, and is not limited to sandy soils like permeation grouting. This method would be the most expensive of the three, and would be the most disruptive to existing operations. Since jet grouting involves the injection of large amounts of grout, spoils are generated and would need to be handled and disposed.

We anticipate that compaction grouting would be the most appropriate of the three grouting methods for this project. This assumes that there are no underground utilities that may be impacted by the compaction grouting. After pilot holes are drilled by the grouting subcontractor, permeation grouting may also be a viable, lower cost alternative if the backfill consists of sandy soils with less than approximately 15 percent fines. With any of the grouting methods, any voids below the slab should be completely filled with grout.

7.6.2.2 Micropiles

A structural system, such as micropiles, may be used to bypass the wall backfill and support the slabs and foundations in the underlying bedrock. As indicated in Section 7.6.2, grouting techniques are likely more suited to the mitigation of soil settlement than micropile installation cost considerations, access limitations, and the unknown areal extent of loose backfill. However, if access can be provided for the limited-access drill rig, micropiles may be a feasible option.

Since the piles will be relatively lightly loaded, we anticipate that the piles will not need to be embedded very deep into the bedrock. For preliminary analysis of the vertical capacity of the micropiles, they can be designed by adding the contributions from shaft friction and end bearing. An allowable frictional resistance of 3,000 psf may be used for the portion of the micropile in bedrock. The frictional resistance in the fill is expected to be minor, and should be ignored. An allowable bearing capacity of 12,000 psf may be used to evaluate the end-bearing component. The micropiles should be embedded at least 5 feet into bedrock. Micropiles should be spaced at least three pile diameters apart, center-to-center. For cost estimating purposes, the top of the bedrock can be assumed to be at El. -23 feet SFCD, though the bedrock will likely be shallower as it slopes upward away from the bottom of the pump pit wall. During construction, the micropiles can be installed to the embedment depth necessary to achieve design capacity. Additional analyses can be performed if higher loads are needed for the seismic upgrade of the structures.

8.0 CLOSURE

The conclusions and recommendations presented herein are professional opinions based on geotechnical and geologic data and the project as described. A review by this office of any foundation, excavation, grading plans and specifications, or other work product that relies on the content of this report, together with the opportunity to make supplemental recommendations is considered an integral part of this study. Should unanticipated conditions come to light during project development or should the project change from that described, we should be given the opportunity to review our recommendations.

The findings and professional opinions presented in this report are presented within the limits prescribed by the client, in accordance with generally accepted professional engineering and geologic practices. There is no other warranty, either express or implied.

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APPENDIX A

HISTORIC PROJECT DRAWINGS

AWSS Pump Station No. 1
San Francisco, California

APPENDIX A

HISTORIC PROJECT DRAWINGS

Historic project drawings were prepared by the San Francisco Department of Public Works (SFDPW) in 1909 for the original construction, 1973 for the conversion of the existing AWSS PS1 pumps in the basement to a diesel drive system, and 1993 for the remodeling of the PS1 building into a three-story San Francisco Fire Department Headquarters building. Other historic drawings included undated structural drawings by Culley Associates, Inc. (CAI) for the conversion of the PS1 building into a three-story SFFDHQ building. Selected drawings that provide plans, profiles (sections), and details are compiled in this appendix including the following:

- Culley Associates, Inc. (CAI), Undated, SFFD Headquarters at Pump Station No. 1, 698 Second Street, San Francisco, California, Job No. 5228A.
    - Exterior Elevations, Sheet Nos. S3.01 and S3.02
    - Longitudinal Section, Sheet No. S3.03
    - Concrete Wall Details, Sheet No. S5.2

- San Francisco Department of Public Works (SFDPW), 1909b, Plans for an Auxiliary Water Supply System for Fire Protection, Contract No. 27 – Sec. H for the Construction and Installation of Mechanical Equipment for Pumping Station No. 1
    - Plan, Sheet No. 1
    - Longitudinal Section of Building, Sheet No. 2
    - Transverse Section of Building, Sheet No. 3

- SFDPW Bureau of Engineering, 1973, Fire Protection High Pressure Pump Station #1, Convert to Diesel Drive, October 24.
    - Vault Plan, Sheet 2 of 21, File B-38, 507
    - Building Cross Sections, Sheet 3 of 21, File B-38, 508
    - Pit and Wall Details, Sheet 6 of 21, File B-38, 511

- SFDPW Bureau of Architecture, 1993, SFFD Headquarters at Pump Station No. 1, 698 Second Street, Job No. 5228A, November 30.
    - Basement Plan, Sheet A2.1
    - Transverse Building Sections, Sheet A3.3
    - Longitudinal Building Section, Sheet A3.6

Oil Tank #5 Capacity-432 Brls.
Oil Tank #4 Capacity-432 Brls.
Oil Tank #3 Capacity-432 Brls.
Oil Tank #2 Capacity-432 Brls.
Oil Tank #1 Capacity-432 Brls.
Oil suction
Oil return
Blow off
Vent
Manhole
Oil Heater
Oil Pumps
Safety Valve
Stanford Street
Second Street
Townsend Street
Deep Well Pumps
Drain Pump
Boiler Feed Pumps
Feed Water Heaters
200 lb Gal Tank
4" Overflow
Electric Generating Set
3'x5' Sewer
Earth Fills
Return To Mechanical Section - 1550 Evans

PLAN
Scale: 1/8 in to 1 foot

DEPARTMENT OF PUBLIC WORKS
CITY AND COUNTY OF SAN FRANCISCO CALIFORNIA

PLANS FOR AN AUXILIARY WATER SUPPLY SYSTEM FOR FIRE PROTECTION

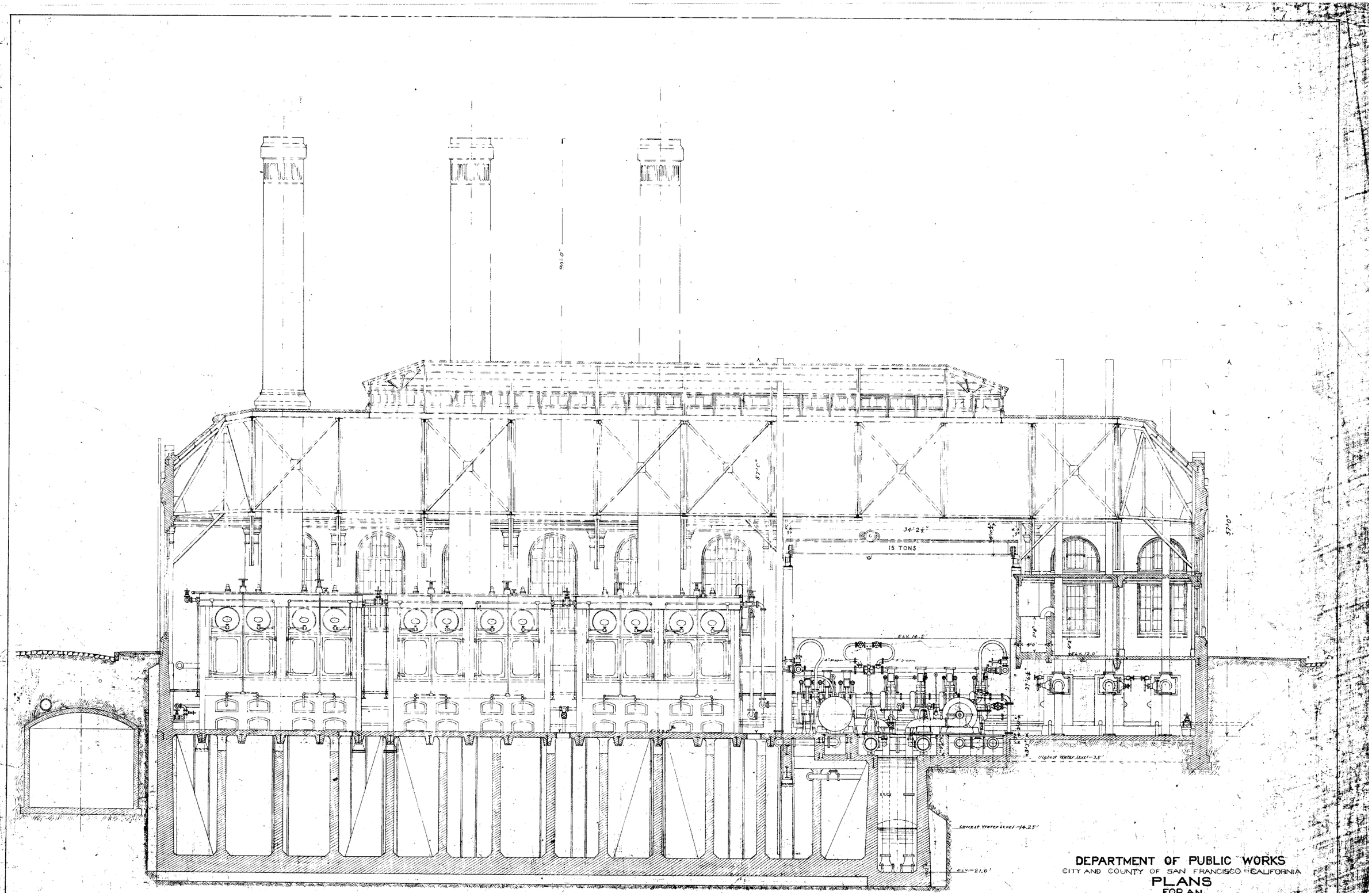
CONTRACT Nº 27-SEC. H

FOR THE CONSTRUCTION AND INSTALLATION OF MECHANICAL EQUIPMENT FOR

PUMPING STATION Nº 1

PREPARED BY MARSDEN MANSON CITY ENGINEER
UNDER DIRECTIONS FROM BOARD OF PUBLIC WORKS
SAN FRANCISCO CALIFORNIA 1909
3 SHEETS SHEET Nº 1

Case B Drawing 1656



SECTION ON LINE A-A

Scale: ⅛ in. to 1-foot.

Longitudinal Section of Building

DEPARTMENT OF PUBLIC WORKS

CITY AND COUNTY OF SAN FRANCISCO CALIFORNIA

PLANS

FOR AN

AUXILIARY WATER SUPPLY SYSTEM

FOR FIRE PROTECTION

CONTRACT Nº 27-SEC. H.

FOR THE CONSTRUCTION AND INSTALLATION OF

MECHANICAL EQUIPMENT FOR

PUMPING STATION Nº 1

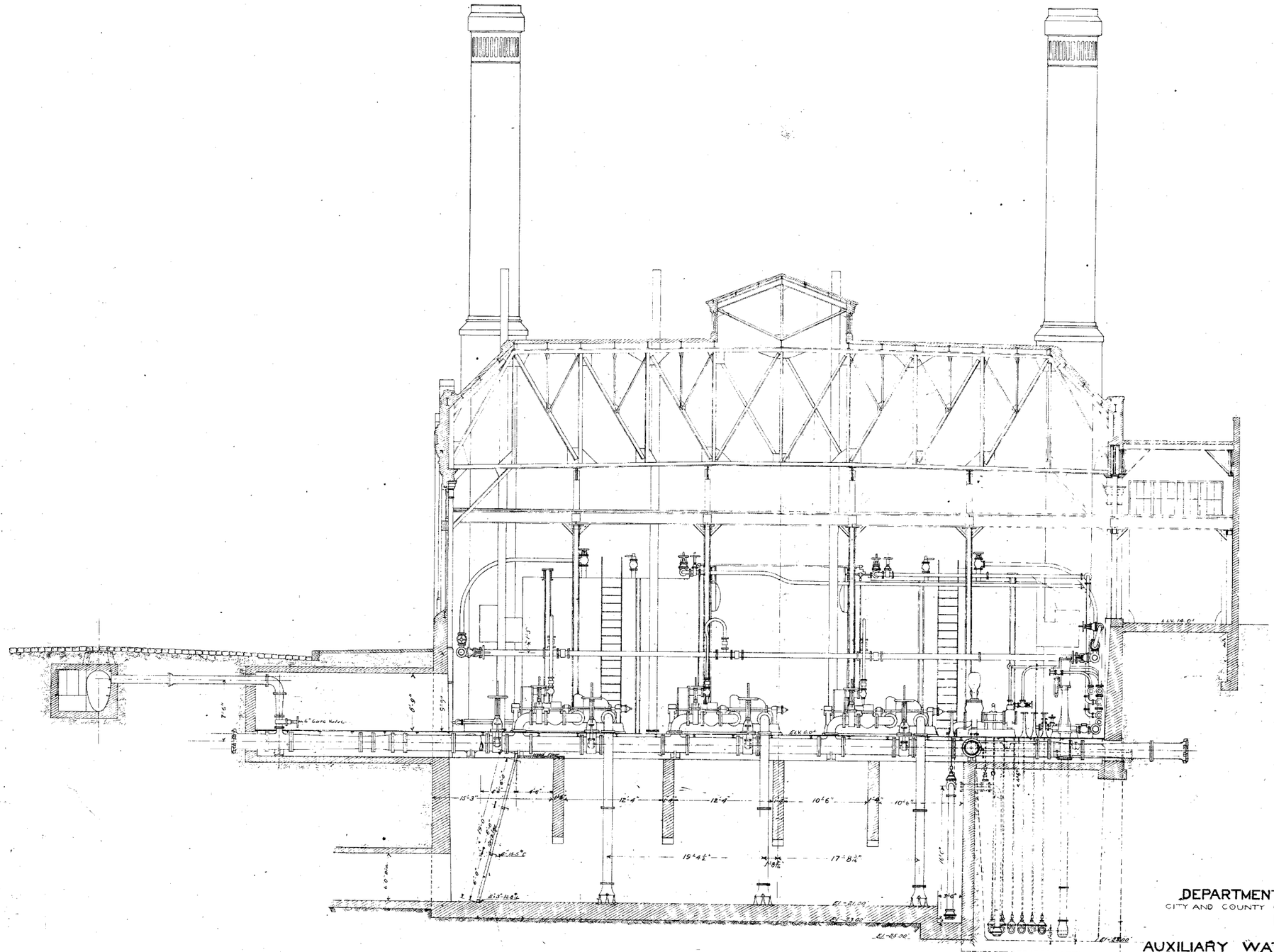
PREPARED BY MARSDEN MANSON CITY ENGINEER

UNDER DIRECTIONS FROM BOARD OF PUBLIC WORKS

SAN FRANCISCO CALIFORNIA

13 SHEETS SHEET Nº

Case A Drawing No.



SECTION ON LINE B-B
Scale 1/8 in. to 1 foot

Transverse Section of Building.

DEPARTMENT OF PUBLIC WORKS.
CITY AND COUNTY OF SAN FRANCISCO CALIFORNIA

PLANS
FOR AN
AUXILIARY WATER SUPPLY SYSTEM FOR FIRE PROTECTION
CONTRACT Nº 27 - SEC. H.
FOR THE CONSTRUCTION AND INSTALLATION OF MECHANICAL EQUIPMENT FOR
PUMPING STATION Nº 1

PREPARED BY MARSDEN MANSON, CITY ENGINEER
UNDER DIRECTIONS FROM BOARD OF PUBLIC WORKS
SAN FRANCISCO, CALIFORNIA
1908

3 SHEETS

8

Face of tile wall

Deep Well Pump Pit

℄ of columns

Pit № 8

Pit № 8

Floor slab on grade

Pit № 7

Underground Water Storage Tank

Col."G"

Col."H"

8'-2"

34'-5"

Pit № 6

Col."E"

Pit № 3

Col."F"

6'-0"

Pit № 5

Pit № 7

℄ of stairway & Building

2-Manholes to underground water storage

Pump Foundation № 3

Col."C"

PH № 2

Pump Foundation № 4

Col."D"

℄ of channel

Pit № 4

Floor of tile wall

Underground Water Storage Tank

To ℄ of tank wall

Pump Foundation № 1

Col."A"

Pump Foundation № 2

Col."B"

Pit № 1

Floor slab on grade

Face of tile wall

2½" = wall tile face to ℄ of column.

DIMENSION NOTES

1) Dimensions are taken from existing plans which are available to the contractor.
2) Dimensions which vary slightly from existing plans are those measured at the project site.
3) All dimensions shall be verified in the field by the contractor.

NOTES:

1) For location of this partial plan see Plan B-38,506

2) Sectional Plan taken just below pump room floor level.

For Details on filling pits ① through ⑧ with reinf. concrete See Plan B-38,511

Face of curb

TOWNSEND ST.

Venturi Meter Chamber

℄ of columns

Building Line

10'-0" Sidewalk

EXISTING PARTIAL PLAN

Note: Wall shifts 5" at col. "G" for details see Plan B-38,511

To face of existing wall

No wall footing for this 4'-0" construct 10" wall to fin. surface only.

End Wall

Deep Well Pump Pit

Fill this underground water storage tank with compacted sand backfill

Steel exhaust pipe support For Details See Plan B-38,512

℄ Tank & center wall

Manhole openings to underground water storage tanks.

Fill this underground water storage tank with compacted sand backfill

Day tank support For Details See Plan B-38,512

To face of existing wall

Earthquake vault for pumps and operational equipment

℄ of Building & stairway

Plan B-38,509

℄ of channel (6'-3" wide)

See Plan B-38,511

Day tank support For Details See Plan B-38,512

℄ Columns

Building Line

End Wall

Venturi Meter Chamber

Ladder for underground Exit, For Details See Plan B-38,512

See Plan B-38,508

Not to scale approx. 4'-0" clearance. Dist. = 52'-0" from building line. See Plan B-38,506

DIESEL FUEL STORAGE TANK № 2

DIESEL FUEL STORAGE TANK № 1

℄ of cradle

Steel fuel tanks are not shown.

Plan B-38,512

24'-0" 46'-0" 22'-0"

TOWNSEND ST.

Plan B-38,508

MODIFIED PARTIAL PLAN

FIELD PRINTS
CORRECTIONS AS BUILT
OFFICIAL COPY - RETURN TO ROOM 348E

NOTES

1. All pits ① through ⑧ shall be filled with concrete and the floor surface shall be tiled to match the adjacent existing floor tile surface.
2. All new floor construction shall be tiled as mentioned above.
3. Underground water storage tanks shall be backfilled with compacted sand through the existing manhole openings or the contractor may remove a portion of the 8" concrete floor slab, taking care not to cut existing reinforcing bars, and the backfill may be placed through the floor slab openings. The concrete floor slab and tiled surface shall be replaced all as designated by the engineer.
4. Cut & adjust existing steel cover plate over pits to fit new construction as directed by the Engineer.

PLANS ARE TO BE RETURNED WITH CORRECTIONS MADE AND CERTIFIED TO IN RED INK AND DATED BY THE INSPECTOR IN CHARGE UPON COMPLETION OF THE WORK BEFORE FINAL PAYMENT CAN BE MADE.

SIGNED- INSPECTOR

Gilbert G. Bender Oct. 24, 1973

REFERENCES INSERT FILE NOS OF SURVEYS, PLANS, ETC., USED

Spec." 23,528

CITY AND COUNTY OF SAN FRANCISCO
DEPARTMENT OF PUBLIC WORKS
BUREAU OF ENGINEERING

FIRE PROTECTION HIGH PRESSURE PUMPING STATION #1 CONVERT TO DIESEL DRIVE

VAULT PLAN

SURVEYED BY	J.T.A.	June '73
DRAWN BY	N.H.	June '73
TRACED BY		
CHECKED BY	J.T.A.	

SCALE 3/16"=1'-0" SHEET 2 OF 21 SHEETS

APPROVED
SECTION ENG. [illegible] 10/24/73
DIVISION ENG. [illegible] 10/24/73

APPROVED Richard C. Low DATE 4/16/74 CITY ENGINEER

FILE B-38,507 CHANGE

TABLE OF CHANGES
CAUTION: CHECK WITH TRACING TO SEE IF YOU HAVE LATEST REVISION

NO.	DATE	DESCRIPTION	BY

Townsend St.

Note: For cradle details See Plan B-38512

Backfill with sand

New 30,000 Gal. Steel Tank

Tank cradle

Note: The contractor shall satisfy himself as to the quantity of rock excavation involved and also the location of the ground water table.

Building Line

Face of existing wall

Plan B-38509

Daytank support For Details See Plan B-38512

Fill pit with cone. See Plan B-38511

Fill pit with concrete

Existing Steel girders set into concrete deep girder

See Detail "A"

Backfill these two water storage tanks with compacted sand.

Soffit of existing deep girder

Deep Well Pump Pit

Backfill with compacted sand.

Pump room floor level

Building loading ramp

End wall 10" min.

Add 2-"6 x 7'-0" lg. at (total 4) over door opening

Face of existing wall

Cols. ℄ of Building Cols.

SECTION "B-B"

3/16" = 1'-0" Plan B-38507

Typical top slab reinf. "6 @ 4" e.w. e.f.

Add 2-"5 x 5'-0" lg. at (total 4) over door opening

Add 2-"6 x 6'-6" lg. at (total 4) over dr. opening.

Top of pipes sealed with welded cover plate and all edges ground smooth. (Typical)

See Typical Ladder Details

End wall 10" min

See Detail "A"

8 Equal Spaces @ 1'-3" = 10'-0"

Typical wall reinf. "6 @ 12" e.w. e.f.

Daytank support For Details See Plan B-38512

Pump room floor level

Backfill where void spaces are encountered under exist. floor slab

To face of existing wall

Fill pits with concrete

Note: Typical all openings in vault wall. 1st reinf. bar shall be 1½" from opening.

SECTION "C-C"

3/16" = 1'-0" Plan B-38507

℄ Ladder

Top of pipes sealed with welded cover plate and all edges ground smooth.

Top of roof slab

8 Equal Spaces @ 1'-3" = 10'-0"

Steel Rung

Face of out side wall

Pump room floor level

TYPICAL LADDER

½" = 1'-0"

SECTION "D-D"

½" = 1'-0"

2½" steel Galv. pipe extra strength

½" dia. Galv. steel bar

SECT. "E-E"

1" = 1'-0"

FIELD PRINTS
CORRECTIONS AS BUILT
OFFICIAL COPY - RETURN TO ROOM 3586

10" ± Min. wall stem

Existing wall

"4 @ 12"

8-"4 continuous

New matching tile floor surface.

Exist. pump room floor level

½" square continuous slot filled with sealant.

"6 @ 12"

2'-6"

This typical footing shall be used at both end walls unless otherwise shown.

Footing slab shall be cut into existing floor slab. No reinf. bars on structural steel shall be cut unless otherwise order by Engineer.

PLANS ARE TO BE RETURNED WITH CORRECTIONS MADE AND CERTIFIED TO IN RED INK AND DATED BY THE INSPECTOR IN CHARGE UPON COMPLETION OF THE WORK BEFORE FINAL PAYMENT CAN BE MADE.

SIGNED: INSPECTOR: Gilbert A. Bending Oct. 24, 1973

DETAIL "A"

1" = 1'-0"

10" Wall

Exist. floor tile

See Detail "A"

New matching floor tile

Exist. floor tile

For Pit reinf. See Plan

Typical Pit

Varies

½" square continuous slot around pit filled with sealant

DETAIL "F"

No scale

NOTE: Where the footing slab is located in one of the pits. The reinf. bars from the wall shall be bent to fit the pit as shown.

REFERENCES (INDEX FILE NO'S OF SURVYS, PLANS, ETC., USED)

Spec. # 28528

DESIGNED BY	J.T.A.	July 1973
DRAWN BY	M.F.	July 1973
TRACED BY		
CHECKED BY	J.T.A.	"

APPROVED
SECTION ENG. [illegible] 10/25/73
DIVISION ENG. [illegible] 2/14/74
DATE 2/12/74

TABLE OF CHANGES
CAUTION: CHECK WITH TRACING TO SEE IF YOU HAVE LATEST REVISION

CITY AND COUNTY OF SAN FRANCISCO
DEPARTMENT OF PUBLIC WORKS
BUREAU OF ENGINEERING

FIRE PROTECTION HIGH PRESSURE PUMPING STATION #1 CONVERT TO DIESEL DRIVE

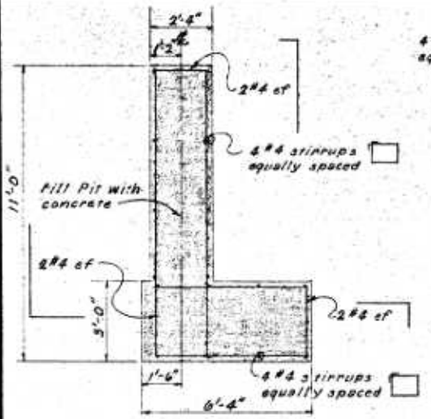
BUILDING CROSS SECTIONS

SCALE: As shown

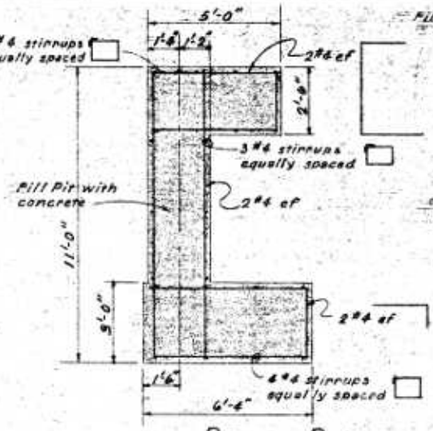
SHEET 3 OF 21 SHEETS

FILE B-38,508

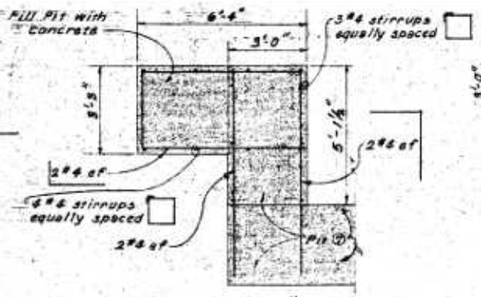
CHANGE



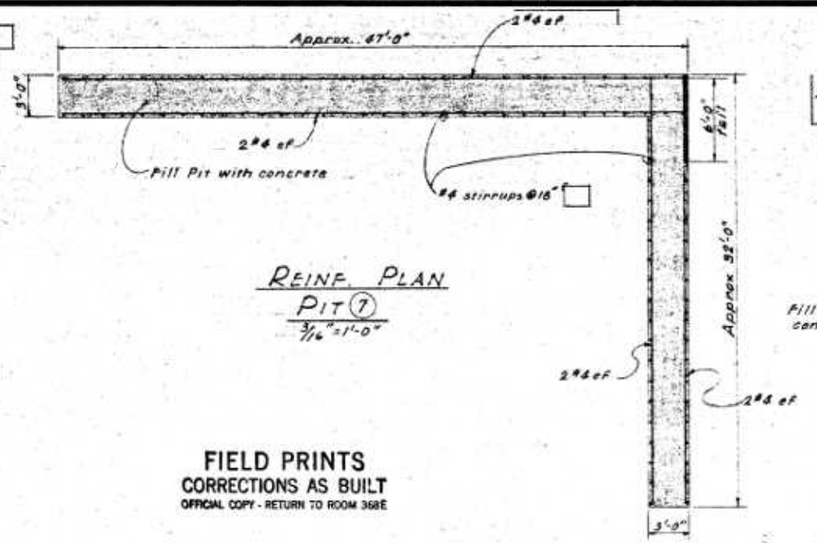
REINF. PLAN
PITS ① AND ②
3/8" = 1'-0"



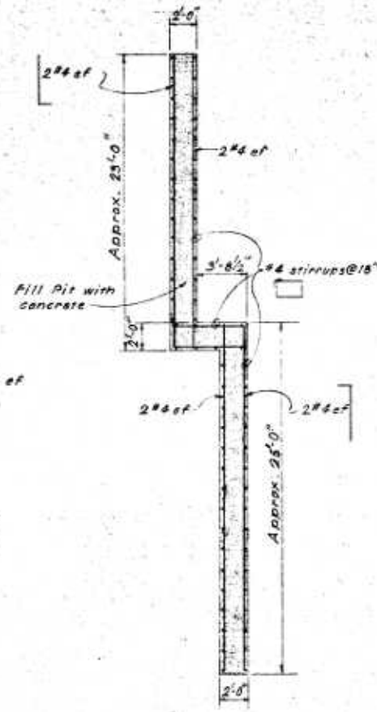
REINF. PLAN
PIT ③
3/8" = 1'-0"



REINF. PLAN
PITS ④, ⑤ AND ⑥
3/8" = 1'-0"

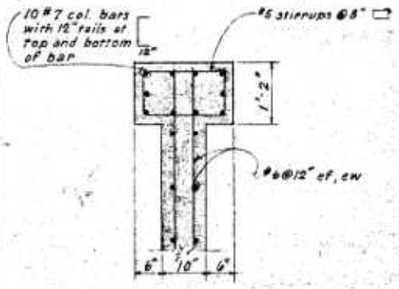


REINF. PLAN
PIT ⑦
3/16" = 1'-0"

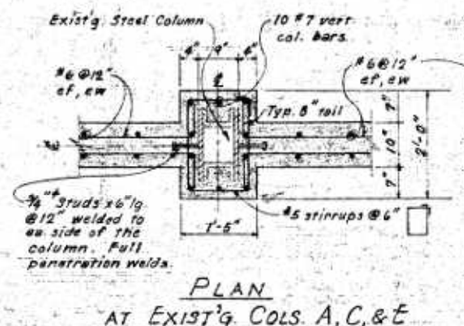


REINF. PLAN
PIT ⑧
3/16" = 1'-0"

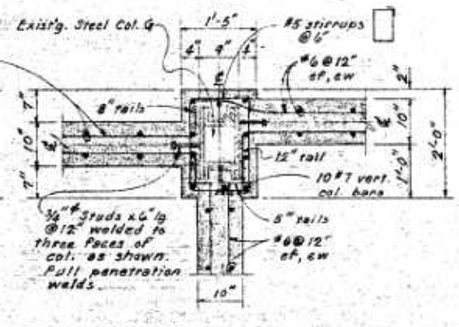
FIELD PRINTS
CORRECTIONS AS BUILT
OFFICIAL COPY - RETURN TO ROOM 368E



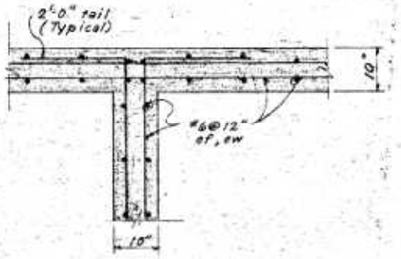
PLAN
TYPICAL TEE COLUMN SUPPORT
3/4" = 1'-0"



PLAN
AT EXIST'G. COLS. A, C, & E
3/4" = 1'-0"



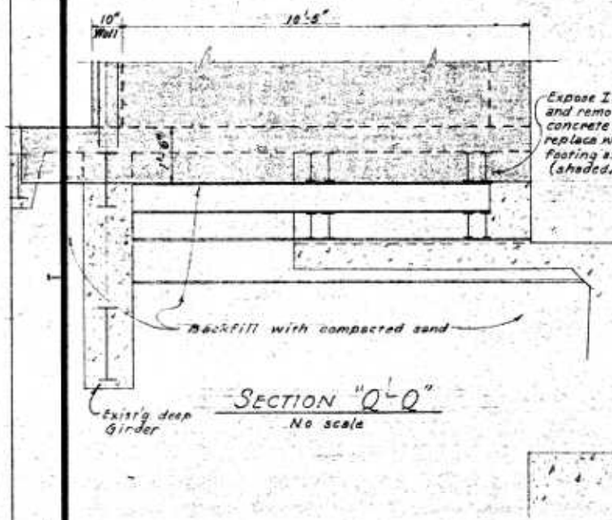
PLAN
AT EXIST'G. COL. G
3/4" = 1'-0"



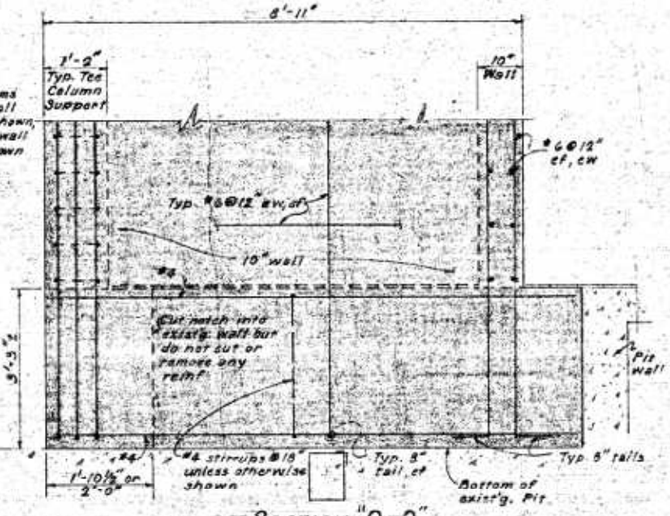
PARTIAL PLAN
TYP. VAULT WALL INTERSECTION
3/8" = 1'-0"

GENERAL NOTES:

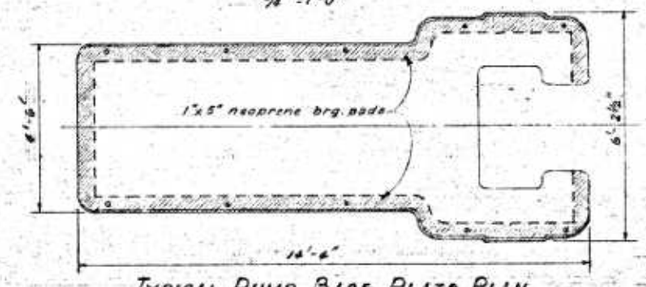
1. Where Pits ① thru ⑧ are filled with concrete the top surface shall be tiled over to match the exist'g. floor tile.
2. Dimensions shown for pits are taken from exist'g. plans. The intent of this contract is to fill the exist'g. pits completely with reinf. concrete to Pit Field conditions regardless of dimension shown. #4 bar reinf. shall conform to field conditions.
3. Exist'g. pump base bolts which are rusted, broken or declared unservicable by the Engineer during construction of the 1½" x 5" groove under the base plate shall be replaced by removing the exist'g. concrete in a 6" radius around the bolt and 12" into concrete foundation. A new galvanized bolt shall be replaced and the replaced concrete shall be an approved epoxy type concrete.



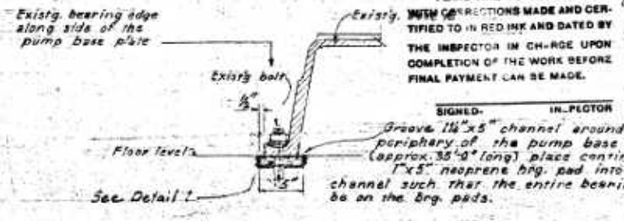
SECTION "Q-Q"
No scale



SECTION "R-R"
TYPICAL SECTION THRU PIT AT WALL
3/4" = 1'-0"

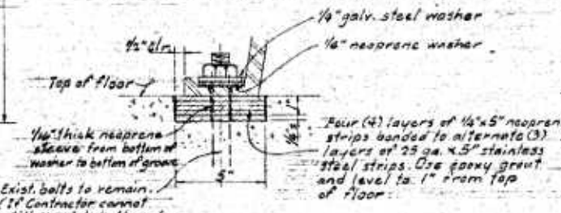


TYPICAL PUMP BASE PLATE PLAN
No scale



TYPICAL BEARING UNDER PUMP BASE PLATE
No scale

Total:
4 pump base plates.



DETAIL 1
3" = 1'-0"

PLANS ARE TO BE RETURNED WITH CORRECTIONS MADE AND CERTIFIED TO IN RED INK AND DATED BY THE INSPECTOR IN CHARGE UPON COMPLETION OF THE WORK BEFORE FINAL PAYMENT CAN BE MADE.

SIGNED- INSPECTOR

Milford G. Bentley
Oct. 24, 1973

REFERENCES SHEET FILE NO'S OF SURVEYS, PLANS, ETC., USED
Spec. # 23,525

DESIGNED BY	J.T.A	Aug '73
DRAWN BY	L.Q.W	Aug '73
TRACED BY		
CHECKED BY	J.T.A	
APPROVED SECTION ENGR.		10/25/73
APPROVED DIVISION ENGR.		2/14/74

CITY AND COUNTY OF SAN FRANCISCO
DEPARTMENT OF PUBLIC WORKS
BUREAU OF ENGINEERING

FIRE PROTECTION HIGH PRESSURE PUMPING STATION #1 CONVERT TO DIESEL DRIVE

PIT AND WALL DETAILS

SCALE As noted — SHEET 6 OF 21 SHEETS

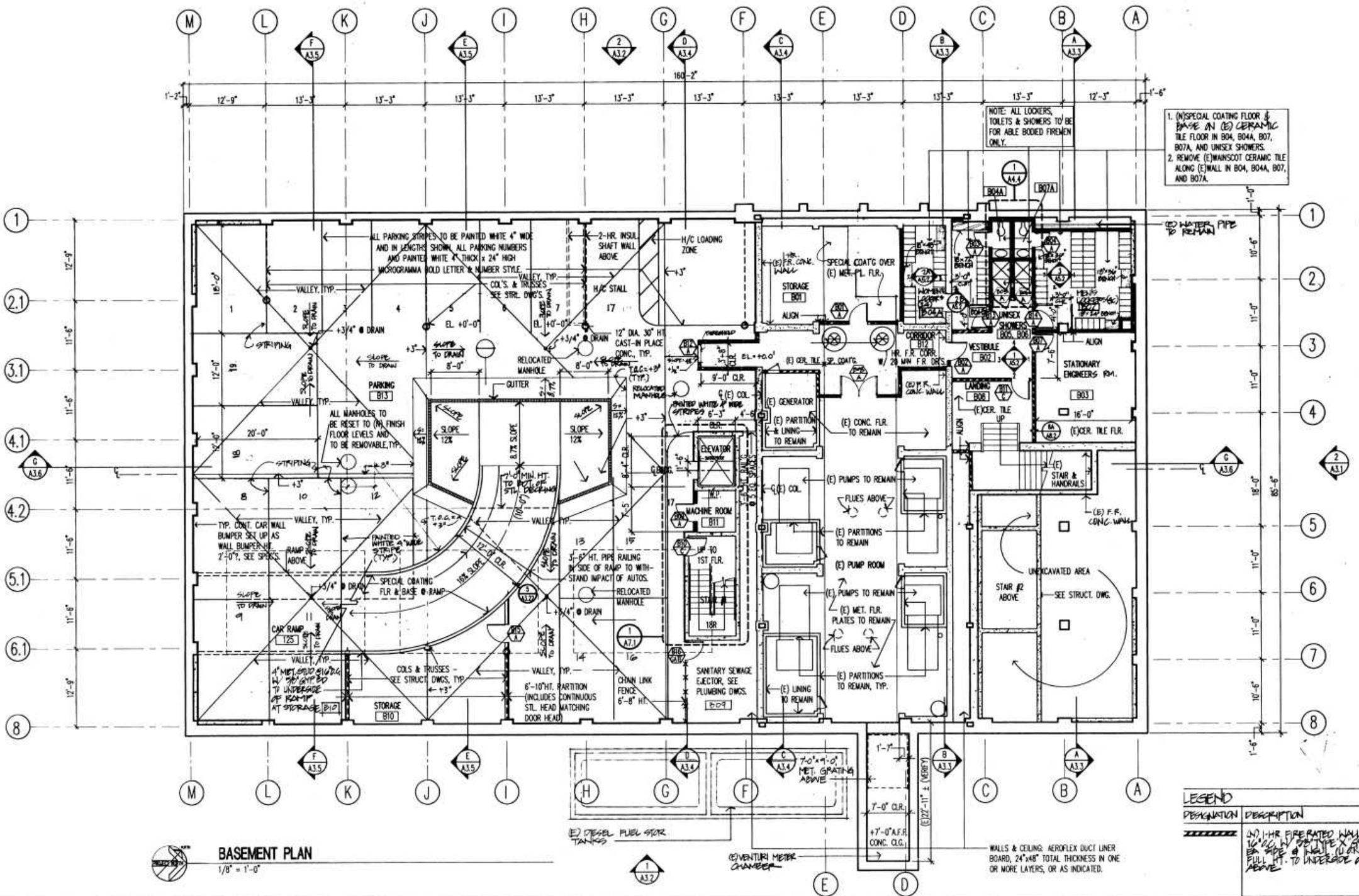
APPROVED [signature] DATE 2/14/74 — ENGINEER

FILE B-38,511 — CHANGE

NO.	DATE	DESCRIPTION	BY

TABLE OF CHANGES
CAUTION: CHECK WITH TRACING TO SEE IF YOU HAVE LATEST REVISION

SA-T 55-68



BASEMENT PLAN

1/8" = 1'-0"

BUREAU OF ARCHITECTURE
DEPARTMENT OF PUBLIC WORKS
CITY AND COUNTY OF SAN FRANCISCO

CITY ARCHITECT: N. KARASICK	DATE: 100%- 11/30/93
DIV. ARCHITECT: R. WONG	PROJ. ARCHITECT: C. COHEN
CHECKED: C. COHEN	DRAWN: A. GIN/S. FAN/B. WONG

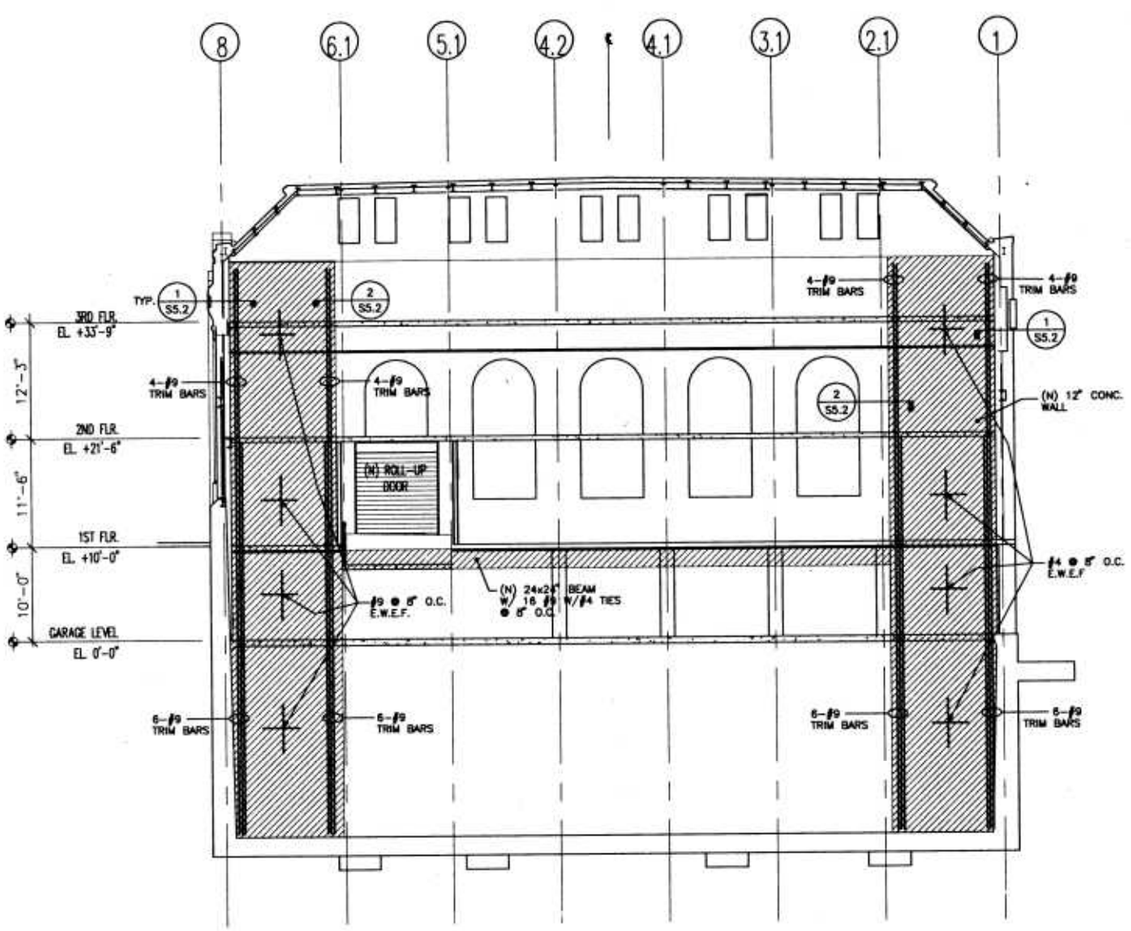
PROJECT TITLE
SFFD HEADQUARTERS AT PUMP STATION NO. 1
698 SECOND STREET, SAN FRANCISCO, CALIFORNIA

DRAWING TITLE
BASEMENT PLAN

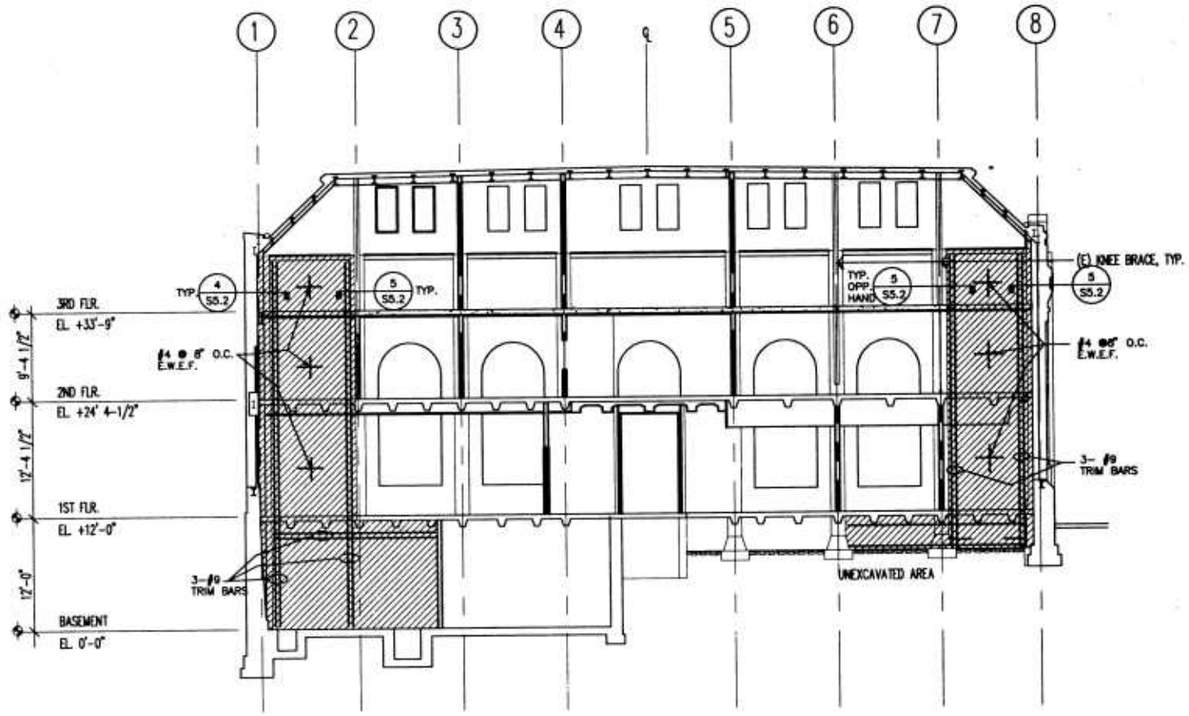
SHEET **A2.1**
6 OF SHEETS
JOB NO. **5228A**

REVISED: 11-18-93

EBI Revisions 11-30-93



1 S3.01 STANFORD STREET ELEVATION LINE M
1/8" = 1'-0"



2 S3.01 SECOND STREET ELEVATION LINE A
1/8" = 1'-0"

CULLEY
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N. KARABECK
R. WONG
C. COHEN
C. COHEN
A. GINES FANEL WONG

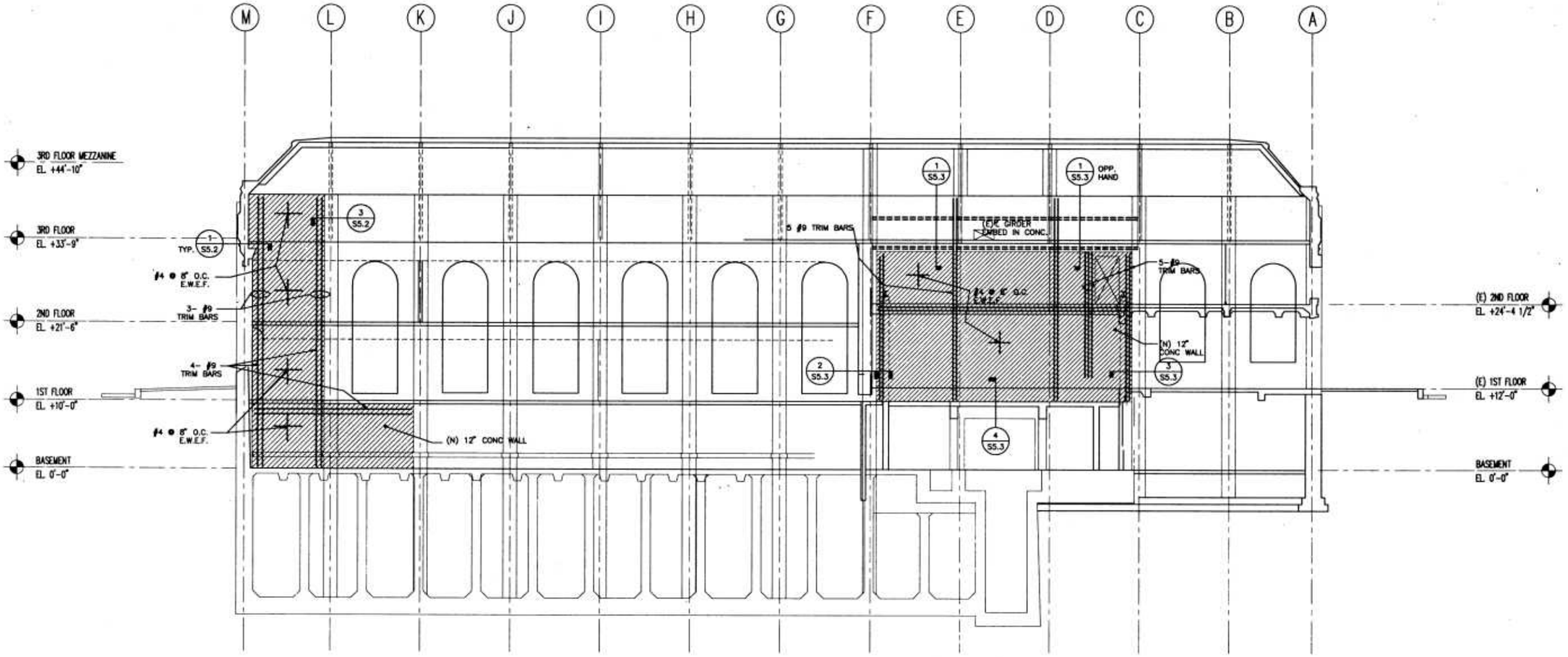
SFFD HEADQUARTERS AT PUMP STATION NO. 1
698 SECOND STREET, SAN FRANCISCO, CALIFORNIA

EXTERIOR ELEVATIONS

C.A.I.# 92037

S3.01

5228A



1 S3.02 REAR WALL ELEVATION LINE 1

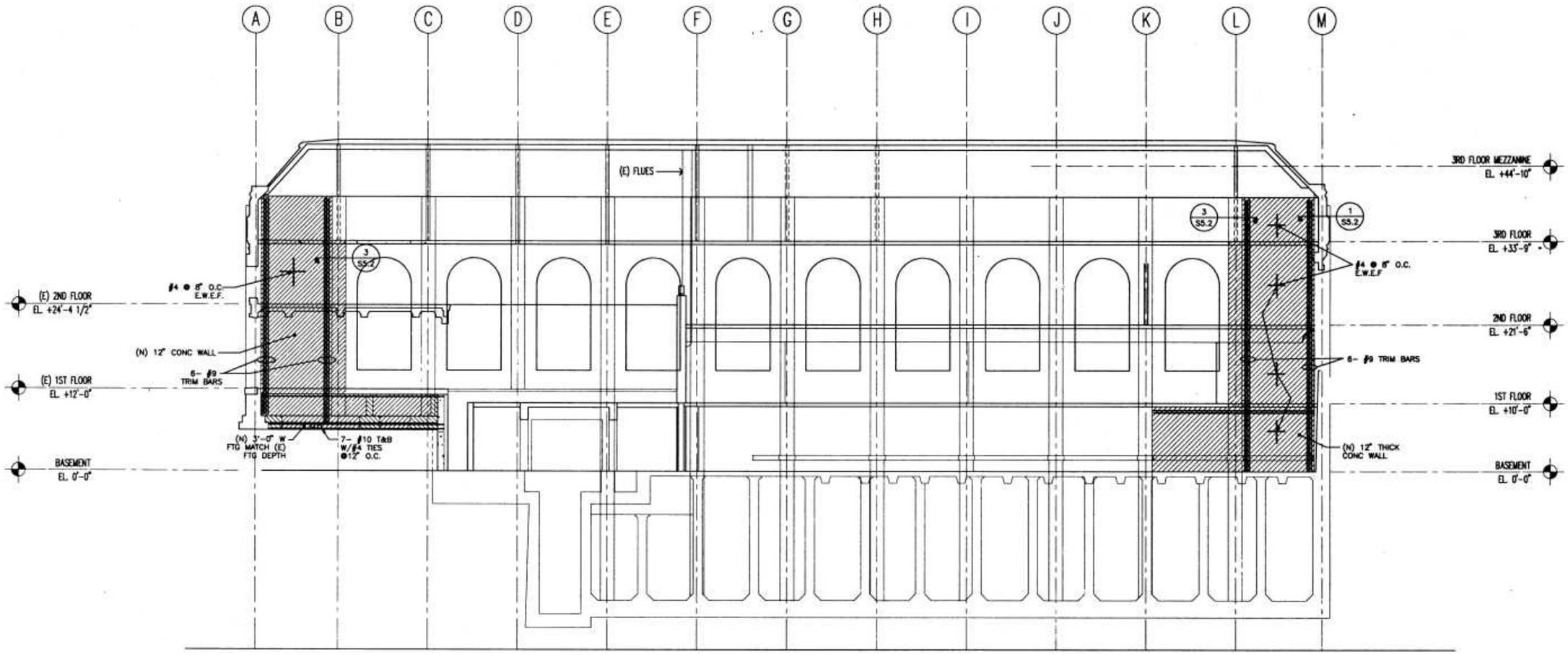
1/8"=1'-0"

CULLEY
Consulting Structural Engineers
CULLEY ASSOCIATES INC.
303 2nd Street, Suite 850
San Francisco, California 94107
Tel:(415)543-0780 Fax:(415)495-5357

R. KARASICK	12-19-02	SFFD HEADQUARTERS AT PUMP STATION NO. 1
R. WONG	C. COHEN	698 SECOND STREET, SAN FRANCISCO, CALIFORNIA
C. COHEN	A. GNIR, FAWR, WONG	EXTERIOR ELEVATIONS

S3.02

CALE 92037 5228A

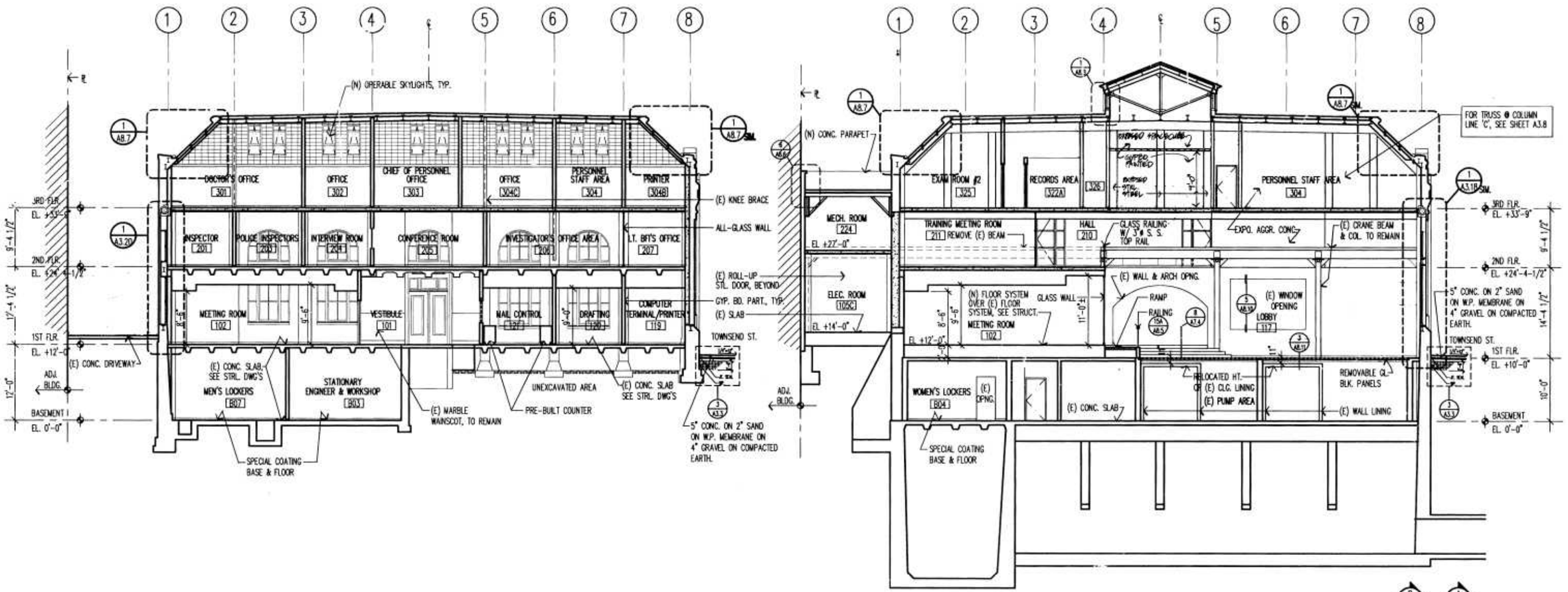


1 / S3.03 TOWNSEND STREET WALL ELEVATION LINE 8

1/8"=1'-0"

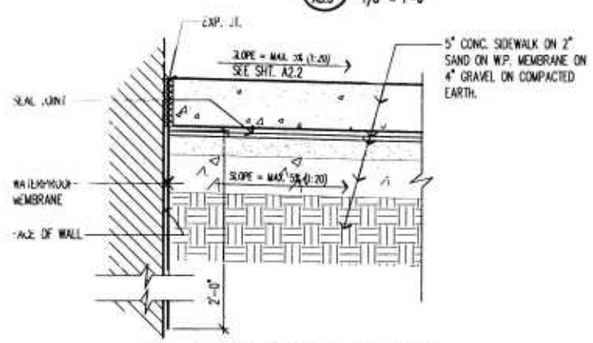
CULLEY
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CULLEY ASSOCIATES INC.
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San Francisco, California 94107
Tel:(415)543-8760 Fax:(415)495-5357

K.KAWABOX		SFFD HEADQUARTERS AT PUMP STATION NO.1	S3.03
R.WONG	C.COHEN	698 SECOND STREET, SAN FRANCISCO, CALIFORNIA	
C.COHEN	A.GNAR, FAIA/R.WONG	LONGITUDINAL SECTION	C.A.I.# 92037 5228A

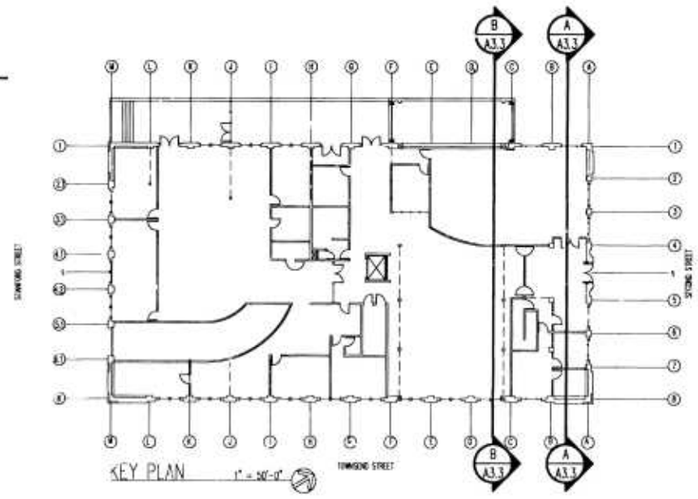


1 A3.3 **BUILDING SECTION A-A**
1/8" = 1'-0"

2 A3.3 **BUILDING SECTION B-B**
1/8" = 1'-0"



3 A3.3 **TYP. CONC. SLAB @ SIDEWALK**
1 1/2" = 1'-0"



BUREAU OF ARCHITECTURE
DEPARTMENT OF PUBLIC WORKS
CITY AND COUNTY OF SAN FRANCISCO

CITY ARCHITECT: N. KARASICK	DATE: 100%- 11/30/93
DIV. ARCHITECT: R. WONG	PROJ. ARCHITECT: C. COHEN
CHECKED: C. COHEN	DRAWN: A. GIN/S. FAN/B. WONG

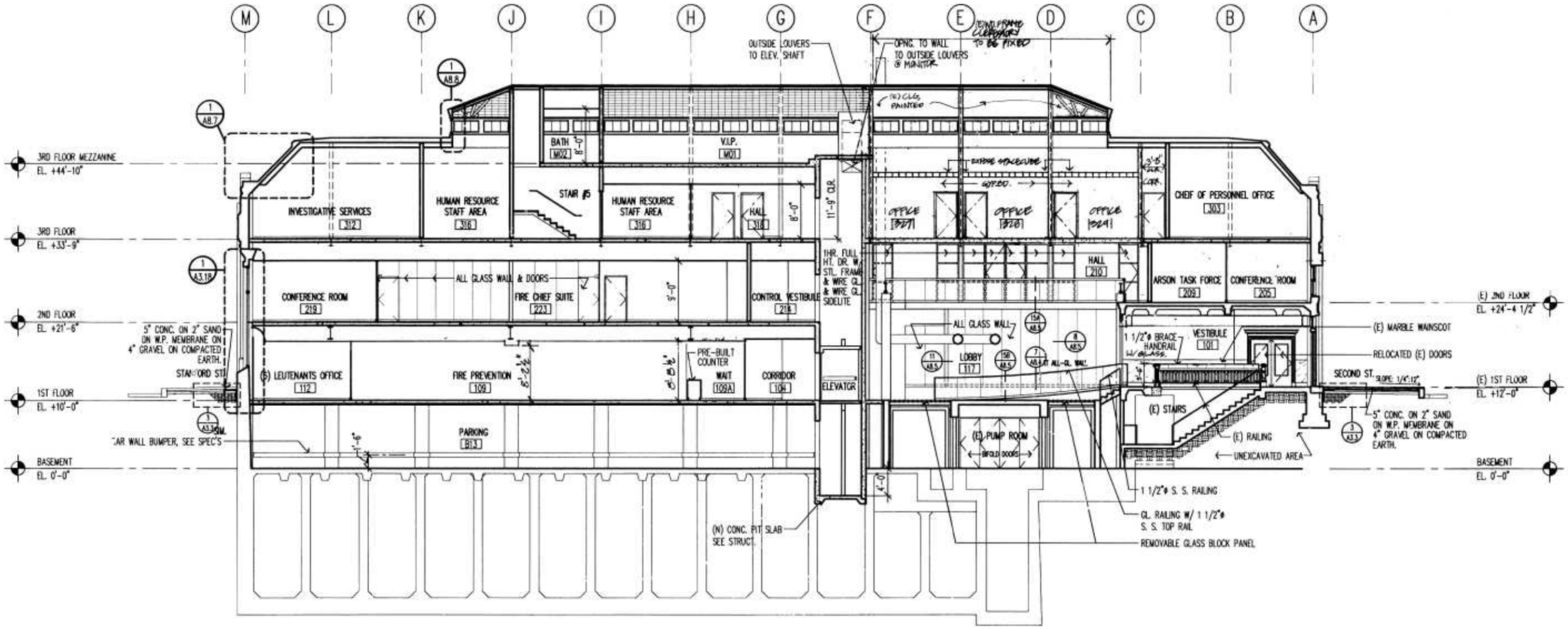
PROJECT TITLE: SFFD HEADQUARTERS AT PUMP STATION NO. 1
698 SECOND STREET, SAN FRANCISCO, CALIFORNIA

DRAWING TITLE: TRANSVERSE BUILDING SECTIONS

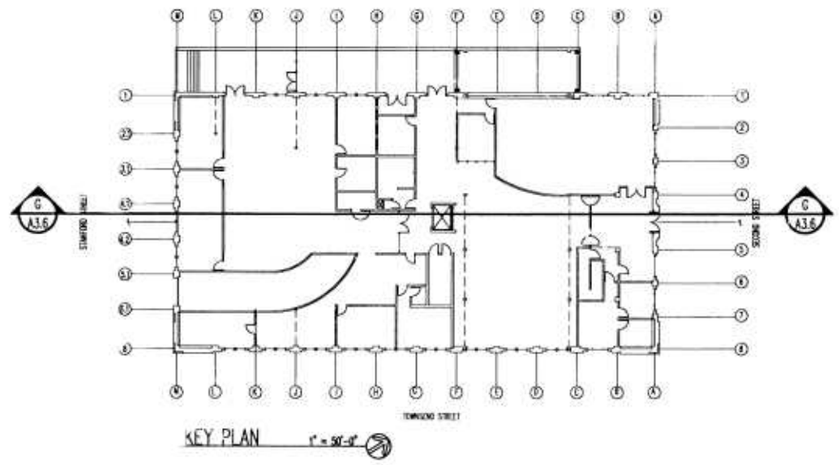
SHEET: A3.3
17 OF SHEETS
JOB NO. 5228A

HBN RESUBMITTAL 11-30-93

REVISED 10-2-93



1 / A3.6 **BUILDING SECTION G-G**
1/8"=1'-0"



BUREAU OF ARCHITECTURE
DEPARTMENT OF PUBLIC WORKS
CITY AND COUNTY OF SAN FRANCISCO

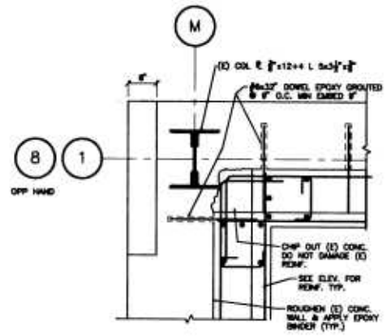
CITY ARCHITECT: N. KARASICK	DATE: 100%- 11/30/93
DIV. ARCHITECT: R. WONG	PROJ. ARCHITECT: C. COHEN
CHECKED: C. COHEN	DRAWN: A. GIN/S. FAN/B. WONG

PROJECT TITLE: SFFD HEADQUARTERS AT PUMP STATION NO. 1
698 SECOND STREET, SAN FRANCISCO, CALIFORNIA

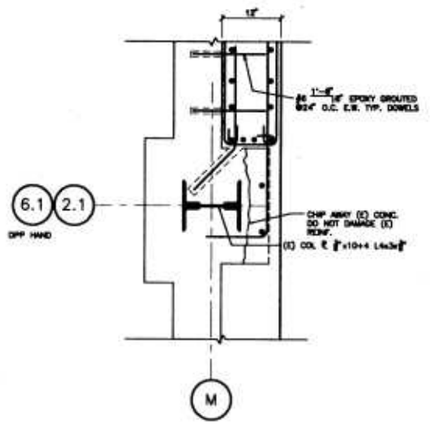
DRAWING TITLE: LONGITUDINAL BUILDING SECTION

SHEET: A3.6
20 OF SHEETS
JOB NO. 5228A

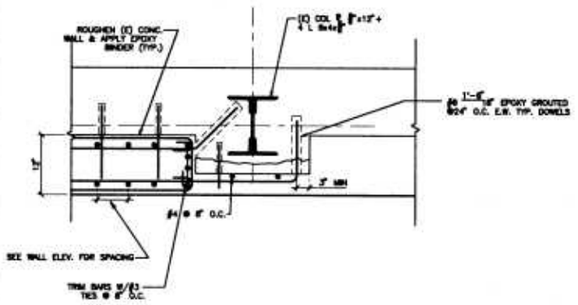
BPI RESUBMITTAL 1-20-93



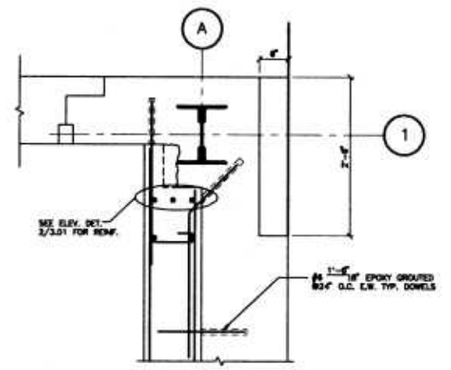
1 S5.2 (N) CONC WALL CORNER TO (E) WALL & PILASTER 3/4"=1'-0"



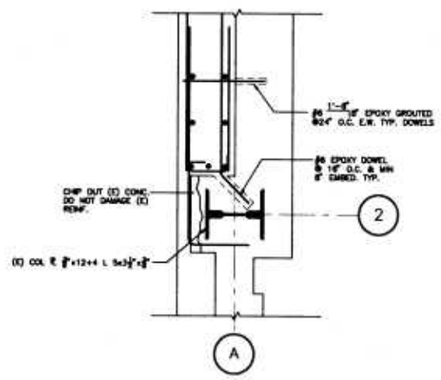
2 S5.2 (N) CONC WALL END TO (E) WALL & PILASTER 3/4"=1'-0"



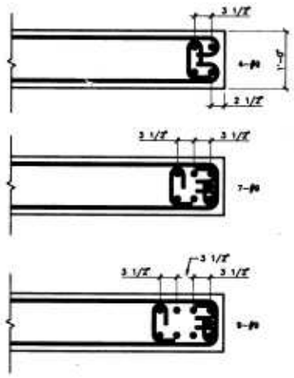
3 S5.2 (N) CONC WALL END TO (E) WALL & PILASTER 3/4"=1'-0"



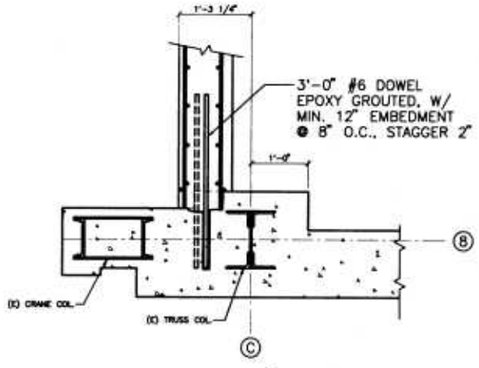
4 S5.2 (N) CONC WALL END TO (E) WALL & PILASTER 3/4"=1'-0"



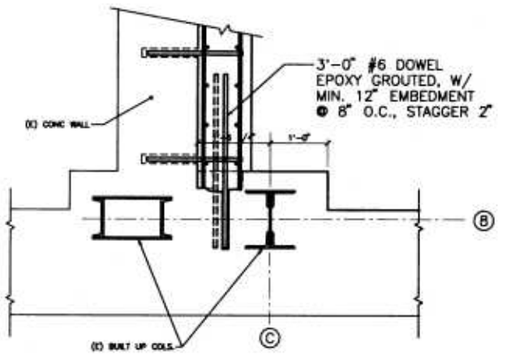
5 S5.2 (N) CONC WALL END TO (E) WALL & PILASTER 3/4"=1'-0"



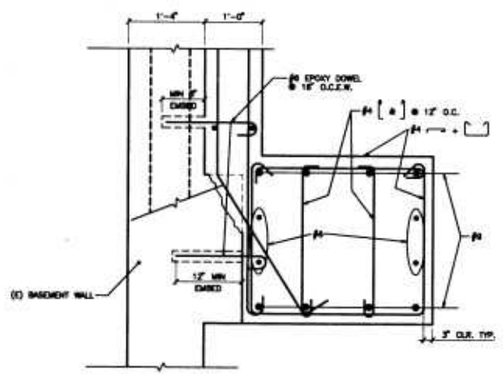
8 S5.2 TRIM BAR LAYOUT (SEE WALL ELEVATIONS) FOR SIZE & SPACING 3/4"=1'-0"



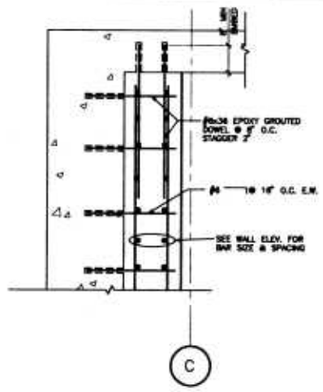
7 S5.2 (N) CONC WALL TO (E) PILASTER 3/4"=1'-0"



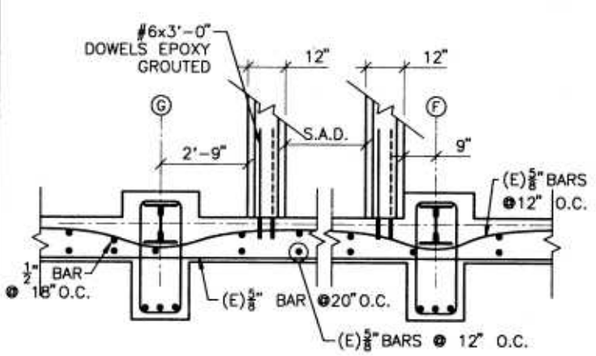
8 S5.2 (N) CONC WALL TO (E) PILASTER 3/4"=1'-0"



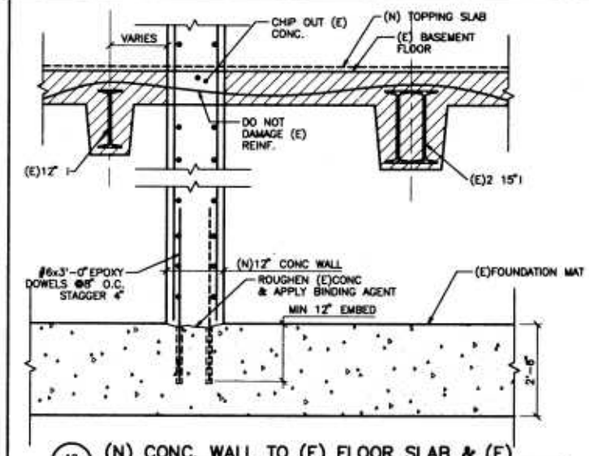
9 S5.2 (N) CONC. WALL FND. TO (E) BASEMENT WALL 3/4"=1'-0"



10 S5.2 (N) CONC. WALL TO (E) CONC FDN. WALL 3/4"=1'-0"



11 S5.2 (N) CONC. WALL TO (E) CONC FDN. WALL 3/4"=1'-0"



12 S5.2 (N) CONC. WALL TO (E) FLOOR SLAB & (E) FOUNDATION MAT 3/4"=1'-0"

CULLEY
Consulting Structural Engineers
CULLEY ASSOCIATES INC.
303 2nd Street, Suite 680
San Francisco, California 94107
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N. KARABOK

R. WONG C. COHEN

C. COHEN A. GIMBEL PAWEL WONG

SFFD HEADQUARTERS AT PUMP STATION NO. 1
698 SECOND STREET, SAN FRANCISCO, CALIFORNIA

CONCRETE WALL DETAILS

C.A.I.# 92037

S5.2

5228A

APPENDIX B

SITE-SPECIFIC ACCELERATION RESPONSE SPECTRA

AWSS Pump Station No. 1
698 2nd Street, San Francisco, California

APPENDIX B

SITE-SPECIFIC ACCELERATION RESPONSE SPECTRA

We evaluated the site-specific horizontal ground acceleration response spectra at 5% critical damping for the purpose of seismic design in accordance with IBC 2009/CBC 2010, and SFPUC GSR (Revision 2, October 6, 2009).

The site-specific acceleration response spectra were developed for the various SFPUC seismic performance levels such as, the Seismic Performance Classes (SPC) I, II and III, and Basic Safety Earthquake (BSE) -1 and -2 (SFPUC, 2009). For each seismic performance level, the site-specific acceleration response spectra were developed based on the lesser of the probabilistic seismic hazard assessment (PSHA) and deterministic seismic hazard assessment (DSHA), and subject to deterministic limits of ASCE7-05 (ASCE, 2005) and SFPUC GSR (SFPUC, 2009).

The site-specific horizontal ground acceleration response spectra at 5% critical damping are presented in **Figure B1 – Site-Specific Horizontal ARS**. Results from evaluation of site-specific ground acceleration response spectra are presented in **Table B1 – Site-Specific Horizontal Ground Accelerations for Pump Station 1**.

PROBABILISTIC SEISMIC HAZARD ASSESSMENT

Probabilistic seismic hazard assessment (PSHA) calculates the seismic hazard at the project site by combining the probabilities of earthquake scenarios of different magnitudes and site-to-fault distances with predictions of the resulting ground motion intensity (McGuire, 2004; Kramer, 1996). Uncertainties in ground motion predictions are addressed in the Next Generation Attenuation (NGA) ground motion prediction equations (GMPEs) which are documented in EERI (2008). Aleatory (random) uncertainties are treated by considering earthquake events with all possible magnitudes and distances. Epistemic uncertainties (due to inadequate knowledge) are accounted for in the GMPEs through a logic tree that assigns a specific weight to each of the GMPEs (Petersen et al., 2008). Uncertainties in earthquake scenarios and GMPEs are incorporated in the PSHA.

PSHA was carried out to evaluate horizontal spectral ground (geometric mean) accelerations in accordance with the 2008 update of the U.S. National Seismic Hazard Maps (NSHM) (Petersen et al., 2008) and the companion 2008 NSHM PSHA interactive seismic de-aggregation available at https://geohazards.usgs.gov/deaggint/2008/index.php. PSHA was evaluated at three recurrence intervals: 475-year return period (10% probability of exceedance in 50 years) for BSE-1; 975-year return period (5% probability of exceedance in 50 years) for SPC

I and II; and 2,475-year return period (2% probability of exceedance in 50 years) for BSE-2 and SPC III. The horizontal spectral accelerations were developed at 5% critical damping for periods of 0, 0.1, 0.2, 0.3, 0.5, 1, 2, 3, 4 and 5 seconds. Based on review of available shear wave velocity data from geophysical (suspension) logging performed by GeoVision and Norcal Geophysical Consultants for several deep borings located in the vicinity of Pump Station No. 1 (Arup, 2009), an average shear wave velocity of 1,100 m/s for the upper 30 m was used to generate the site-specific PSHA spectral accelerations.

Results from the PSHA at 5% critical damping are presented for the three site-specific horizontal Uniform Hazard Spectra at 475-, 975- and 2,475-year return periods in **Figure B2 – Site-Specific Probabilistic & Deterministic Horizontal ARS**.

DETERMINISTIC SEISMIC HAZARD ASSESSMENT

Deterministic seismic hazard assessment (DSHA) was performed for the project site to develop site-specific horizontal acceleration response spectra at 5% critical damping for the maximum earthquake in each of the active faults described in Appendix A of the SFPUC GSR (SFPUC, 2009). In developing the site-specific response spectra, we considered the major earthquakes faults (San Andreas, Hayward and Calaveras faults) as defined on page 6 of Chapter 3 of the Request for Proposals for the Planning Support Services for the AWSS, Agreement No. CS-199 by SFPUC, dated May 11, 2011.

DSHA was carried out using the five 2008 NGA GMPEs: Abrahamson-Silva (AS08), Boore-Atkinson (BA08), Campbell-Bozorgnia (CB08), Chiou-Youngs (CY08), and Idriss (I08). The horizontal spectral accelerations were developed at 5% critical damping for periods between 0 and 5 seconds. The site-specific deterministic spectra at 1.5 times the median and the 84th percentile (median plus one standard deviation) levels were developed for the major earthquake faults described above using an average shear wave velocity of 1,100 m/s for the upper 30 m developed based on review of available geophysical (suspension) logging data from borings located in the vicinity of Pump Station No. 1 (Arup, 2009). The 84th percentile deterministic level represents the upper limit as defined by SFPUC (2009).

Results from the DSHA are presented in **Figure B2** for the horizontal site-specific deterministic spectra at 5% critical damping for the Northern San Andreas (M7.9, R 13 km) and Hayward (M6.9, R 16 km) faults.

SITE-SPECIFIC HORIZONTAL GROUND ACCELERATION RESPONSE SPECTRA

For each seismic performance level, site-specific acceleration response spectra were developed for horizontal motions at 5% critical damping. Results are presented for these site-specific acceleration response spectra in **Figures B1**, and summarized in **Table B1**.

Site-specific horizontal acceleration response spectra were developed for each seismic performance level based on the lesser of the probabilistic and deterministic (1.5 times median) spectra. Additionally, the site-specific response spectra are subject to the deterministic lower limit of ASCE7-05: Figure 21.2-1 (ASCE, 2005), and the deterministic upper limit at 84th percentile as defined in SFPUC GSR (SFPUC, 2009). Due to proximity (within approximately 13 km) of the dominant earthquake scenario (Northern San Andreas fault) to the project site, site-specific ground acceleration response spectra for periods greater than 0.6 second were adjusted for near-fault directivity effects using methods developed by Somerville (1997) and updated by Abrahamson (2000).

Results for the site-specific horizontal acceleration response spectra at 5% critical damping are presented for the three seismic performance levels without directivity adjustment in **Figure B3**, and comparison with and without directivity adjustment in **Figure B4**. The site-specific horizontal acceleration response spectra developed with directivity adjustment (**Figure B1**, and **Table B1**) should be used in seismic design for the project.

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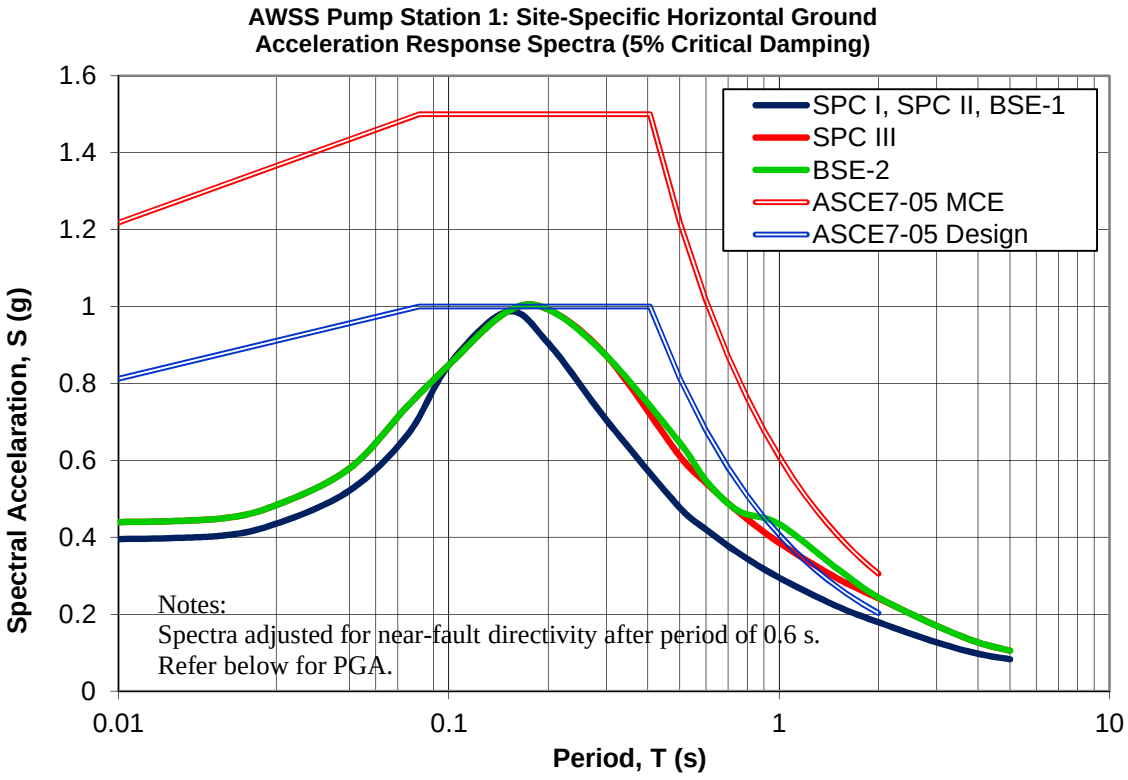
United States Geological Survey (USGS), 2009, 2008 National Seismic Hazard Mapping Program (NSHMP) Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation Web Site, http://geohazards.usgs.gov/deaggint/2008/index.php.

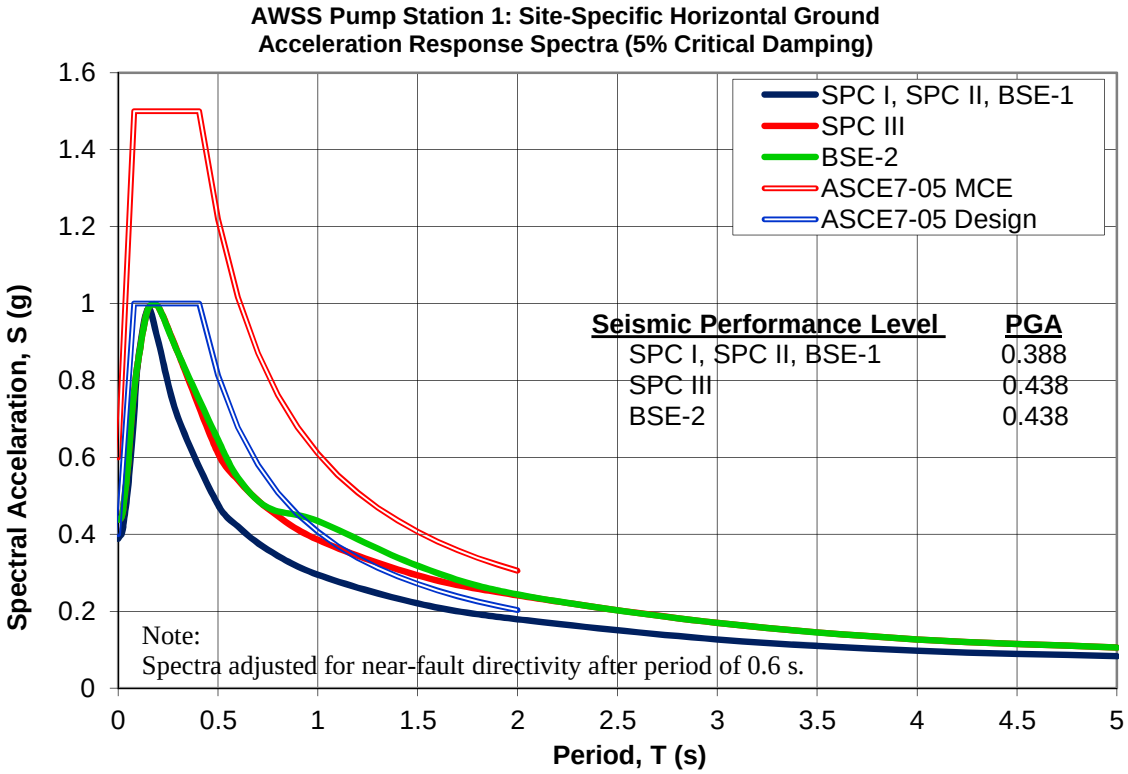
TABLE B1 – SITE-SPECIFIC HORIZONTAL GROUND ACCELERATIONS FOR PUMP STATION NO. 1

Period (s)	Site-Specific Horizontal Spectral Ground Acceleration 5% Critical Damping (g)		
	SFPUC Seismic Performance Level [1]		
	BSE-1, and SPC I, II	SPC III	BSE-2
0	0.388	0.438	0.438
0.01	0.396	0.440	0.440
0.02	0.403	0.448	0.448
0.03	0.435	0.484	0.484
0.05	0.521	0.579	0.579
0.075	0.667	0.741	0.741
0.1	0.848	0.848	0.848
0.15	0.986	0.986	0.986
0.2	0.904	0.992	0.992
0.25	0.788	0.942	0.942
0.3	0.705	0.872	0.872
0.4	0.566	0.714	0.748
0.5	0.477	0.611	0.646
0.6	0.418	0.539	0.548
0.75	0.356	0.462	0.469
1.0	0.289	0.378	0.434
1.5	0.213	0.284	0.318
2.0	0.172	0.231	0.244
3.0	0.120	0.165	0.170
4.0	0.091	0.127	0.127
5.0	0.078	0.105	0.105

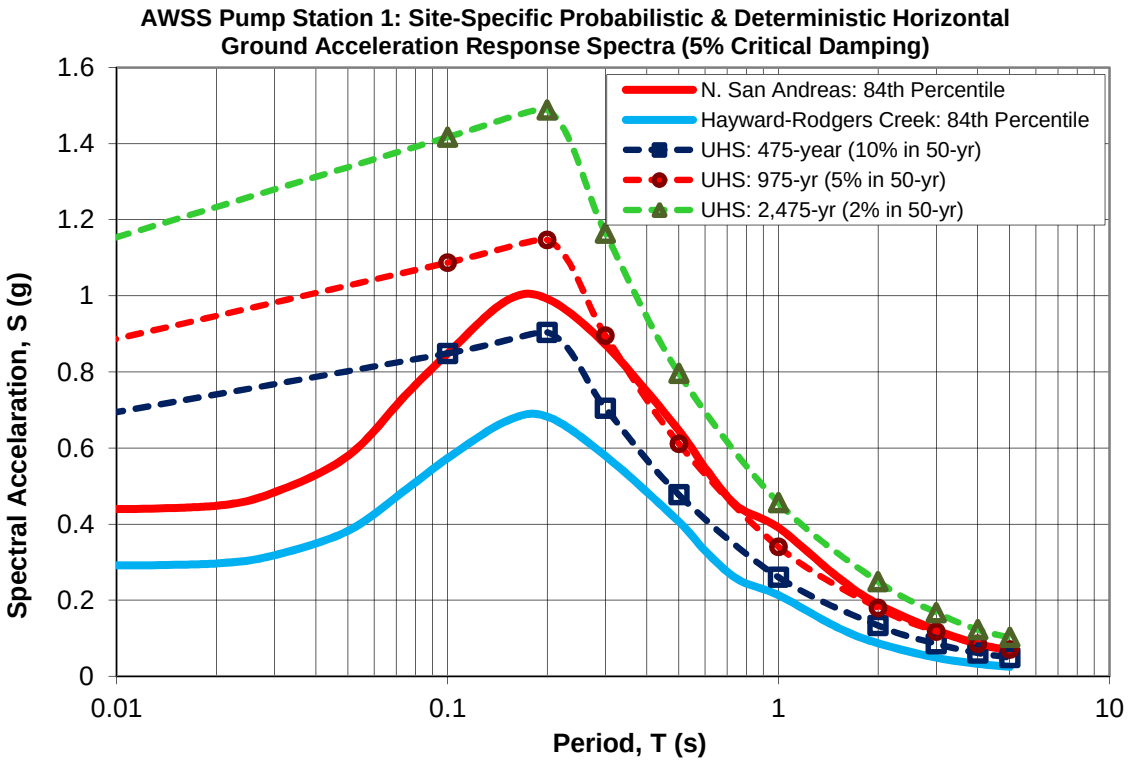
Note:

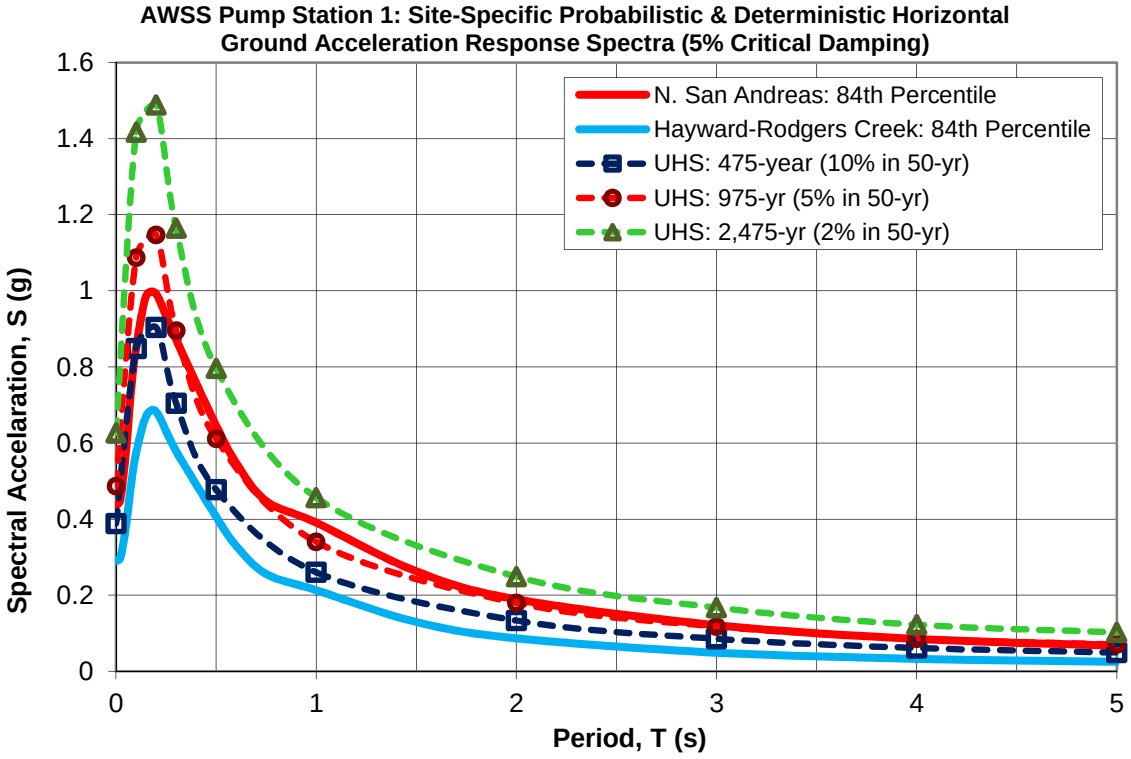
1. Seismic Performance Classes (SPC) I and II and Basic Safety Earthquake-1 (BSE-1) correspond to MCE developed from the probabilistic seismic hazard at a 475-year return period (10% P.E. in 50 years). SPC III corresponds to MCE developed from a combination of probabilistic seismic hazard at a 975-year return period (5% P.E. in 50 years) and the deterministic hazard at 84th percentile. BSE-2 corresponds to MCE developed from the deterministic hazard at 84th percentile. The site-specific MCE are subject to the requirements for upper and lower limits per SFPUC GSR (dated 10/6/2009) and ASCE7-05.



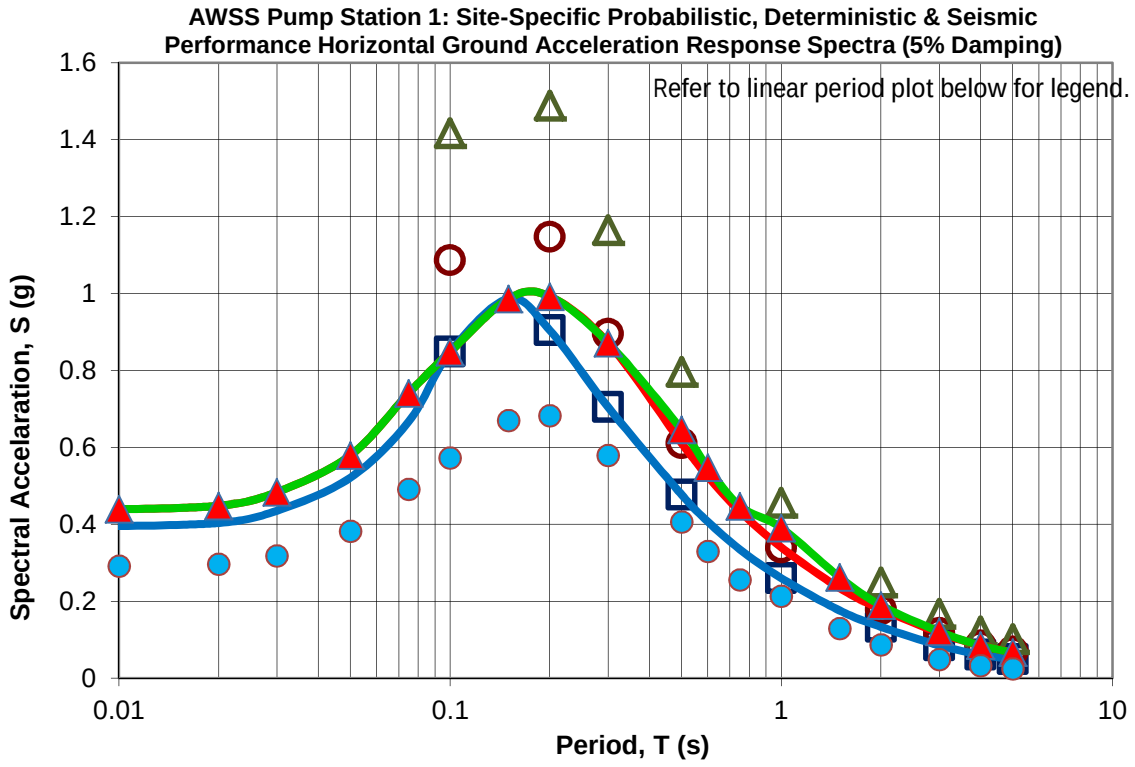


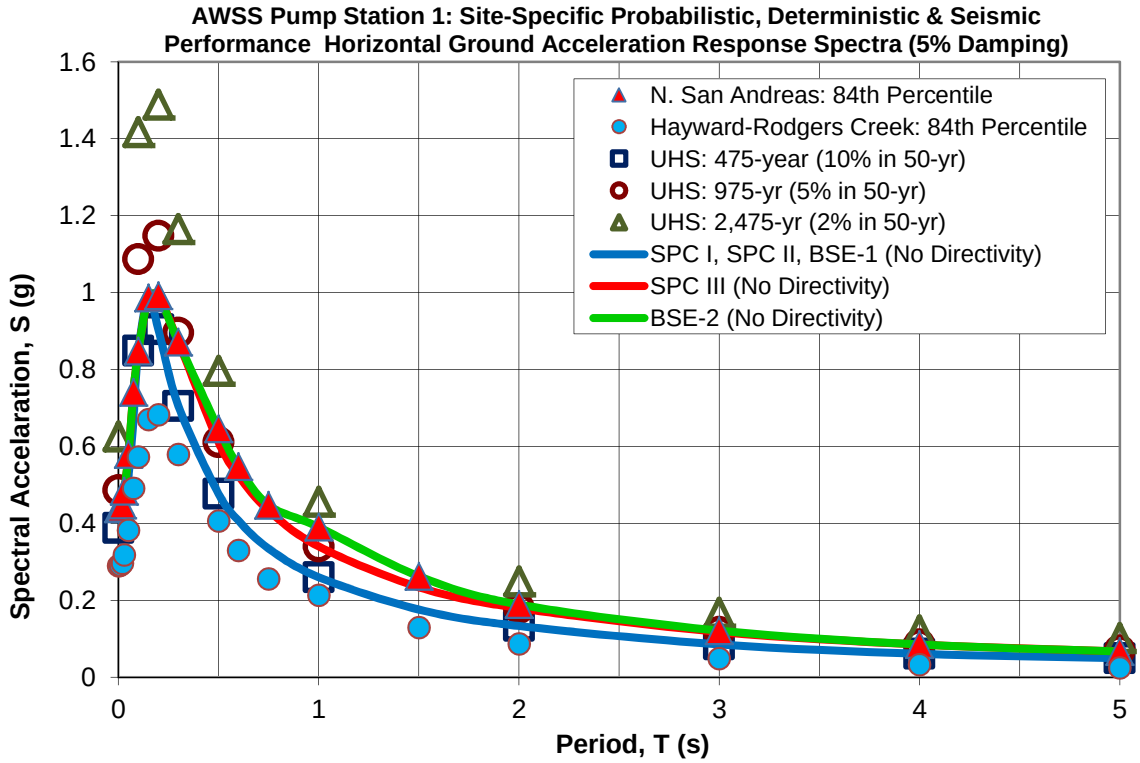
GEOTECHNICAL CONSULTANTS, INC. 500 Sansome Street, Suite 402 San Francisco, California 94111	SITE-SPECIFIC HORIZONTAL ARS	FIGURE
	AWSS PUMP STATION 1	B1
	JANUARY 2012	SF11024



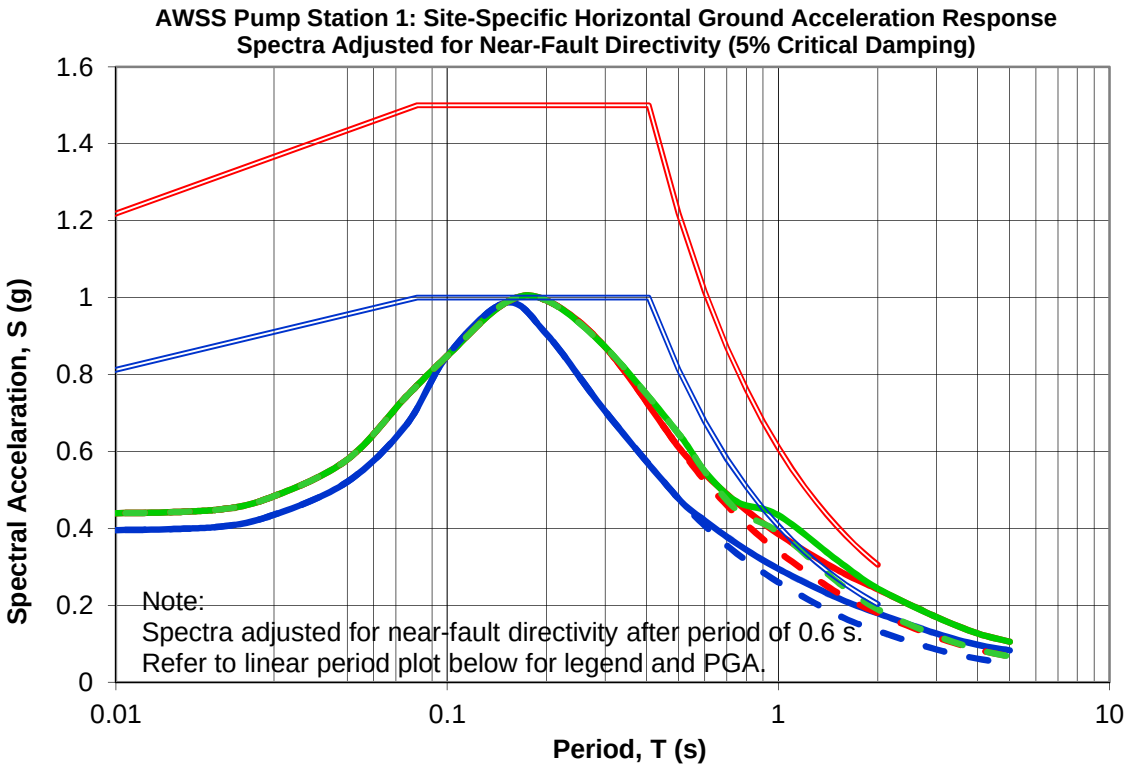


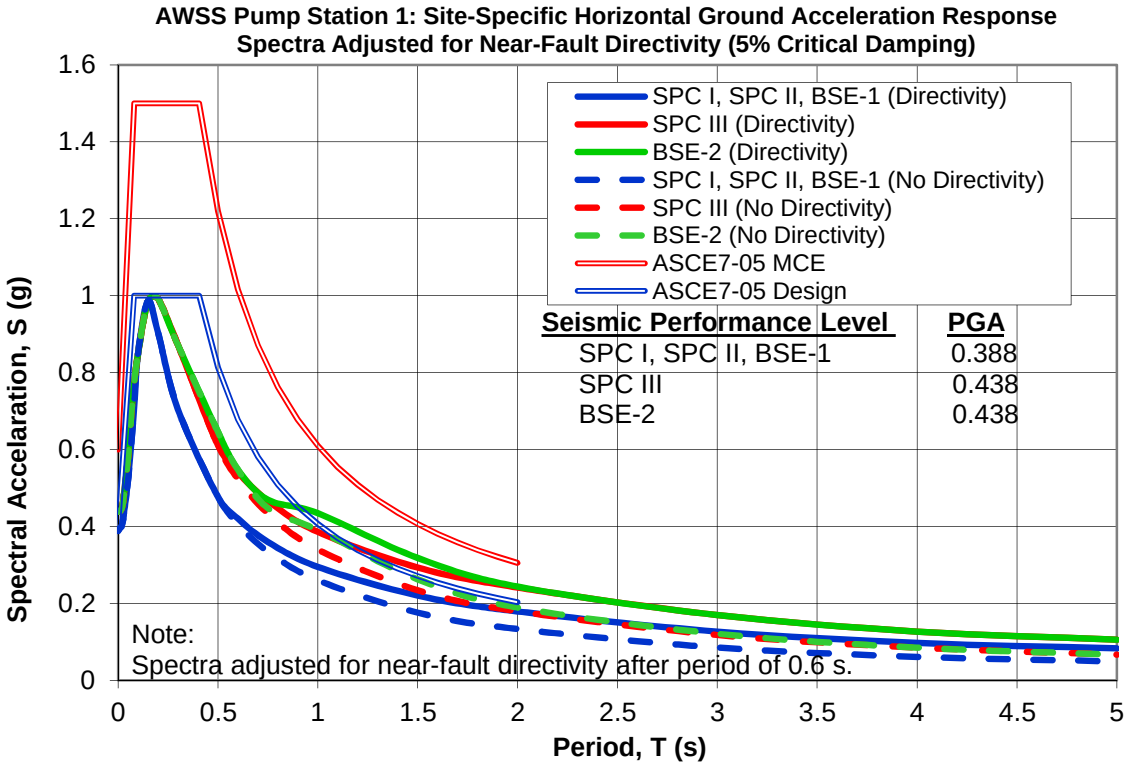
GEOTECHNICAL CONSULTANTS, INC. 500 Sansome Street, Suite 402 San Francisco, California 94111	**SITE-SPECIFIC PROBABILISTIC AND DETERMINISTIC HORIZONTAL ARS**	**FIGURE**
	AWSS PUMP STATION 1	**B2**
	JANUARY 2012	**SF11024**





GEOTECHNICAL CONSULTANTS, INC. 500 Sansome Street, Suite 402 San Francisco, California 94111	**SITE-SPECIFIC PROBABILISTIC, DETERMINISTIC & SEISMIC LEVEL ARS**	**FIGURE**
	AWSS PUMP STATION 1	**B3**
	JANUARY 2012	**SF11024**





GEOTECHNICAL CONSULTANTS, INC. 500 Sansome Street, Suite 402 San Francisco, California 94111	SITE-SPECIFIC HORIZONTAL ARS (DIRECTIVITY)	FIGURE
	AWSS PUMP STATION 1	B4
	JANUARY 2012	SF11024

APPENDIX C

BROCHURES FOR GROUTING AND MICROPILES

AWSS Pump Station No. 1
San Francisco, California

APPENDIX C

BROCHURES FOR GROUTING AND MICROPILES

The following brochures are provided for general information related to grouting techniques and micropile installation:

- Keller Ground Engineering Pty Ltd (KGE), Undated, Specialist Grouting Brochure, 7 pages.
- KGE, Undated, Case History, Cement Permeation Grouting to Support Foundations for a New Basement Excavation – Watson's Bay, Sydney, 1 page.
- Keller Group plc (KG), Undated, Compaction Grouting, Brochure 66-01E, 12 pages.
- KG, Undated, The Soilcrete® - Jet Grouting Process, Brochure 67-03E, 12 pages.
- Hayward Baker, Inc., 2010, Micropiles Brochure, 8 pages, www.HaywardBaker.com.
- Sonic Drilling Ltd., Klemm KR704 Drill Rig Data Sheet, 1 page.

KELLER
GROUND ENGINEERING

Specialist Grouting

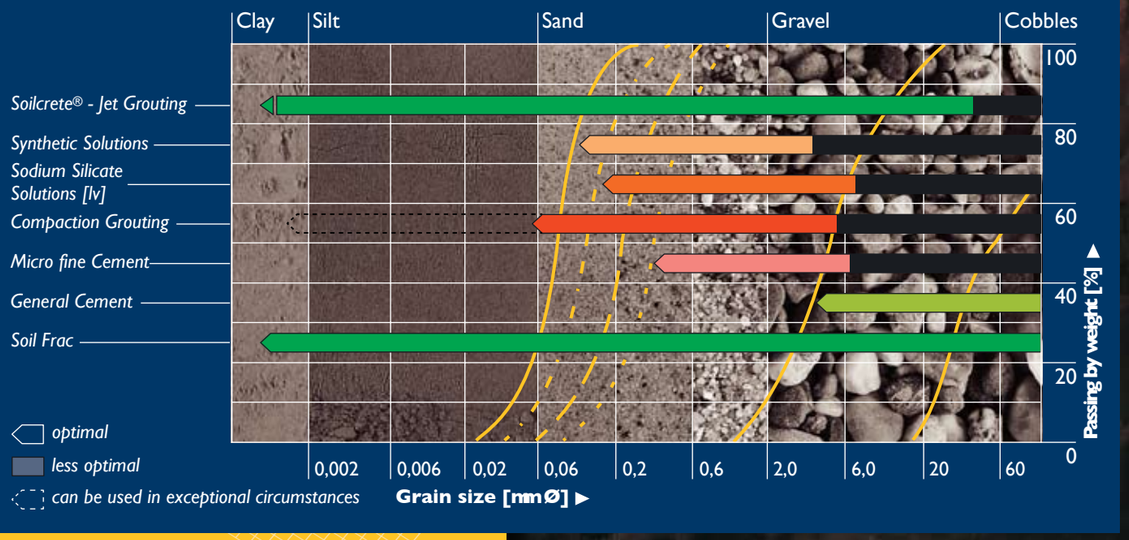


KELLER GROUND ENGINEERING

Keller Ground Engineering has sixty years grouting experience. Whether a common application or one that draws upon our unparalleled experience and creativity throughout Australia, New Zealand and the South Pacific we assist engineers, contractors and owners with identifying and implementing the right solution from outline concept to detailed design and execution for any situation.



LIMITS FOR GROUTING TECHNIQUES:



GROUTING SOLUTIONS

KELLER MEETS YOUR ENGINEERING NEEDS

GROUND MODIFICATION SOLUTIONS

APPLICATIONS	SOILFRAC®	SOILCRETE®	PERMEATION	COMPACTION	INFILL
Ground Water Control		◆	◆		
Underpinning	◆	◆	◆	◆	
Settlement Control	◆	◆	◆	◆	
Ground Strengthening/ Consolidation	◆	◆	◆	◆	◆
Settlement Reversal	◆			◆	
Void Grouting			◆		◆
Dam Grouting		◆	◆		
Excavation Support		◆	◆		
Leachate Control		◆	◆		
Mine & Shaft Infilling	◆				◆
Pile Base & Shaft Grouting	◆		◆		
Piping Failure Control		◆	◆		
Sealing Slabs		◆			
Solution Features				◆	◆
Swallow Holes				◆	◆
Shaft Bases		◆	◆		
Tunnelling Support	◆	◆	◆		
Underslab Grouting			◆	◆	◆
Liquefaction Prevention	◆			◆	
Ground Compaction	◆		◆	◆	
Foundation Support	◆	◆	◆	◆	◆

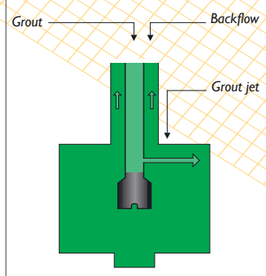


SOILCRETE® JET GROUTING

THE PROCESS

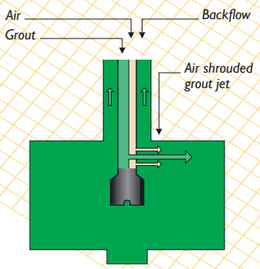
Jet Grouting is a Ground Modification System used to create insitu, cemented formations of soil. Using high velocity (>100m/s) cutting jets of water or cement suspension (grout) the system erodes and insitu mixes the soil to form the soil grout composite material.

Soilcrete® Jet Grouting can be carried out using three different systems. The choice of method is determined by the soil conditions, the required geometry, the required Soilcrete® parameters and the application.



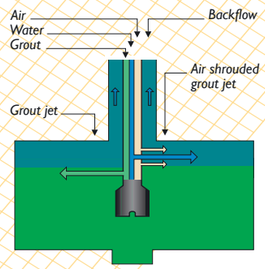
Single System:

Single System operates with a grout jet for simultaneous cutting and mixing of the soil. The Single System is used for small to medium sized Soilcrete® columns in low density soils.



Double System:

The Double System uses a compressed air shroud around the jet nozzle to increase the erosion capability of the jet. The Double System is used in dense soils, installing panel walls, underpinning and mass treatment.



Triple System:

Typically used for underpinning and in sensitive or highly variable soils, the Triple system erodes the soil with an air shrouded water jet and simultaneously injects grout into the eroded soil through an additional nozzle.

SUPER JET

Keller Ground Engineering hold the exclusive licence for the Super Jet system. Developed in Japan and operated by Keller companies in North America, Europe and the Pacific, Super Jet takes advantage of tool efficiencies and increased energy to deliver columns up to 5.0m in diameter. The benefits of Super Jet are technical supremacy, reduced program periods and costs.



Excavated 5.0m Super Jet Column.



Super Jet monitor and nozzle.

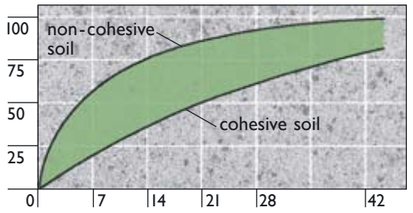
THE PROPERTIES

The compressive strength of Soilcrete® is a function of the cement content of the grout and grading of the portion of soil within the Soilcrete® mass.

The permeability and strength of the Soilcrete® can be controlled by the addition of additives to the grout mix.

Type of Soil	Silt or Clay	Sand	Gravel
UCS (MPa)	≤5	≤10	≤20

▼ Compressive strength in % of the final strength



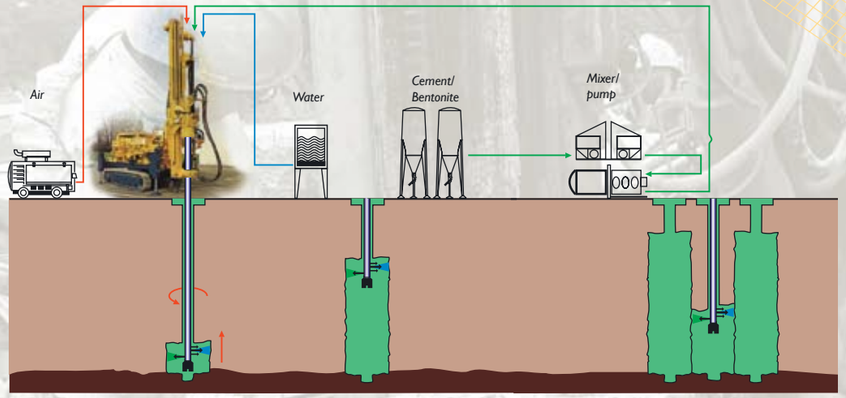
Development of Soilcrete® Strength with time

SOILCRETE® JET GROUTING

CONSTRUCTION SEQUENCE

Soilcrete® Jet Grouting equipment consists of a high pressure pumping unit, cement mixing equipment, storage silo and stores containers.

The grout mixing and pumping unit is connected to the drill rig with high pressure hoses and control cables allowing remote location of the plant. The jetting process can be performed using a wide range of drilling rigs with mast heights varying from 2.5m to 35.0m. The boreholes are normally contained in shallow trenches used to direct the flow of the excess soil-water-cement spoil to tanks or settling ponds.



1. Drilling

Drill rods fitted with jet nozzles are bored to the final depth using grout to stabilize the hole. Jetting commences at the base of the hole.

2. Grouting

Turbulence caused by the jet results in the uniform mixing of grout and soil. Excess water, soil and grout returns to the surface through the space between the drill rod and borehole wall.

3. Continuation

Additional columns can be added fresh on fresh or fresh on firm. Sequence depends upon the jetting system as well as the technical requirements of the structure being treated.

DESIGN AND QUALITY CONTROL

Soilcrete® Jet Grouting can be designed to mix or almost fully replace a soil with grout. For the common application of underpinning, excavation support and groundwater cut-off the design consists of developing a continuous Soilcrete® mass to resist over turning and sliding whilst maintaining the integrity of supported structures or utilities.



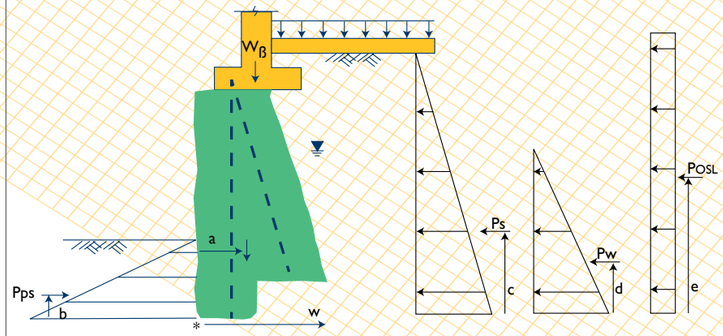
For retaining structures design checks are made for the competency of the soils below the base for bearing and settlement together with an evaluation of the internal shear and bending stresses in the Soilcrete®. The strength of the Soilcrete® is a function of the insitu soils and strength variations are to be expected, for this reason a factor of safety of 2.5 to 3.0 is typically applied for an average allowable strength.

The size of the Soilcrete® mass to be created is dictated by the application, a variety of geometries can be achieved. The width or diameter of each panel or column is determined during the design stage. Reliable descriptions of the soil composition and strength or density allows this assessment to be made with confidence.

Left: Soilcrete® underpinned Heritage Building, Manly NSW
Top: Intimate bond between Soilcrete® and Foundation.
Middle: Double or Triple Rods provide separate flows of grout, air and water.
Bottom: Jet grout pump.



DESIGN AND QUALITY CONTROL



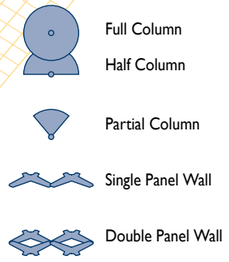
$$F = (W_\beta + W_{SC}) \tan \S + w \cdot Ca$$

$$\text{FS overturning}^* = \frac{(W_\beta + W_{SC})\, a + Pps \cdot b}{(Ps \cdot c) + (Pw \cdot d) + (Posl \cdot e)} \geq 1.5 \qquad \text{FS sliding} = \frac{Pps + F}{Ps + Pw + Posl} \geq 1.5$$

Legend

a,b,c,d,e	Moment arms	Posl	Surcharge force
Ca	Base soil adhesion	Pps	Passive force
§	Base soil friction angle	Pw	Hydraulic force
F	Friction force	W_β	Weight of structure footing load
FS	Factor of safety	W_{SC}	Weight of Soilcrete®
Ps	At-rest or active soil force	W_{SL}	Surcharge load

SOILCRETE® GEOMETRICS



Full Column

Half Column

Partial Column

Single Panel Wall

Double Panel Wall



Sheet Pile Sealing Systems

QUALITY ASSURANCE

Critical aspects of a successful Soilcrete® jet grouting program are quality assurance and quality control. These ensure that sub-surface soils are consistent with design assumptions and that design parameters are met or exceeded.

Typically projects commence with a test section to verify the design geometry and to confirm the quality and strength characteristics of the Soilcrete® material. During the Jet Grouting process a series of inspection items are monitored and documented, these include:

Drilling:	Location, angle, depth;
Batching:	Preparation of grout for consistency in material content and physical and chemical properties;
Jetting:	Checking of drill parameters (lift speed, rotation rate) and injection parameters (pressure and flow of all inputs);
Documentation:	Construction times and correlation to any sampling;
Sampling & Testing:	Retrieval of representative samples for external testing.

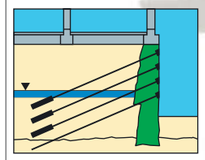


Right: SuperJet Grouting to provide tunnel support at RNA Showgrounds, Brisbane

SOILCRETE® JET GROUTING

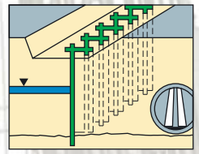
APPLICATIONS

In contrast to conventional grouting Soilcrete® Jet Grouting may be used for stabilization and sealing of all kinds of soil ranging from loose granular sediments to clay. This applies for homogenous soil formations and variable soil layers. Weak rock formations have also been treated with Soilcrete® Jet Grouting.



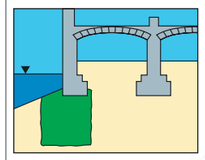
Underpinning

Underpinning by means of low deformation gravity walls, sometimes also used as a ground water seepage barrier, may be safely constructed even from confined working areas. Can be used in conjunction with ground anchors.



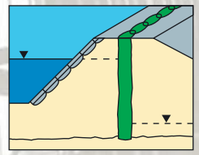
Panel Walls

Soilcrete® panel walls to cut off ground water are used below roads and buildings, for crossing pipelines and to subdivide building pits into different excavation sections. According to the sealing requirements single or multiple panels may be constructed.



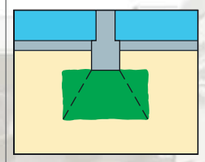
Foundation Restoration

Historic buildings may be endangered in the event of settlements occurring. Soilcrete® provides a safe foundation with the maximum structural protection.



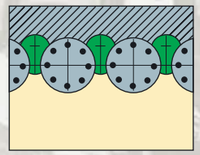
Column Walls

In the event of higher mechanical strain by shear force, danger of undermining or of a high impermeability requirements, cut off walls of intersecting Soilcrete® columns may be constructed.



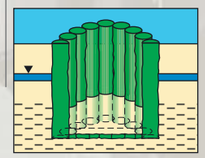
Foundation Improvement

Changes in use or modifications of buildings often require an enlargement or alternation of the foundation. Soilcrete® is an economical and flexible solution for this task.



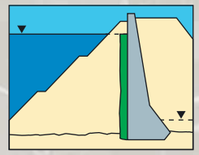
Joint Sealing

For sealing of joints between piles, sheet piles or other construction parts in the ground, Soilcrete® is ideal.



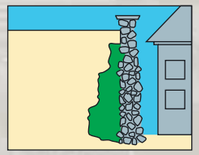
Shaft Supports

Shafts with intersecting Soilcrete® columns are constructed if a vibration free installation is required and/or the shafts enter into ground water bearing strata.



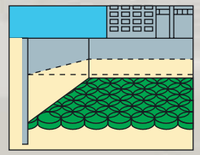
Dam Sealing

Soilcrete® may be used to repair dam cores or enlarge cut-off walls in or below dams.



Earth Pressure Relief

Structures exposed to earth pressures, such as historical walls, abutments, steep slopes protections or quay walls may be relieved by the addition of or connection to a backup Soilcrete® body.



Sealing Slabs

Soilcrete® sealing slabs are constructed by means of overlapping columns within an uplift proof depth. The sealing slabs may be connected to any kind of vertical sealing systems.



Top: Self Drilling jet grout monitor
Middle: Drill head and mast configuration
Bottom: Joint sealing between piles

COMPACTION GROUTING

THE PROCESS

When compaction grout is injected into loose soils, homogeneous grout bulbs are formed that displace, densify and thus strengthen the surrounding soil. Originally developed as a remedial measure for building settlement control the technique has evolved to treat a wide range of sub-surface conditions including rubble and poorly placed fill, loosened or collapsible soils, sinkholes and liquefiable soils.

The grout mix must have specific characteristics; a very low mobility (low slump) mixture that is 'pumpable' but upon installation, exhibits an internal friction enabling it to remain intact and displace the surrounding soil without fracturing it

Compaction grout improves ground conditions by displacement of soil particles.





A high viscosity (low mobility) aggregate grout is pumped into the ground in stages to displace and densify the surrounding soils. By sequencing the grout injections from primary to secondary and tertiary locations this densification process can be performed to achieve the required improvement.

Above: Low mobility compaction grout.
Left: Controlled underpinning of shallow foundations.

DESIGN

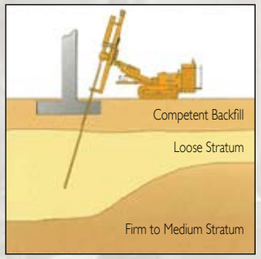
A number of aspects must be considered for compaction grouting to work best:

- The insitu vertical stress in the treatment zone must be sufficient to enable the grout to displace horizontally and hence avoid uncontrolled heave.
- In saturated soils a pore water pressure increase will occur as a result of ground displacement, for effective densification this must dissipate.
- Soils that loose strength during remolding should be avoided
- Greater displacement will occur in weaker soil strata, exhumed grout bulbs confirm that compaction grouting focuses improvement where it is needed most.

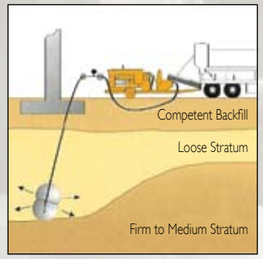
Quality control includes procedural inspection and documentation of work activity, testing to ensure proper mix design and verification works.

COMPACTION GROUTING

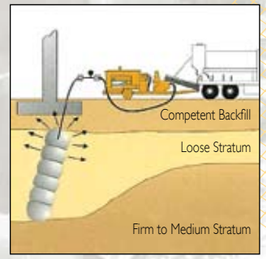
CONSTRUCTION SEQUENCE



Installation Of Grout Pipe:
- Drill or Drive Casing
- Record Ground Information from Casing Installation



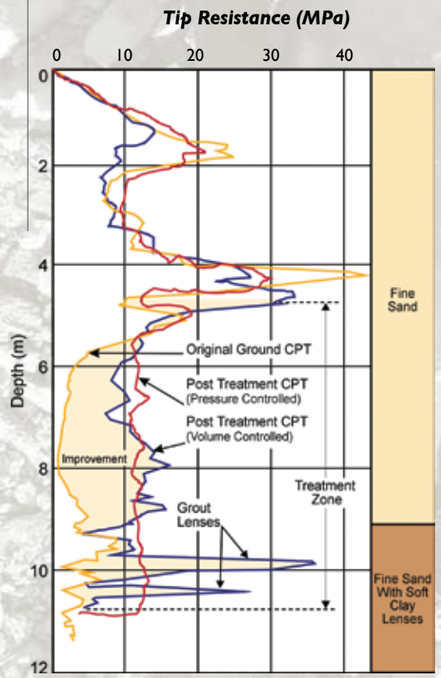
Initiation of Grouting:
- Typically a bottom up process
- Grout quality important
- Slow uniform stage injection



Continuation of Grouting:
- On-site batching of grout can aid control
- Sequencing of plan injection points very important
- Usually injection is pressure or volume limited

ADVANTAGES OF COMPACTION GROUTING

- ***Pinpoint treatment***
- ***Speed of Installation***
- ***Non Hazardous***
- ***No waste disposal***
- ***Can be performed in very restricted access***
- ***No need to connect to existing foundations***
- ***Economic alternative to removal & replacement or piling***
- ***Volume and pressure control options***

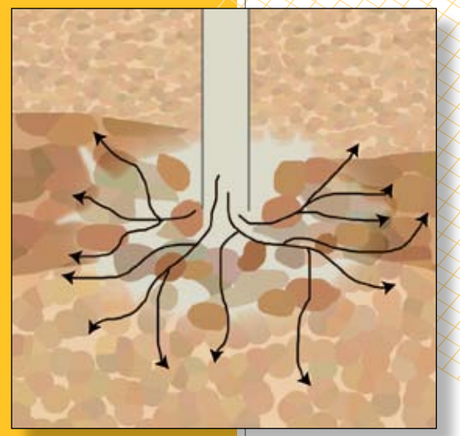




Left: Cone Penetrometer Test results, such as the ones illustrated left for volume cut-off and pressure cut-off, show the degree of improvement achieved by compaction grouting.
Right Top: Grout tubes
Right Centre: Steel point on driving shoe
Right Bottom: Driving grout pipes



PERMEATION GROUTING



Permeation of cement or chemical grout through granular soil.

THE PROCESS

Permeation Grouting is the injection of a fluid grout into granular, fissured or fractured ground to produce a solidified mass to support increased load and/or to fill voids and fissures to control water flow.

Grout Materials include:

- General Cement
- Micro-fine Cement
- Sodium Silicate Solutions

Cement Grouts are often applied with admixtures or additives to improve grout stability, viscosity or durability. Common Admixtures include: bentonite, sand, plasticiser and PFA.

DESIGN

- Dependent upon the preferred grout viscosity and particle size.
- Generally limited to soils containing sand and gravel size particles only.

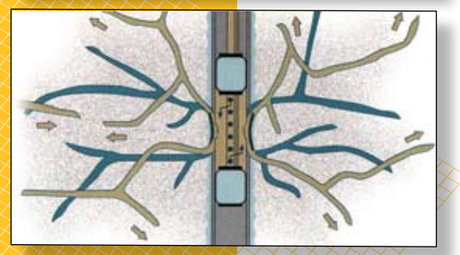
CONSTRUCTION SEQUENCE

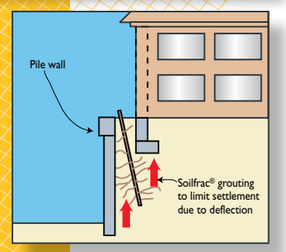
Grout can be injected into the soil by a variety of methods:

- Hand Driven lances or spears.
- Drilled casings with grout injected through the end of the hollow casing.
- Tube-à-manchette (TAM), a series of pipes with regular spaces rubber sleeve covered injection holes.

SOILFRAC® COMPENSATION GROUTING







Top: Grout fractures emanating from TAM pipe in clay
Middle: TAM pipe and Soilfrac® injection
Bottom: Typical application to mitigate potential for settlement damage due to adjacent piling and excavation.

THE PROCESS

Soilfrac® Compensation Grouting is a process used to control or reverse the settlement of structures. It uses similar equipment and grout materials as permeation grouting to induce fractures in the soil thereby causing an expansion to take place counteracting settlement and producing controlled heave of the foundation. The system can be applied in all soil types.

CONSTRUCTION SEQUENCE

Soilfrac® is an observational process involving the installation of grout injection tubes (TAM) in a pre-determined pattern, careful injection of grout through sleeves to fracture the soil and the monitoring of ground or building movements to feed back into the grouting regime.

ADVANTAGES OF SOILFRAC® COMPENSATION GROUTING

- Method is usually performed from outside the building without disruption.
- Repeatable process allows settlement to be controlled on an ongoing basis.
- Cost effective.

ROCK GROUTING

THE PROCESS

Rock Grouting is normally performed in fissured rock masses to reduce the flow of water along the joints and discontinuities in the rock.

- Grout is typically injected in isolated stages from boreholes drilled using rotary percussive drills.
- Reduction in permeability is a function of the grouting material, the rock and grout design.
- Rock Grouting is typically performed with General Purpose, Ultra-fine or Micro-fine Cements.

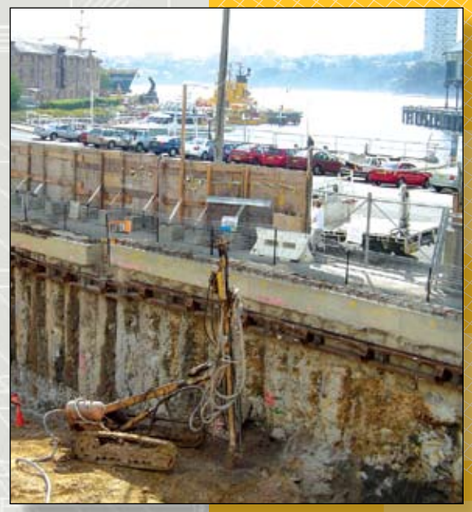
DESIGN AND QUALITY CONTROL

- Injection pressure and volume is recorded during the injection.
- Periodic water testing (lugeon test) is performed to demonstrate the achieved reduction in permeability.

Electronic monitoring and recording of grout injections can be used to provide significant cost savings through ongoing observational design of the grouting.

CONSTRUCTION SEQUENCE

- Drill and Grouting is progressed in either 'Down stage' or 'Up Stage' manner.
- Grout viscosity varied during each stage injection to ensure effective grout penetration.
- Holes are drill and grouted in a split spacing method with a primary, secondary, tertiary, etc approach.



Rock grouting in Hawkesbury sandstone, Sydney NSW.

MINE INFILL GROUTING

THE PROCESS

Coal and mineral extraction has taken place for the past two centuries using a variety of methods. With exhausted workings in various states of collapse careful consideration needs to be given to the most appropriate method of treatment.

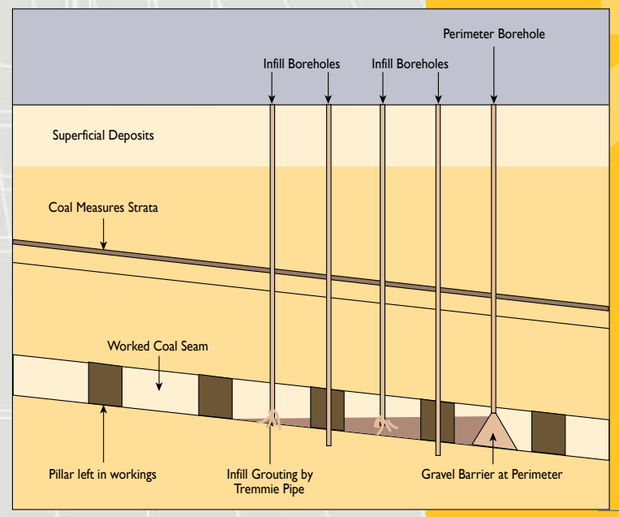
- Consolidation and Void fill grouting is the preferred method when access to the seam is not possible.

CONSTRUCTION SEQUENCE

- A perimeter barrier is installed to prevent grout migration
- Grout is pumped through tremmie pipes placed in pre-drilled holes.
- Temporary casing is used to support the drill hole in unstable ground

DESIGN & QUALITY CONTROL

- Keller Ground Engineering offer a full remediation design and construct service to meet all the needs of the project.
- Works performed in accordance with established practice developed from extensive international experience of similar projects.



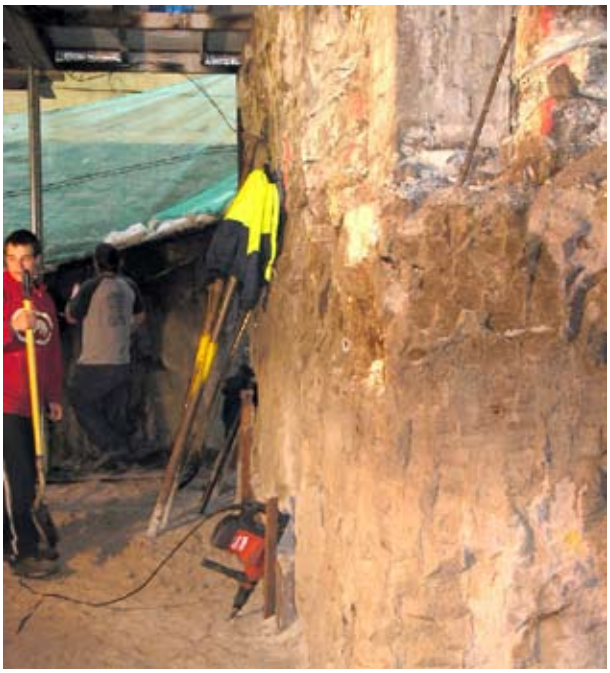
KELLER
GROUND ENGINEERING

Keller Ground Engineering Pty Ltd

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Cement Permeation Grouting to support foundations for a new basement excavation – Watson's Bay, Sydney





The excavation of a new basement below an existing 19th Century cottage required a discrete solution.

Project

The owner of a 100 year old cottage wished to extend his living space underneath his existing floor space. During initial excavation works it became evident that improvement of the soil below the foundations would be required to ensure stability.

Soil Conditions

The property was underlain by dry fine to coarse Aeolian Sands of loose to medium dense consistency.

Solution

Keller Ground Engineering was requested to stabilise the sands to allow the excavation of a vertical face of 1.8m below ground level and 1.2 m below the existing footings.

Keller proposed the use of permeation grouting to cement the sands insitu. Permeation Grouting uses a low pressure injection of a cement suspension grout to fill the voids between the soil particles. In this instance, due to the presence of fine sand particles, a fine grained cement known as 'microfine cement' was used to ensure the grout flow through the small apertures between the fine sand grains.

Construction

Due to extremely limited space on site the project was carried out with light, handheld drilling and injection equipment, while the grout was mixed and pumped remotely using electric colloidal mixers and low controlled volume piston pumps.

Following injection the grouted soil was left for 7 days prior to excavation sufficient time to allow the grout to achieve the required compressive strengths of greater than 1MPa, while 28 day results of >5Mpa were achieved.

Keller's Permeation grouting system can be utilised to increase the strength and bearing capacity of soils and is suitable for the underpinning of structures, improving the bearing capacity of foundations and control of ground water flow.

Specialist Geotechnical Contractor:

Keller Ground Engineering Pty Ltd

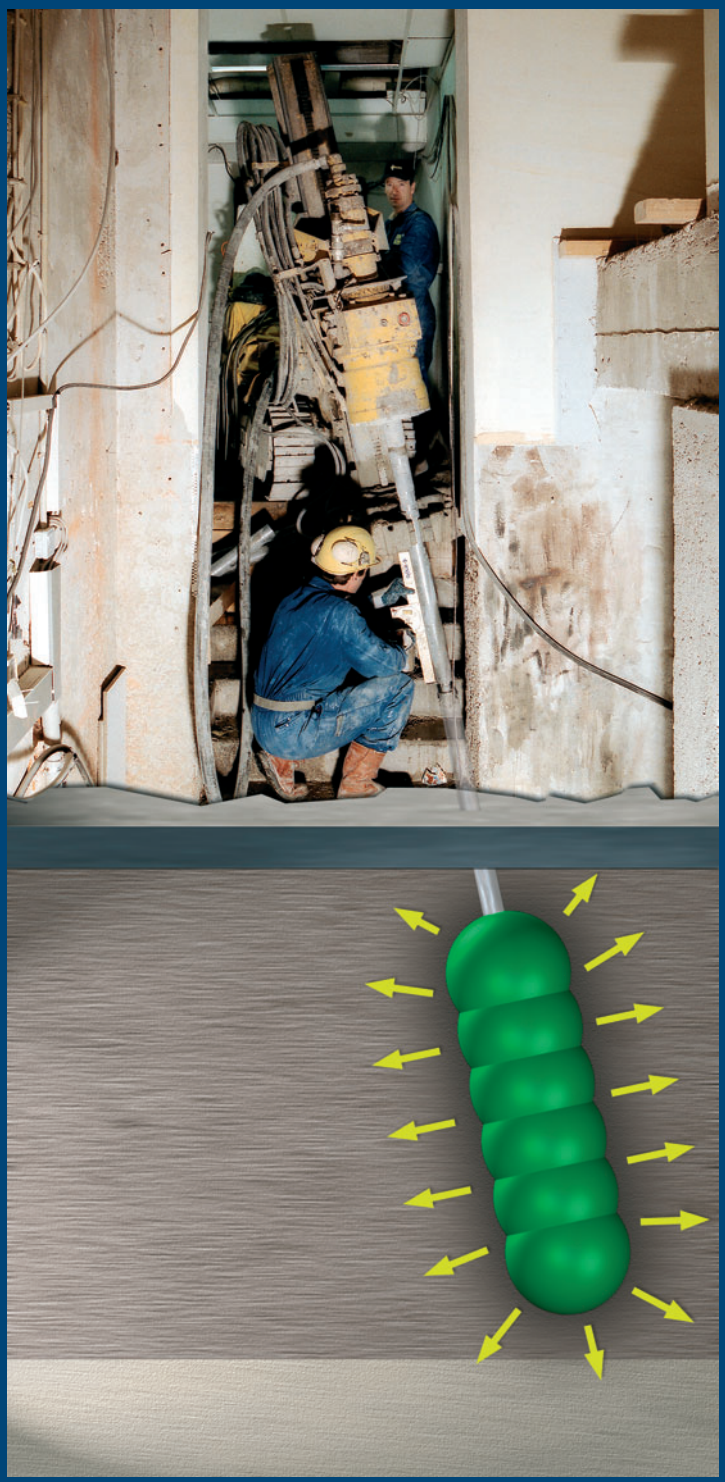
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KELLER GROUND ENGINEERING

Compaction Grouting



Brochure 66-01 E



Contents

The method on the market 3
Process description 4
Soil improvement 6
Foundation rehabilitation 8
Cavity grouting 10

Development

The compaction grouting process, which was first applied in the USA during the 1950ies and further developed, mainly by engineers of Hayward Baker Inc., is gaining more and more acceptance in Europe since the beginning of the 1990ies.
When Keller first applied this technique in Europe, we were able to build upon the rich experience gained by our American sister company and since then have been continuously developing this process.

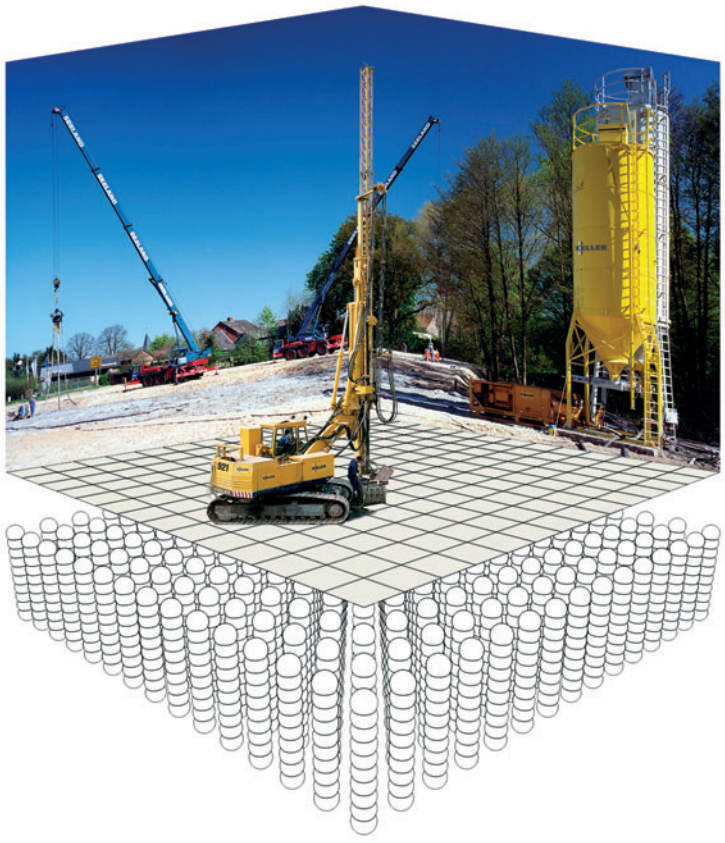
Fields of application / Position on the market

While compaction grouting had originally been used only for the foundation rehabilitation of settlement sensitive structures, the field of application in the meantime has been extended. Nowadays this technique is used in various fields of application – which frequently can be combined in a single project.

- **Soil improvement**
  Improvement of soils with insufficient bearing capacity, respectively with an increase in relative density, e.g. as an alternative or supplement to pile foundations or stone columns.
- **Stabilisation and rehabilitation of foundations**
  Increase or restoration of the bearing capacity of the soil under an existing foundation, e.g. in cases of increase in load or settlement damages. This process is an alternative to Soilcrete® or mini piles and/ or serves as pre-treatment when applying the Soilcrete® and Soilfrac® process.
- **Cavity Grouting**
  Force locking backfill of very porous soils, erosions or cavities, e.g. in backfilled areas which have not sufficiently been compacted, in karst, in case of damages caused by water pipes or below carriageway slabs.

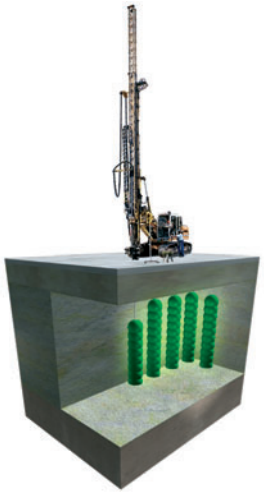
This process constitutes a multi-purpose supplement to existing specialised ground engineering techniques and, in addition, can be combined with almost all known procedures.

COMPACTION GROUTING TODAY



The Compaction Grouting Process

When applying the compaction grouting process usually a stiff to plastic grout is injected into the soil under pressure. It expands in the soil as a relatively homogeneous mass and at the same time is forming almost ball-shaped grout bulbs. The soil surrounding the grouted area is displaced and at the same time compacted. Compared to other grouting techniques, the grout material neither penetrates into the pores of the in-situ soil (as is the case with the classical injection) nor are local cracks formed (as is the case with the Soilfrac® technique).
During the compaction grouting process pressure and grout quantity as well as possible deformations at ground surface, respectively at structures are monitored. Depending on the design requirements, the compaction grouting process will be terminated either when reaching a maximum pressure, a maximum grout volume, when achieving the desired uplift of the structure or in case of grout material flowing out on the site surface. The execution method of the compaction grouting process is laid down in the European Standard EN 12 715.



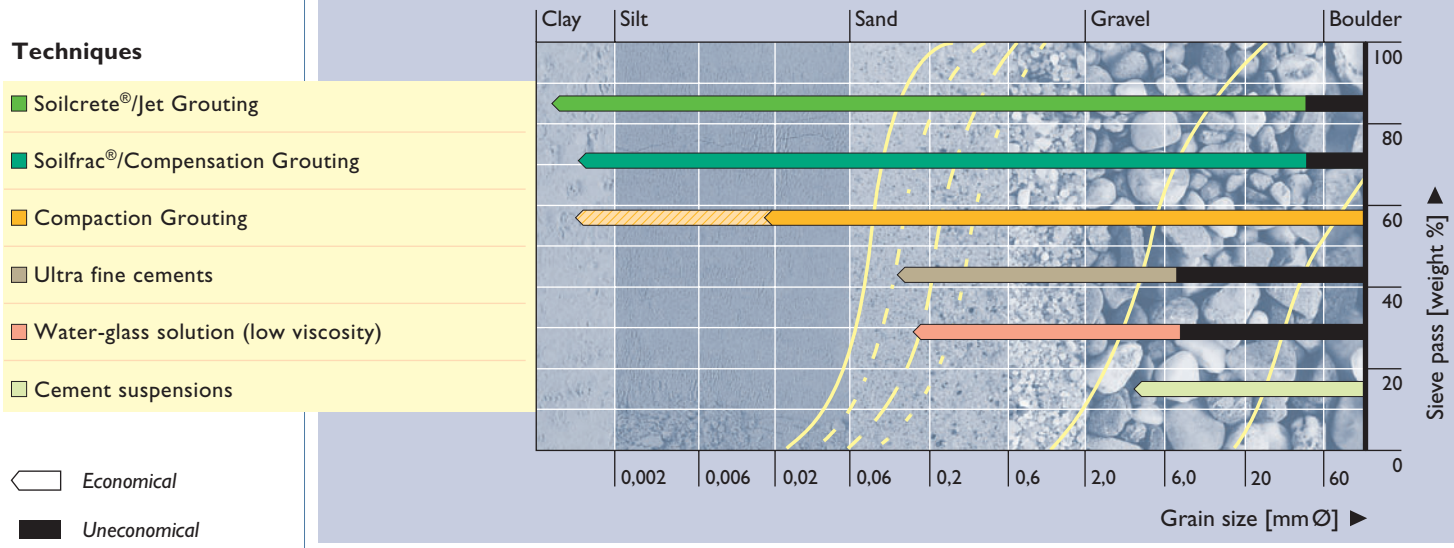
Range of Application

The compaction grouting method may be used for the improvement of non-cohesive soils, especially in cases, where soils of loose to medium density are encountered.
This method is also used in fine-grained soils*) in order to install elements of higher strength and bearing capacity in soils of low bearing capacity, thus improving the load bearing behaviour of the soil.
When using this technique in saturated clayey soil, a temporary increase of the pore water pressure can be observed.

* *Due to the fact that fine-grained soils cannot be compacted from the soil mechanics' point of view – by applying the same technique – strictly speaking, consolidation grouting is carried out.*

TECHNIQUE

Range of Application for Grouting Techniques




1

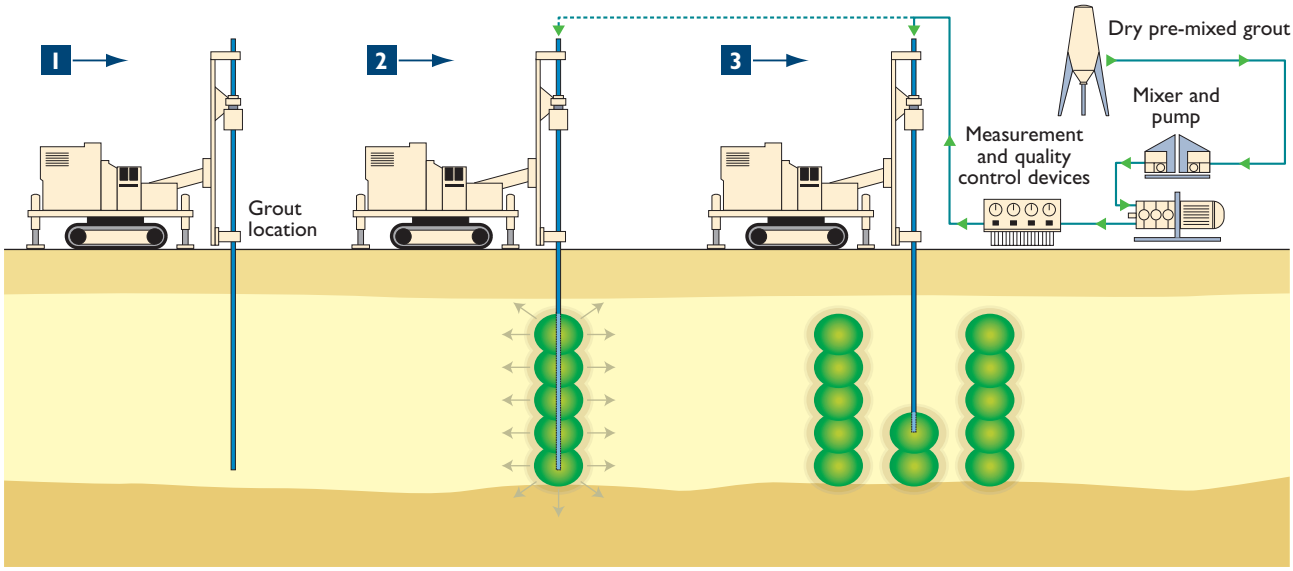

2


3

1 *Installation in a basement*

2 *Installation next to an existing building*

3 *Installation on a construction site*



1 Installation of the Grout Pipe
The grout pipe is either installed by means of a drill rig or a vibro hammer, depending on the soil and on the treatment requirements.

2 Compaction Grouting
The grout paste is prepared in the mixing plant and pressed into the soil by means of a custom-built grout pump.
While gradually pulling or penetrating the grout pipes, individual intersecting grout bulbs are consecutively formed, thus creating column shaped structural elements.

3 Staged Compaction
In order to achieve a uniform compaction of the soil, the injections are at first executed in a large primary grid, and may be compacted further by means of a secondary grid.

Quality Assurance

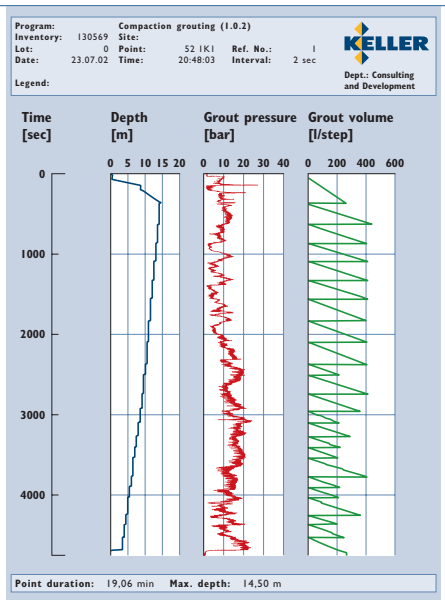


Quality as well as suitability of the fresh grout is constantly assured by measuring its slump.

The degree of compaction can be controlled by the following parameters, depending on the requirements:

- evaluation of the automatically recorded process parameters by means of a Keller in-house software
- deformation measurements at site surface or structures
- soundings (CPT, SPT) before and after the compaction grouting process

Evaluation



M4-print out: depth, pressure and grout volume are continuously recorded and shown on a time axis. Alternatively, recordings can be drawn on a depth axis.

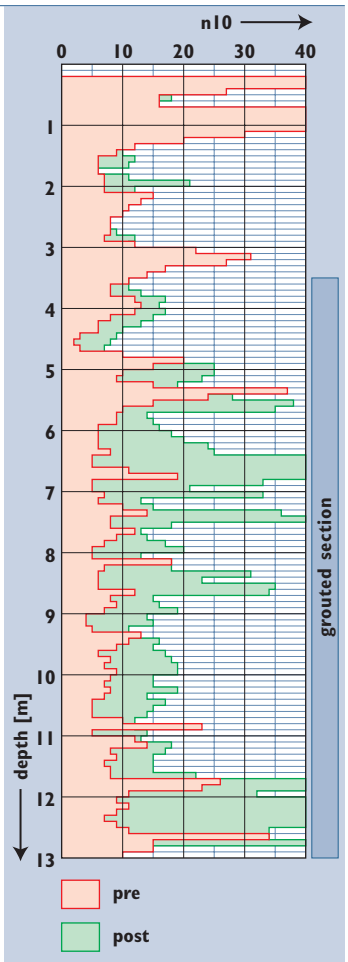


Diagram of a penetrometer test (pre/post) ▶

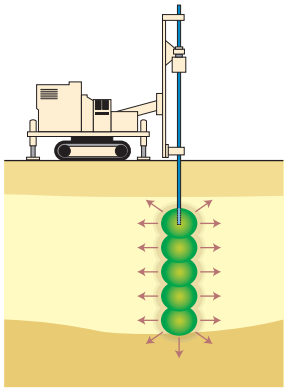
Ground Improvement

Frequently, soil improvement is necessary to increase the bearing capacity of poor soils, in case of a structure which has been proposed to be founded on this soil. In addition to the deep vibro techniques (such as vibro compaction and vibro replacement) which have been applied by Keller for decades, the compaction grouting technique has proven to be a suitable alternative in many cases.
For the execution of compaction grouting works a number of different drill rigs and and vibro hammers can be used. The execution method can therefore be easily adapted to the prevailing local conditions.

In principle, the compaction grouting technique can achieve a similar degree of improvement as by the deep vibro techniques.

The compaction grouting method is particularly well suited as an alternative or supplement to deep vibro techniques in the following cases:

- confined working space
- limited working height
- vibration-free technique required (e.g. because of a highly sensitive structure in the vicinity)
- compaction at very large depths
- for intermittant strong soil layers, which cannot be penetrated by a depth vibrator, thus making its use inefficient.



COMPACTING



1



2

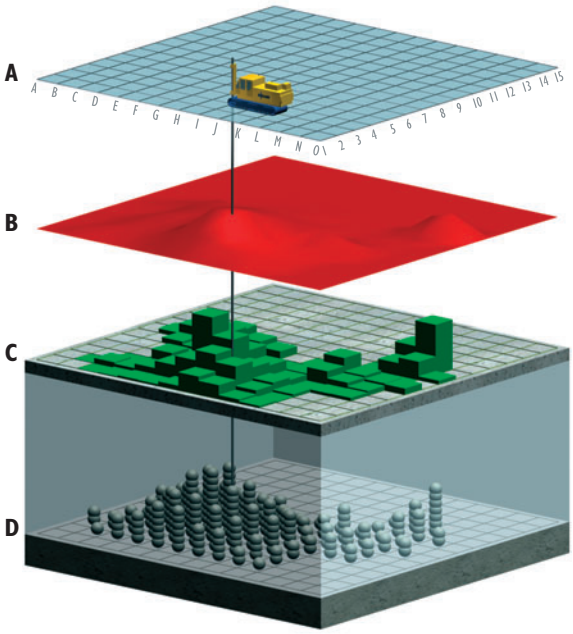


3

4 Visualisation

Each working level is shown separately

A *Working level / grid*
B *Pressure distribution*
C *Quantity distribution*
D *Injections*



1 *Exposed section of a compaction grout column*

2 *Exposed compaction area*

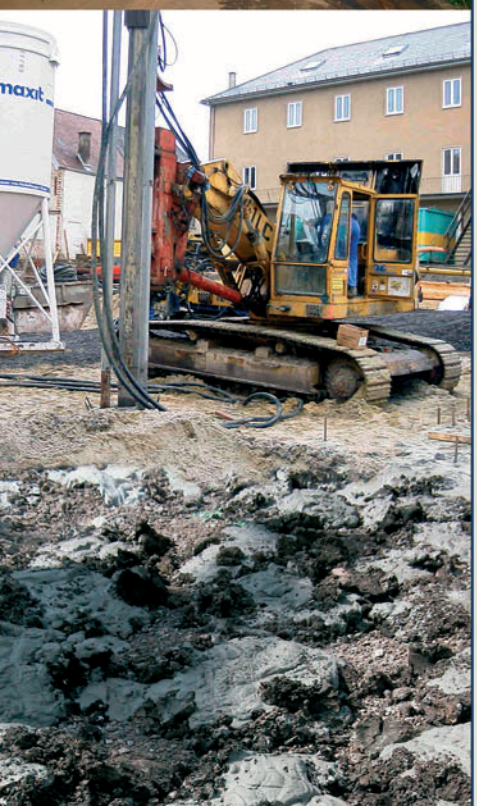
3 *Soil improvement for the construction of a new power plant in Indonesia*

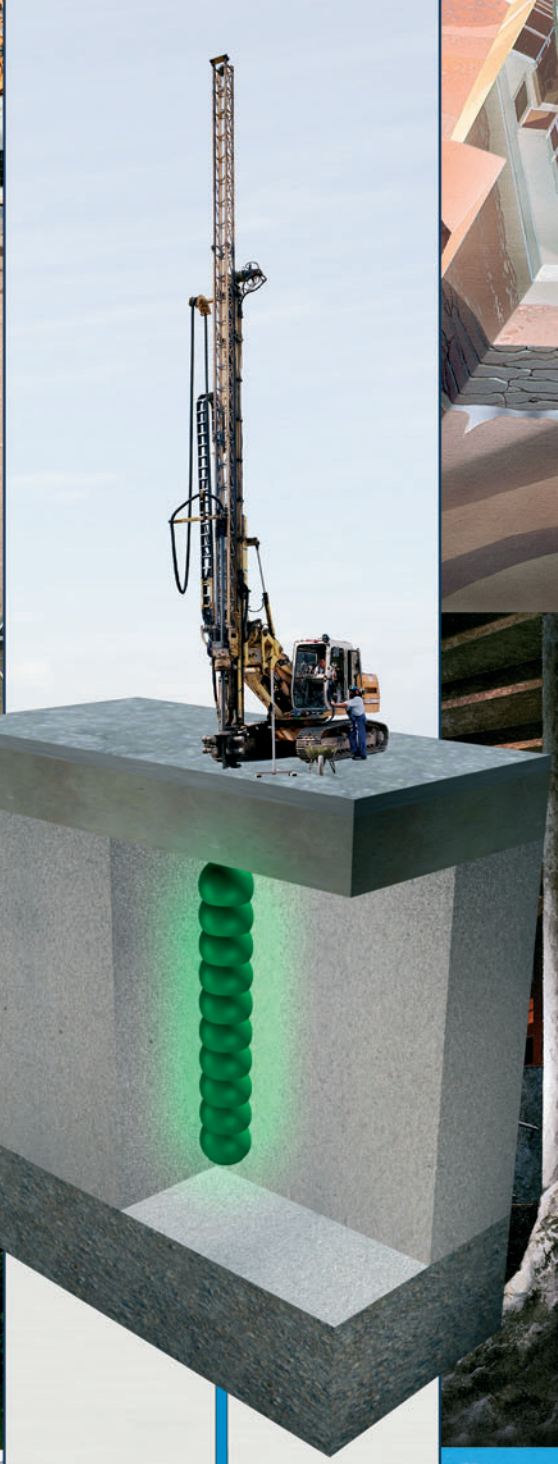
4 *Modern software is used for simulation and visualisation*

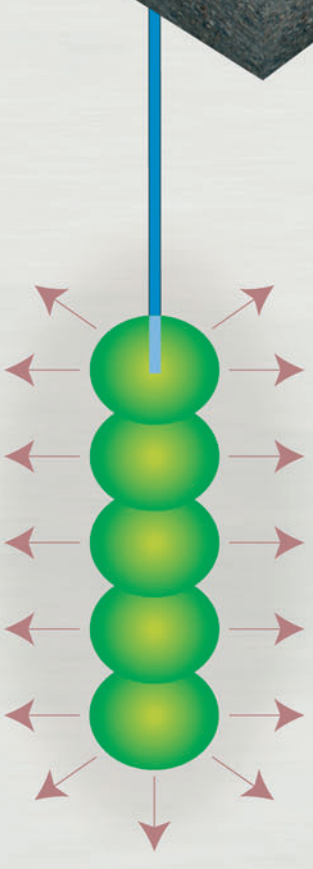



KELLER




maxit









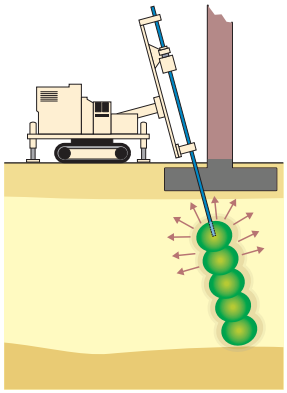

1:55

Foundation Rehabilitation

Subsoil and footing together form the foundation system. Detrimental external influences and long-term processes which affect the subsoil conditions, are known to lead occasionally to a reduction in the bearing capacity of a foundation. Structural modifications of existing buildings often require an increase of the bearing capacity of the foundation.

In all these cases, the compaction grouting process can be sensibly applied for target-oriented rehabilitation of existing footings, in order to stop incompatible settlements and deformations and to reverse them, if necessary, or to increase the bearing capacity of an existing foundation in a controlled manner.

The compaction grouting process also has proven to be an excellent technique for underpinnings, in case of sensitive or non-homogeneous soil conditions are encountered or if the causes of the settlements could not exhaustively be determined.

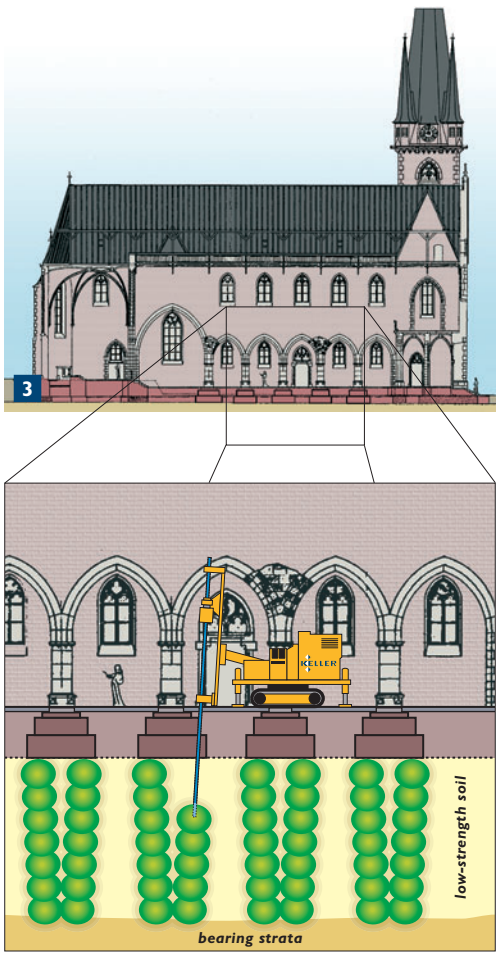


REHABILITATION









1 *Rehabilitation of a road bridge after flood damage*

2 *Rehabilitation of the footings under a residential building*

3 *Improvement of the stability of a church*

4 *Rehabilitation of a footing underneath a historical building*

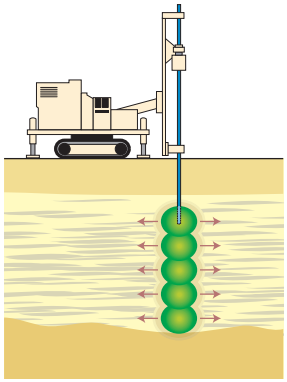


4

Cavity Grouting

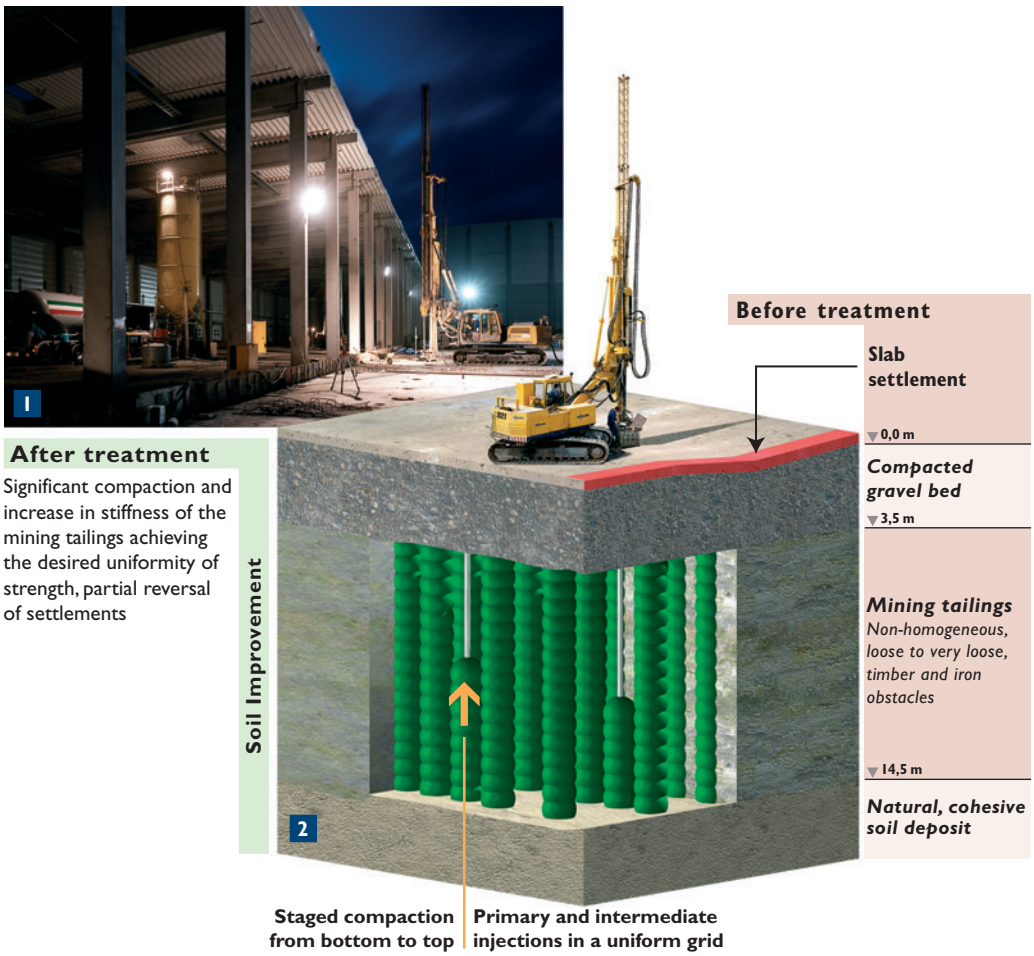
With ressources becoming more and more scarce and prices for land increasing, it becomes increasingly necessary to build structures in areas, which were previously considered to be unsuitable for construction due to their poor soil properties.
In former mining regions, mining tailing backfills were for example sold as cheap construction lots after a certain rest period and are frequently used for the construction of industrial and commercial buildings.

Due to the direct interaction between soil and grout material the compaction grouting process is a particularly suitable technique to avoid or compensate nasty surprises as a result of unforeseen non-homogeneities or cavities occurring during the construction phase when building on poor soils of this kind.



GROUTING





1 *Soil improvement under the slab of a multi-storey warehouse under construction*

2 *Improvement principle: compaction underneath a settled slab*



1

Keller Holding GmbH
Head Office and Overseas Division

Kaiserleistraße 44
63067 Offenbach
Germany

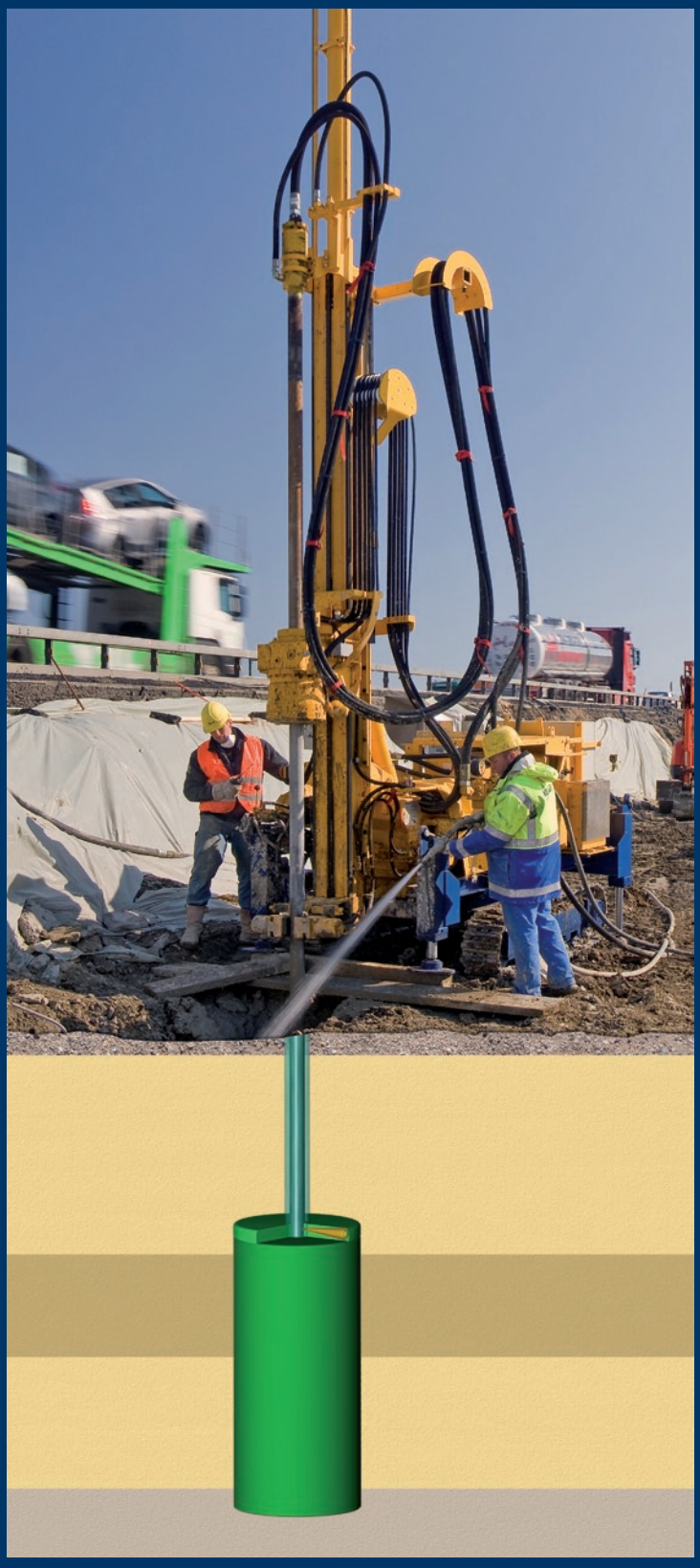
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www.KellerGrundbau.com

KELLER

A company of Keller Group plc

The Soilcrete® – Jet Grouting Process



Brochure 67-03E

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Soilcrete® –
Jet Grouting Process 4

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Addresses 12



Soilcrete® – History



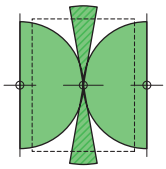
Small scale underpinning works were the beginnings of an unusual way of jet grouting. To reach today's state of the art, additional development was necessary.

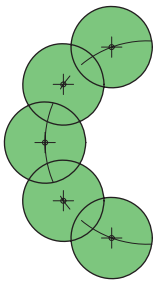
- The process was modified to suit different types of soils.
- The application was developed step by step to provide solutions to a variety of problems.
- Equipment was developed and improved.
- The Soilcrete®-technique has been enhanced constantly.

This brochure reports on the state of the Soilcrete®-technique today.

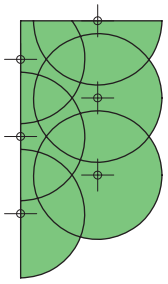
With the aquisition of a licence in 1979 for the jet grouting process and the introduction into Germany under the trademark "Soilcrete®", Keller Grundbau entered into a new field of soil stabilization.

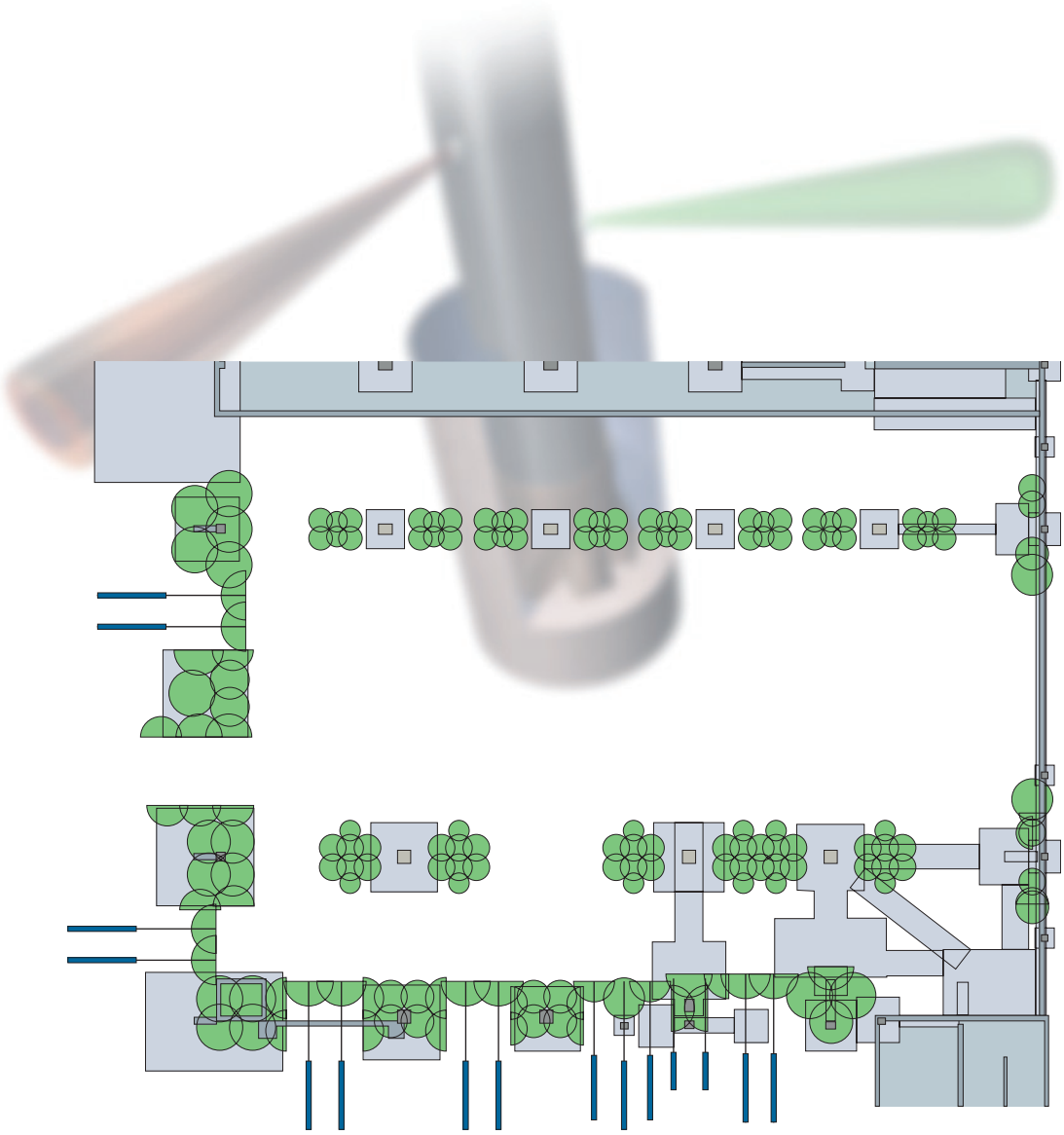




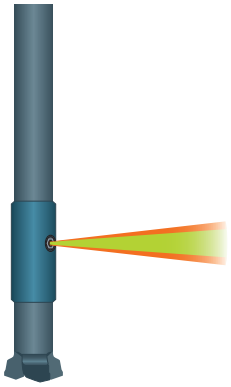








Soilcrete® – Jet Grouting Process



The jet grouting process

The jet grouting process "Soilcrete®" is recognized as a cement soil stabilization. With the aid of high pressure cutting jets of water or cement suspension having a nozzle exit velocity ≥100 m/sec eventually air-shrouded the soil around the borehole is eroded.

The eroded soil is rearranged and mixed with the cement suspension. The soil-cement mix is partly flushed out to the top of the borehole through the annular space between the jet grouting rods and the bore-hole. Different geometrical configurations of Soilcrete®-elements can be produced.

The name "*Soilcrete*" derives from the concept "*soil*" to "*concrete*" a form of soil with a concrete consistence, a description that characterizes the type of soil stabilization.

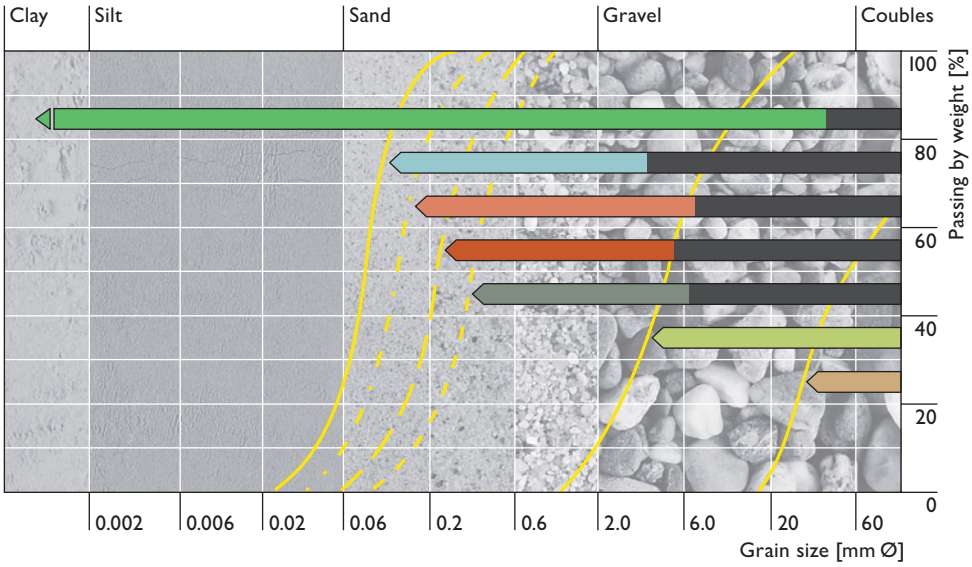
Application limits for grouting techniques

- Soilcrete® – Jet Grouting
- Synthetic Solutions
- Sodium Silicate Solutions [lv]
- Silicate Gel [hv]
- Ultra Fine Cement
- Cement Suspension
- Mortar

nv = low viscous
hv = high viscous

economical

uneconomical



The erosion distance of the jet varies according to the soil type to be treated, the kind of Soilcrete®-process and the jetting fluid being used, and may reach up to 3.5 meter.
This technique is regulated by European Standard EN 12716.

Applications

In contrast with the conventional ground stabilisation methods Soilcrete® may be used for stabilization and sealing of all kind of soil ranging from loose sediments to clay. This applies also for non homogeneous soil formations and changing soil layers, including organic material. Soft rock formations have also been treated by Keller – for example sandstone with weak grain texture.

The Soilcrete®-properties

Soilcrete® acts in the ground according to the specification either as a stabilization or as a sealing structure. A combination of both properties is increasingly required. The compressive strength of Soilcrete® ranges from 2 to 25 N/mm² and is determined by the cement content and the remaining portion of the soil in the Soilcrete® mass. The sealing effect of Soilcrete® against water ingress is achieved by selecting a suitable grout suspension and if necessary by the addition of bentonite. The type and quantity of the injected grout material as well as the kind and volume of the remaining soil particles in the Soilcrete® mass guiding the sealing properties.

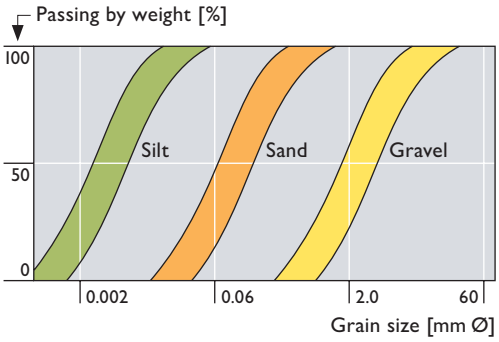




Soilcrete®-sealing mixtures tested in a laboratory

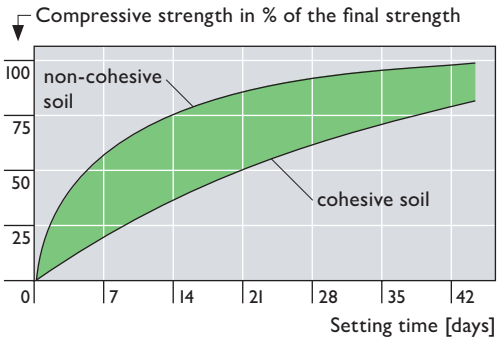
Soilcrete®-pump and control station

Compressive strenght of Soilcrete®



Type of soil	Silt	Sand	Gravel
Compressive strength [N/mm²]	≤ 5	≤ 10	< 25

Development of Soilcrete®-strength



Depending on the nature of soils, Soilcrete®-cut-offs are able to reduce the coefficient of permeability by several decimal powers. High quality requirements in respect of the degree of sealing effect necessitate extensive production efforts. For many applications both the strengthening and sealing characteristics of the Soilcrete®-elements are used. The selected suspensions need to be composited accordingly.



Foundation

Soilcrete®

Soilcrete®-cubatures forming a firm contact with any kind of foundation forms

Soilcrete® – Process Variations

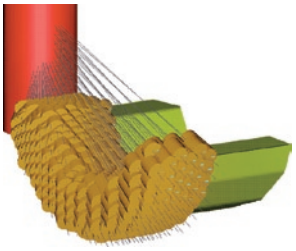


The triple Soilcrete®-rods provide separate flows of air, water and grout to the jet nozzle holder

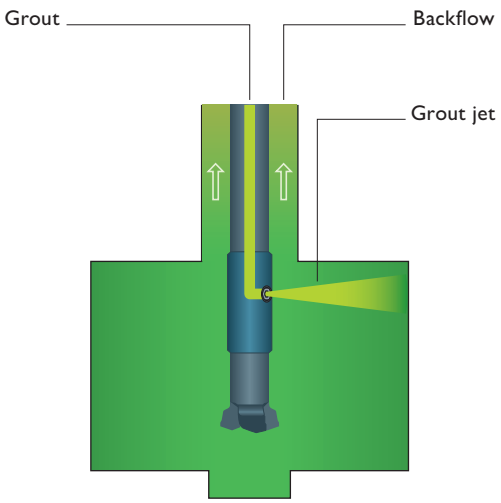


Drill/core bit with the above-installed Soilcrete®-monitor.

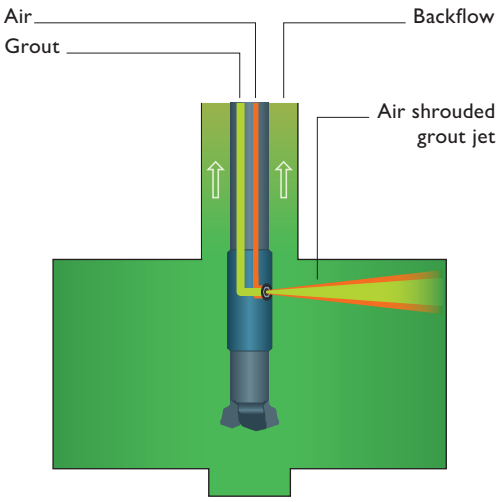
Computer design and optimisation of complicated Soilcrete®-elements by Keller's own software Soiljet®



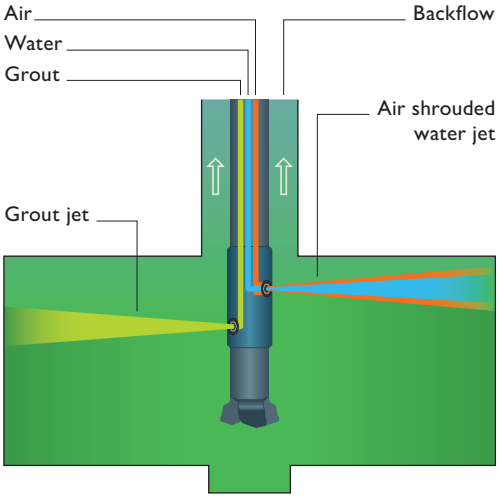
Soilcrete® is produced in three different ways. The method to be used is determined according to the prevailing soil conditions, the geometrical form and the required quality of the Soilcrete®-elements.



Soilcrete®-S
(Single direct process) operates with a grout jet of min. 100 m/sec. exit velocity for simultaneous cutting and mixing of the soil without an air schroud. The S process is used for small to medium sized jet grout columns.

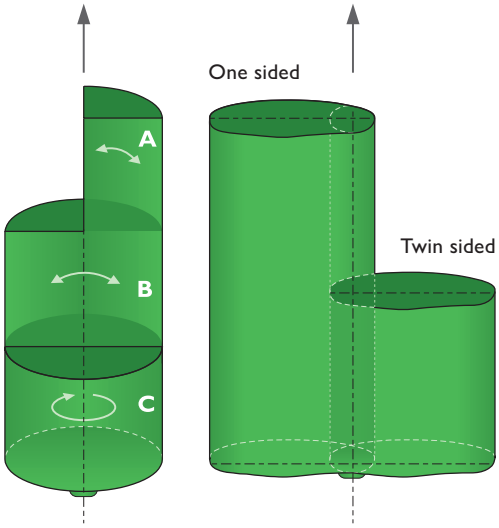


Soilcrete®-D
(Double direct process) operates with a grout jet of min. 100 m/sec exit velocity for simultaneous cutting and mixing of the soil. To increase the erosion capability and the range of the grout jet, air shrouding by means of shaped air jet nozzle is used. The D process is mainly used for panel walls, underpinning and sealing slabs.



Soilcrete®-T
(Triple separation process) erodes the soil with an air shrouded water jet of min. 100 m/sec exit velocity. Grout is injected simultaneously through an additional nozzle located below the water jet nozzle. The grout pump pressure ranges above 15 bar. A variation of this process operates without air shrouding of the water jet. The T process is used for underpinning works, cut off walls and sealing slabs and is mainly used for the treatment of cohesive soils.

Soilcrete® – Construction Forms



Column **Lamella**

A = Quarter Column
B = Half Column
C = Complete Column

The basic geometrical form of Soilcrete®-elements is created through movements of the drill rods:

- Pulling of the drill rods without rotation resulting in panels – if several jet nozzles are used multiple panels may be produced.
- Pulling and swivelling create segments.
- Pulling and rotating create complete columns.

Soilcrete® – basic forms
The Soilcrete® basic forms may be arranged and combined in any way to form each type of element.

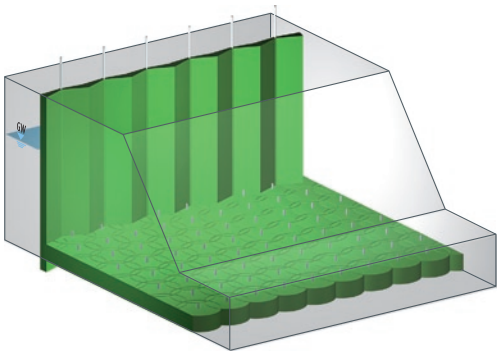


Exposed test field with several Soilcrete®-disks for a sealing slab

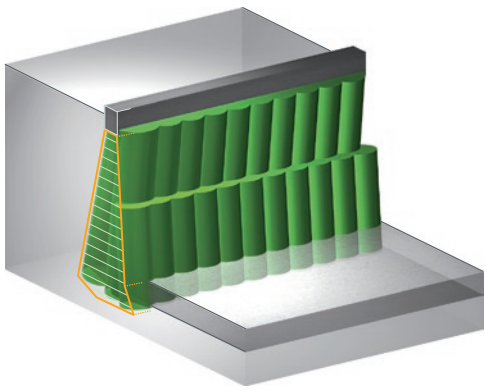


Exposed Soilcrete®-elements of an underpinning wall in sandy soils

Interlinked Soilcrete®-elements

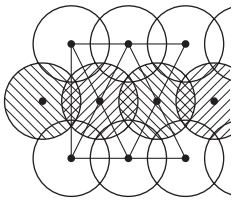


Panel wall with sealing slab

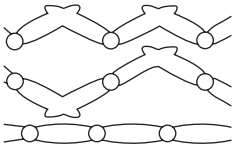


Underpinning elements

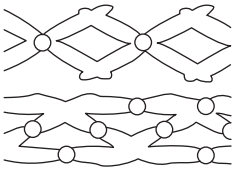
Interlinked bodies – constructed from uniform elements



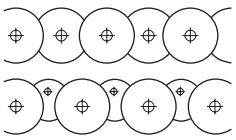
Sealing slabs



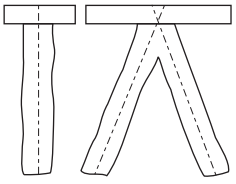
Panel walls



Box walls



Column walls



Deep foundation

Soilcrete® – Construction Sequenses

Small drilling rigs manufactured by Keller provide access to confined working places

Soilcrete®-site installations consist of storage containers, silos and a compact mixing and pumping unit. Several high pressure hose connections and control cable lines connect the pumping unit with the Soilcrete® drilling rig at the installation point. The mast length of the rig varies from 2.0 m in cellars or shaft sand up to 35.0 m in open areas. The bore points are normally located in small trenches equipped with sludge pumps. From there the excess material, water-cement-soil mixture is pumped to setting ponds or tanks.

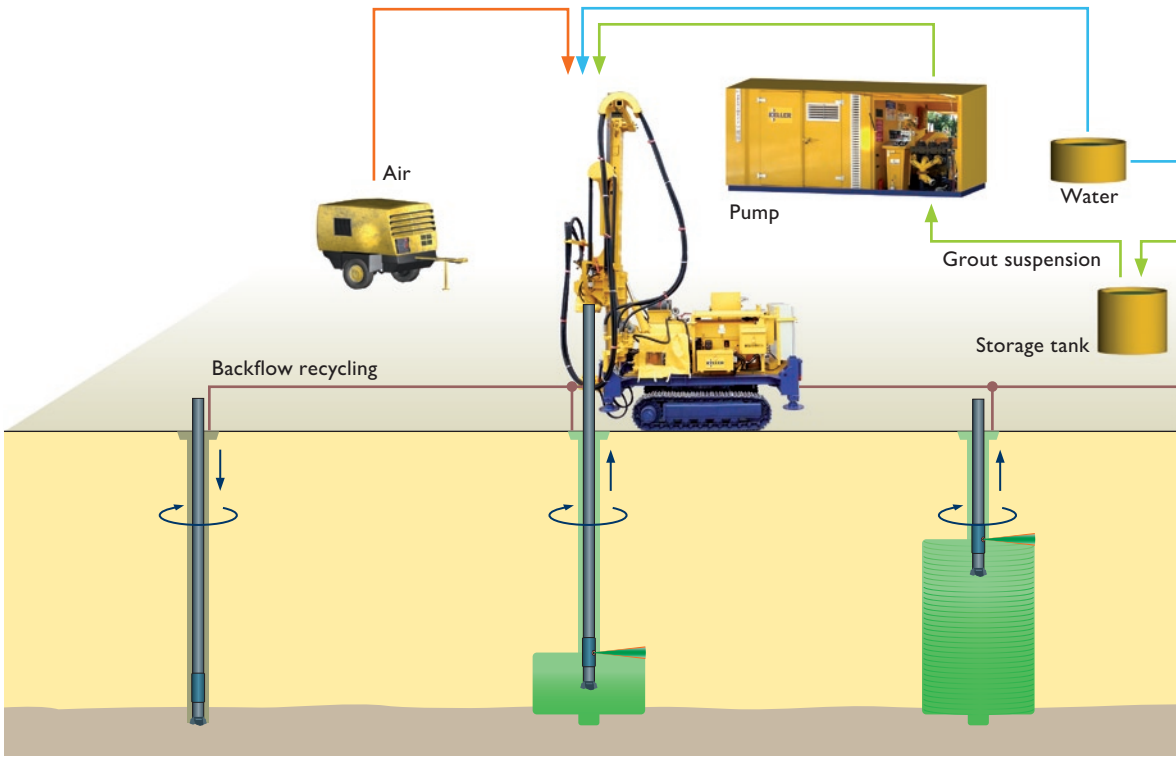












1 Drilling

Drill rods equipped with jet nozzle holder and drill bit are used to drill the jet grouting hole down to the required depth. Normally the jet grout mixture is used as drill flushing to stabilize the borehole during the drilling operation. In masonry and concrete, special drilling bits are used.

2 Jetting

The disolution of the grain texture with a powerful fluid jet starts at the lower end of the Soilcrete®-element. The excess water soil cement mixture is removed to the surface through the annular space between drill rod and bore-hole wall. The pre-selected production parameters are constantly monitored.

3 Grouting

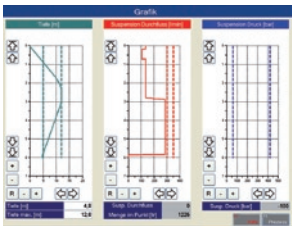
For all Soilcrete®-variations cement suspension is injected under pressure simultaneously with the erosion of the soil. The turbulences caused by the jetting technique results in the uniform mixing of the grout with the soil within the treatment zone. Until the grout in the Soilcrete®-elements starts setting hydro-static pressure in the borehole is kept by backfilling grout into the hole from time to time.

Process monitoring

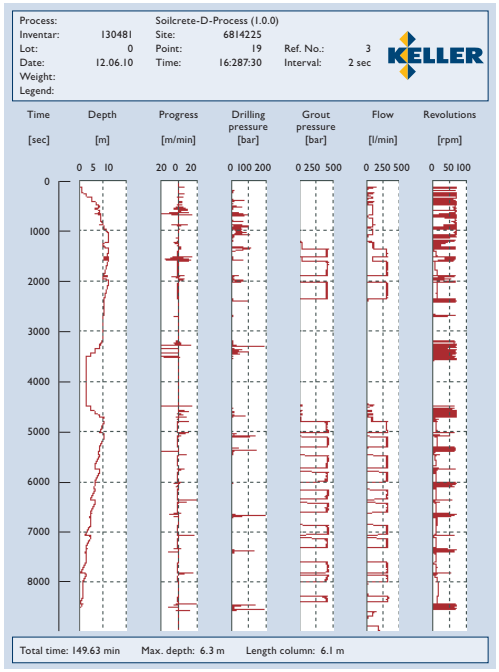
Up to 12 different parameters for the construction of Soilcrete®-elements may be recorded and used by the engineer-in-charge of supervision and control.



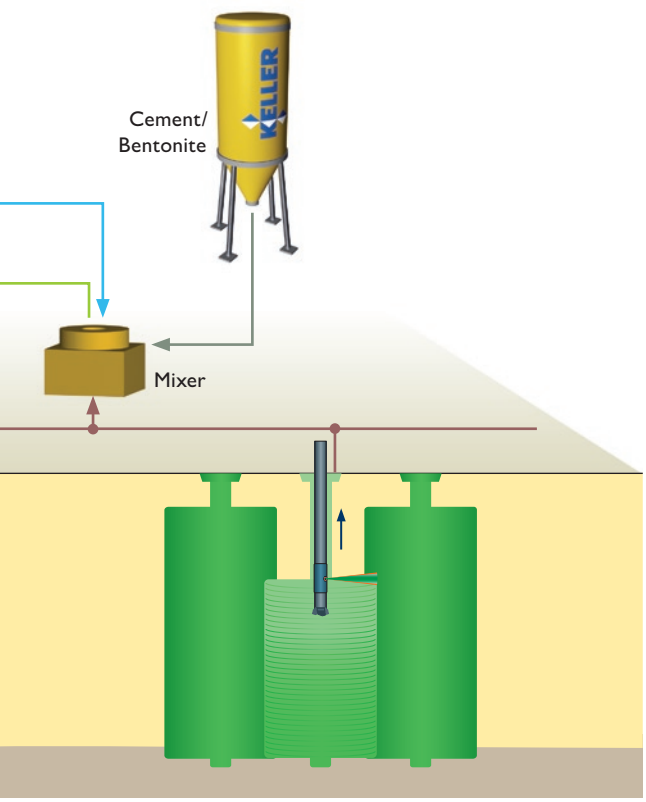




Recording unit M5



Important construction parameters may be electronically recorded



4 Extension

Soilcrete®-elements of each form may be constructed fresh on fresh as well as fresh against firm and combined and connected in a variety of ways. The working sequence follows the technical requirements and the conditions of the structure to be treated.

Optical and manual checking as well as controls and tests according to the requirements laid down in the European standard EN 12716 ensure quality workmanship.



Core recovery from Soilcrete®-elements

Soilcrete® – Applications





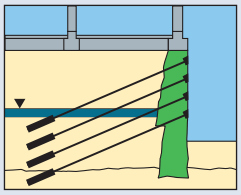






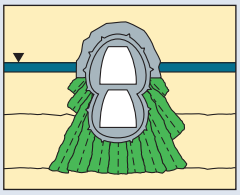
Soilcrete® for stabilisation
Underpinning work for adjacent construction pits is one of the main tasks of soil stabilisation – followed by foundation modifications and restoration. Soilcrete® opens new ways for tunnel constructions in loose soils.

Underpinning



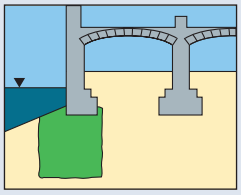
Underpinning by means of low de-formation gravity walls sometimes also used as a ground water sea-page barrier, may be safely constructed even from confined working areas.

Tunnel protection



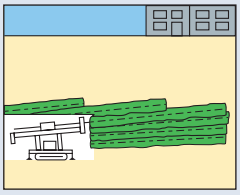
Soilcrete®-tunnel protection is mainly constructed in loose soils below or close to endangered structures, sometimes with the aim of reducing the groundwater ingress into the tunnel excavation.

Foundation restoration



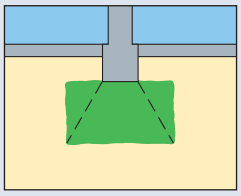
Historical buildings may be endangered in the event of settlements occurring. Soilcrete® provides a safe foundation with the maximum structural protection.

Soilcrete® – Horizontal



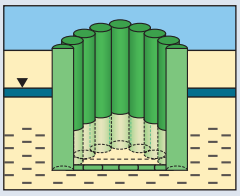
Horizontal Soilcrete®-columns protect tunnel drives in loose soil formations. They are constructed from working faces and are horizontal or slightly inclined.

Foundation modification



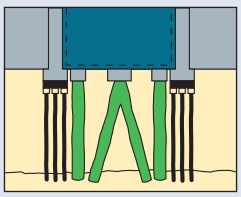
Changes in utilisations or modifications of buildings often require an enlargement or alteration of the foundation. Soilcrete® is an economical and flexible solution for this task.

Shaft supports



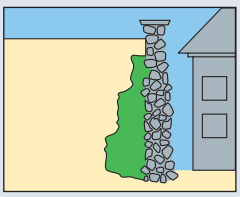
Shafts with intersecting Soilcrete®-columns are constructed if a vibration free installation is required and/or the shafts enter into groundwater bearing strata.

Deep foundation



Soilcrete® is used for new foundations which require special care inview of nearby existing structures such as historical buildings and other shock-sensitive constructions.

Earthpressure relief



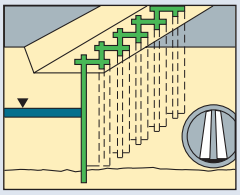
Structures exposed to earth pressures, such as historical walls, abutments, avalanche galleries, steeps lopes protections or quay walls may be relieved by the addition of or connection to a statically calculated backup Soilcrete® body.

Soilcrete® for sealing

Building pit walls constructed of deformation and ground water resistant Soilcrete®-underpinning elements, combined with low permeable Soilcrete®-slabs enables the execution of deep building pits without the need for large scale ground water lowering. Environmental safe mineral binder materials are used for Soilcrete®.

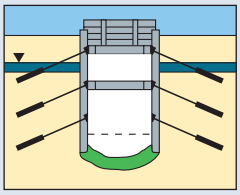


Panel walls



Soilcrete®-panel walls to cut off ground water are used below roads and buildings, for crossing pipelines and to subdivide building pits into different excavation sections. According to the sealing requirements single or multiple panels may be constructed.

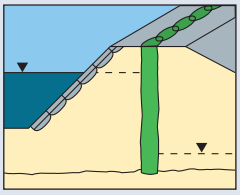
Vault slabs



For small building pits and shafts with reduced width Soilcrete®-slabs are used as prevention against the water uplift pressure, reducing the required depth of slabs normally required to resist the hydrostatic uplift.

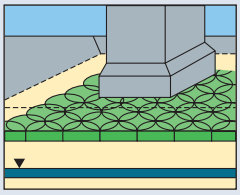


Column walls

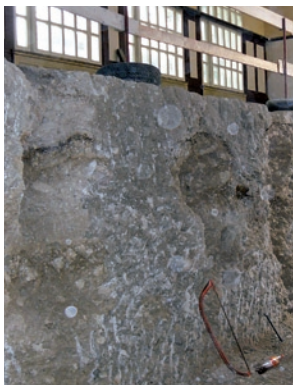


In the event of higher mechanical strain by shearforce, danger of undermining or of a high impermeability requirements, cut off walls of intersecting Soilcrete®-columns may be constructed.

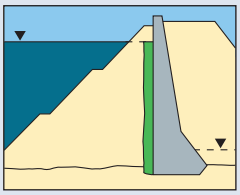
Sealing cover



The Soilcrete®-cover protects the groundwater below buildings against affects from construction activities and old toxic waste de-posits.

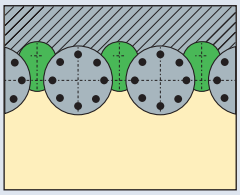


Dam sealing



Soilcrete® may be used to repair dam cores or enlarge cut-off walls in or below dams.

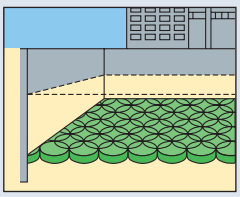
Joint sealing



For sealing of joints between piles, sheet piles or other construction parts in the ground the Soilcrete®-wing-jet is applied.

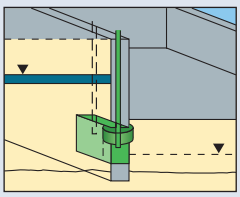


Sealing slabs

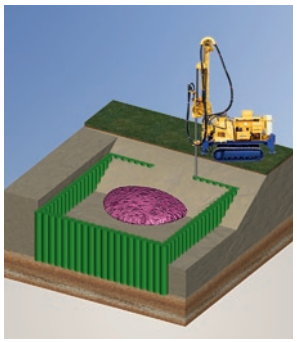


Soilcrete®-sealing slabs are constructed by means of overlapping columns within an uplift proof depth. The sealing slabs may be connected to any kind of vertical sealing systems.

Groundwater exits



Sealing walls are often used as temporary ground water barriers. The reinstatement of the permeability may be reached using the Soilcrete®-process to wash out the binder material from predetermined sections.



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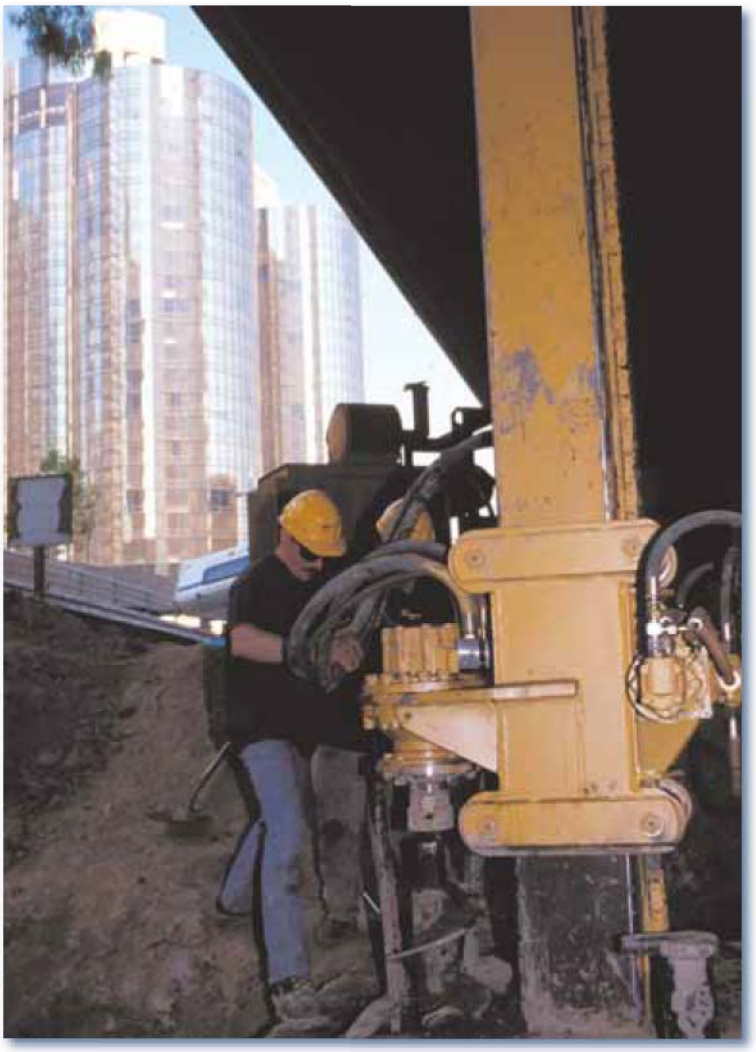
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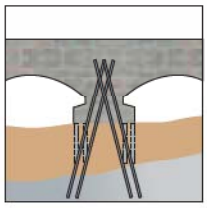
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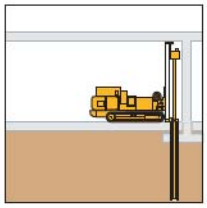
MICROPILES

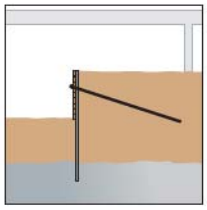
Project Support From The Ground Down

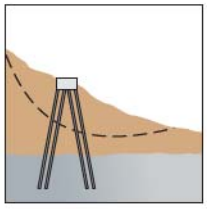


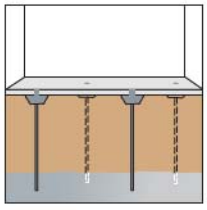
Susan Loftwasser

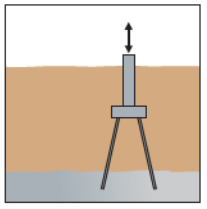


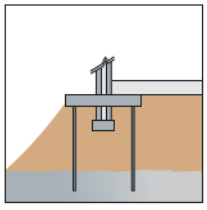












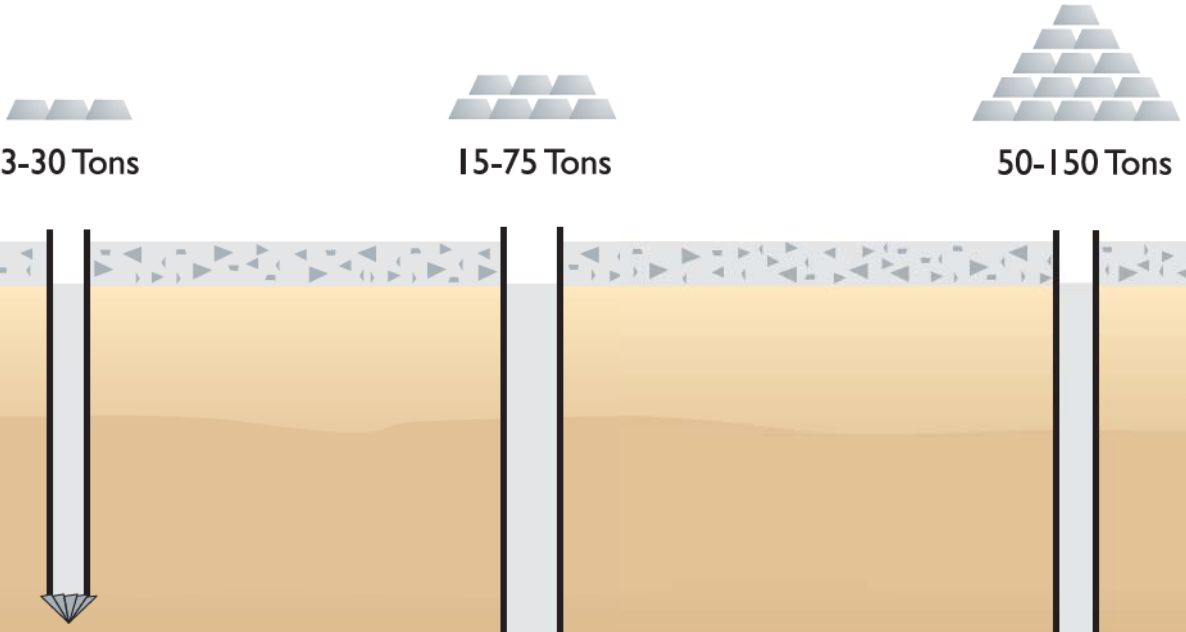
Micropiles are small diameter piles that can be installed in almost any type of ground where piles are required, with design loads as small as three tons and as high as 500+ tons. Also known as minipiles, pin piles, needle piles or root piles, micropiles can offer a viable alternative to conventional piling techniques, particularly in restricted access or low headroom situations.

Hayward Baker brings the same high quality technical expertise to its micropile technology as it does to all of its ground improvement techniques. This expertise, coupled with extensive equipment resources and proven project experience, allows us to respond quickly to provide the right micropile solution for your needs. Our commitment to excellence is reflected in the successful completion of projects ranging from residential underpinning to preservation of historic structures, to seismic retrofit, and to foundation upgrade for large manufacturing plants.

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Micropile Designs Are Tailored To The Problems They Solve . . .



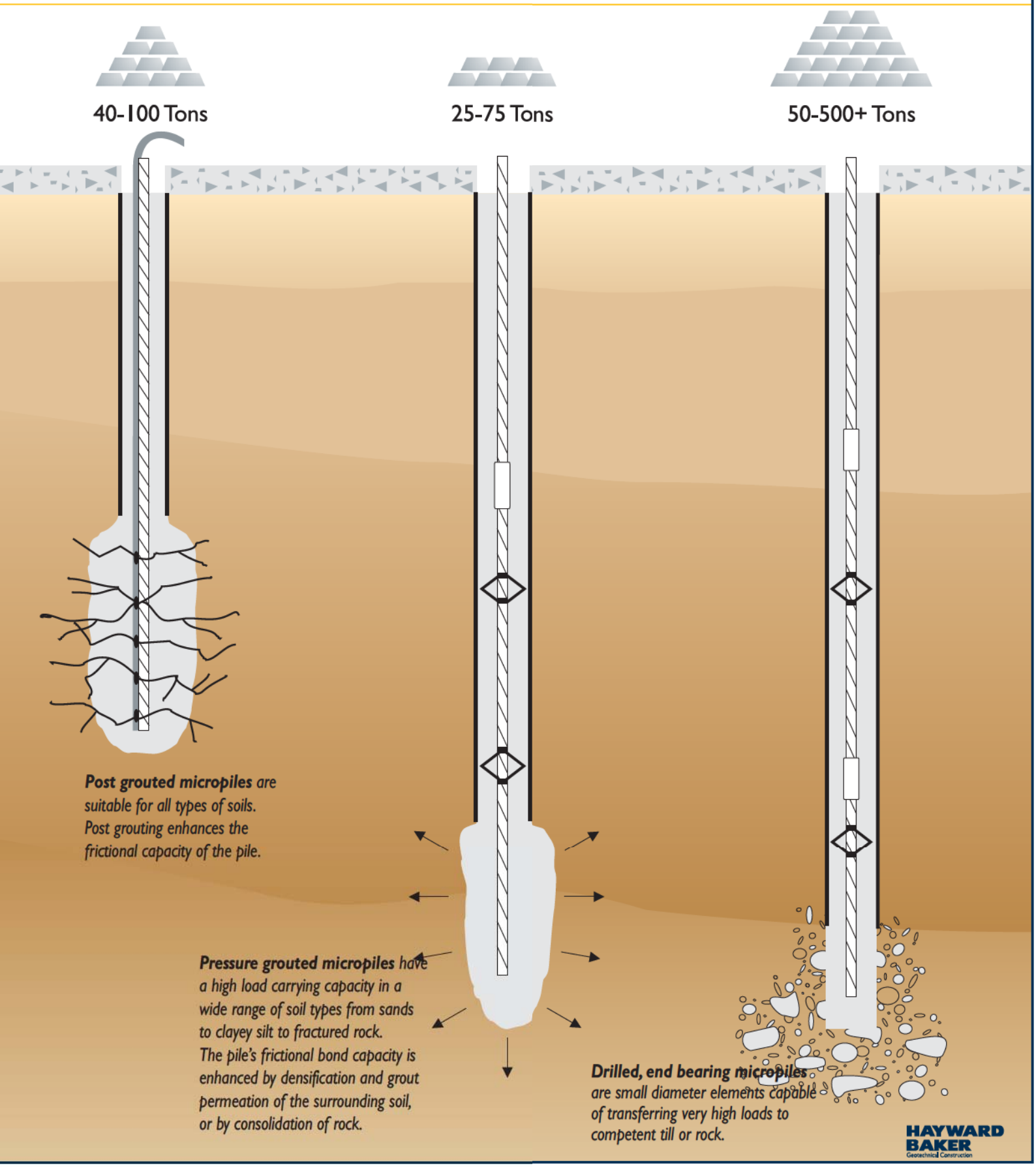
Pushed or Driven micropiles *are typically installed through loose/soft materials to competent bearing strata to carry light loads such as are required for slab support.*

Compaction grouted micropiles *are able to develop loads at shallow depth in loose, sandy soils. The expanded base geometry and densification of the soil provide enhanced load capacity.*

Jet grouted micropiles *have a high load carrying capacity and are ideal for all types of soils where sufficient enhancement of soil strength cannot be achieved by direct pressure grouting or post grouting.*

Selection of the right micropile to meet design objectives is primarily a function of soil conditions and load transfer requirements. Since soils and structural loads will vary tremendously from project to project, an equally varied range of micropile options is necessary. Hayward Baker offers the range of micropiles needed for the most appropriate technical solution, together with innovative methods of connecting piles to existing foundations, and a variety of corrosion protection methods when needed. We also have the capability of combining our micropile technology with one or more of our other ground improvement techniques to meet unique or complex project requirements cost-effectively and efficiently.

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Post grouted micropiles *are suitable for all types of soils. Post grouting enhances the frictional capacity of the pile.*

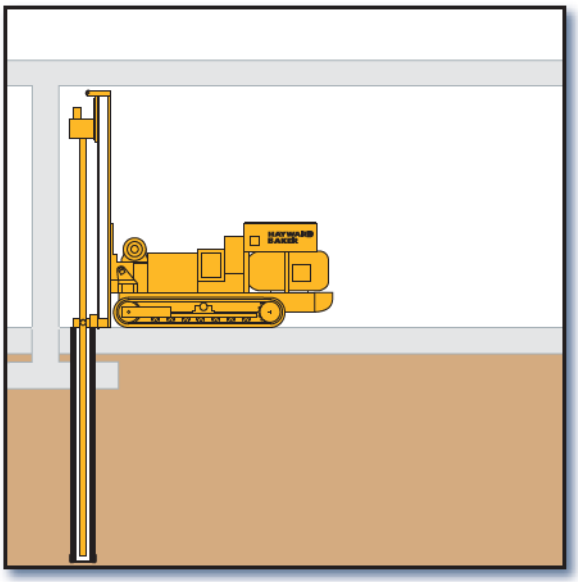
Pressure grouted micropiles *have a high load carrying capacity in a wide range of soil types from sands to clayey silt to fractured rock. The pile's frictional bond capacity is enhanced by densification and grout permeation of the surrounding soil, or by consolidation of rock.*

Drilled, end bearing micropiles *are small diameter elements capable of transferring very high loads to competent till or rock.*

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Micropiles: Solutions For Tough Geotechnical Problems . . .



◄ Supporting New Loads in Congested Areas

Hayward Baker's micropile drill rigs allow installation in restricted access, low headroom interiors, permitting facility up-grade with minimal disruption to normal operations.

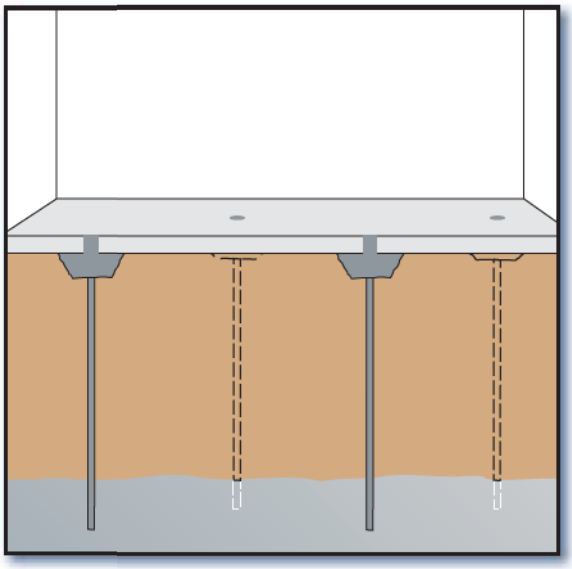


▲ Resisting Uplift/D

Micropiles can be post-g
frictional forces, thus pro
resistance to tensile load

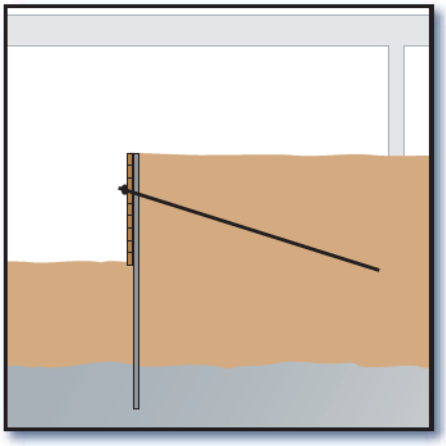
Arresting Structural Settlement ►

Micropiles bypass the poor load bearing soils to arrest structural settlement problems.



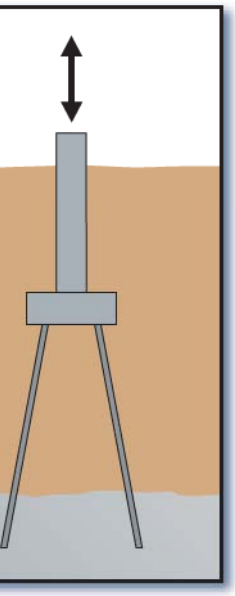
Excavation Support in Confined Areas ►

Micropiles connected by wooden lagging can form walls for excavation support in basements and other confimed areas.



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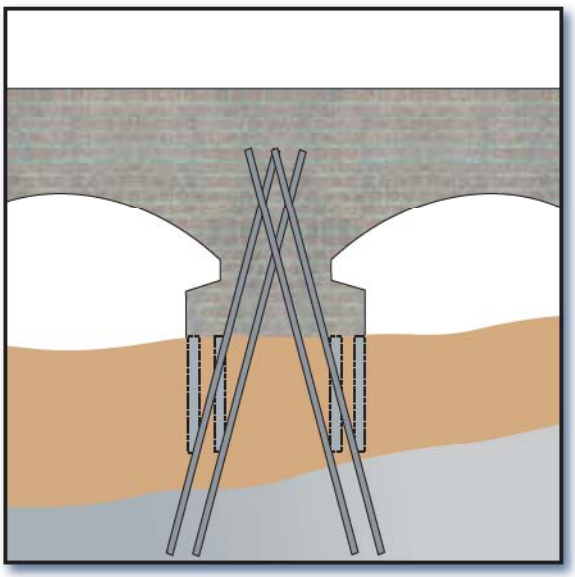


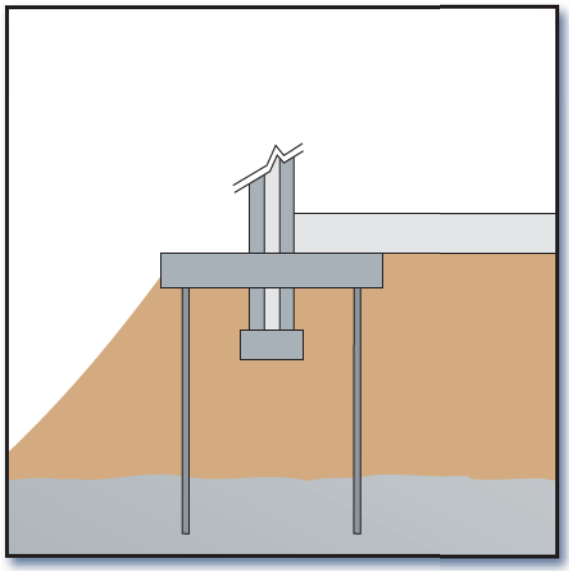
ynamic Loads

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Seismic Retrofit ►

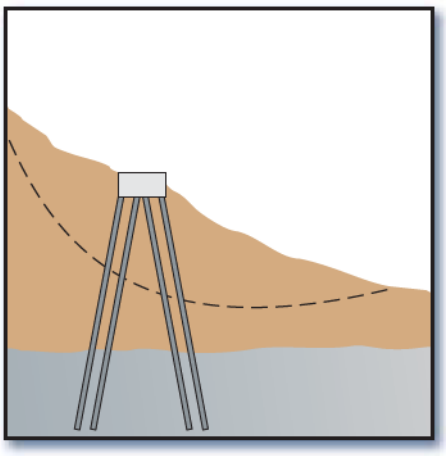
Micropiles transfer loads through liquefiable soils to competent bearing strata to conform to seismic design requirements.





◄ Underpinning

Micropiles can be used as a form of underpinning to stabilize foundations adjacent to future excavations.



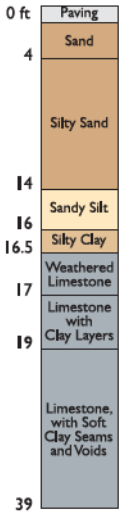
◄ Stabilizing Slopes

Micropiles can serve to "stitch" the soil together, focusing on predicted shear zones to enhance mass stability.

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Case Histories...



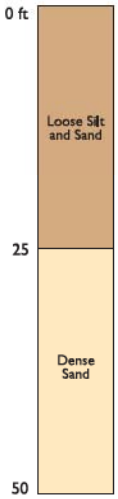
Dorney Park Rollercoaster
Allentown, Pennsylvania

As part of a maintenance upgrade at Dorney Park in Allentown, PA, battered micropiles were installed to support 14 piers along the outboard side of the roller-coaster track curve to increase the factor of safety and stability of the structure. Working from the roller-coaster track itself, part of which traversed a man-made lake, a 15-ton, small diameter micropile was socketed into competent rock at each pier location. Despite a challenging access situation, the project was completed in just one week, allowing the ride to open on schedule.





Susan Lohwasser



Odenton Commuter Station
Odenton, Maryland

As a value engineered alternative to an originally specified cast-in-place piling technique, 35-ton micropiles were selected to support a new platform at the Odenton commuter station. Selection of the micropiles was influenced by several critical factors. Low overhead, 12,000 volt catenary cables precluded access for conventional piling equipment. Also, 50% of the piles were to be installed within 5 ft of an operating track. Although the work was to be performed at night and most of the rail traffic could be diverted, there would still be occasions when the track would be used. This would require clearing the work zone of personnel and equipment within minutes, an exercise not feasible with cast-in-place piling. A further benefit was that the nine-inch diameter micropiles could be easily installed around existing underground utilities.



Five Star Hotel, Hawaiian Islands

Differential settlements of over three inches, attributed to poorly compacted backfill, had caused unsightly and extensive damage to a luxury resort in the Hawaiian Islands. Several structures, terraces and entryways surrounding the main, multi-story structure were affected. In a fast-paced underpinning program designed to minimize impact on the hotel's guests, 935 micropiles were installed into bedrock at depths varying from 15 to 35 ft. The galvanized piles were designed and tested to 24 tons for floor slabs and 40 tons for foundations. Although pile installation involved working in formal dining rooms, tropical gardens, terraces and the main building, cooperation between engineers, contractors and hotel staff ensured that the resort's prestigious image was upheld.


Francis Gularte

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"Dry Soil Mixing permits construction on sites previously considered unsuitable. Soft coastal soils can now be rendered buildable land."

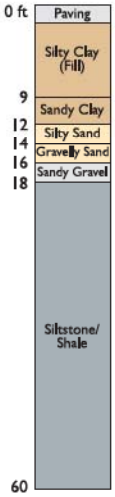


Westwood Connector Bridge
Orlando, Florida

Expansion of the Orange County Convention Center required a new bridge between the new exhibit hall and hotels located on opposite sides of the busy Bee Line Expressway. Two of the planned bridge pier locations were underlain by numerous, closely spaced underground utilities which could not be interrupted or relocated. The planned driven precast piling was determined risky for the utilities. A reduced number of higher capacity micropiles was selected to solve the problem. Through careful probing at each location, the micropiles were threaded past the utilities. The nominal 10-inch diameter micropiles were drilled and grouted into the limestone formation at an approximate depth of 140 ft. A load test was successfully performed to twice the design load of 165 tons.


Vince Hull


George Burke



Fourth Street Viaduct
Los Angeles, California

Part of the City of Los Angeles' seismic retrofit program included upgrading of the Fourth Street Viaduct, a major traffic artery into the downtown business district. Small diameter tiedown piles (225 tons ultimate design load) were to be founded into the underlying silts and clays and keyed into existing pile caps. Performance testing in both tension and compression was required to ensure adequate uplift as well as compression capacity. Design input from Hayward Baker resulted in the acceptance of seven inch diameter, high capacity, post-grouted micropiles. Following successful load testing, 160 piles were installed to an approximate depth of 60 ft, meeting the seismic design requirement.



ITT Rayonier Paper Mill
Fernandina Beach, Florida

Over the course of several years, structural distress had occurred in a pulp machine building at an ITT Rayonier paper mill. Original, untreated timber piles had settled or deteriorated away from the pile caps by up to 18 inches. Due to the poor condition of the timber piles, an underpinning program utilizing micropiles was implemented. Pile design capacity of 44 tons included a 12-ton per pile allowance in compression for downdrag. Prior to construction, a non-production pile was load tested to 100 tons in compression to verify design. In an operation that required tunneling beneath the building, Hayward Baker installed 36 pressure grouted piles to depths of approximately 70 ft, constructed pile caps, and connected them to the existing columns.

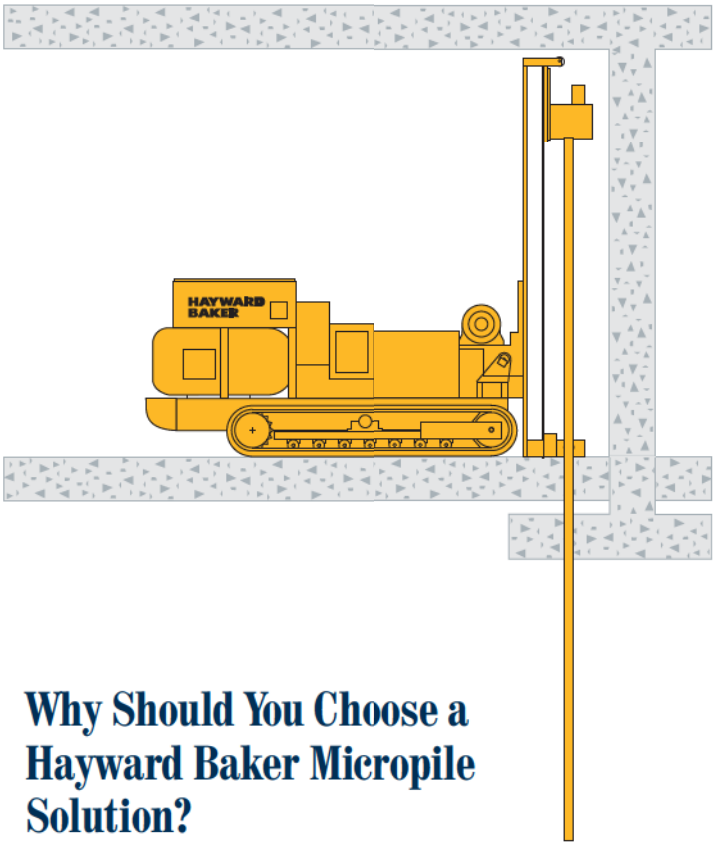

Nigel Osborn

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Advantages of Hayward Baker Micropiles

- **High performance**
- **Design loads from 3 to 500+ tons**
- **Can be readily designed for tension/uplift loads**
- **Appropriate for a wide range of ground conditions**
- **Suitable for low headroom and restricted access**
- **Low noise and vibration**
- **Can penetrate obstructions**



Why Should You Choose a Hayward Baker Micropile Solution?

For owners and design engineers contemplating restricted access underpinning projects, the initial low cost of a cast-in-place piling system may often be substantially increased by the expense of extensive utility re-routing, creation of adequate access, waste disposal, and sometimes even a shutdown in facility operations. With Hayward Baker's micropiles, these complications are rarely an issue. Our small diameter piles can be easily installed around existing utilities. Hayward Baker works closely with engineers, contractors and owners to ensure that micropile installation minimizes impact on facility operations. Moreover, micropiles greatly alleviate the quality assurance concerns associated with cast in place piling in weak soils.

As the leading specialty geotechnical contractor in North America, with more than 30 years experience in ground improvement technology, Hayward Baker has the experience, expertise, equipment and resources to provide the quality solution to your foundation problem.

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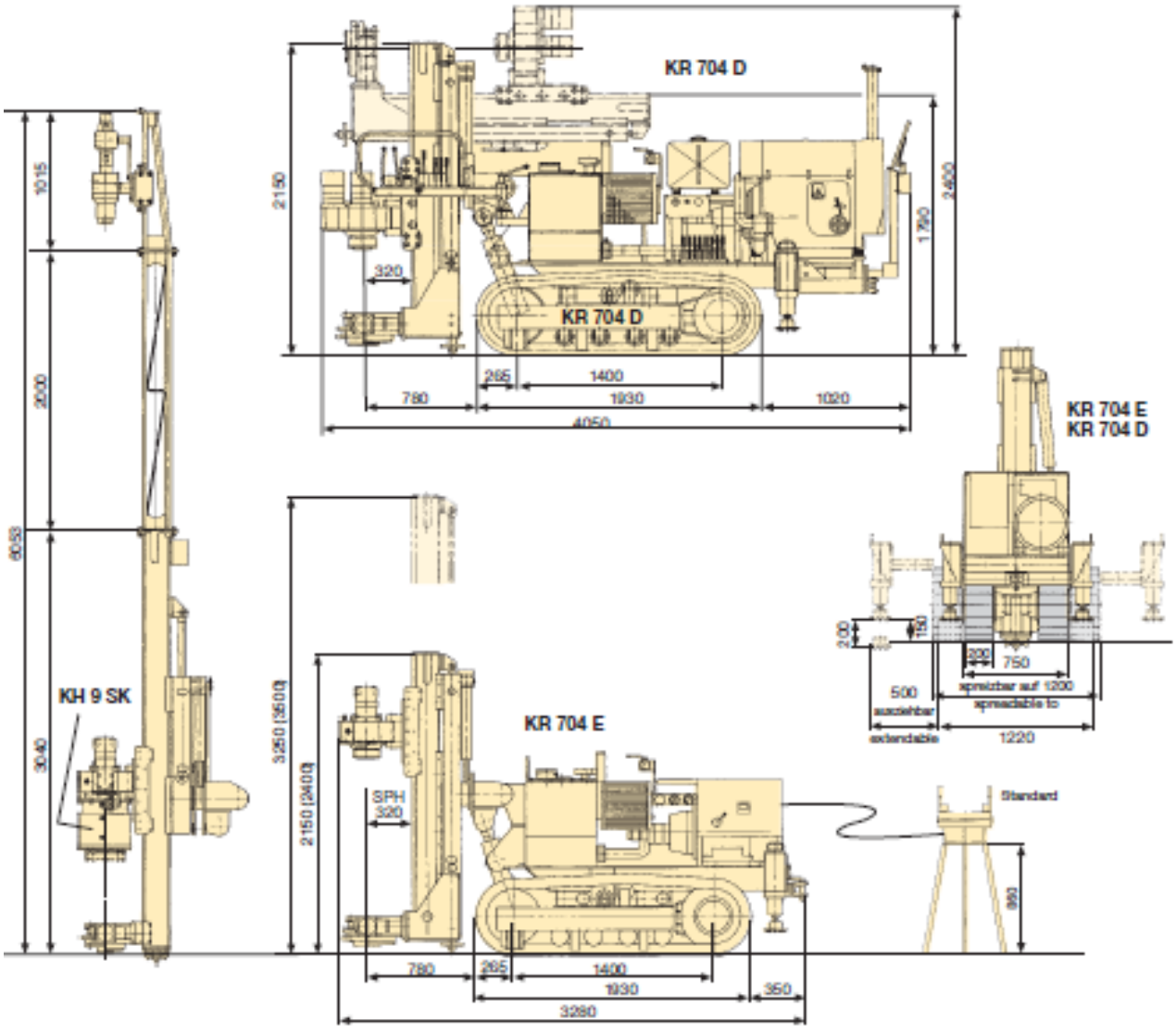
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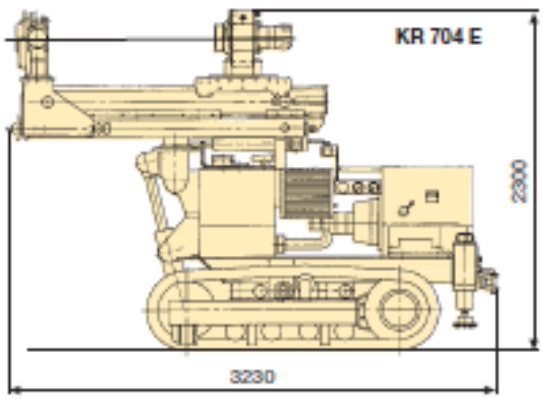
HAYWARD BAKER
Geotechnical Construction
KELLER

KLEMM KR704 DRILL RIG

Data Sheet



Transportabmessungen — Transport Dimensions



		KR 704 E	KR 704 D
Gesamtlänge	*overall length:*	3.230 mm	3.900 mm
Gesamtbreite	*overall width:*	1.220 mm	1.220 mm
Höhe	*overall height:*	2.300 mm	2.400 mm
Gewicht	*overall weight:*	4.000 kg	4.300 kg

APPENDIX D

DRAFT GEOTECHNICAL REPORT REVIEW COMMENTS AND RESPONSES

AWSS Pump Station No. 1
San Francisco, California

SAN FRANCISCO PUBLIC UTILITIES COMMISSION
AWSS Pump Station No. 1
Geotechnical Report
Review Comments

Project: AWSS Pump Station No. 1
Design Consultant: Geotechnical Consultants, Inc.
SFPUC Project Manager: David Myerson
SFPUC Project Engineer: Victor Shih
Review Item: Draft Geotechnical Report

Acceptance Codes:
1a – Accepted – already implemented
1b – Accepted
2 – Will be dealt with later
3 – Unacceptable

Date: January 24, 2012
Comment Status Codes:
1 – Resolved
2 – Resolution pending
3 – Unresolved
4 – Rolled over to next submittal

Comment No.	Comment By	Section or Drawing No.	Review Comment	Responsible Parties	Response	Acceptance Code	Comment Status Code	Notes
DRAFT GEOTECHNICAL REPORT								
1	Angus Chen	Pg. 13, 15: Sec. 7.2.1	Liquefaction – Section 7.2.1 concludes that the liquefaction is very low. Section 7.3 recommends that a groundwater elevation of zero (SFCD) be used for design. Is the higher of groundwater the more risky the soil will be subject to liquefaction? Is liquefaction analysis performed based on laboratory data? Please verify.	GTC	Based on our review of past geotechnical borings at the site, Pump Station No. 1 is founded on bedrock. Bedrock is not liquefiable independent of the groundwater elevation. Backfill soils behind basement walls and in utility trenches, if not properly compacted, are potentially liquefiable. The report will be updated to address this distinction.	1b		
2	Angus Chen	Pg.15: Sec. 7.4	Heavy equipment access – Section 7.4 indicates the bedrock should be able to be excavated using heavy equipment due to the fracturing in the rock. The basement has only limited space. Please advise the minimum space/path for heavy excavation equipment access.	GTC	Bedrock excavation is only anticipated for the micropile retrofit alternative that can be installed using limited access equipment. Limited access micropile rigs vary between contractors. The dimensions shown for the Klemm KR704E, which is provided on the last page of Appendix C, can be used for preliminary planning and review of suitable alternatives.	1b		
3	Angus Chen	Pg. 19 - 20: Sec. 7.6.2.1	Grouting methods – Section 7.6.2.1 introduces three grouting methods for the sites which are permeation grouting with ultrafine cement, compaction grouting, and Jet grouting. The compaction grouting is most recommended. Report indicates that the compaction grouting will impact/damage any underground utilities at grouting zone. We will try to perform USA survey once for the basement defected slab area. The permeation grouting is only applied for the sandy soil. According to the appendix C, the jet grouting equipment is big and may have access problem.	GTC	Comments acknowledged. Jet grouting equipment has also been adapted for limited access, so although the amount and size of equipment is larger than for compaction and permeation grouting, the equipment at the grout injection point can be of similar size.	1b		