COAST & HARBOR
ENGINEERING

DRAFT
Coastal Engineering Analysis
West Trail Shoreline Protection, Pillar Point Harbor, CA

1. Introduction

Coast & Harbor Engineering, Inc. (CHE) was contracted by GHD, Inc. (GHD) to perform coastal engineering analysis and develop design criteria for shoreline protection along the West Trail, Pillar Point Harbor, CA. Figure 1 shows the location of the portion of the West Trail characterized by existing damage (bluff and shoreline erosion). Figure 2 shows pictures of the eroded bluff.

Figure 1. Location of West Trail damage

Figure 2. Ground photos of eroded bluff (looking south)

2. Coastal Conditions Evaluation

CHE evaluated coastal conditions at the project site including bathymetry/morphology, tides, winds, wind-waves and offshore waves and developed design criteria for shoreline protection.

2.1. Bathymetry/Morphology

The project site is located inside the outer breakwaters at Pillar Point Harbor, CA. A hydrographic survey of the vicinity of the project site was performed in 2006 (Gahagan & Bryant, 2006). Modeling domains were constructed in areas outside the harbor using a NOAA Digital Elevation Model (NOAA, 2011). Bathymetry figures are included in Section 3. Bottom morphology in the areas surrounding the project site was not included in the analysis.

2.2. Tides

No on-site tide measurements or datum information were available for the analysis. The goal of tidal analysis here is only to provide estimates of tidal datums such as MHHW and MHW relative to MLLW for the purposes of the analysis. Therefore, a tidal datum table was developed for the site based on linear interpolation between tidal datum information from two sites on the open coast surrounding the project site. The National Oceanic and Atmospheric Administration (NOAA) datum information at Point Reyes (9415020) and Monterey (9413450) stations were used to develop appropriate tidal elevations to be used in the wind-wave and offshore wave numerical modeling efforts. Table 1 shows the tidal datum information available for the two stations, and an approximate tidal datum table developed based on linear interpolation taking into account distance from each site. Figure 3 shows the location of the two tidal stations with respect to the project site.

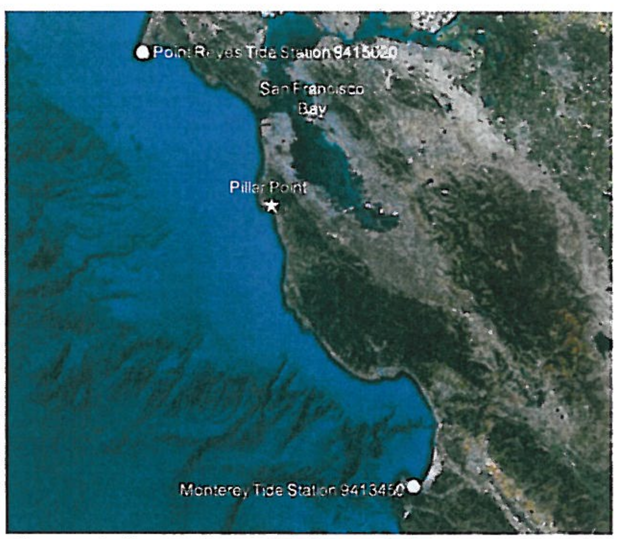


Figure 3. Location of tide stations

Table 1. Tidal Datum Elevations

Datum	Point Reyes Elevation [ft, MLLW]	Monterey Elevation [ft, MLLW]	Pillar Point Approximate Elevation [ft, MLLW]
Highest Observed Water Level	8.5	7.9	8.3
MHHW	5.8	5.3	5.6
MHW	5.1	4.6	4.9
MTL	3.1	2.9	3.0
MSL	3.1	2.8	3.0
MLW	1.2	1.1	1.1
MLLW	0.0	0.0	0.0
Lowest Observed Water Level	-2.7	-2.4	-2.6

2.3. Sea Level Rise

Sea level rise in San Francisco Bay based on measured trends was evaluated using the San Francisco NOAA tide station (9414290) for the 110-year period from 1897 through 2006. Monthly mean sea levels during the period 1897 to 2006 were evaluated and only the sea level data following the 1906 earthquake were used. The sea level rise trend at San Francisco is presently 2 mm/year relative to datums for the NOAA tidal epoch 1983-2001. Evidence of accelerated sea level rise due to effects of global climate change is not yet detectable in the measured tide record. The NOAA data are the only direct measurements of the local relative sea level trend. Based on the measured sea level rise of 2 mm/year, the sea level rise at the proposed project site over a 50-year period is estimated to be 0.33 ft (4 inches).

Other entities such as the International Panel on Climate Change (IPCC) have developed a wide range of estimates of long-term eustatic sea level rise. Upon request from GHD, CHE also evaluated the design storm wave conditions assuming an additional 4 feet of sea level rise had occurred by the time of the 100-year storm event.

2.4. Winds

Wind analysis was performed to develop extreme winds to be used for wind-wave growth and transformation numerical modeling. Wind analysis was conducted using wind data collected at the Half Moon Bay buoy (NDBC Buoy 46012) for the period 1981-2011. Half Moon Bay buoy wind data includes hourly wind speeds and directions that were later adjusted to 10m elevation and two-minute averages. These data were analyzed and the largest measured wind events were extracted. The storm events were fit to a Weibull distribution, and sustained wind speeds were predicted for extreme events with recurrence intervals ranging from 2 to 100 years for all directions. Figure 4 shows the predicted extreme wind speeds for all return periods

7+91 and 8+12 causes saturation of the materials underling the trail surface and rill erosion of the outer trail edge.

The concrete swale drains to a 24-inch diameter corrugated steel pipe (csp) culvert upslope of 6+30. The swale is in poor condition with many cracks and separations. Vegetation is also growing in the separations. The culvert discharges into an earth basin adjacent to the trail at 6+00. Also at 6+00 are a 12-inch csp culvert and a deteriorated 18-inch reinforced concrete culvert that cross under the trail and have outlets at the shoreline. The trench backfill in the bank at this location has been eroded and the culverts are exposed. There is also a void in the trench backfill that extends approximately 6 feet under the trail. The void area is currently taped off. It appears that the culvert inlets are buried beneath the basin bottom and were originally intended to receive the water discharged from the 24-inch csp culvert. It appears that at least some of the backfill erosion is caused by water seeping from the basin, through the trench and toward the shore.

4.0 GEOLOGIC AND SUBSURFACE CONDITIONS

The west trail and adjoining ridge are underlain at shallow depth by the Pliocene-age Purisima formation bedrock, typically consisting of highly fractured, variably soft to medium hard mudstone, siltstone, and sandstone. Bedrock outcrops and exposures were observed at several locations on the hillside upslope of the trail, in the steep bank adjacent to the trail outer edge, and along the shoreline. Bedrock was also exposed at several locations in cuts in the hillside made for the trail alignment. A limited mapping of bedrock exposures is included on Figure 1. Cross sections showing limited subsurface conditions are shown on Figures 2 through 6.

Our observations of the earth materials exposed in the bank adjacent to the shoreline indicate that between 5+25 and 6+85 and between 7+15 and 8+25 the outer edge of the trail is typically underlain by 2 to 3 feet, but it some areas as much as 5 feet, of stiff sandy and clayey fill containing gravel and large rock and concrete fragments. The fill is typically underlain by bedrock, but in some areas a thin veneer of stiff clayey native soil is exposed between the fill and the bedrock. Bedrock is expected to underlie the surface sand and gravel at shallow depth in shoreline areas near the base of the bank. Between 6+85 and 7+15 bedrock underlies the trail and extends approximately 30 feet east from the trail toward the water.

Our observations of the earth materials composing the upslope area between stations 8+50 and 10+00 indicate that the slope surface has 3 to 9 inches of loose material derived from the underlying bedrock. This surficial material is coarse silty sand composed of small bedrock particles. Highly fractured fissile mudstone bedrock underlies the loose veneer.

Our observations of the earth materials exposed in the steep bank between stations 9+90 and 10+60 indicate that the outer edge of the trail is underlain by at least 4 feet of gravelly fill at this location. Friable bedrock was exposed at the base of the upslope cut and is expected to underlie the outer edge of the trail at depths ranging between 6 and 8 feet.

The trail section is within a State of California Earthquake Fault Zone[4] designated for the San Gregorio-Seal Cove fault. Based on our preliminary review of the fault maps, the mapped fault locations do not cross the trail section under consideration. Because the project is a repair of a recreational trail and no structures for human occupancy are involved, the impact of fault proximity is no different than previously existed for the trail and is judged to be not applicable for this repair project.

Other geologic hazards potentially affecting the trail section under consideration include seismic shaking, tsunami inundation, landslides, and rockfalls. Because the project is a repair to an existing recreational trail, the impacts of these hazards to the project are no different than existed previously for the trail. Seismic shaking should be addressed in the design of the repair; however, tsunami inundation, landslides, and rockfall hazards are judged to be not applicable for this repair project.

5.0 REPAIR CONCEPTS

Repair alternatives are presented in the following sections and are not listed in order or ranking. In general, our discussion of advantages and disadvantages considered aesthetics, design limitations, constructability, relative cost, encroachment into tidal zone, and future maintenance.

The repair alternatives for shoreline and trail edge erosion generally involve repair limits and/or construction equipment being within the tidal zone. The impacts to the tidal zone vary among alternatives and are only mentioned in this report in very general terms. Further evaluation of the impacts is outside the scope of this study and should be done by others practicing in the appropriate disciplines.

It is anticipated that the bedrock materials that would be encountered during construction of the repair alternatives, as presented in the following sections, can be excavated using conventional excavation and drilling equipment intended for weak and weathered rock sites.

---

[4] California Division of Mines and Geology, 1976, State of California Special Studies Zones, Half Moon Bay Quadrangle, California, 7.5 minute series.

5.1 Shoreline Erosion 5+25 to 8+25

For all repair alternatives, it will need to be decided whether to excavate the bedrock outcrop between 6+85 and 7+15 or to leave it in place and have the repair terminate on each side of the outcrop. Advantages of leaving the bedrock outcrop in place include lower cost and access from trail to shore. Disadvantages include future erosion which would change the contact between the outcrop and the repair, and which would require maintenance.

Trail access by the public during construction hours will be difficult to maintain with any of the repair alternatives. There may be periods where public trail access during construction hours is possible, but the access would be through a construction site with stockpiled materials and equipment. The harbor district will need to consider if limited public use of the trial during construction hours is feasible from safety, legal, production, and cost perspectives.

An access alternative would be to create a "detour" trail that connects the upper portion of West Point Avenue with the existing trail at about 8+50. The "detour" trail could be incorporated into the permanent path included in the hillside erosion repair alternative discussed in section 5.2.3 of this report.

5.1.1 Riprap Revetment

A riprap revetment would consist of placing large rock boulders against the bank and at an angle to transition to the shoreline. We understand that a riprap slope of 1.5H:1V will be used, based on a preliminary analysis by GHD. A toe key or trench would be excavated to provide stability of the revetment. It is anticipated that the key will extend 3 feet below the existing shoreline and that bedrock will be exposed in the base of the majority of the key. We understand that the median size of the riprap is estimated by GHD to be 3.5 feet. A geotextile fabric would be placed against the bank prior to rock placement and smaller facing class rock would be used to fill the voids between the bank and the larger rocks. This repair alternative is shown on Figure 7, Riprap Revetment Alignment Concept 5+25 to 8+25, and on Figure 8, Riprap Revetment Concept Cross Sections 5+25 to 8+25.

Advantages

- Aesthetically neutral, similar to existing breakwater.
- Potentially aesthetically pleasing, if voids are filled with soil and planted with vegetation, providing it is allowed by permits.

- Alignment easily conforms to existing bank.
- Ends conform well to existing bank with the possible exception of 6+85 and 7+15 if the rock outcrop is left in place.
- Relatively quick construction.
- Relatively low cost.

Disadvantages

- Excavation of the bedrock outcrop at 6+85 to 7+15 may be required to provide a stable revetment (open ends are more susceptible to movement by wave action).
- Access for excavation and hauling equipment will be most likely from the shoreline because the trail will not support the repeated heavy loads of hauling equipment. Equipment access from the trail to the shore will require damaging existing vegetation and the small bank. Existing trail segment from parking area to shore will sustain more damage from this alternative than from other alternatives. Protective measures such as cribbing could be undertaken to reduce or prevent trail damage; however, subgrade repair and reconstruction of the trail surface may be more cost effective.
- Culvert at 6+00 will require extension to provide and outlet beyond the riprap.
- Pipeline at 7+80 to 8+00 will require protection prior to placing riprap over it.
- Riprap will extend horizontally as much as 15 to 18 feet east of the top of bank (for the highest bank sections), will require excavation and rock placement within the tidal zone, and may be subject to permit restrictions.
- Riprap is subject to movement under storm wave impact and future maintenance of riprap and backfill may be required.
- High level of disturbance to adjacent areas.

5.1.2 Retaining Walls

Retaining walls may be constructed either at the current edge of the bank or at a distance from the current edge of the bank to gain additional clearance for the trail depending on the desired trail width, avoidance of conflict with the pipeline, and permit requirements. Engineered backfill would be placed to fill the space between the wall and the bank. In all alternatives the wall would have:

- a toe that extends 2 to 3 feet below the existing base of the bank and bears on bedrock, except sheet pile wall.
- varying amounts of concrete poured near the marine environment.
- construction equipment that could work from the existing trail (with varying difficulty), except the sheet pile wall.

- the trail open to emergency vehicles during construction, assuming a minor delay to move construction equipment.
- materials satisfying requirements for marine environments.

The retaining wall alternatives are shown on Figure 9, Retaining Wall Alignment Concept 5+25 to 8+25 and Figure 10, Retaining Wall Concept Cross Sections 5+25 to 8+25.

5.1.2.1 Soldier Pile and Lagging - A soldier pile and lagging wall would consist of either timber or precast reinforced concrete lagging placed between steel soldier piles. Concrete lagging can be colored to blend with the adjacent environment. The use of steel piles would need to consider corrosion over the design life of the project.

The lagging would need to be secured from movement at the top of the pile because of wave impact. Untreated timber lagging will have a short life. Treated timber lagging (pressure treated with chemical preservatives) will have a longer life, though its use may be restricted in water and recreational environments. Precast, reinforced-concrete lagging will have the longest life. All lagging will also need to be designed for abrasion and minor impact.

Advantages

- Designed and installed with either straight or non-linear alignment.
- Tolerates small amounts of movement.
- Discrete sections can be repaired with less difficulty than other alternatives.
- Aesthetically pleasing marine appearance.
- Low to moderate cost.

Disadvantages

- Wall ends require additional slot excavation into bank and concrete backfill to provide erosion resistant contact (conform) between wall ends and bank (particularly at 8+25, and possibly at 6+85 and 7+15).
- Corrosion to the steel piles (aesthetic only, if designed for sacrificial thickness for corrosion).
- Future maintenance to backfill area after severe storms (some loss of backfill should be expected from water pressure by wave impact transferred through gaps in lagging).

5.1.2.2 Cast-in-Place Reinforced Concrete Pile-Supported Wall – This alternative would consist of a conventional cast-in-place reinforced-concrete wall supported by drilled cast-in-place, reinforced-concrete pier foundations.

Advantages:

- Designed and installed with any alignment, curved or linear.
- Wall ends conform well with existing bank.
- Less future backfill maintenance after storm wave impact.
- Low to moderate cost. Depending on design, potentially the least costly wall alternative.

Disadvantages:

- Aesthetically less pleasing unless texture, design, or color added at additional cost.
- More involved construction effort with regard to formwork and timing of tides.
- Not easily repaired.

5.1.2.3 Shotcrete and Soil Nail Wall - A reinforced-shotcrete soil nail wall would consist of installing one to two rows of soil nails[5] in the bank, placing steel reinforcing mesh and bars against the bank, supporting the mesh by attaching it to soil nails, and pumping shotcrete (low-slump concrete) under high pressure through hoses and a nozzle, and directing it against the bank. The shotcrete can be colored at the batch plant and can be textured or sculpted after placement and before setting. Nearby sculpted wall projects include Highway 92 Pillarcitos Canyon, Half Moon Bay and Highway 1 - Devils Slide Tunnel project, Pacifica and Montara. Similar shoreline projects include Pleasure Point seawalls-Santa Cruz.

Advantages

- Aesthetically pleasing, wide variety of options for wall surfaces including smooth, textured, or sculpted patterns, such as mimicking on-site bedrock.
- Alignment conforms to existing bank easily.
- Can be designed and constructed to span over the existing pipeline, if desired.
- Wall ends conform well with existing bank.
- Relatively small equipment can be used and can operate from existing trail.
- Very quick construction time.
- No backfill required.
- Least disruptive to shoreline and adjacent areas.
- Less future backfill maintenance after storm wave impact.
- Repaired with less difficulty than other alternatives.

[5] A soil nail is a solid, continuous steel reinforcing bar installed in a pre-drilled hole in soil or rock, often drilled at a downward angle of about 15° below horizontal. The soil nail is grouted into the hole and functions as a ground anchor. The soil nails are usually attached to a facing such as a reinforced-concrete facing in the case of retaining walls, or high strength steel mesh in the case of soil or rock slope stabilization.

- Low to moderate cost depending on surface sculpting, texturing, and coloring. Low cost if minimal texture and color.

Disadvantages
- Wall is slightly more costly if constructed away from the bank (using formwork) to avoid the existing pipeline (7+80 to 8+00). Some backfill would also be required.
- Shotcrete placement in a marine environment could be subject to permit restrictions.

5.1.2.4 Sheet Pile Wall - A steel sheet pile wall is only marginally feasible and depends entirely on the soundness of the bedrock at depth. The wall would be installed by driving interlocking sheet piles. Pre-drilling at piling locations would be required to penetrate the bedrock. If the bedrock materials remain weak and intensely fractured within the 10 to 15 foot depth below the shore surface, then it is possible that the pile could achieve enough penetration.

Advantages
- High damage resistance.
- Long design life.
- Less future backfill maintenance after storm wave impact.
- Less concrete poured near the marine environment.

Disadvantages
- May not fit in aesthetically.
- High level of uncertainty regarding the ability to drive piles to design depth.
- Large pile driving equipment required to be within tidal zone, smaller equipment would be ineffective from existing trail.
- Wall ends require additional slot excavation into bank and concrete backfill to provide erosion resistant contact (conform) between wall ends and bank (particularly at 8+25, and possibly at 6+85 and 7+15).
- Very high cost.

5.2 Hillside Erosion 8+50 to 10+00

Based on observations during several visits, we conclude the primary cause of the hillside erosion is from foot traffic and from use as a play area. The lower slope area is virtually bare with 3 to 9 inch thick loose cover material. The mid-slope area has sparse to moderate vegetation and has more established and stable soil cover.

The following three repair alternatives involve re-vegetation measures and are presented in increasing effectiveness and cost.

5.2.1 Temporary Fencing and Hydoseeding

This alternative would include fencing off the bare lower slope area on all accessible sides with metal fence posts driven into the weak bedrock surface and mesh fencing material attached to the posts. The area within the fence would be hydroseeded with a native seed mixture that is erosion and damage resistant. It would also include the wild strawberry and bunch grass ground cover which are growing prolifically upslope. The mid-slope area would also be hydroseeded. The fence would remain for one to two years.

Advantages
- Area will have new vegetation and be aesthetically improved.
- Lower cost alternative.

Disadvantages
- Some erosion by runoff will occur before vegetation is established.
- Some loss of seed will occur by birds and wind.
- Full coverage by vegetation may not be achieved.
- Area subject to future damage from foot traffic.

5.2.2 Temporary Fencing, Hydoseeding, and Erosion Control Mesh

This alternative would include all items in the above alterative and would include applying a layer of mulch after the hydoseeding was completed, installing a long-term erosion control mesh stapled to the slope, and placing fiber rolls on the mesh at intervals secured by staking.

Advantages
- Area will have new vegetation and be aesthetically improved.
- Higher probability for full coverage of vegetation.
- Lower to moderate cost alternative.

Disadvantages
- Area subject to future damage from foot traffic.

5.2.3 Temporary and Permanent Fencing, Hydoseeding, Erosion Control Mesh, and New Path

This alternative would include all of the elements of the previous alternative and would include a permanent wood fence (split rail or similar) along the bottom and top of the slope with appropriate "keep off" signage, and a new established path from the bottom of the slope to West Point Avenue at the top of the ridge.

Advantages
- Area will have new vegetation and be aesthetically improved.
- Higher probability for full coverage of vegetation.
- Longer-term erosion protection of the slope.
- Reduced potential for future damage from foot traffic.

Disadvantages
- Higher cost alternative.
- Some future damage from foot traffic still possible. Additional fencing on the accessible perimeter would be required to further reduce the potential for future damage.

5.3 Trail Edge Erosion 9+90 to 10+60

Erosion of the outer or down slope trail edge is occurring in this trail section. The erosion appears to be the result of wave action, although other factors may be contributing. This area was not within the initial focus of this study; however, the trail width in this area is 11 feet with a 3-foot high steep bank (0.66H:1V) at the trail edge. This trail section is at risk of losing vehicle access.

5.3.1 Hillside Cut

This alternative would consist of widening the trail by cutting into the hillside 2 to 3 feet. The cut would likely be expose severely weathered rock in the bottom portion and 2 to 3-foot thick soil veneer in the upper portion. The 5 to 6-foot high cut would be at an inclination of 0.75H:1V.

Advantages
- Aesthetically neutral.
- Improved trail width.
- Work is outside the tidal zone.
- Very low initial cost.

Disadvantages

- Periodic maintenance required to clear the trail of sloughed material from the cut slope.
- Does not mitigate bank erosion, only postpones mitigation.

5.3.2 Buried Riprap Buttress

A riprap buttress would consist of placing large rock boulders against the bank at a 1.5H:1V inclination. A toe key or trench would be excavated to provide stability of the buttress. It is anticipated that the key will extend 3 feet below the existing shoreline. We estimate that the size of the riprap could be smaller than the 3.5 feet median size to be used in the other sections and could be similar to the existing riprap used previously on a trail section further south. A geotextile fabric would be placed against the bank and smaller facing class rock, large gravel and sand would be used to fill the voids between the larger rocks. The buttress would be covered by 1 to 2 feet of sand or native soil and planted with native vegetation. This alternative is shown on Figure 11, Riprap Buttress Alignment and Section Concept 9+90 to 10+60.

Advantages

- Aesthetically pleasing.
- Mitigates bank erosion and stabilizes trail width.

Disadvantages

- Work is close to or within tidal zone.
- Materials will need to be hauled to and from the site on the existing trail in small quantities using smaller equipment to avoid trail damage.
- Higher cost.

5.4 Drainage Improvements

The following drainage improvements may be considered:

- Excavate sediment and clear culvert inlet(s) at 6+00 to restore flow. Inlet improvements may also include an inlet structure or small headwalls/wingwalls.
- If the filling of the inlet was intentional in order to reroute flow, then the culverts should be properly abandoned by removing pipes and plugging the trench to stop water migration.
- Repair backfill erosion and void at 6+00.
- Create positive surface drainage gradients on trail between 5+25 and 8+25.

- Repair concrete swale by clearing vegetation and loose concrete, and relining in place.

6.0 LIMITATIONS

Our work was performed according to generally-accepted geotechnical engineering practice in the San Francisco Bay Area for similar engineering studies at this time. No other warranty, either expressed or implied, is made.

The conclusions and recommendations to be presented in this preliminary study are based on our interpretation of limited site and subsurface data acquired by Fisher Geotechnical and existing site data provided by others. Actual site subsurface conditions could vary from those assumed in this study and additional evaluations not included in the scope of this study could be required. If you have any questions regarding this report, please contact us. Thank you for the opportunity to provide services for this project.

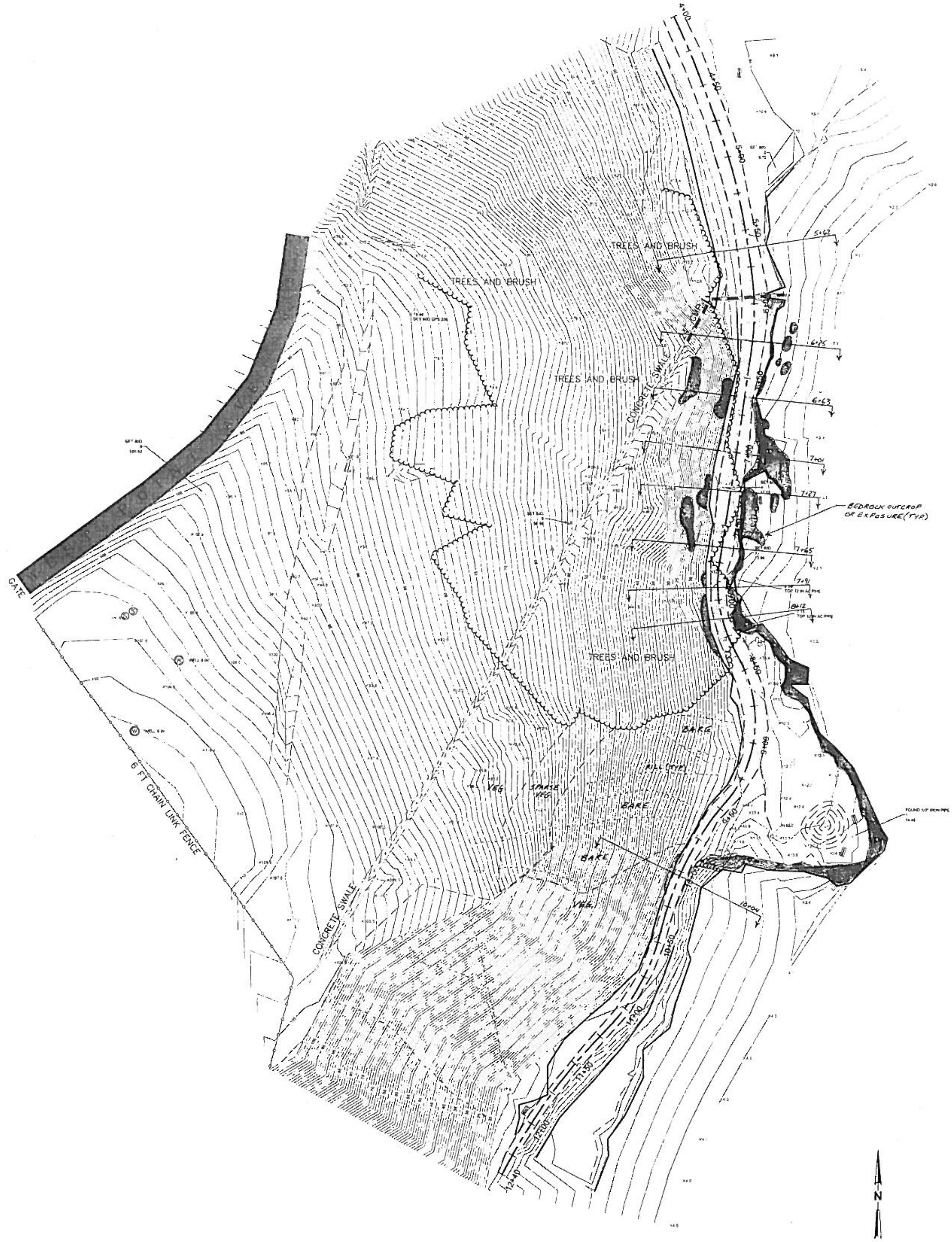
Sincerely,

Raymond L. Fisher

Raymond L. Fisher, P.E., G.E.
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
RAYMOND L. FISHER
3-31-14
GEOTECHNICAL
STATE OF CALIFORNIA

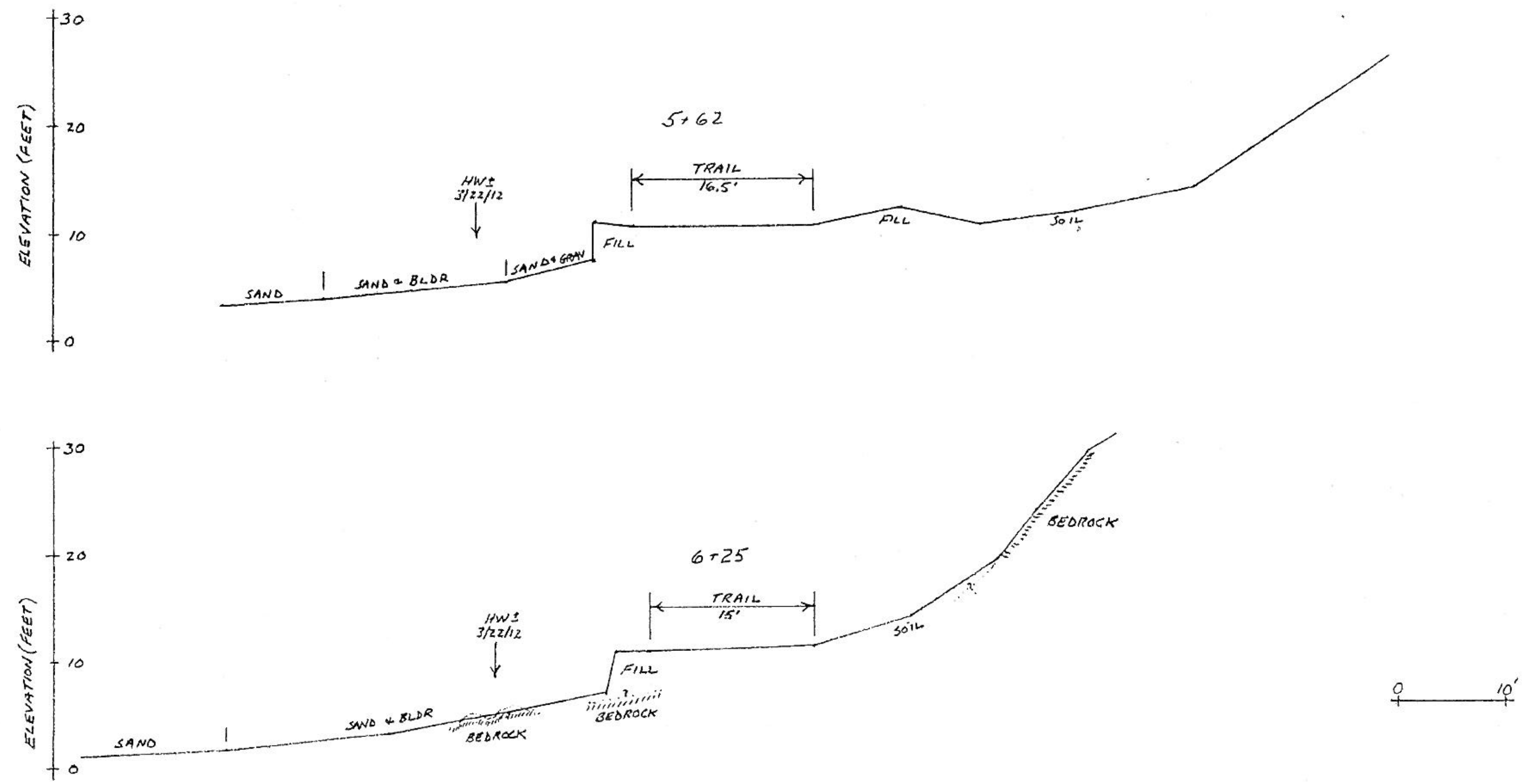
Copies: Addressee (1)
Attachments: Figure 1, Site Plan
Figure 2, Cross Sections at 5+62 and 6+25
Figure 3, Cross Sections at 6+63 and 7+01
Figure 4, Cross Sections at 7+27 and 7+65
Figure 5, Cross Sections at 7+91 and 8+12
Figure 6, Cross Section at 10+04
Figure 7, Riprap Revetment Alignment Concept 5+25 to 8+25
Figure 8, Riprap Revetment Concept Cross Sections 5+25 to 8+25
Figure 9, Retaining Wall Alignment Concept 5+25 to 8+25
Figure 10, Retaining Wall Concept Cross Sections at 5+25 to 8+25
Figure 11, Riprap Buttress Alignment and Section Concept 9+90 to 10+60



SITE PLAN

Fisher Geotechnical Civil and Geotechnical Engineering	PILLAR POINT WEST TRAIL EROSION REPAIR *PRINCETON, SAN MATEO CO., CALIFORNIA*

PROJECT NO.	DATE	FIGURE
3330	Mar 2012	1



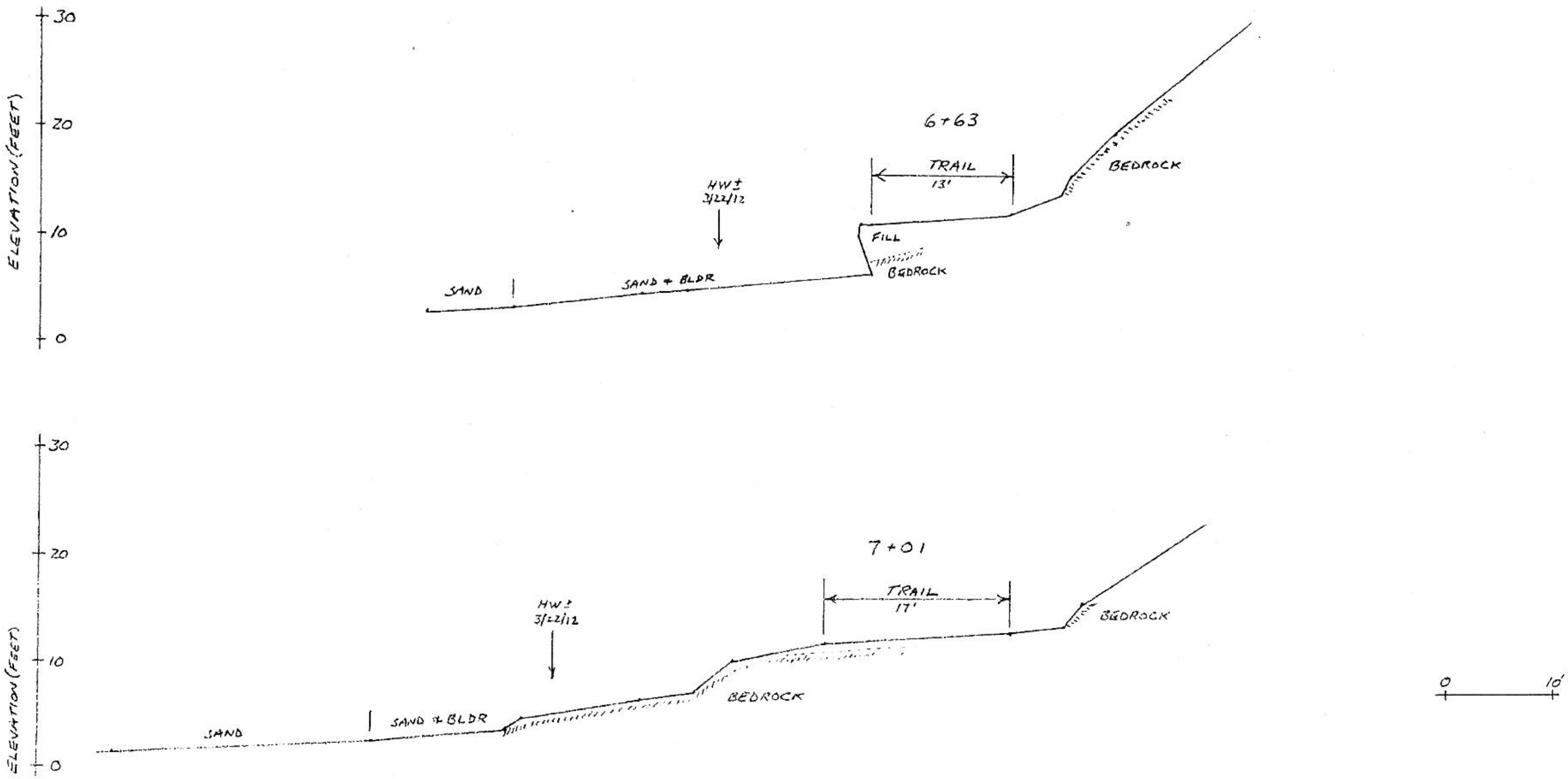
CROSS SECTIONS 5+62 AND 6+25

NOTES:
1. CROSS SECTIONS ARE BASED ON MEASUREMENTS WITH HAND LEVEL AND TAPE AND SHOULD BE CONSIDERED APPROXIMATE.
2. SURFACE AND SUBSURFACE CONDITIONS SHOWN ARE INFERRED FROM LIMITED SITE OBSERVATIONS AND SHOULD BE CONSIDERED APPROXIMATE.

Fisher Geotechnical
Civil and Geotechnical Engineering

PILLAR POINT WEST TRAIL EROSION REPAIR
PRINCETON, SAN MATEO CO., CALIFORNIA

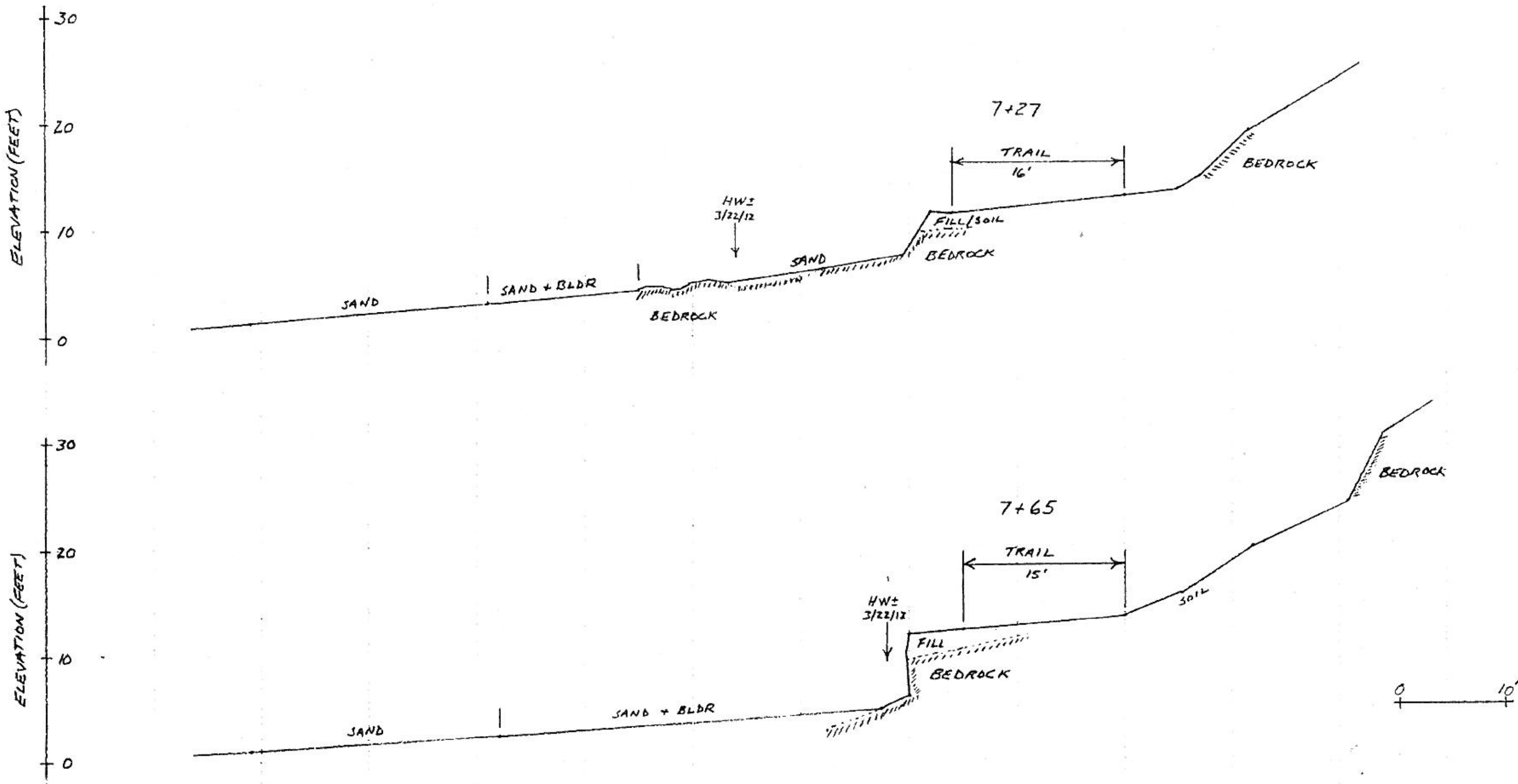
PROJECT NO.	DATE	FIGURE
3330	Mar 2012	2



CROSS SECTIONS 6+63 AND 7+01

NOTES:
1. CROSS SECTIONS ARE BASED ON MEASUREMENTS WITH HAND LEVEL AND TAPE AND SHOULD BE CONSIDERED APPROXIMATE.
2. SURFACE AND SUBSURFACE CONDITIONS SHOWN ARE INFERRED FROM LIMITED SITE OBSERVATIONS AND SHOULD BE CONSIDERED APPROXIMATE.

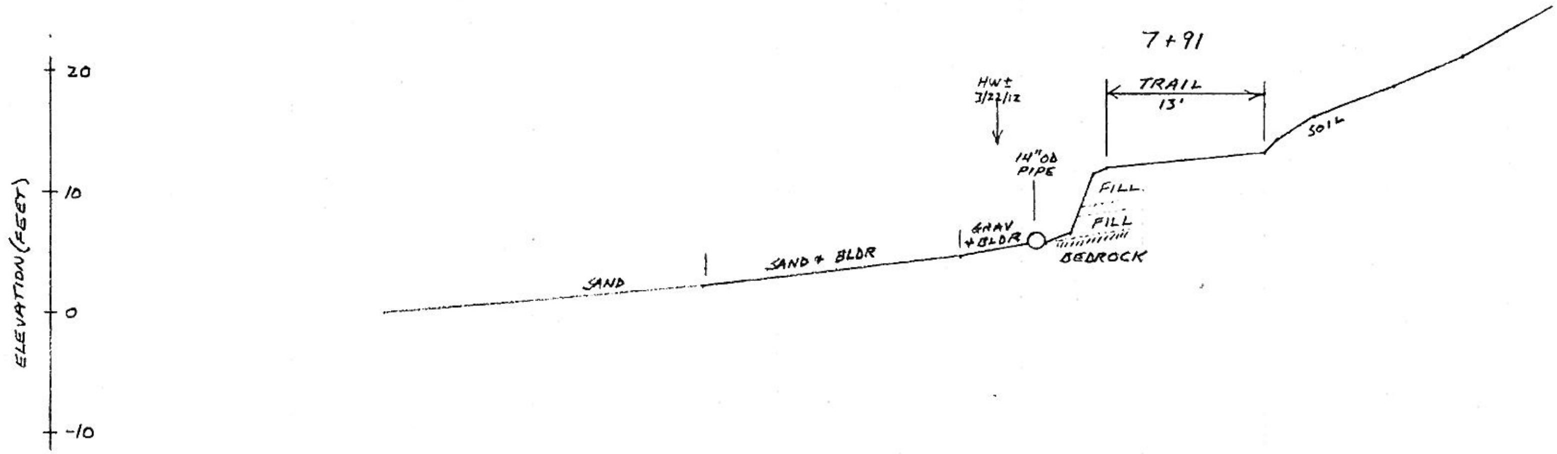
Fisher Geotechnical Civil and Geotechnical Engineering	PILLAR POINT WEST TRAIL EROSION REPAIR PRINCETON, SAN MATEO CO., CALIFORNIA		
	PROJECT NO. 3330	DATE Mar 2012	FIGURE 3

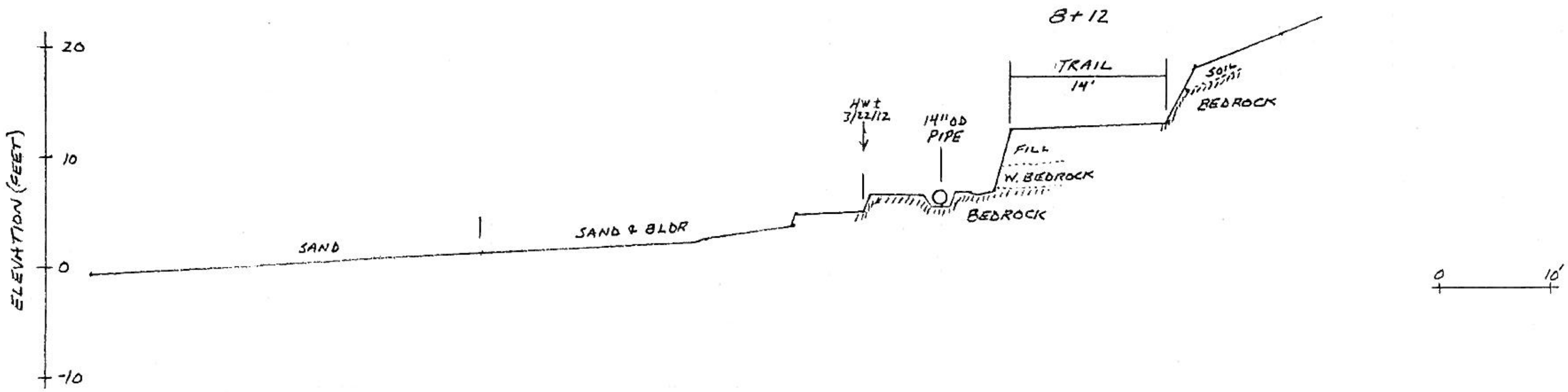


CROSS SECTIONS 7+27 AND 7+65

NOTES:
1. CROSS SECTIONS ARE BASED ON MEASUREMENTS WITH HAND LEVEL AND TAPE AND SHOULD BE CONSIDERED APPROXIMATE.
2. SURFACE AND SUBSURFACE CONDITIONS SHOWN ARE INFERRED FROM LIMITED SITE OBSERVATIONS AND SHOULD BE CONSIDERED APPROXIMATE.

Fisher Geotechnical Civil and Geotechnical Engineering	PILLAR POINT WEST TRAIL EROSION REPAIR PRINCETON, SAN MATEO CO., CALIFORNIA		
	PROJECT NO. 3330	DATE Mar 2012	FIGURE 4



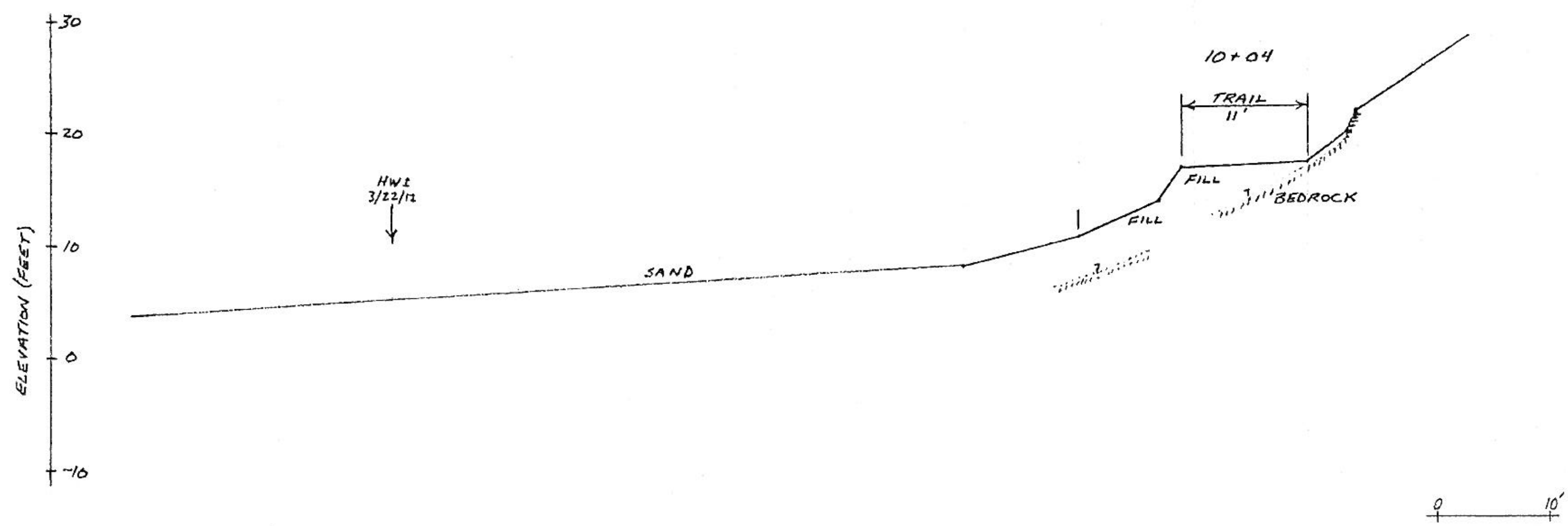


CROSS SECTIONS 7+91 AND 8+12

NOTES:

1. CROSS SECTIONS ARE BASED ON MEASUREMENTS WITH HAND LEVEL AND TAPE AND SHOULD BE CONSIDERED APPROXIMATE.
2. SURFACE AND SUBSURFACE CONDITIONS SHOWN ARE INFERRED FROM LIMITED SITE OBSERVATIONS AND SHOULD BE CONSIDERED APPROXIMATE.

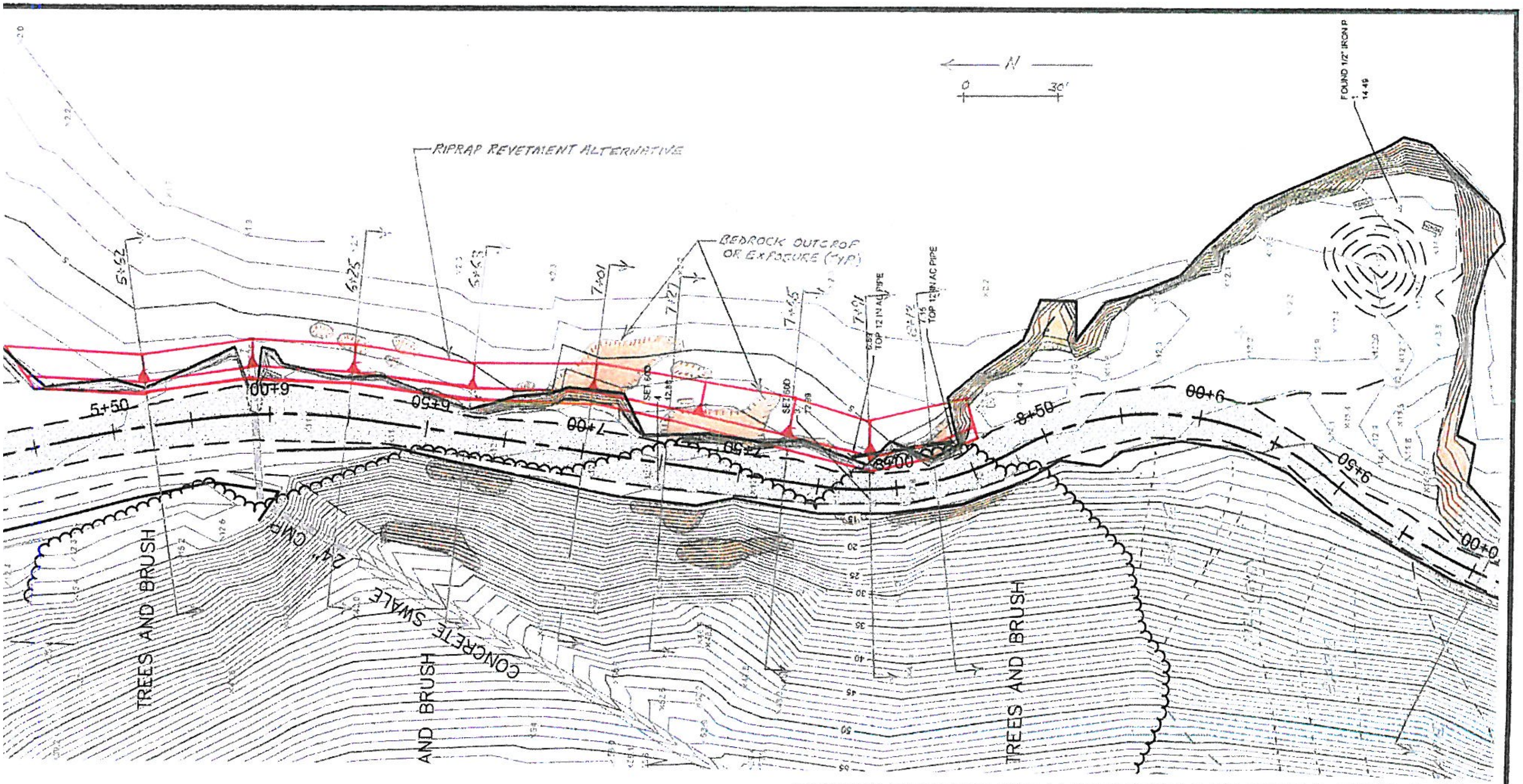
Fisher Geotechnical Civil and Geotechnical Engineering	PILLAR POINT WEST TRAIL EROSION REPAIR PRINCETON, SAN MATEO CO., CALIFORNIA		
	PROJECT NO. 3330	DATE Mar 2012	FIGURE 5



CROSS SECTION 10+04

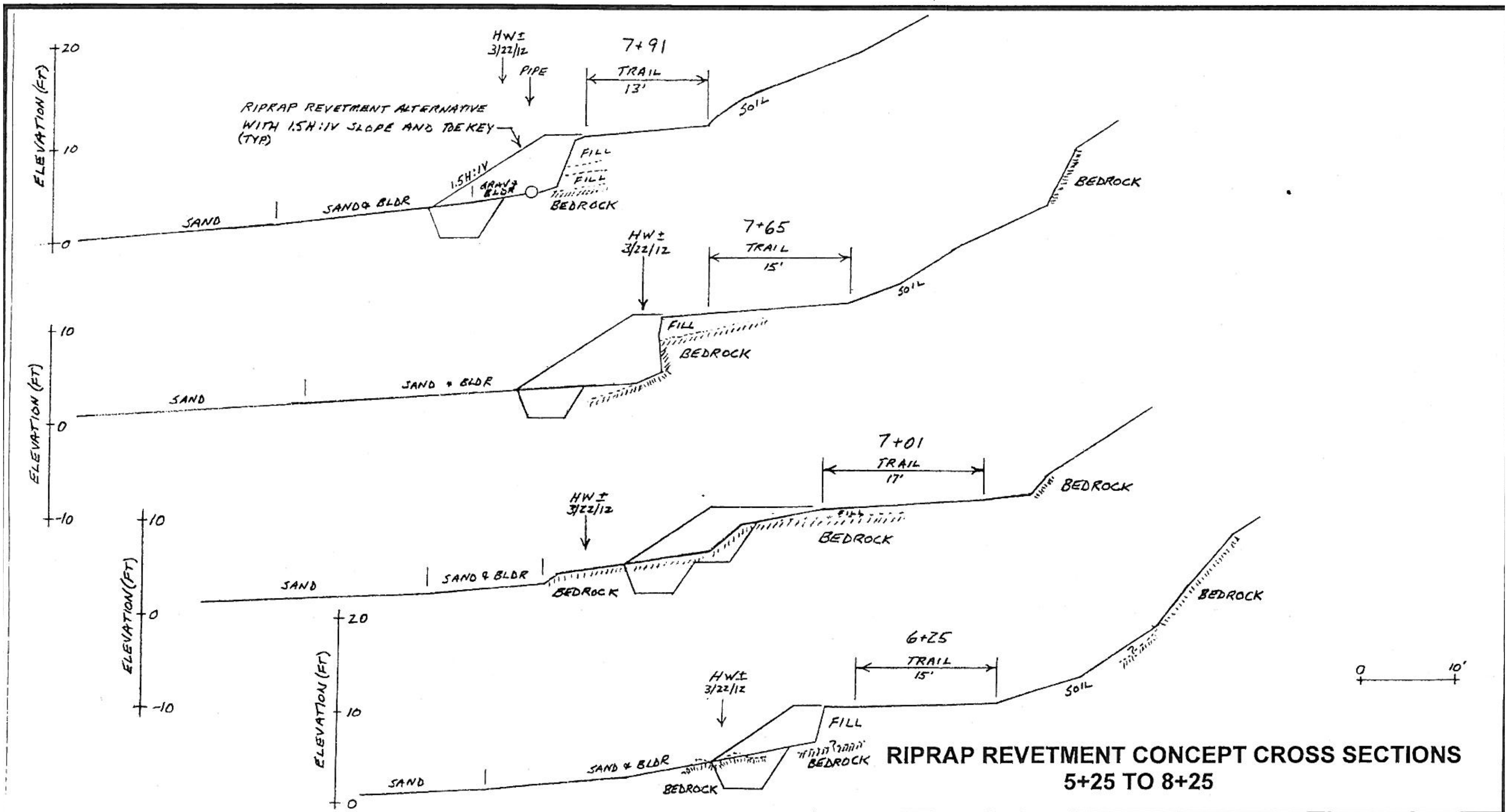
NOTES:
1. CROSS SECTIONS ARE BASED ON MEASUREMENTS WITH HAND LEVEL AND TAPE AND SHOULD BE CONSIDERED APPROXIMATE.
2. SURFACE AND SUBSURFACE CONDITIONS SHOWN ARE INFERRED FROM LIMITED SITE OBSERVATIONS AND SHOULD BE CONSIDERED APPROXIMATE.

Fisher Geotechnical Civil and Geotechnical Engineering	PILLAR POINT WEST TRAIL EROSION REPAIR PRINCETON, SAN MATEO CO., CALIFORNIA		
	PROJECT NO. 3330	DATE Mar 2012	FIGURE 6



RIPRAP REVETMENT ALIGNMENT CONCEPT 5+25 TO 8+25

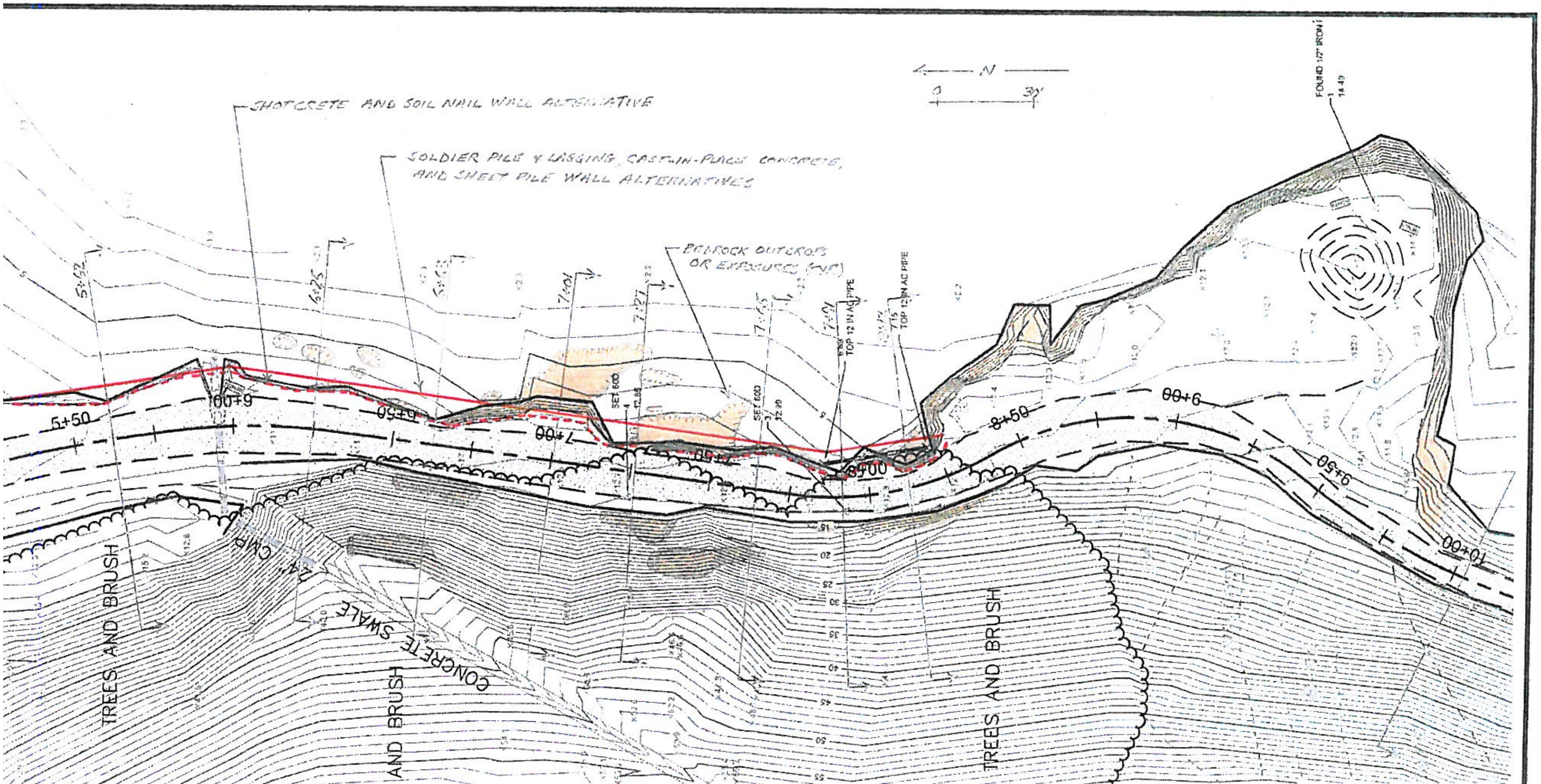
Fisher Geotechnical Civil and Geotechnical Engineering	PILLAR POINT WEST TRAIL EROSION REPAIR PRINCETON, SAN MATEO CO., CALIFORNIA		
	PROJECT NO. 3330	DATE Mar 2012	FIGURE 7



RIPRAP REVETMENT CONCEPT CROSS SECTIONS
5+25 TO 8+25

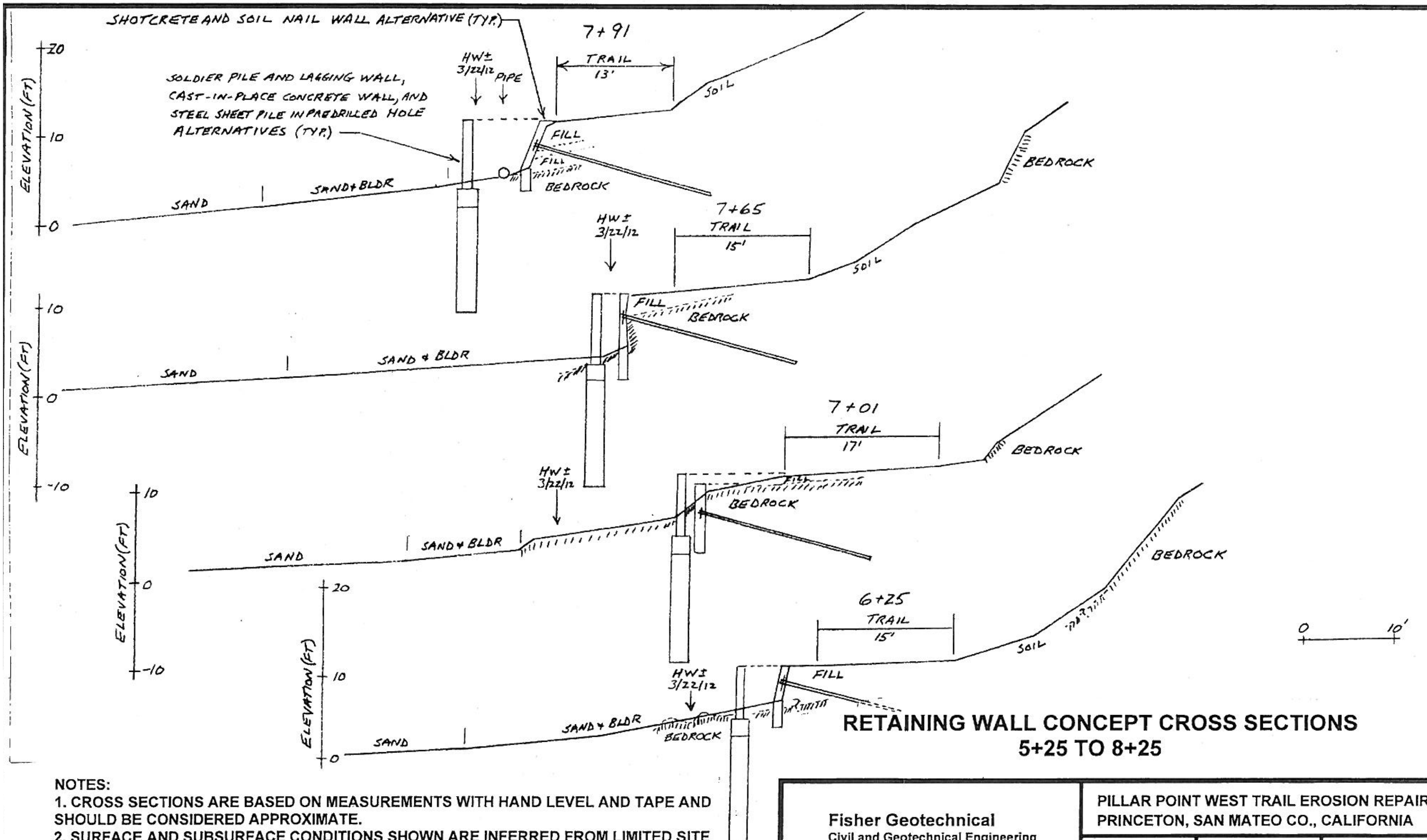
NOTES:
1. CROSS SECTIONS ARE BASED ON MEASUREMENTS WITH HAND LEVEL AND TAPE AND SHOULD BE CONSIDERED APPROXIMATE.
2. SURFACE AND SUBSURFACE CONDITIONS SHOWN ARE INFERRED FROM LIMITED SITE OBSERVATIONS AND SHOULD BE CONSIDERED APPROXIMATE.

Fisher Geotechnical Civil and Geotechnical Engineering	PILLAR POINT WEST TRAIL EROSION REPAIR PRINCETON, SAN MATEO CO., CALIFORNIA		
	PROJECT NO. 3330	DATE Mar 2012	FIGURE 8



RETAINING WALL ALIGNMENT CONCEPT 5+25 TO 8+25

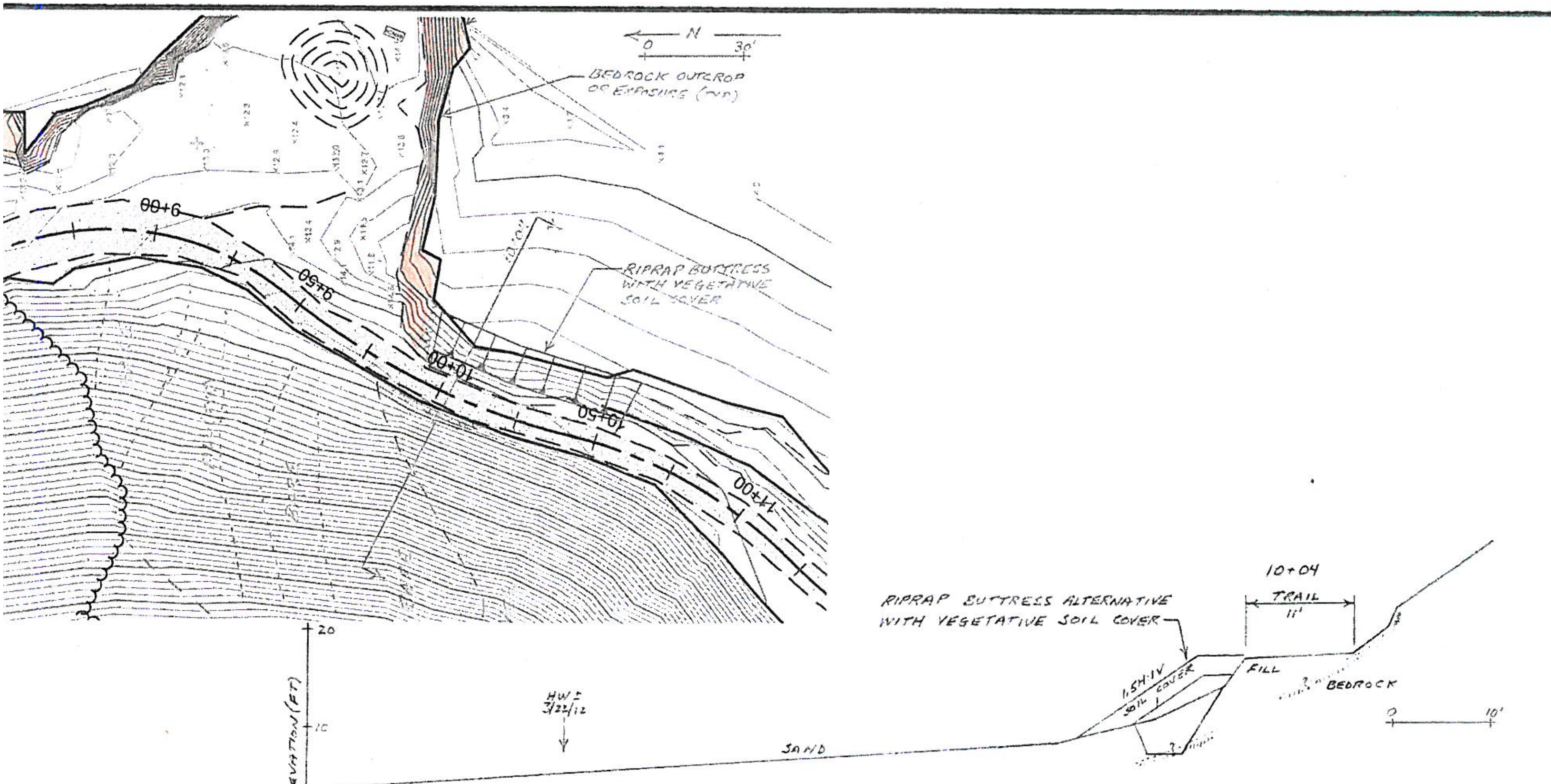
Fisher Geotechnical Civil and Geotechnical Engineering	PILLAR POINT WEST TRAIL EROSION REPAIR PRINCETON, SAN MATEO CO., CALIFORNIA		
	PROJECT NO. 3330	DATE Mar 2012	FIGURE 9



RETAINING WALL CONCEPT CROSS SECTIONS
5+25 TO 8+25

NOTES:
1. CROSS SECTIONS ARE BASED ON MEASUREMENTS WITH HAND LEVEL AND TAPE AND SHOULD BE CONSIDERED APPROXIMATE.
2. SURFACE AND SUBSURFACE CONDITIONS SHOWN ARE INFERRED FROM LIMITED SITE OBSERVATIONS AND SHOULD BE CONSIDERED APPROXIMATE.

Fisher Geotechnical Civil and Geotechnical Engineering	PILLAR POINT WEST TRAIL EROSION REPAIR PRINCETON, SAN MATEO CO., CALIFORNIA		
	PROJECT NO. 3330	DATE Mar 2012	FIGURE 10



RIPRAP BUTTRESS ALIGNMENT AND SECTION CONCEPT
9+90 TO 10+60

NOTES:
1. CROSS SECTIONS ARE BASED ON MEASUREMENTS WITH HAND LEVEL AND TAPE AND SHOULD BE CONSIDERED APPROXIMATE.
2. SURFACE AND SUBSURFACE CONDITIONS SHOWN ARE INFERRED FROM LIMITED SITE OBSERVATIONS AND SHOULD BE CONSIDERED APPROXIMATE.

Fisher Geotechnical Civil and Geotechnical Engineering	PILLAR POINT WEST TRAIL EROSION REPAIR PRINCETON, SAN MATEO CO., CALIFORNIA		
	PROJECT NO. 3330	DATE Mar 2012	FIGURE 11