



Design Report and Site Life Optimization

March 2022

Prepared for

**Avalon Environmental Services
a CR&R Incorporated Company**

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Prepared by

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Use of this Report and Limitations

This report has been prepared for the exclusive use of CR&R Inc. for specific application to the construction of the proposed re-designed final elevation of the Pebble Beach Landfill (PBL). The purpose of this re-design is to optimize the site life of the landfill through an increase in the height of the top deck via a “piggyback” option. The engineering design and evaluations for the re-design of PBL top deck were completed in general accordance with currently accepted standard engineering principles and practice in California at the time the report was prepared. No other warranty, expressed or implied, is made. These services were performed consistent with our agreement with our client. This report is solely for the use and information of our client unless otherwise noted. Any reliance on this report by a third party is at such party’s sole risk.

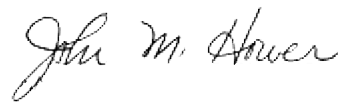
Opinions and recommendations contained in this report apply to conditions existing when services were performed and are intended only for the client, purposes, locations, time frames, and project parameters indicated. We are not responsible for the impacts of any changes in environmental standards, practices, or regulations subsequent to performance of services. By referencing studies, designs, or reports of other firms, Geo-Logic Associates (GLA) is not certifying, warranting, or guaranteeing their accuracy or completeness. We do not warrant the accuracy of information supplied by others, or the use of segregated portions of this report.

The findings of this report conclude that the design of the proposed site life optimization study complies with the applicable requirements of Federal Subtitle D regulations, Title 27 of the California Code of Regulations (CCR), and the general intent of the facility’s current Waste Discharge Requirements (WDR) Order No. R4-2016-0140. This design report was prepared under the supervision of the undersigned California Registered Civil Engineer and Certified Engineering Geologist.

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Date signed: March 14, 2022

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1. Introduction

1.1 Description

This Design Report, prepared by Geo-Logic Associates (GLA) is for the Pebble Beach Disposal Site, also known as Pebble Beach Landfill (PBL). The capacity of PBL is diminishing and the remaining site life is very limited. The purpose of this design report is to optimize site life within the existing landfill boundary (approximate limit of waste) and to present modifications to the current design that will be presented to regulatory agencies for approval.

PBL is owned by the City of Avalon and operated by CR&R, Inc. (CR&R) doing business as Avalon Environmental Services (AES). The site was formerly operated by Seagull Sanitation Systems, a subsidiary of Consolidated Disposal Service, L.L.C. from October 1991 to July 1, 2013.

PBL is located in an unincorporated area of Los Angeles County, on the southeast edge of Santa Catalina Island (Catalina), approximately 2.3 miles south of the City of Avalon. Catalina is located approximately 26 miles off the coast of southern California, south westerly of the City of Long Beach. Because the site is located on an island offshore of the continent, United States Geological Survey (USGS) does not provide a Section, Township, Range, Baseline, or Meridian location reference. The location of the disposal site is 36°19'18" N Latitude and 118°19'4" W Longitude. Access to the site is from the north, via Pebble Beach Road, which traverses the eastside of the disposal site. The airport on the Island is located approximately 8 miles to the north of the landfill and the flight pattern is not near the landfill. There is also a heliport terminal located approximately one-half mile north of the landfill.

PBL and the associated Materials Recovery Facility (MRF) are permitted to accept mixed municipal solid wastes as defined and regulated by California Department of Resources Recycling and Recovery (CalRecycle) and the California Regional Water Quality Control Board – Los Angeles Region (LARWQCB). Wastes accepted at the facilities include: residential and commercial refuse (primarily from the City of Avalon); green materials; recyclables; inert fill; residue and grit from sewer cleaning and treatment processes; and dewatered sewage sludge generated by the City of Avalon's wastewater treatment plant. No high liquid content wastes, untreated medical wastes, designated wastes, hazardous wastes or other wastes requiring special handling are accepted at the site. Construction/ demolition debris and salvageable materials are placed in roll-off bins, and recyclable materials recovered in the MRF are baled for recycling and are not landfilled. All material not suitable for recycling or diversion is baled in the MRF and then landfilled.

1.2 Site History

PBL site was originally a rock quarry that was used to provide riprap for coastal marine projects in the southern California area and elsewhere. The excavation into the existing steep natural terrain was conducted as a part of mining activities that were underway at the site before 1926. The site was graded level prior to the beginning of the landfill operations, sometime in the 1950's.

The most current waste composition data for the City of Avalon was obtained by field sampling conducted by Seagull Sanitation Systems during April, May and June of 1995. The field samples were obtained from both commercial waste and residential waste collected for disposal from the City of Avalon. The results were compiled and analyzed to determine the quantities of various recyclable materials in the City's waste stream. The outcome of that study was used to design the current waste collection and processing activities and infrastructure.

The 1998 Conditional Use Permit 96-162 (4) issued by the Los Angeles County Regional Planning Board granted a vertical expansion of PBL from the previous limit of 230-ft. MSL to 260-ft. MSL. Based on historic topographical information from the December 2, 2019 aerial survey, the proposed preliminary final grading plan (ES Engineering, Inc, May, 2015), and calculations presented in Appendix D of the RDSI, the total remaining gross site capacity, as of the December 2, 2019 flyover, was approximately 75,461 cubic yards resulting in a closure date of 2nd quarter 2028.

2. Project Description

PBL is an approximately 7.6-acre site with a waste disposal footprint of approximately 5.6 acres within the existing limit of the property boundary. Currently, the site has less than seven years of airspace remaining. The slopes of the existing waste fill are typically 2:1 (H:V), with local variation. GLA proposes to extend the life of the site primarily through the construction of an 18-foot-high retaining wall placed at the toe of slope (eastern most boundary of the limit of waste). Waste will be filled against the retaining wall and project back to the top deck at an average grade of 2:1 H:V, allowing sufficient space to construct a final cover system that will tie into a concrete v-ditch installed along the top of the wall to intercept and divert surface water to the original discharge locations as shown in Section 4 of this report. Seismic hazard and slope stability analyses were conducted and are summarized in Section 3 of this report.

2.1 Regulatory Requirements

The primary regulatory oversight is by CalRecycle (formerly known as the California Integrated Waste Management Board or CIWMB) as delegated to the County of Los Angeles Department of Public Health (LACDPH) Local Enforcement Agency (LEA). The facility is identified by CalRecycle as SWIS (Solid Waste Information System) number 19-AA-0061. Some of the other regulatory agencies providing oversight of PBL are the LARWQCB and the California Coastal Commission (CCC).

2.2 Permitted Closure System

The closure system of PBL shall comply with California Code of Regulations (CCR) Title 27 Section 21090. This includes a 2-foot-thick foundation layer, a low-conductivity layer and an erosion resistant layer. However, regulations contained in §21140 of CCR Title 27 provide general requirements for landfill covers and present guidelines for regulators to consider engineered alternatives. In addition, regulations contained in §20080(b) and (c) of CCR Title 27 provide the criteria for consideration of engineered alternatives by the LARWQCB. While the current Preliminary Closure Post Closure Maintenance Plan (PCPCMP), prepared by ES Engineering, Inc. (May, 2015), proposed a 3-foot thick evapotranspirative (ET) cover, GLA also considered utilizing a geosynthetic turf cover with negligible thickness strictly from the perspective of maximizing the remaining airspace available and calculating life of site.

2.3 Volume Calculations

PBL has maintained comprehensive historical records on incoming/outgoing material volumes including waste landfilled, recyclable materials diverted, sludge and green materials processed and dirt imported. This data (Table 1) along with the December 2, 2019 aerial topographic map, the September 29, 2021 aerial topographic map, the current final grading plan, and proposed modifications (this document) to the final grading plan, was used to calculate the various remaining airspace scenarios for the site.

Table 1 – Site Historic Tonnage Data

(From RDSI Appendix B dated June 26, 2020 by Montrose Environmental)							Calculations		
Bale Dimensions- 45"W X 30"H X 62"L/3.75' X 2.5' X 5.167 1.8 CY/Bale									
Month	Total Waste Received* Tons	Recycled/ Diverted* Tons	Cover Material Tons	Cover (sludge)	Baled Waste (Tons)	Number of Bales	Volume CY	AUF^ Tons/CY	Bale Density^^ (Tons)
January 2010	348.68	19.98	100.43	0	191.18	204	367.20	0.52	0.94
February 2010	356.78	123.08	75.3	0	176.28	176	316.80	0.56	1.00

(From RDSI Appendix B dated June 26, 2020 by Montrose Environmental)							Calculations		
Bale Dimensions- 45"W X 30"H X 62"L/3.75' X 2.5' X 5.167 1.8 CY/Bale									
Month	Total Waste Received* Tons	Recycled/ Diverted* Tons	Cover Material Tons	Cover (sludge)	Baled Waste (Tons)	Number of Bales	Volume CY	AUF^ Tons/C Y	Bale Density^^ (Tons)
March 2010	394.85	43.31	28.79	0	232.58	244	439.20	0.53	0.95
April 2010	460.53	87.87	151.76	0	243.34	270	486.00	0.50	0.90
May 2010	461.45	41.23	0	0	263.31	298	536.40	0.49	0.88
June 2010	478.77	21.44	14.41	57.88	288.94	294	529.20	0.55	0.98
July 2010	555.87	58.96	363.34	13.34	392.56	404	727.20	0.54	0.97
August 2010	566.58	107.63	18.1	23.79	385.91	398	716.40	0.54	0.97
September 2010	453.64	42.44	18.63	0	287.82	304	547.20	0.53	0.95
October 2010	390.99	95.19	69.43	0	227.26	244	439.20	0.52	0.93
November 2010	366.26	19.88	65.24	8.25	201.85	250	450.00	0.45	0.81
December 2010	314.66	42.19	15.84	0	192.87	220	396.00	0.49	0.88
Totals 2010	5,149.06	703.20	921.27	103.26	3,083.90	3,306	5950.80	0.52	0.93
2010 Average Per Day	14.18	1.94	2.54	0.28	8.50	9.11	16.40	0.52	0.93
2010 Total Landfilled (Sludge and Baled Waste) =			3,187.16						
January 2011	406.35	89.57	181.05	0.00	182.74	208	374.40	0.49	0.88
February 2011	393.93	136.43	14.33	0.00	155.66	200	360.00	0.43	0.78
March 2011	492.22	151.24	0.90	0.00	187.26	220	396.00	0.47	0.85
April 2011	537.06	217.13	9.75	8.01	211.07	264	475.20	0.44	0.80
May 2011	575.13	161.94	7.73	0.00	232.81	220	396.00	0.59	1.06
June 2011	681.84	204.60	377.73	10.90	262.62	170	306.00	0.86	1.54
July 2011	713.50	163.41	21.67	19.28	329.93	364	655.20	0.50	0.91
August 2011	690.82	143.39	25.78	32.95	363.26	396	712.80	0.51	0.92
September 2011	615.89	219.48	16.64	27.21	255.59	293	527.40	0.48	0.87
October 2011	512.84	134.68	296.96	5.82	200.30	244	439.20	0.46	0.82
November 2011	414.56	129.21	47.12	32.28	174.86	228	410.40	0.43	0.77
December 2011	408.03	17.57	57.24	15.84	122.21	164	295.20	0.41	0.75
Totals 2011	6,442.17	1,768.65	1,056.90	152.29	2,678.31	2,971	5347.80	0.50	0.90
2011 Average Per Day	17.75	4.87	2.91	0.42	7.38	8.18	14.72	0.50	0.90
2011 Total Landfill (Sludge and Baled Waste)			2,830.60						
January 2012	413.16	166.02	0.00	20.66	141.98	184	331.20	0.43	0.77
February 2012	434.25	158.71	0.45	7.97	141.96	182	327.60	0.43	0.78
March 2012	523.67	263.87	285.19	31.25	163.62	230	414.00	0.40	0.71
April 2012	554.41	142.53	0.00	11.60	191.29	194	349.20	0.55	0.99
May 2012	594.62	114.94	320.80	35.98	306.59	280	504.00	0.61	1.09
June 2012	659.70	137.50	0.32	52.60	264.15	232	417.60	0.63	1.14
July 2012	687.08	127.68	109.00	12.80	423.55	386	694.80	0.61	1.10
August 2012	724.55	120.42	504.40	19.14	417.25	354	637.20	0.65	1.18
September 2012	564.47	128.37	308.40	13.68	276.40	248	446.40	0.62	1.11
October 2012	483.01	141.97	394.80	28.24	212.93	184	331.20	0.64	1.16
November 2012	533.28	150.38	16.87	10.62	175.38	172	309.60	0.57	1.02

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Month	Total Waste Received* Tons	Recycled/ Diverted* Tons	Cover Material Tons	Cover (sludge)	Baled Waste (Tons)	Number of Bales	Volume CY	AUF^ Tons/C Y	Bale Density^^ (Tons)
December 2012	573.05	288.05	168.60	0.00	141.25	128	230.40	0.61	1.10
Totals 2012	6,745.25	1,940.44	2,108.83	244.54	2,856.35	2,774	4993.20	0.57	1.03
2012 Average Per Day	18.53	5.33	5.79	0.67	7.85	7.62	13.72	0.57	1.03
2012 Total Landfill (Sludge and Baled Waste)			3,100.89						
January 2013	537.40	146.64	91.10	0.00	180.06	176	316.80	0.57	1.02
February 2013	491.76	293 .11	0.15	24.01	140.06	136	244.80	0.57	1.03
March 2013	499.24	146.10	5.93	0.00	151.57	143	257.40	0.59	1.06
April 2013	595.70	255 .98	86.70	36.23	205.28	204	367.20	0.56	1.01
May 2013	577.58	243.42	137.66	19.51	239.76	218	392.40	0.61	1.10
June 2013	569.05	334.20	26.60	36 .10	273.85	242	435.60	0.63	1.13
July 2013	783.38	0.00	30.22	4.52	375.01	319	574.20	0.65	1.18
August 2013	616.50	247 .38	86.47	0.00	420.98	352	633.60	0.66	1.20
September 2013	683.74	155.04	93.11	36.27	288.54	226	406.80	0.71	1.28
October 2013	802.47	324.26	221.49	15.34	274.37	242	435.60	0.63	1.13
November 2013	584.99	315.45	788.98	8.84	196.14	179	322.20	0.61	1.10
December 2013	465.62	165.93	1389.90	11.96	201.30	194	349.20	0.58	1.04
Totals 2013	7,207.43	2,627 .51	2,958 .31	192 .78	2,946.92	2,631	4735.80	0.62	1.12
2013 Average Per Day	19.86	7.24	8.15	0.53	8.12	7.25	13.05	0.62	1.12
2013 Total Landfill (Sludge and Baled Waste)			3,139.70						
January 2014	480.58	168.21	271.69	20.98	205.58	198	356.40	0.58	1.04
February 2014	419.92	331.36	200 .48	1.12	176.67	162	291.60	0.61	1.09
March 2014	673.75	186.52	335.00	18.50	239.93	202	363.60	0.66	1.19
April 2014	554.67	152.14	233 .02	10.17	271.17	220	396.00	0.68	1.23
May 2014	729.97	160.65	148.47	5.81	280.01	242	435.60	0.64	1.16
June 2014	869.23	565.91	260 .62	5.25	324.47	256	460.80	0.70	1.27
July 2014	777.40	384.55	132.03	19.52	384.82	320	576.00	0.67	1.20
August 2014	744.35	198.27	12.68	11.22	406.71	324	583.20	0.70	1.26
September 2014	652.28	348.43	149.35	23.10	331.37	286	514.80	0.64	1.16
October 2014	613.96	297.40	254.56	4.77	276.92	210	378.00	0.73	1.32
November 2014	567.47	142.29	124.94	5.41	224.45	204	367.20	0.61	1.10
December 2014	450.73	348.19	42.98	2.33	233.52	188	338.40	0.69	1.24
Totals 2014	7,534.31	3,283.92	2,165.8 2	128.18	3,355.62	2,812	5061.60	0.66	1.19
2014 Average Per Day	20.70	9.02	5.95	0.35	9.22	7.73	13.91	0.66	1.19
2014 Total Landfill (Sludge and Baled Waste)			3,483.80						
January 2015	506.59	275.09	3.48	3.7	227.74	202	363.60	0.63	1.13
February 2015	467.35	209 .05	442.94	14.92	216.97	198	356.40	0.61	1.10
March 2015	500.23	203 .57	1043 .9	21.83	278.49	227	408.60	0.68	1.23
April 2015	513.12	149.04	301	11.54	299.5	258	464.40	0.64	1.16
May 2015	537.96	177.35	240 .9	12.73	314.74	268	482.40	0.65	1.17

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June 2015	634.19	317.77	26.36	8.32	339.75	300	540.00	0.63	1.13
July 2015	465.06	320.31	169 .2	9.77	465.06	382	687.60	0.68	1.22
August 2015	722.09	182.9	210 .9	18.71	442.06	354	637.20	0.69	1.25
September 2015	567.63	193.98	226.9	19.67	369.87	310	558.00	0.66	1.19
October 2015	534.59	190.26	376 .7	17.05	299.59	256	460.80	0.65	1.17
November 2015	473.52	180.68	601.7	25.3	258.91	238	428.40	0.60	1.09
December 2015	395.24	253 .84	22.26	12.81	221.22	206	370.80	0.60	1.07
Totals 2015	6,317.57	2,653.84	3,666.24	176.35	3,733.90	3,199	5758.20	0.65	1.17
2015 Average Per Day	17.40	7.31	10.10	0.49	10.29	8.81	15.86	0.65	1.17
2015 Total Landfill (Sludge and Baled Waste)			3,910.25						
January 2016	435.8	116.99	90.42	24.17	204.78	194	349.20	0.59	1.06
February 2016	457.39	144.36	29.62	18.47	241.35	223	401.40	0.60	1.08
March 2016	511.2	222.5	64.89	2.65	263.34	248	446.40	0.59	1.06
April 2016	543.66	2.86	543 .89	43.59	264.37	254	457.20	0.58	1.04
May 2016	617.28	277.41	249.66	50.22	295.8	281	505.80	0.58	1.05
June 2016	593.49	238 .85	17.11	32.88	310.98	284	511.20	0.61	1.10
July 2016	700.18	11.25	787.97	23.85	437.54	381	685.80	0.64	1.15
August 2016	664.95	231.25	963.48	29.45	391.63	352	633.60	0.62	1.11
September 2016	529.92	113.33	224.95	21.14	304.36	286	514.80	0.59	1.06
October 2016	558.52	121.55	2387.2	30.36	263.73	238	428.40	0.62	1.11
November 2016	511.66	257 .29	27.5	31.01	22.04	220	396.00	0.06	0.10
December 2016	474.72	150.08	53.88	14.29	199.45	206	370.80	0.54	0.97
Totals 2016	6,598.77	1,887.72	5,440.57	322.08	3,397.37	3,167	5700.60	0.60	1.07
2016 Average Per Day	18.13	5.19	14.95	0.88	9.33	8.70	15.66	0.60	1.07
2016 Total Landfill (Sludge and Baled Waste)			3,719.45						
January 2017	428.6	111 .42	0	0	215.78	202	363.60	0.59	1.07
February 2017	402.9	129.73	52.94	0	203.07	184	331.20	0.61	1.10
March 2017	570.68	120	2.77	66.06	272.07	270	486.00	0.56	1.01
April 2017	578.49	116.55	20.94	25.88	257.68	246	442.80	0.58	1.05
May 2017	780.24	276.73	66.54	24.11	283	281	505.80	0.56	1.01
June 2017	812.7	352 .38	799 .18	9.36	344.16	318	572.40	0.60	1.08
July 2017	750.48	153.37	608.1	22.26	430.77	435	783.00	0.55	0.99
August 2017	749.1	263 .06	225 .1	27.84	394.01	410	738.00	0.53	0.96
September 2017	572.57	127.3	26.99	17.49	276.6	442	795.60	0.35	0.63
October 2017	554.07	117.03	47.24	20.34	239.42	256	460.80	0.52	0.94
November 2017	525.07	121.78	150.95	4.75	203.12	216	388.80	0.52	0.94
December 2017	495.95	137.26	316.1	26.38	174.99	196	352.80	0.50	0.89
Totals 2017	7,220.85	2,026.61	2,316 .85	244.47	3,294.67	3,456	6220.80	0.53	0.95
2017 Average Per Day	19.89	5.58	6.38	0.67	9.08	9.52	17.14	0.53	0.95

(From RDSI Appendix B dated June 26, 2020 by Montrose Environmental)							Calculations		
Bale Dimensions- 45"W X 30"H X 62"L/3.75' X 2.5' X 5.167 1.8 CY/Bale									
Month	Total Waste Received* Tons	Recycled/ Diverted* Tons	Cover Material Tons	Cover (sludge)	Baled Waste (Tons)	Number of Bales	Volume CY	AUF^ Tons/C Y	Bale Density^^ (Tons)
2017 Total Landfill (Sludge and Baled Waste)			3,539.14						
January 2018	447.72	251.08	88.51	9.62	194.79	230	414.00	0.47	0.85
February 2018	452.72	87.7	39.54	26.22	212.51	210	378.00	0.56	1.01
March 2018	513.1	111.73	43.38	4.87	275.12	267	480.60	0.57	1.03
April 2018	585.47	216.77	157.83	31.53	275.06	281	505.80	0.54	0.98
May 2018	547.5	224.88	255.35	1.96	255.1	278	500.40	0.51	0.92
June 2018	641.91	168.61	365.11	38.88	311.96	302	543.60	0.57	1.03
July 2018	774.67	117.97	300.99	15.64	462.58	423	761.40	0.61	1.09
August 2018	731.69	240.38	121.67	19.81	428.47	409	736.20	0.58	1.05
September 2018	615.79	123.51	85.88	17	325.67	341	613.80	0.53	0.96
October 2018	635.45	207.75	148.81	13.58	254.86	308	554.40	0.46	0.83
November 2018	657.38	276.72	154.58	6.29	230.3	254	457.20	0.50	0.91
December 2018	490.73	122.13	158.06	16.32	254.03	218	392.40	0.65	1.17
Totals 2018	7,094.13	2,149.23	1,919.71	201.72	3,480.45	3,521	6,337.80	0.55	0.99
2018 Average Per Day	19.54	5.92	5.29	0.56	9.59	9.70	17.46	0.55	0.99
2018 Total Landfill (Sludge and Baled Waste)			3,682.17						
January 2019	514.59	108.85	25.33	0	272.59	232	417.60	0.65	1.17
February 2019	522.47	260.93	341.92	0	255.95	208	374.40	0.68	1.23
March 2019	543.12	121.54	378.88	8.18	279.31	242	435.60	0.64	1.15
April 2019	683.78	130.83	368.96	33.35	337.27	290	522.00	0.65	1.16
May 2019	609.5	230.4	418	18.06	369.76	306	550.80	0.67	1.21
June 2019	627.73	342.89	111.74	14.21	354.14	308	554.40	0.64	1.15
July 2019	785.19	113.9	58.37	19.78	433.3	344	619.20	0.70	1.26
August 2019	655.35	93.44	215.28	16.91	466.4	374	673.20	0.69	1.25
September 2019	698.57	204.43	124.54	15.2	425.05	321	577.80	0.74	1.32
October 2019	543.92	91.06	568.52	5.65	276.0	234	421.20	0.66	1.18
November 2019	535.73	65.29	46.47	21.81	367.22	295	531.00	0.69	1.24
December 2019	499.42	111.33	122.79	15.57	259.63	226	406.80	0.64	1.15
Totals 2019	7,219.37	1,874.89	2,780.80	168.72	4,096.62	3,380	6,084.00	0.67	1.21
2019 Average Per Day	19.78	5.14	7.62	0.46	11.22	9.26	16.67	0.67	1.21
2019 Total Landfill (Sludge and Baled Waste)			4,265.34						
* Waste includes all MSW, C& D, compost, sludge, green waste, & metal from AES, Public and City. Does not include Cover Dirt ** Recycled includes C&D, grease, E-waste, metal, plastic, etc. Does not include green waste/ compost. ^ AUF is Airspace Utilization Factor for waste only and is calculated by Tons waste/Vol Waste. ^^Tons per Bale is calculated by Waste tons/# of Bales									
10 yr Avg Daily Waste	18.59	5.76	6.97	0.53	9.06	8.59		0.58	1.05

			Landfilled Waste by Month (From CR&R email dated 10/19/2021)			Calculations		
	Total Waste Received* Tons	Recycled/ Diverted* Tons	Cover Material and Sludge Volume Cubic Yards***	Baled Waste Tons	Number of Bales Landfilled	Volume of Bales CY	AUF Tons/CY	Bale Density Tons/Bale
January 2020	NA	NA	87.84	269.58	244	439.20	0.61	1.10
February 2020	NA	NA	79.92	235.81	222	399.60	0.59	1.06
March 2020	NA	NA	80.64	247.83	224	403.20	0.61	1.11
April 2020	NA	NA	70.56	220.54	196	352.80	0.63	1.13
May 2020	NA	NA	69.12	206.75	192	345.60	0.60	1.08
June 2020	NA	NA	62.64	185.08	174	313.20	0.59	1.06
July 2020	NA	NA	125.28	362.27	348	626.40	0.58	1.04
August 2020	NA	NA	126	396.06	350	630.00	0.63	1.13
September 2020	NA	NA	115.92	348.37	322	579.60	0.60	1.08
October 2020	NA	NA	120.96	369.08	336	604.80	0.61	1.10
November 2020	NA	NA	97.56	291.86	271	487.80	0.60	1.08
December 2020	NA	NA	70.92	202.16	197	354.60	0.57	1.03
Totals 2020			1107.36	3,335.39	3,076	5536.80	0.60	1.08
2020 Average Per Day			3.03	277.95	256.33	461.40	0.60	1.08
2020 Total Landfill (Sludge and Baled Waste)								
January 2021	NA	NA	72	210.02	200	360.00	0.58	1.05
February 2021	NA	NA	75.6	205.28	210	378.00	0.54	0.98
March 2021	NA	NA	99.72	271.45	277	498.60	0.54	0.98
April 2021	NA	NA	116.64	319.1	324	583.20	0.55	0.98
May 2021	NA	NA	174.24	353.29	484	871.20	0.41	0.73
June 2021	NA	NA	133.2	368.18	370	666.00	0.55	1.00
July 2021	NA	NA	158.76	471.69	441	793.80	0.59	1.07
August 2021	NA	NA	143.28	428.82	398	716.40	0.60	1.08
September 2021	NA	NA	98.28	286.14	273	491.40	0.58	1.05
October 2021						0.00		
November 2021						0.00		
December 2021						0.00		
Totals 2021			1071.72	2,913.97	2,977.00	5358.60	0.54	0.98
2021 Average Per Day			3.93	323.77	330.78	595.40	0.54	0.98
2021 Total Landfill (Sludge and Baled Waste)								

*** Volumes estimated by site. Waste to cover ratio assumed at 5:1 then cover volumes calculated

**** Remaining airspace calculated from difference between Closure Plan and 9/29/2021 topo

***** Calculated from Closure Plan Top Deck Area multiplied by proposed alternative cover thickness

$$= [(69,500 \text{ sf})(3\text{ft})]/27 = 7,725\text{CY}$$

AUF of 0.58 is from waste bales only

AUF of 0.63 is difference between topos and includes cover and sludge

2019 Topo Vs. 2021 Topo	
Month	Total Tons Waste Landfilled
Dec-19	259.63
Jan-20	269.58
Feb-20	235.81
Mar-20	247.83
Apr-20	220.54
May-20	206.75
Jun-20	185.08
Jul-20	362.27
Aug-20	396.06
Sep-20	348.37
Oct-20	369.08
Nov-20	291.86
Dec-20	202.16
Jan-21	210.02
Feb-21	205.28
Mar-21	271.45
Apr-21	319.1
May-21	353.29
Jun-21	368.18
Jul-21	471.69
Aug-21	428.82
Sep-21	286.14

2.4 Current Remaining Airspace

The remaining airspace and, consequently, the remaining site life depend on a variety of factors including, but not limited to, the daily volume of incoming waste to be landfilled and the corresponding amount of cover material needed to cover the waste, the amount of waste diverted as recyclables, and the density of the waste (sometimes referred to as AUF, or airspace utilization factor expressed in tons/CY). Other factors affecting airspace include the amount of material used for daily and intermediate cover (cover ratio), and the final cover thickness.

The first calculation GLA made was to compare the 2019 topographic map and 2021 topographic map to calculate the volume of airspace consumed between those dates (refer to the bottom portion of Table 2). Then, using the tonnage records provided by the site during that time frame, an AUF and cover ratio was established, i.e. the density of the landfilled waste and the volume of cover material used. These values were then used to project remaining capacity under existing conditions. The remaining volume for the site is the calculated difference between the current final grading plan and the September 29, 2021 topography. The

calculations suggest that approximately 47,640 CY of total airspace remain, which translates to 75 months (6.2 years) of life or that the site will run out of space around the first quarter of 2028. This corroborates the previous calculations reported in the 2019 Remaining Permitted Capacity Report prepared by X Engineering and Consulting, Inc. (January, 2020) as included in Appendix D of the 2020 RDSI and Section 1.7-Site Life Projection of the 2020 RDSI (Montrose Env, July, 2020).

2.5 Site Life Optimization

Due primarily to the physical constraints at the site, a lateral step out of the landfill boundary beyond the existing waste footprint is not a feasible option to optimize/increase site life. It was decided that staying within the permit and property boundaries was the only realistic means to increase site life. As such, GLA investigated a mechanically stabilized earth wall and/or retaining wall constructed at the toe of the landfill to increase capacity within the existing horizontal extent of the landfill. The proposed final grading plan as shown on Drawing C01 of Appendix A utilizes a concrete retaining wall running North to South along the toe of the landfill. GLA also considered "piggybacking" over the top of the current final grades to the maximum extent possible. The proposed final grading plan was created with that in mind. This grading plan was compared to the September 29, 2021 topography to calculate the increase in available airspace. Two specific airspace volumes were calculated; 1) remaining airspace discounting a 3-foot thick ET final cover and, 2) remaining airspace using a geosynthetic cover of negligible thickness (Table 2). Under the ET cover scenario, the results show approximately 183,500 CY remaining which translates to 288 months or 24 years, extending the closure date to 2045. Under the geosynthetic cover scenario, approximately 210,450 CY remain, which equates to a site life of 330 months, or about 27.5 years putting closure around 2048 assuming that existing waste disposal and soil usage rates remain the same during this period.

Under the proposed final grading plan, access to the site will be achieved from an access ramp in the middle portion of the retaining wall with a secondary access utilizing the neighboring cement plant for closure and post closure maintenance. Final slopes are designed at 2:1 H:V. It was thought that the interlocking effect of the baled waste might provide for additional slope stability and hence a steeper final grade, however, we observed that the waste bales break down shortly after emplacement in the landfill. As a result, our calculations conservatively and realistically assume that the waste baling operation provides no benefit to waste stability when compared to traditional waste fill. Thus, strength characteristics were analyzed using common strengths for municipal waste and the final slopes were designed at 2:1 H:V.

Table 2 – Life of Site Calculations

	Current Final Grading Plan Vs. 2021 Topo		Proposed Final Grading Plan Vs. 2021 Topo		
	Final Cover Area* (SF)	3' E.T. Volume (CY)	Final Cover Area** (SF)	3' E.T. Volume (CY)	Turf Volume (CY)
Total Airspace Remaining from CADD 3" Final E.T. Cover Turf Closure Total Airspace Remaining Subtotal		64,639	242,430	210,450 26,937 183,513	210,452 0 210,450
Daily Cover CY (4:1 ratio)*** Waste Airspace Remaining (CY) Waste Airspace Remaining (Tons) = CY x AUF where AUF=0.58 Tons/CY calculated from records Avg. Landfilled Tons/Mo****		9,528 38,111 296		36,703 146,811 296	42,090 168,360 296
Months Remaining Years Remaining		85 6.2		288 24.0	330 27.5

* As reported from July 2020 RDSI

** Area calculated from Proposed Final Grading Plans

Dec. 2, 2019 Topo Vs. Sept. 29, 2021 Topo	
Airspace from CADD (CY)	14,164
Volume waste landfilled from Tonnage Report (CY)	11,302
Calc. Cover Volume = Airspace-Volume Waste Landfilled	2,862
Calc. Waste to Cover Ratio	4

**** Average monthly tonnage reported between aerial flyover dates Dec. 2, 2019-Sept. 29, 2021

Operational Efficiencies

Improving operational efficiencies to increase AUF, and improving waste to cover ratios can also impact site life. While it may not add a tremendous amount, it can make a difference especially for a site nearing its end of useful life.

It was found that the waste lifts at PBL were placed at approximately seven to eight feet or three bales high (one bale is approximately 30" tall by 45" wide by 60" long). Cover material was then placed on top of the bales to create a deck area. If an additional bale was placed making a four-bale lift then the waste to cover ratio could be increased significantly (up to 33%), ultimately increasing sight life. One foot of cover soil over a three-bale lift or 7.5 feet of waste is approximately 11.8% of the total lift height. One foot of cover over a four-bale lift 10 feet of waste is approximately 9.1% of the lift height.

Increasing bale density will consume less airspace. The current bales dimensions are 30" tall by 45" wide by 60" long, or 1.8 CY per bale. The average reported bale weight is approximately 1.04 tons per bale which gives an AUF of 0.58 tons per CY (1160lbs/CY). The baler produces fixed bale sizes so increasing the individual bale density is a function of the mechanical compaction effort of the baler and may not be realistic.

Bale placement is also important to maximize density. The operator should make sure there is little to no airspace between bales when placed in the landfill. Any gaps between bales is unused airspace and costly.

Unfortunately, it is both difficult to quantify impacts associated with operational efficiencies and to rely on those modifications to operations. As such GLA did not consider these in remaining life of site calculations.

3. Seismic Hazard Evaluation

3.1 General

Seismic hazard evaluation is required to develop site-specific seismic hazard parameters for the evaluation of seismic stability of gravity retaining walls (cantilever-type retaining walls) and waste fill slopes at the site. Additional interpretation of these parameters (e.g., acceleration response spectrum) may be required for structural evaluations at the site.

3.2 Seismic Source Model

The United States Geological Survey (USGS) national seismic hazard model for the Western United States, as implemented by Risk (2021) into EZ-FRISK (V 8.07), was used to define the seismic sources for the seismic hazard analysis presented herein. The shallow crustal faults that govern the seismic hazard at the site (e.g., the Santa Cruz-Santa Catalina Ridge, the Palos Verdes Fault and the Newport Inglewood Fault Zone are indicated on a fault map that is enclosed in Appendix B.7.) The subduction zones, also included in our evaluation, are outside of a 60-mile (100-km) radius off the site and therefore are not included in Appendix B.7.

3.2.1 Ground Motion Prediction Equations Employed

NGA West-2 Ground Motion Prediction Equations (GMPEs), as implemented in EZ-FRISK, were used in this analysis. In particular, Boore and Atkinson (2008); Campbell and Bozorgnia (2014); and Chiou and Youngs (2014); were used for shallow crustal events. A standard USGS set of

GMPEs for subduction events including, Atkinson and Boore (2003); and Youngs et al. (1997), was also used.

3.2.2 Seismic Site Conditions

The time-averaged shear wave velocity in the upper 30 meters of the soil profile, V_{s30} , is a key index parameter to account for seismic site conditions for a site. McPhillips et al. (2020) has compiled measured V_{s30} funded by the USGS and other governmental agencies for 4,389 sites across the United States. The data is presented as an interactive map on the USGS website (<https://earthquake.usgs.gov/data/vs30/us/>). The map includes V_{s30} measurements for two sites near the PBL on Santa Catalina Island. The V_{s30} measurements are from 505 m/s to 756 m/s. To provide a conservative assessment for the seismic hazard analyses, the V_{s30} was selected as 505 m/s (1,657 ft/s) for the landfill subgrade. These assumed site conditions correspond to the National Earthquake Hazard Reduction Program (NEHRP) boundary of Site C (Very dense soil and soft rock).

3.2.3 Seismic Hazard Analysis Results

The seismic hazard analysis documented herein is Probabilistic. The Probabilistic Seismic Hazard Analysis (PSHA) was performed to calculate the acceleration response spectrum with a 2% probability of exceedance in 50 years, which corresponds to an event with a 2,475-year return period and is representative of a Maximum Credible Earthquake (MCE) at the site.

The PSHA spectrum calculated by EZ-FRISK is shown in Appendix B.6, Figure 1. De-aggregated Mean Moment Magnitude for the site is **M** 6.46, as presented in Appendix B.2. This de-aggregated moment magnitude is representative of MCE at the site and is consistent with approximate values for the moment magnitude assigned to the Santa Cruz-Santa Catalina Ridge alt 2 fault by USGS. This moment magnitude is representative of MCE at the site and is consistent with values for the Santa Cruz-Santa Catalina Ridge alt 2 fault reported by USGS. The corresponding Maximum Probable Earthquake (MPE) moment magnitude was conservatively evaluated as MCE **M** 6.46 – 0.5 = **M** 5.96.

Additional details related to the evaluations explained above are provided in Appendix B.1 (the echo of input data for the EZ-FRISK program), B.2 (de-aggregation of seismic hazard at zero period computed using USGS unified hazard tool), and B.3 (probabilistic seismic hazard analysis response spectrum, the output of the EZ-FRISK). Note that the EZ-FRISK “echo” file is over 980 pages long; therefore, only the first five pages are enclosed as an appendix.

3.2.4 Evaluation of Seismic Coefficient

Seismic coefficient (k_s) is an empirical constant required for pseudo-static evaluations. In the State of California, $k_s = 0.05$ is assigned to faults within the magnitude range of **M** 5.5 – 6.4, $k_s = 0.1$ is assigned to faults within the magnitude range of **M** 6.5 – 7.4, and $k_s = 0.15$ is assigned to faults within magnitude greater than **M** 7.5. For seismic evaluations at this site, to provide a conservative basis for design, commensurate with the estimated moment magnitude for MCE, $k_s = 0.10$ is selected.

3.3 Design Basis

The design basis and the stability criteria for the design of PBL were originally established by Golder (1999). This information has been updated and expanded in accordance with the current regulations and design standards in California and now include:

- Static Factor of Safety (FS): $FS \geq 1.5$;
- Pseudo-static Factor of Safety (FS): $FS \geq 1.5$;
- Design Moment Magnitude: MCE $M = 6.46$; MPE $M = 5.96$; and
- Design Seismic Coefficient: $k_s = 0.10$ (for MCE)¹.

3.4 Gravity Retaining Walls

3.4.1 General

During the design stage of this project, several waste retention options were considered. These included construction of: (i) Mechanically Stabilized Earth (MSE) walls; (ii) conventional gravity retaining walls; and (iii) cantilever-type retaining walls. The cantilever-type retaining walls were selected as the most economical option. The layout of these walls along the southern perimeter of the site is shown in Appendix B.6, Figures 2 to 5. Also shown is an access ramp that will be constructed in the same fashion. Including the access ramp, the height of cantilever-type retaining walls (i.e., wall stem height) ranges from near-zero to 18 feet.

GLA selected standard Caltrans design for cantilever-type retaining walls to be constructed at the site. Relevant Caltrans drawings and Technical Specifications are presented in Appendix B.4. A summary of relevant information, including reference to the Caltrans sheet numbers, is provided in Appendix B.6 Figures 2 to 5. Given the standard design, no special considerations /

¹ Note that even higher seismic coefficient was used to develop standard retaining wall design by CalTrans.

design basis is required. However, the global static and seismic stability of the wall-waste backfill system has been evaluated in subsequent sections.

Wall stem water- (leachate-) proofing, surface water, and wall backfill drainage details are presented in Appendix B.6, Figure 3.

3.4.2 Discussion

Cantilever-type retaining walls design by Caltrans is reasonably conservative for applications at this site. This is because:

- Caltrans standard seismic design is based upon seismic coefficient $k_s = 0.2$. Seismic coefficient of this intensity is applicable for all locations across the State and is significantly higher than $k_s = 0.10$ evaluated for this site;
- Caltrans design assumes backfill material unit weight of 120 pcf, typical of compacted soil. Backfill at this site is waste, with unit weight of 85 pcf (MSW) and 105.7 pcf (incinerator ash).
- Caltrans design assumes that the wall is founded in soil. Proposed retaining walls are founded in granitic bedrock.

3.5 Material Parameters

3.5.1 General

The following material parameters are required to evaluate static and seismic stability of waste fills, and backfills: (i) shear strength of waste; (ii) unit weight of waste; and (iii) interface shear strength (base and side-slope liner systems).

At this site, two kinds of waste have been disposed of: (i) incinerator ash; and (ii) baled waste. The incinerator ash is a result of incineration of Municipal Solid Waste (MSW). It was disposed of from the inception of landfill in the early 1920's to around 2000. Upon cessation of the incineration activities, the site acquired a baler and started disposing of baled MSW.

Material parameters of the cantilever-type retaining wall are not discussed herein. That is because GLA adopted a standard Caltrans design for gravity retaining walls (see discussion above) which includes wall geometry, keying requirements, and material specification.

3.5.2 Incinerator Ash

Schoenberger and Fungaroli (1971), Poran and Ahtchi-Ali (1989), and Goh and Tay (1993) studied the geotechnical properties of MSW incinerator ash. The review of the references revealed that:

- When incinerator ash is compacted using standard procedures, the dry density of the material would be lower than the dry density of typical soil with similar gradation. The dry unit weight of incinerator ash is typically from 78.9 pcf to 102 pcf. A typical wet density incinerator ash of 105.7 pcf, as reported by Poran and Ahtchi-Ali (1989), was used in the slope stability analyses.
- The friction angle of incinerator ash is typically higher than the friction angle of soils having a similar texture. The friction angle of incinerator ash ranges from 43 to 45 degrees. To provide a conservative basis for design, friction angle of 43 degrees was assigned to incinerator ash and was further employed in the slope stability analyses. Cohesion intercept was assumed to be zero. The shear strength envelope for the incinerator ash is shown in Appendix B.6, Figure 6.

3.5.3 Municipal Solid Waste

GLA selected a coherent set of MSW parameters that accounts for waste composition, age, and density. The particular set of MSW parameters was developed by testing of waste recovered from the Operating Industries Inc. (OII) landfill by Kavazanjian et al. (2013) (see Attachment B.5). The OII landfill is in Monterey Park, California.

The testing of OII MSW by Kavazanjian et al. (2013) was performed under the United States Environmental Protection Agency (US EPA) Technical Review Panel (TRP) oversight. The testing was performed in a large-diameter (18-inch) device at normal stress as high as 1,750 kPa.

The Kavazanjian et al. (2013) shear strength envelope is reproduced in Appendix B.6, Figure 6. The average of this strength envelope is used herein. This model is characterized by a cohesion intercept of 900 psf and a friction angle of 31 degrees.

3.5.4 Waste-Bedrock Interface

Incinerator ash was placed over a prepared bedrock (granite) subgrade. Subgrade preparation likely consisted of proof-rolling excavated area. No interface shear tests are available. Therefore, to provide a conservative basis for design, GLA assumed that interface resistance is equal to 2/3 of the shear strength of the weaker material. Given that the shear strength of

incinerator ash is lower than that of its bedrock counterpart, a friction angle of $2/3 \times 43 \text{ deg} = 28 \text{ deg}$ was assigned to the interface (i.e., the base of the landfill).

3.6 Waste Fill Stability Assessment

3.6.1 Method of Analysis

For stability evaluations documented herein, GLA used the conventional limit equilibrium approach. In particular, GLA employed Morgenstern and Price (1965) method, as implemented in the computer program SLOPE/W (GSI, 2021; www.geo-slope.com). Results are reported for the critical calculate failure surface, i.e., failure surface, which yielded the lowest calculated Factor of Safety (FS).

The results of static and pseudo-static evaluations are presented, for each stability configuration, and they are evaluated in the form of critical (static) failure surface and the corresponding lowest calculated FS. Based on historic topographic maps and interviews with site operations staff, the thickness of the incinerator ash is approximately 50 feet (roughly 170 ft MSL to 220 ft MSL). To provide a conservative assessment, a parametric study was performed to evaluate the effect of the ash layer thickness on slope stability. The range of the elevation of the ash layer in the parametric study was from about 175 ft to 250 ft MSL.

3.6.2 Representative Cross Sections

Representative cross-sections A-A', B-B', and C-C' developed through the proposed waste fill plan are shown in the plan view in Appendix B.6 Figure 2 and in the profile view in Figure 7. These cross-sections were developed to engage the thickest waste fill configuration (cross-section B-B'), the highest waste fill slope (cross-sections A-A', B-B', and C-C'), and a waste fill slope with minimal buttressing (cross-sections A-A' and C-C').

Figure 8 in Appendix B.6 shows the isopach drawing for the proposed landfill top deck re-configuration for site life optimization.

3.6.3 Results of Stability Evaluations

The parametric study results of the static and pseudo-static slope stability evaluations are presented in Appendix B.6, Figures 9 through 14. The results of these evaluations are further summarized in Table 3.

Table 3 – Parametric Study Results of Static and Pseudo-static Slope Stability Evaluations

Cross Section	FS_{STAT}	FS_{P-STAT} for k_s = 0.10	Reference Figures
A-A'	From 2.28 to 2.42 > 1.5 O.K.	From 1.80 to 1.91 > 1.5 O.K.	9 and 10
B-B'	From 2.20 to 2.41 > 1.5 O.K.	From 1.72 to 1.84 > 1.5 O.K.	11 and 12
C-C'	From 1.99 to 2.61 > 1.5 O.K.	From 1.60 to 1.95 > 1.5 O.K.	13 and 14

FS_{STAT} = Static Factor of Safety; FS_{P-STAT} = Pseudo-static Factor of Safety; k_s = Seismic Coefficient.

4. Surface Water Analysis

The following section describes the conceptual design for the conveyance and erosion control features of the surface water management system (SWMS) for PBL. The overall SWMS at landfill closure is presented on Drawing C01.

The purpose of this section is to describe the existing and future hydrologic conditions of the proposed landfill re-design. The PBL watershed is approximately 11.8 acres of which 4.2 acres is run-on from adjacent properties. Elevations vary between 161 ft MSL at the northern point of the property and 730 ft MSL at the southern point of Subarea D3 as shown on Figure 2 of Appendix C.1. Slopes adjacent to the landfill are steep and mostly bare given the site’s history as being a quarry prior to landfilling operations. The climate is Mediterranean with the majority of rainfall occurring between November and May.

The proposed drainage for PBL is shown on Drawing C01. The runoff will be collected in a series of open channels and pipes and conveyed to existing discharge locations on the east side of the property.

4.1 Design Criteria and Methodology

4.1.1 Design Storm

27 CCR 20365(f) requires that drainage structures for landfill sites be designed to accommodate peak flow resulting from the design storm event. For Class III MSW landfills, the design storm event is the 100-year frequency, 24-hour storm. The proposed surface water management system features have been designed to accommodate the anticipated precipitation and resulting run-off generated during the peak 100-year, 24-hour rainfall event (design storm). The rainfall data used in the hydrologic analysis was obtained from the National Oceanic and

Atmospheric Administration (NOAA) Atlas 14, Volume 6, Version 2 rainfall frequency maps. Upon review of the rainfall frequency maps, the 100-year, 24-hour storm event is estimated to be 6.72 inches as shown in Appendix C.2. While Catalina Island is located within Los Angeles County, there is no information about the Catalina Island in the Los Angeles County Department of Public Works Hydrology Manual. Therefore, GLA used the TR-55 Tabular Hydrograph method within Autodesk Storm and Sanitary Sewer Analysis (SSA) to calculate the peak runoff generated from the design storm event.

4.1.2 Drainage Areas and Peak Run-off and Run-on Determination

Figure 1 in Appendix C.1 shows the existing conditions of the site and the existing drainage areas. This figure is included to illustrate that the drainage discharge locations remain the same at the site. The design only includes the addition of surface water conveyance structures to direct the flow to the existing discharge points.

The 11.8-acre watershed was divided into 15 drainage areas. Peak discharges were calculated for each drainage area as shown in Appendix C.5. Figure 2 in Appendix C.1 delineate the drainage areas used in surface water calculations. The drainage areas were then used to calculate the run-on and run-off volumes for the project in order to size each system's drainage structures.

The peak discharge flow rate at critical points of the system is calculated by determining the contributing drainage areas and applying the Tabular Hydrograph method (USDA, 2011). GLA used the TR-55 Tabular Hydrograph method within SSA to calculate the peak runoff generated from the design storm event. This method is widely accepted for calculating surface water runoff for small watersheds and can be used to describe a heterogeneous watershed that is divided into a number of homogeneous sub-watersheds. The Tabular Hydrograph method relates rainfall depth, a runoff curve number (CN), time of concentration (Tc), and drainage area to calculate the peak runoff from a drainage area.

The following assumptions were made by GLA in estimating the peak runoff:

1. In calculating the surface hydrology, GLA assumed that all the precipitation impacting a particular SA is eventually diverted into designed drainage channels. We also assumed that the design storm event will have a duration that exceeds the time of concentration of overland flow.
2. A runoff curve number was assigned by GLA for each SA based on the hydrologic soil group, cover type, soil treatment, hydrologic condition, and antecedent runoff condition. Specifically, based on a hydrologic soil group of type "B" (determined from the U.S.

Department of Agriculture [USDA] Web Soil Survey in Appendix C.3) and established vegetation, an overall curve number of 86 was assigned for existing (undisturbed) areas and proposed areas.

3. The time of concentration was calculated by GLA using Autodesk SSA (2022) for each SA. In general, the drainage path that will result in the longest time of concentration will be the one consisting of greater lengths of sheet and shallow flow.
4. After a maximum of 100 feet, sheet flow typically becomes shallow concentrated flow. The average velocity for this flow can be obtained from Figure 3-1 in the TR-55 Manual, or directly from within the Autodesk SSA program, in which average velocity is a function of watercourse slope and surface lining. After establishing the average velocity using Figure 3-1, SSA uses the equation $T_t = L/3600V$ to estimate travel time for the shallow concentrated flow segment, where T_t is the concentrated flow travel time (hr.), L is the flow length (ft), and V is the velocity (ft/s).
5. The following is an overview of how SSA calculates the time of concentration and flow:

For sheet flow less than 100 feet, Manning’s kinematic solution is used to compute T_t :

$$T_t = (0.007(nL)^{0.8})/((P_2)^{0.5}s^{0.4})$$

where: T_t = sheet flow travel time (hr.)

n = Manning’s roughness coefficient

L = flow length (ft)

P_2 = 2-year, 24-hour rainfall (in)

s = slope of hydraulic grade line (land slope, ft/ft)

See Table 5 in Appendix C.5 for a summary of the drainage areas and peak discharge. The peak discharge will be used to size the open channel as described in the following sections. See Tables 6, 7, and 8 in Appendix C.6, C.7, and C.8 for a detailed list of design parameters for each structure.

4.1.3 Open Channel Design Approach

Open channels, including v-ditches, bench channels, a down-chute and drive-through swales were sized for the site design using SSA. Assumptions were made for anticipated flow based on Manning’s coefficient, geometry, and slope. The assumptions and evaluations are further discussed in Section 4.2. Existing channels and culverts were modeled in order to accurately quantify flow entering the PBL and the proposed structures.

See Tables 6 and 7 in Appendix C.6 and C.7 for the open channel and pipe and culvert design summaries.

4.1.4 Rip Rap Calculations

Rip rap is commonly used to protect soil from erosion in areas of concentrated runoff. Rip rap is utilized in this design at the discharge locations in order to dissipate the flow before it leaves the PBL. The rip rap calculation shown in Appendix C.9 includes calculations to dissipate energy. The calculations include stone sizing, thickness, grout, and apron dimensions.

4.2 Proposed Surface Water Improvements

4.2.1 Overview

Figure 2 shows the proposed drainage improvements associated with PBL. Once constructed, the PBL top deck will capture surface water along bench ditches that drain into a series of drive through swales, down-chutes and perimeter v-ditches. A perimeter v-ditch along the proposed retaining wall will also collect water and convey it to a drive through swale which will outlet onto rip rap aprons at the discharge points.

Using SSA, GLA modeled the entire PBL surface water conveyance system. The proposed improvements were designed considering material type, velocity, flow rates, flow volume, slope, and ease of maintenance. The output from SSA can be found in Appendix C.4.

4.2.2 Assumptions

The following assumptions were made for the surface water improvements associated with PBL:

- Design storm: 100-year, 24-hour storm
- Material types and Manning's Roughness Coefficients:
 - ◇ Concrete or Gunitite Ditches and Drive Through Swale: these channels are lined with reinforced concrete or gunitite lining. A Manning's roughness coefficient of 0.019 is used for the ditches because it is a rough finish concrete or gunitite.
 - ◇ HDPE Culverts and Pipes: the proposed culverts and pipes were designed using a Manning's roughness of 0.01 consistent with that of corrugated HDPE.

4.2.3 Open Channel Design

All ditches, pipes, and culverts were analyzed in segments due to changing of slopes and material type. Each structure was broken into subparts during analysis in order to accurately model the hydraulic gradient throughout the system. This included all slope transitions, proposed inlets, and proposed convergences of ditches, drive through swales, pipes and culverts. The subparts of all structures and their corresponding junctions can be found in Figure 3. The culverts, pipes, and ditches analyzed include:

- Drive-Through Swales
- Bench Ditches
- Down-chute
- Perimeter V-Ditches
- Pipe
- Culvert

Table 3 below, summarizes the proposed dimensions for the drainage structures. These structures and locations are shown on Figure 3. The detailed calculations for open channels and pipes/culverts including dimensions, contributing subareas, flow rates, flow volumes, velocities, and capacities are included in Tables 6 and 7 of Appendix C.6 and C.7, respectively.

Table 4 – Open Channel and Pipe and Culvert Design Summary Table

Description	Associated Elements	Dimensions
Drive-Through Swale	Drive-Through Swale-1, Drive-Through Swale-2 (A-B)	Trapezoidal channel with ~10% side-slopes, 5-ft-wide bottom width, 0.5 ft deep, and concrete or gunnite lined
Bench Ditch	Bench-Ditch – 1, Bench-Ditch – 2, Bench-Ditch – 3, Bench-Ditch – 4, Bench-Ditch – 5, Bench-Ditch – 6	Bench ditch that is 12 ft wide and 0.5 ft deep and concrete or gunnite lined
Down-chute	Down-chute – 1 (A-B)	Trapezoidal channel that has a 1-foot bottom width, 2:1 (H:V) sideslopes and is 0.5 ft deep and concrete or gunnite lined
Perimeter V-Ditch	V-Ditch - 1, V-Ditch - 2, V-Ditch - 3, V-Ditch - 4, V-Ditch - 5, V-Ditch - 6, V-Ditch - 7, V-Ditch - 8, V-Ditch - 9, V-Ditch - 10, V-Ditch - 11, V-Ditch - 12, V-Ditch - 13, V-Ditch - 14	V-ditch with 1.33:1 (H:V) side-slopes, 1.5 ft deep, and concrete or gunnite lined
24"Ø Pipe	Pipe – 1 (A-C)	24-inch-diameter corrugated HDPE pipe with concrete wall inlet
18"Ø Culvert	Culvert - 1	18-inch-diameter corrugated HDPE culvert with a mitered to slope inlet

4.2.4 Rip Rap Design

The grouted rip rap after the drive-through swale calculations are shown in Appendix C.9. The dimensions are based on the 24" diameter pipe and 4' wide v-ditch outletting onto the aprons.

The width of the apron at the pipe outlet is three times the diameter of the pipe as shown on the calculations in Appendix C.9. Refer to Figure 2 for rip rap apron location.

5. Conclusions and Preliminary Recommendations

The results of static and seismic stability evaluations documented herein indicate that the final waste fill configuration, as proposed herein, meets the design basis stability criteria established for PBL. A new landfill top deck configuration design for PBL including an increase to 300 ft MSL is technically feasible and can be used to apply for permit revisions required to implement the plan in accordance with CCR T14 and T27 regulations. Regulatory approvals will be required from the County, LEA, Cal Recycle, LARWQCB, and CCC.

The results of the surface water analysis indicate that the surface water can be routed in the proposed conveyance structures to outlet at the same discharge points at the site that were being utilized prior to improvements. The conveyance structures have all been designed to contain the 100-year, 24-hour storm per Title 27.

The preliminary recommendations for the next phase of work include the following tasks:

1. Prepare a draft revised RDSI/JTD...
2. Prepare a draft updated /revised PCPCMP....
3. Prepare Project Description
4. Assist County with preparation of CEQA documents including an Initial Study and MND (if required)
5. Compile Joint application Form for revised SWFP and WDR Update
6. Other Permits and Approvals, if required.

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Appendix A

Construction Drawings

PEBBLY BEACH LANDFILL

SITE LIFE OPTIMIZATION DESIGN REPORT DRAWINGS

FEBRUARY 2022

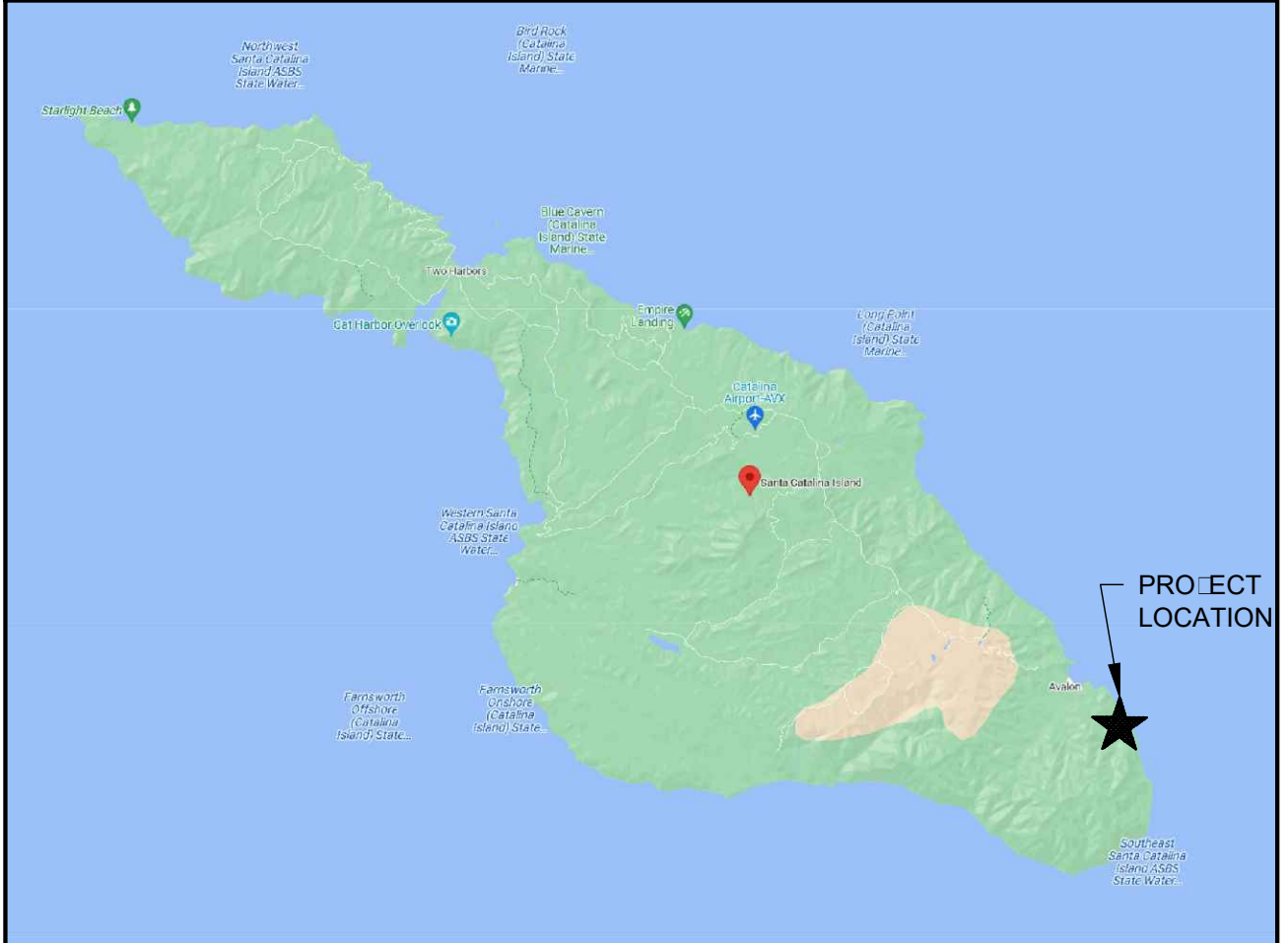


CALIFORNIA COUNTIES

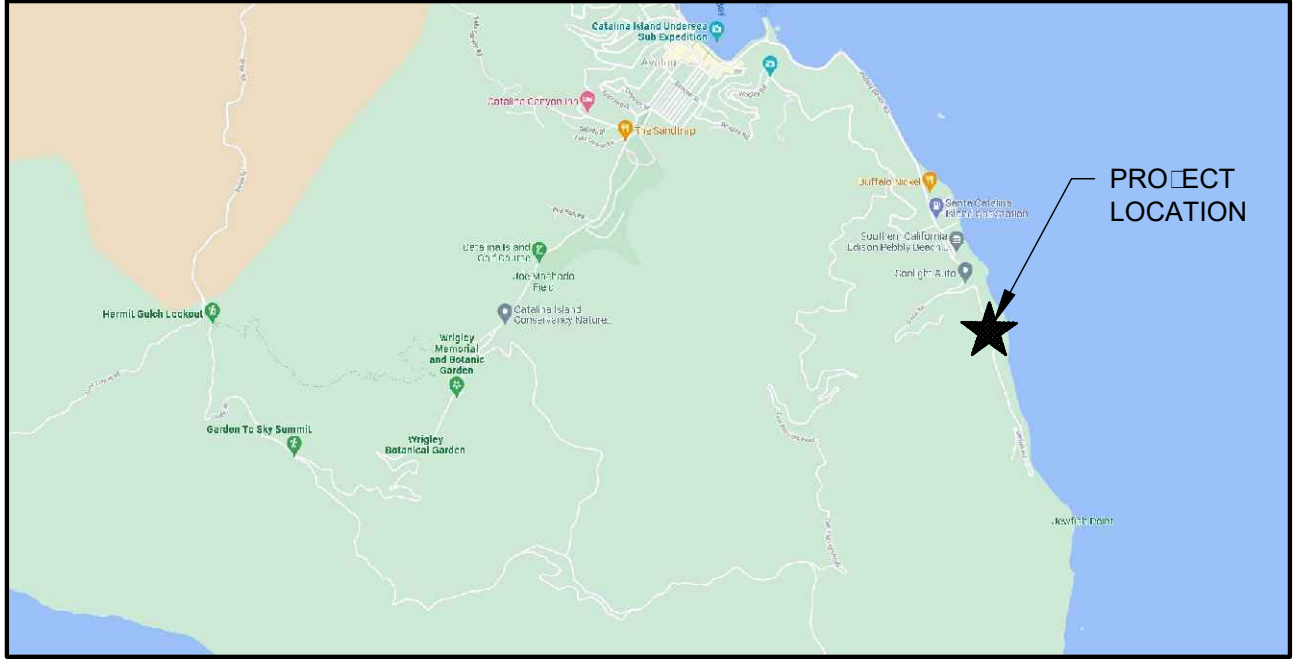


SHEET INDEX

- G01 TITLE
- G02 EXISTING SITE CONDITIONS AND PROJECT SITE PLAN
- C01 PROPOSED FINAL GRADING PLAN
- C02 PROPOSED FINAL GRADING PLAN ISOPACH
- C03 REPRESENTATIVE CROSS SECTION
- C04 GRADING DETAILS
- C05 GRADING DETAILS



REGIONAL MAP



VICINITY MAP

PRIVATE ENGINEER'S NOTICE TO CONTRACTORS

THE EXISTENCE AND LOCATION OF ANY UNDERGROUND UTILITY PIPES, CONDUITS OR STRUCTURES SHOWN ON THESE PLANS WERE OBTAINED BY A SEARCH OF AVAILABLE RECORDS. TO THE BEST OF OUR KNOWLEDGE THERE ARE NO EXISTING UTILITIES EXCEPT AS SHOWN ON THESE PLANS. THE CONTRACTOR IS REQUIRED TO TAKE DUE PRECAUTIONARY MEASURES TO PROTECT THE UTILITIES SHOWN AND ANY OTHER LINES OR STRUCTURES NOT SHOWN ON THESE PLANS.

CONTRACTOR FURTHER AGREES THAT HE SHALL ASSUME SOLE AND COMPLETE RESPONSIBILITY FOR JOB SITE CONDITIONS DURING THE COURSE OF CONSTRUCTION OF THIS PROJECT, INCLUDING SAFETY OF ALL PERSONS AND PROPERTY THAT THIS REQUIREMENT SHALL APPLY CONTINUOUSLY AND NOT BE LIMITED TO NORMAL WORKING HOURS AND THAT THE CONTRACTOR SHALL DEFEND, INDEMNIFY AND HOLD GOVERNMENT AGENCIES, THE OWNER AND THE ENGINEER HARMLESS FROM ANY AND ALL LIABILITY, REAL OR ALLEGED, IN CONNECTION WITH THE PERFORMANCE OF WORK ON THIS PROJECT, EXCEPTING FOR LIABILITY ARISING FROM THE SOLE NEGLIGENCE OF THE OWNER OR ENGINEER.

GEOTECHNICAL ENGINEER AND ENGINEERING GEOLOGIST STATEMENT

THIS PLAN HAS BEEN REVIEWED AND CONFORMS TO RECOMMENDATIONS OF SOILS ENGINEERING/GEOLOGIC REPORTS DATED MARCH 2020 WITH JUNE 2020 ADDENDUM.

DATE: _____

DATE: _____

SIGNATURE: _____

SIGNATURE: _____

CIVIL ENGINEER

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DATE OF ISSUE: MARCH 2022
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CAD DESIGN BY: J. AMAYA
CHECKED BY: D. HARICH
APPROVED BY: D. HARICH



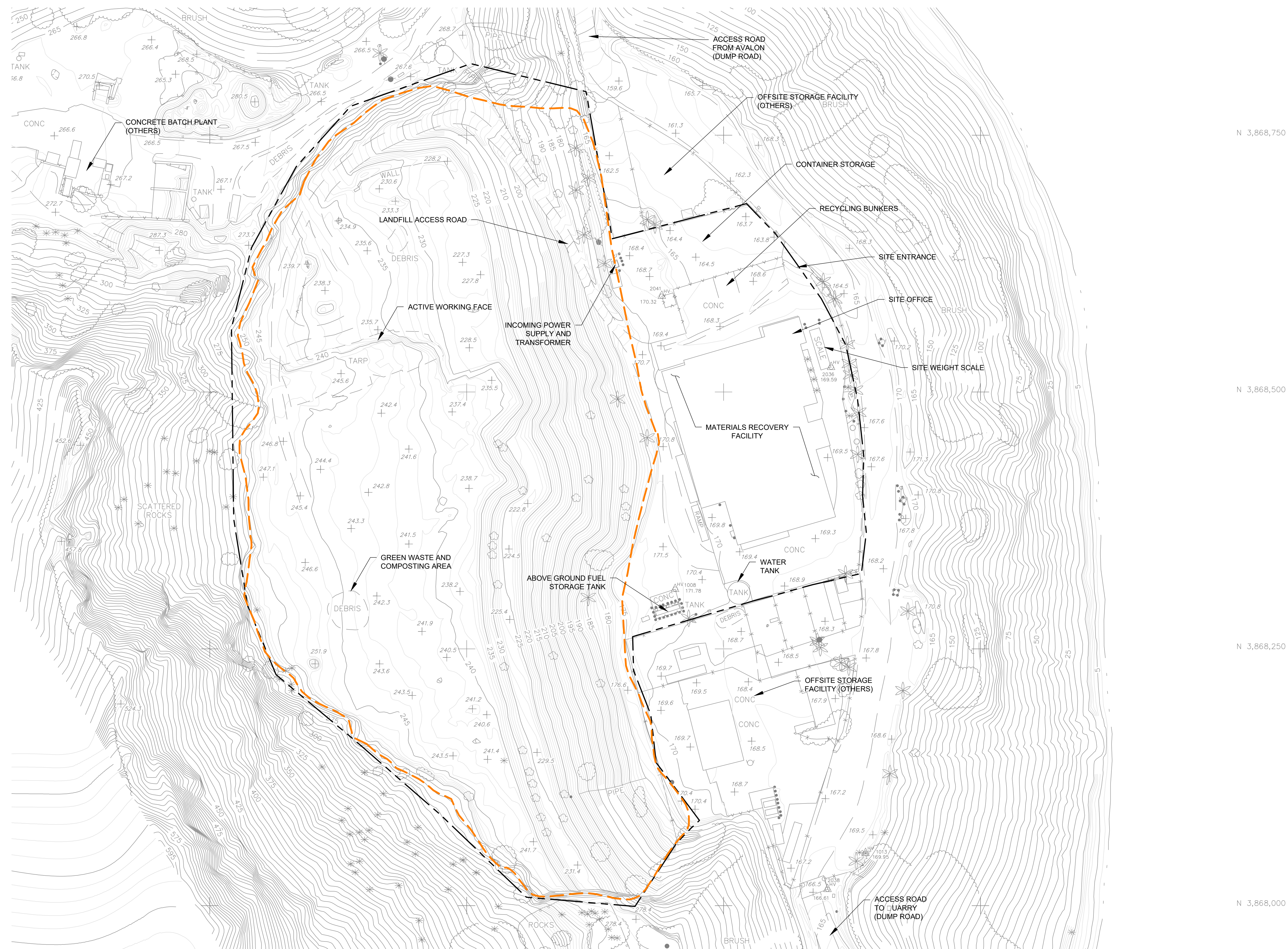
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PEBBLY BEACH LANDFILL, AVALON, CA
SITE LIFE OPTIMIZATION
DESIGN REPORT DRAWINGS
TITLE

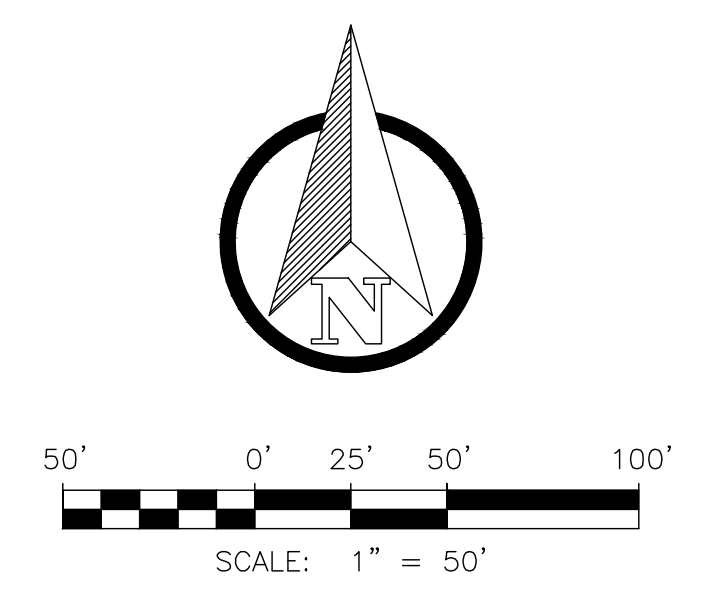
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PROJECT NO. S021.1196

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LEGEND	
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	LIMIT OF WASTE
	E. GROUND CONTOUR, FEET
	E. UNPAVED ROAD
	E. FENCE

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 N 3,868,500
 N 3,868,250
 N 3,868,000



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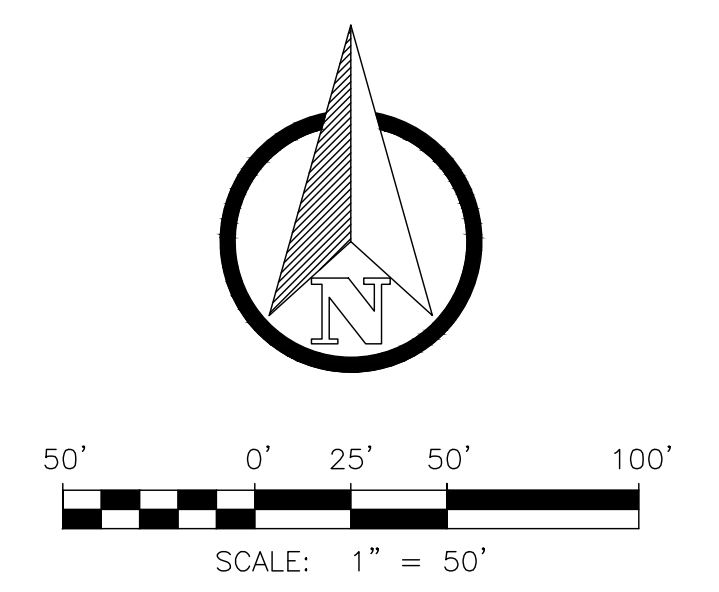
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 - E.G. FENCE
 - PROJECT GRADING LIMITS
 - GRADING HINGE
 - RETAINING WALL
 - DESIGN GRADE DIRECTION
 - DETAIL OR SECTION NUMBER
 - SHEET NUMBER FOR DETAIL LOCATION

- FILL PATTERN**
- GRADING WASTE LIMITS
 - PROPOSED ACCESS AND BENCHES
 - CONCRETE CHANNEL

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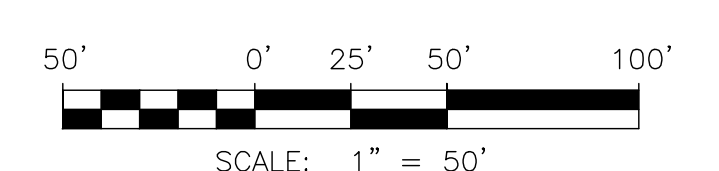
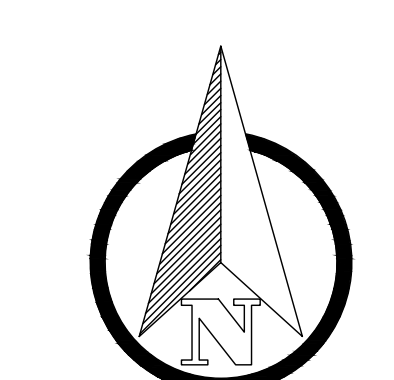
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C01
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 - FILL DEPTH IN FEET (16' TO 10' GRID)

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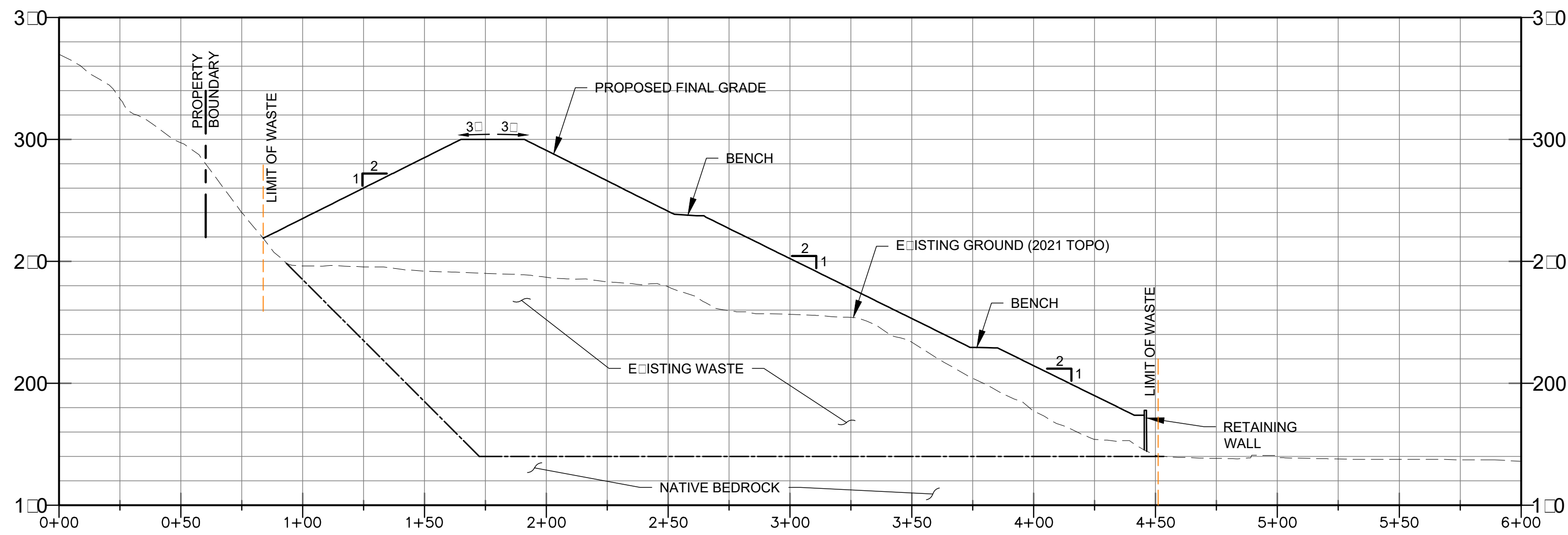


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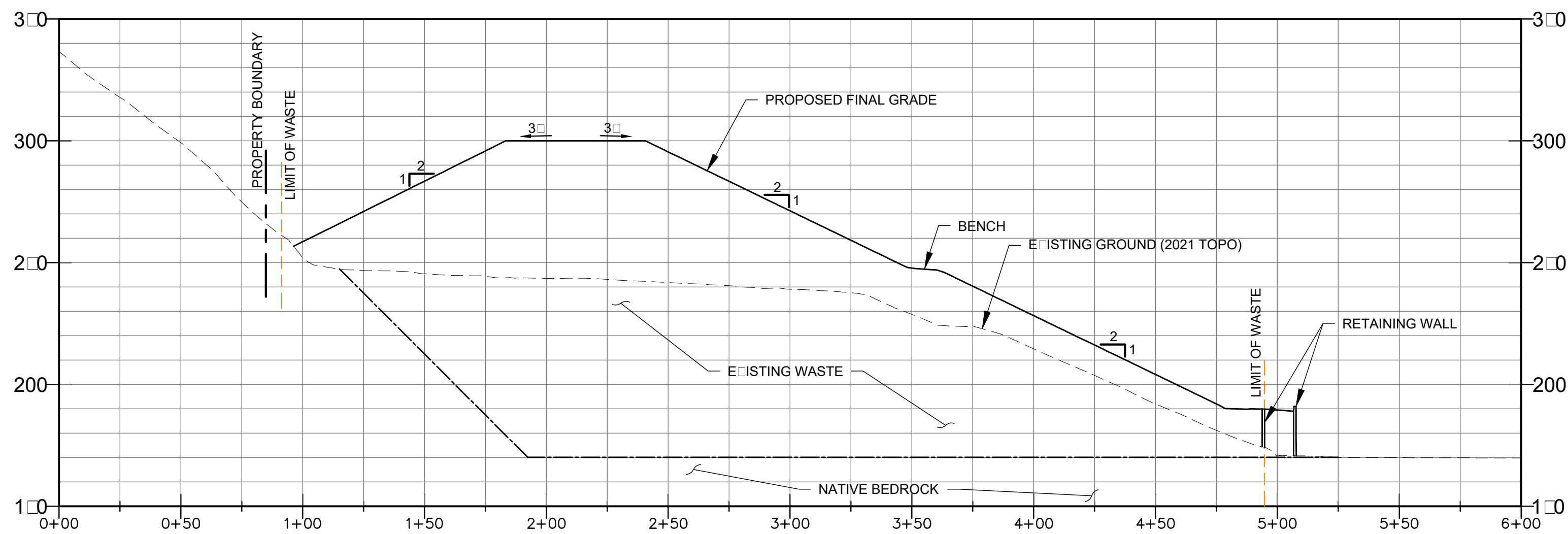
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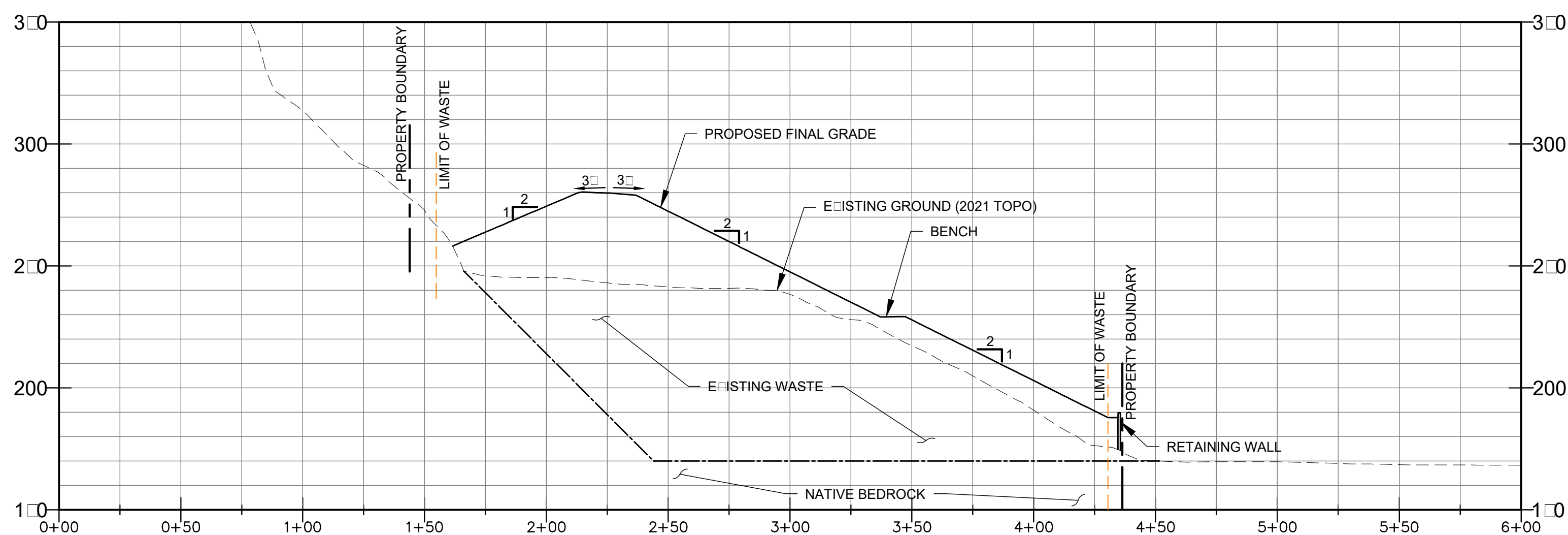
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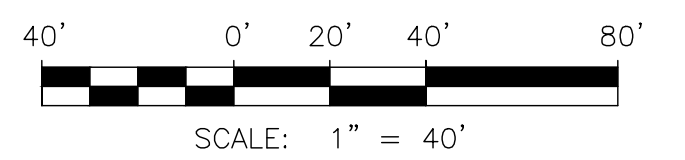
A SECTION



B SECTION



C SECTION



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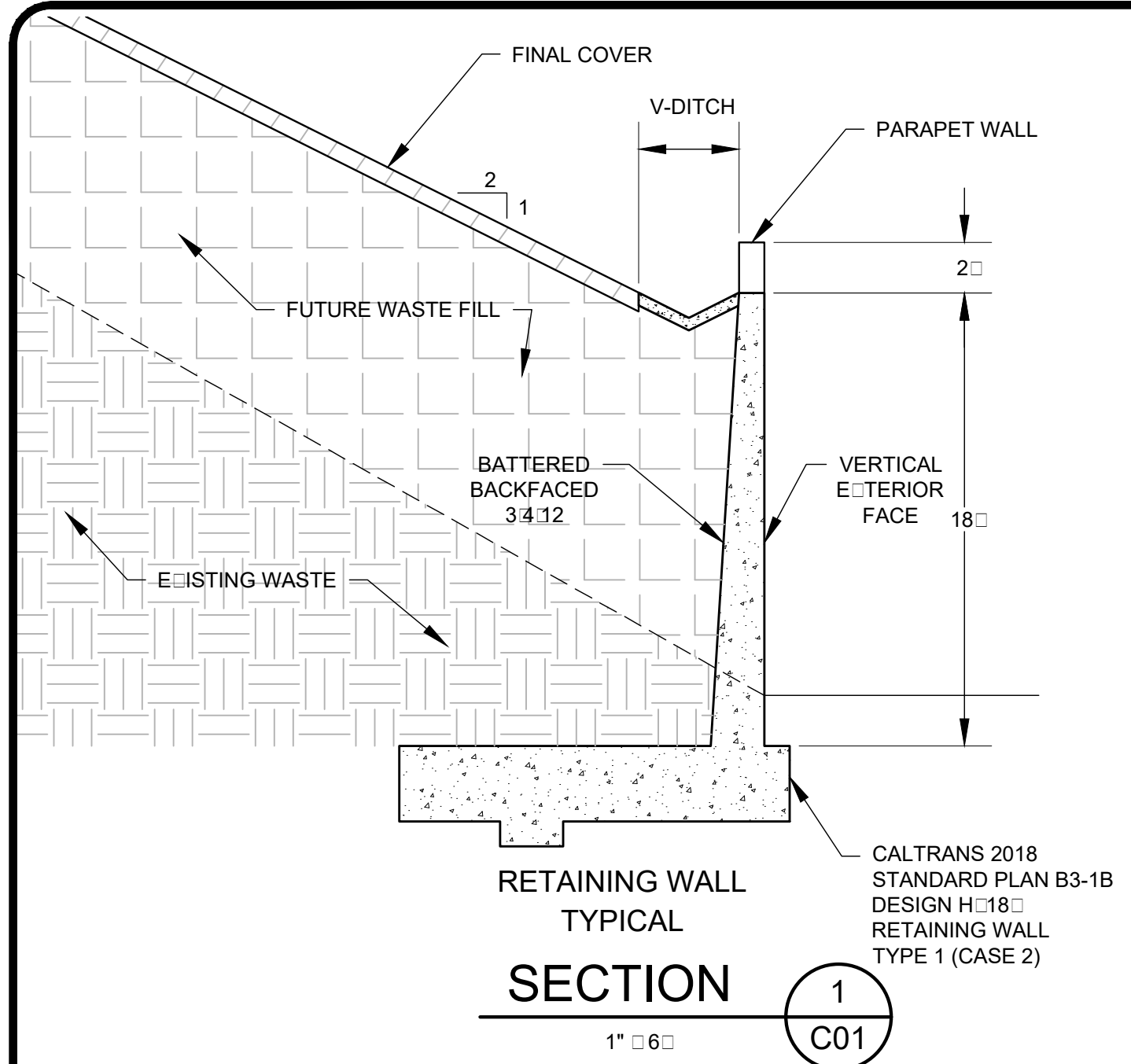
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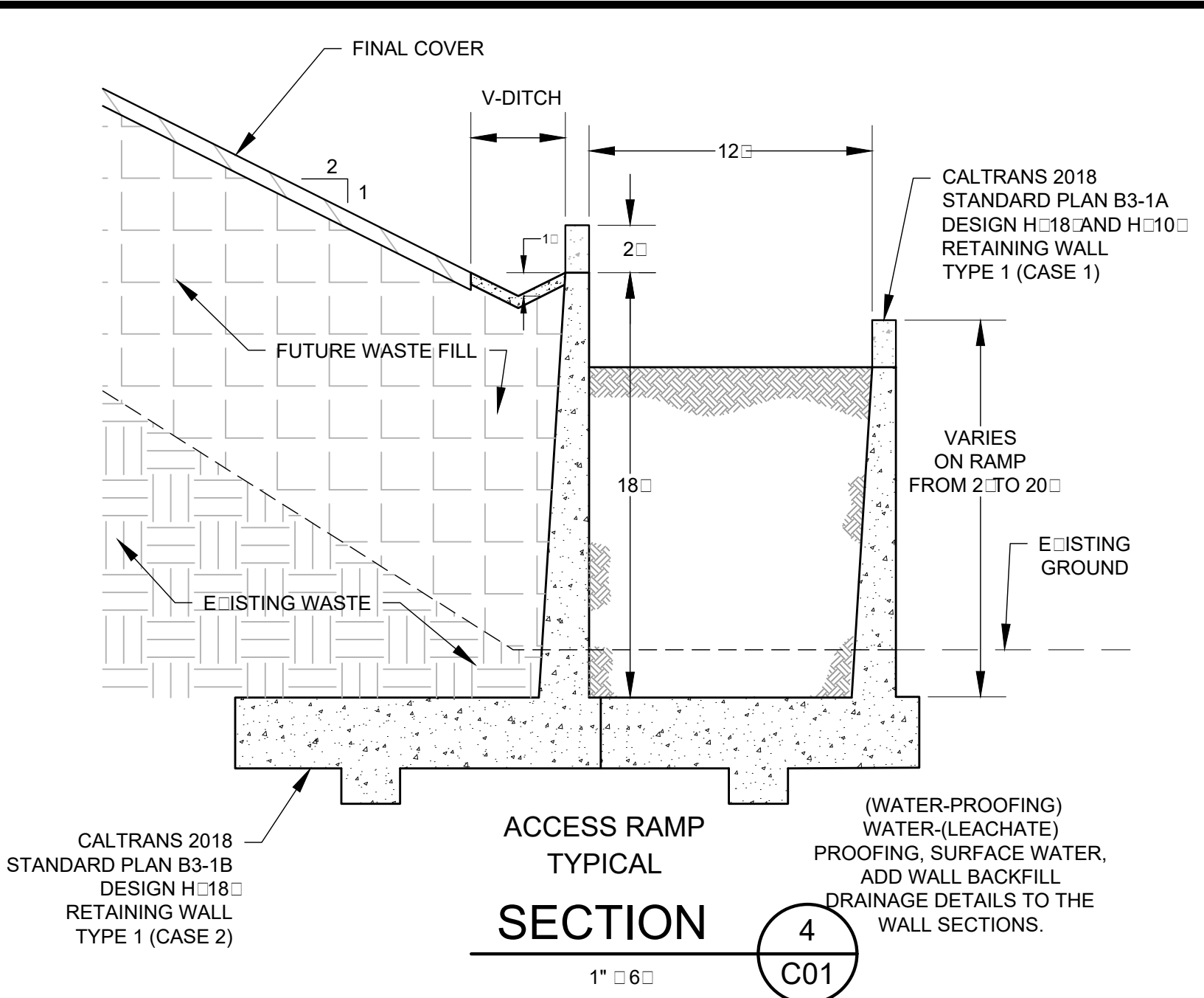
PEBBLY BEACH LANDFILL, AVALON, CA
 SITE LIFE OPTIMIZATION
 DESIGN REPORT DRAWINGS
 REPRESENTATIVE CROSS SECTION

DWG NO. C03
 PROJECT NO. S021.1196

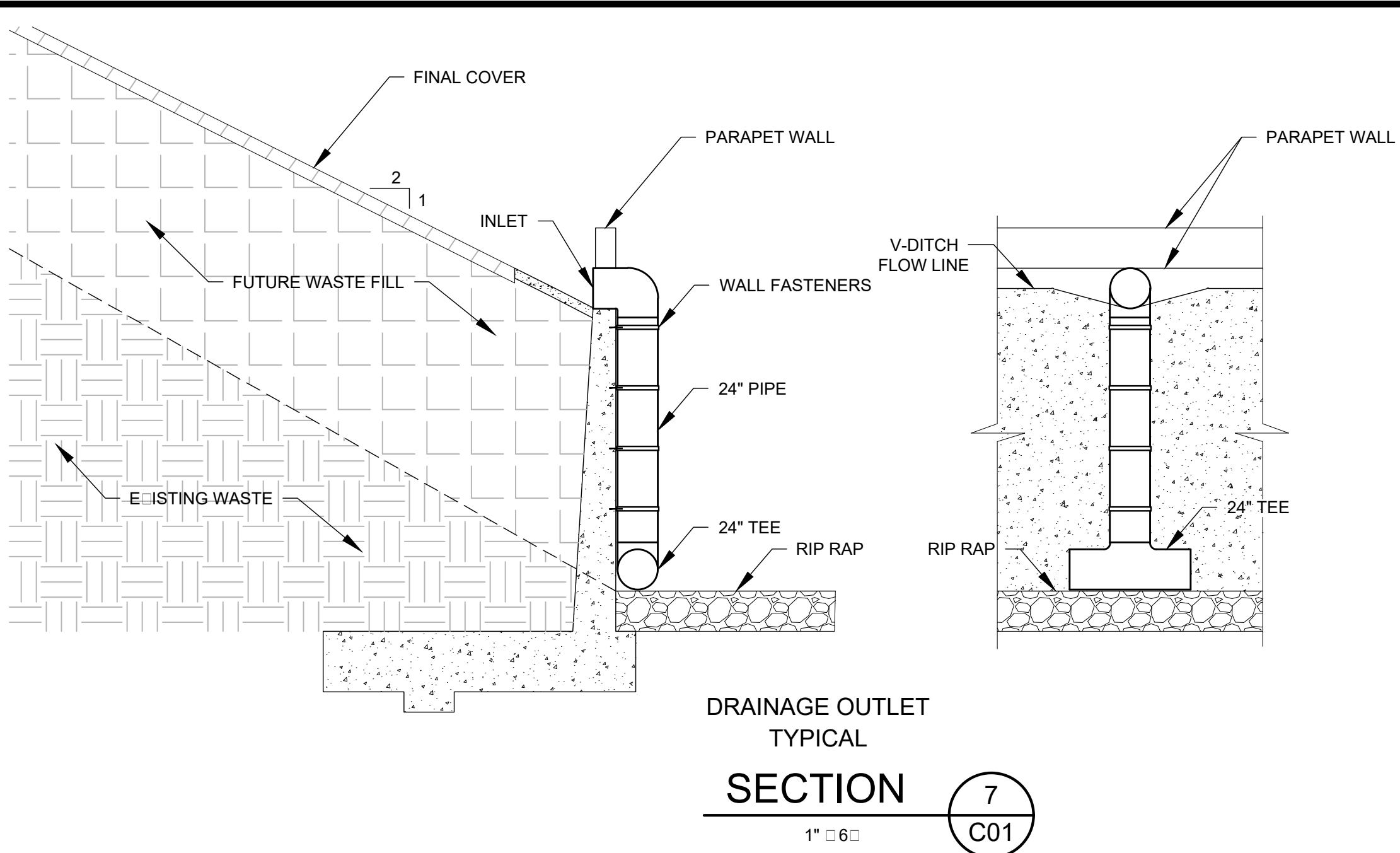
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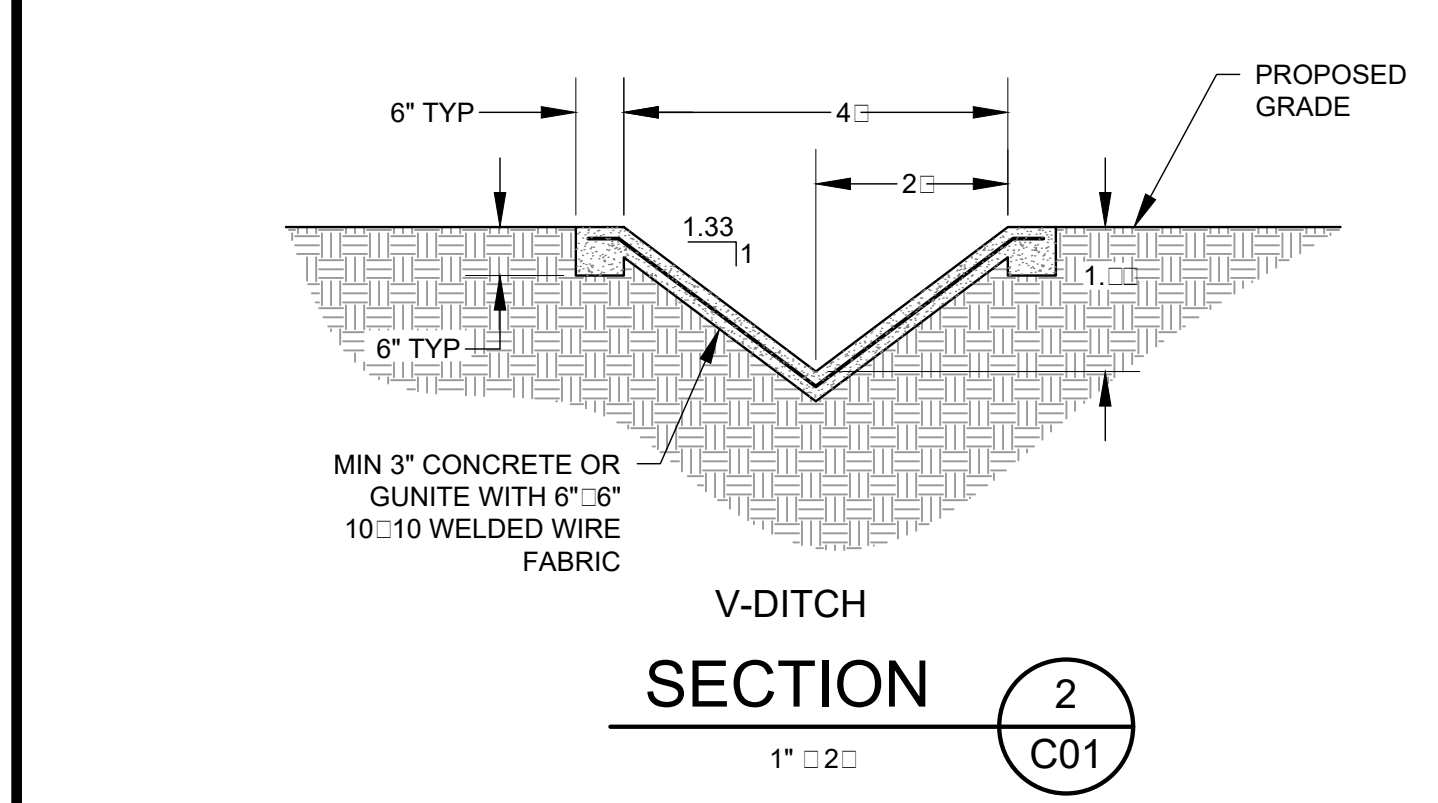
RETAINING WALL TYPICAL SECTION 1
1" = 6'
C01



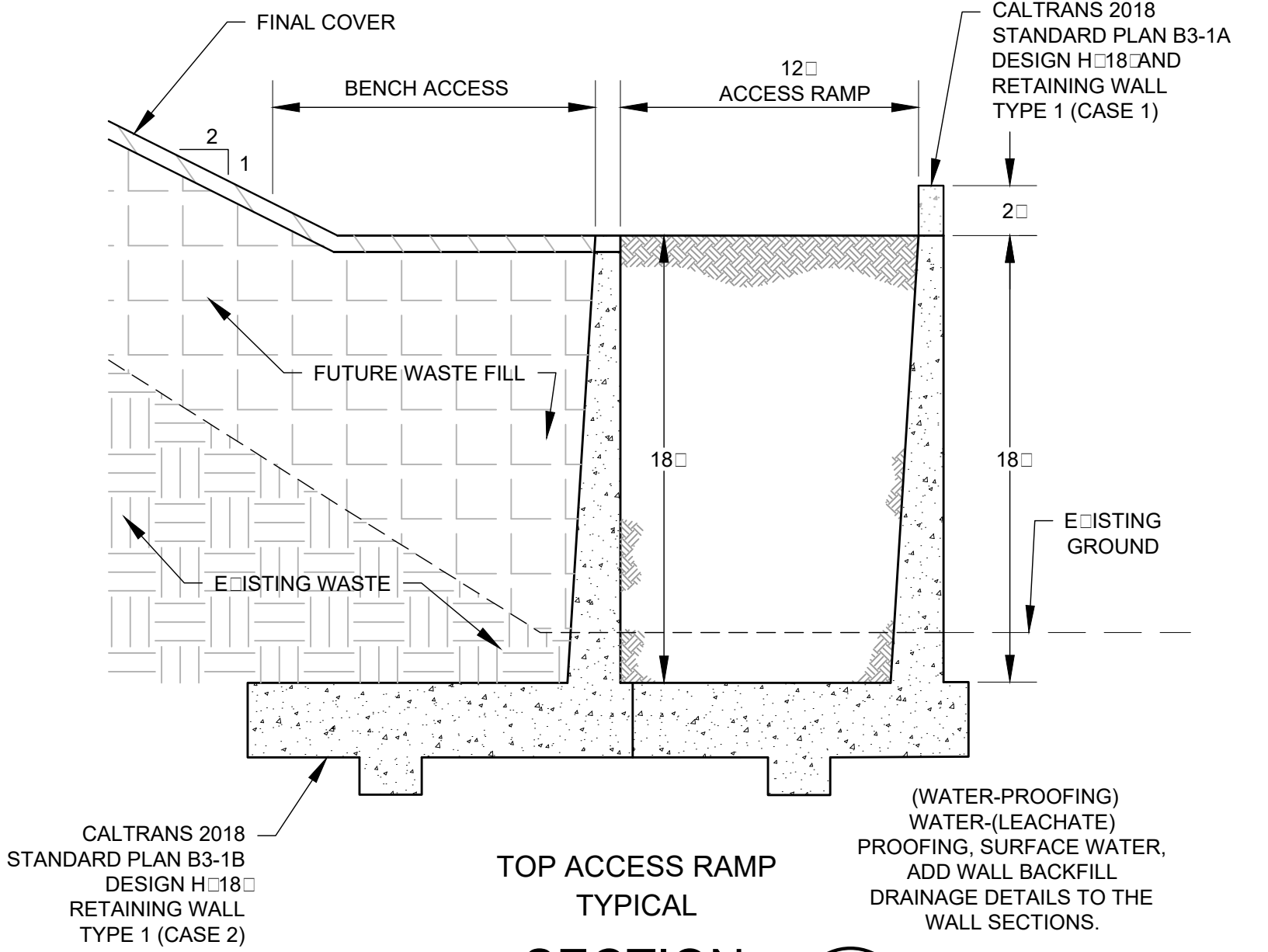
ACCESS RAMP TYPICAL SECTION 4
1" = 6'
C01



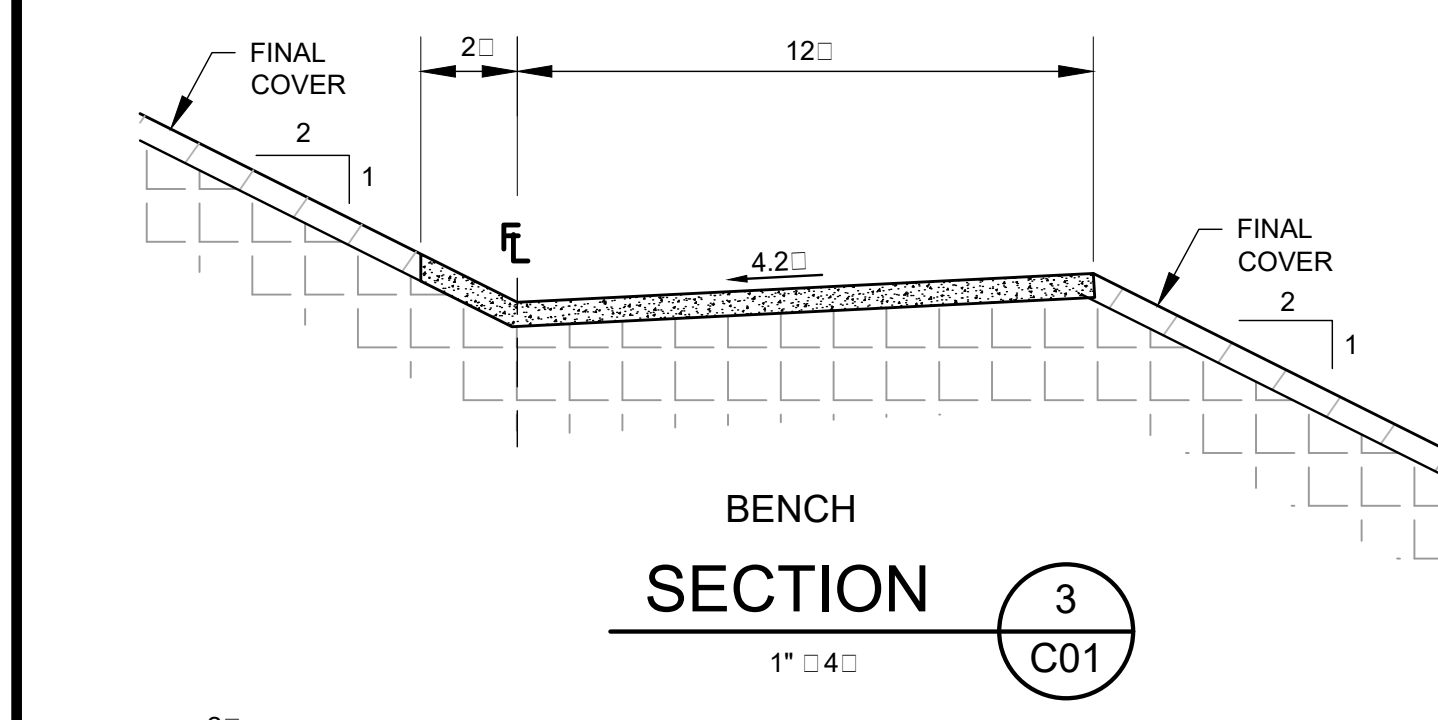
DRAINAGE OUTLET TYPICAL SECTION 7
1" = 6'
C01



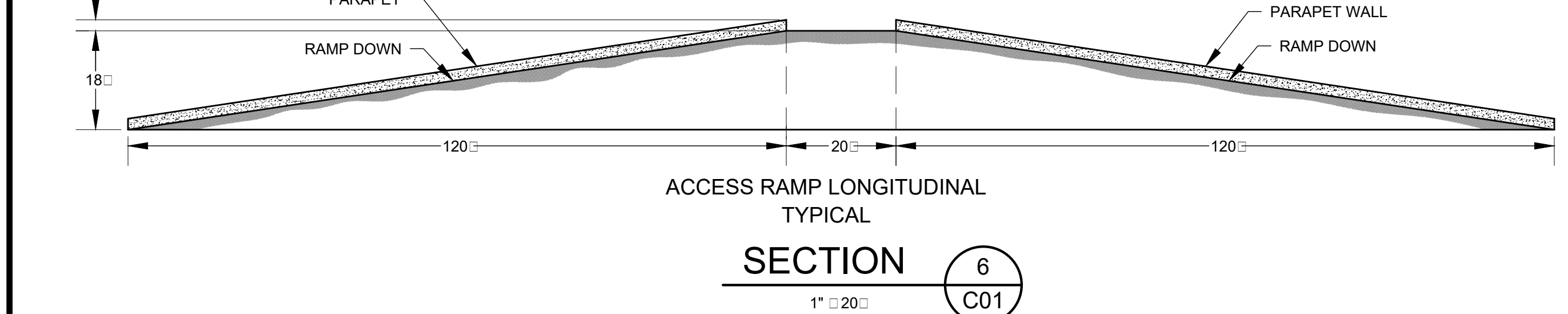
V-DITCH SECTION 2
1" = 2'
C01



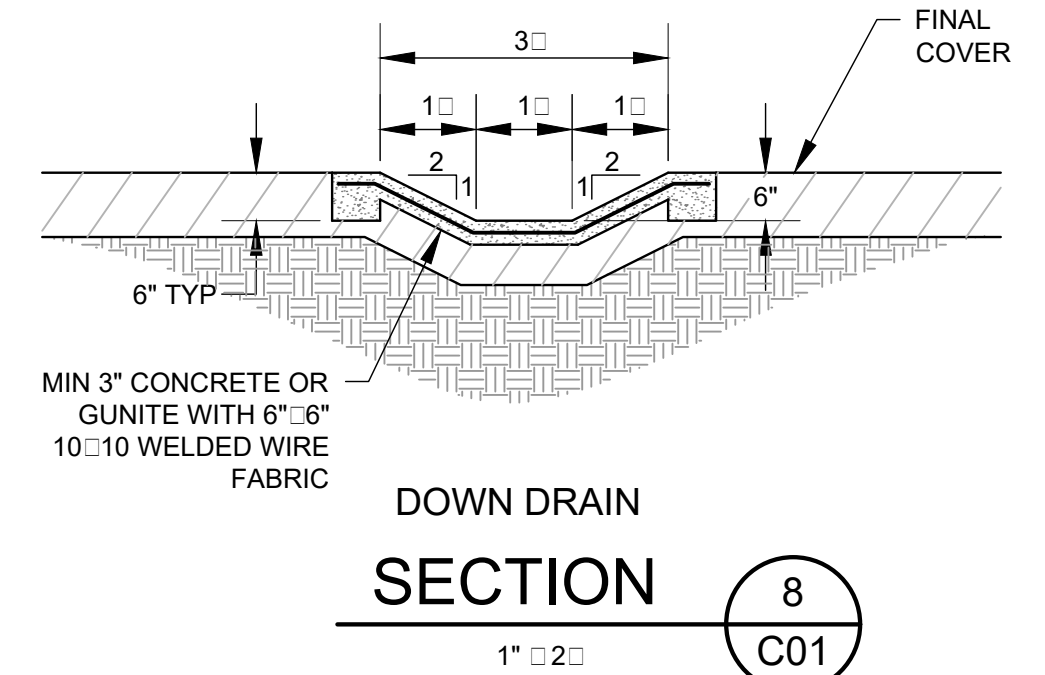
TOP ACCESS RAMP TYPICAL SECTION 4
1" = 6'
C01



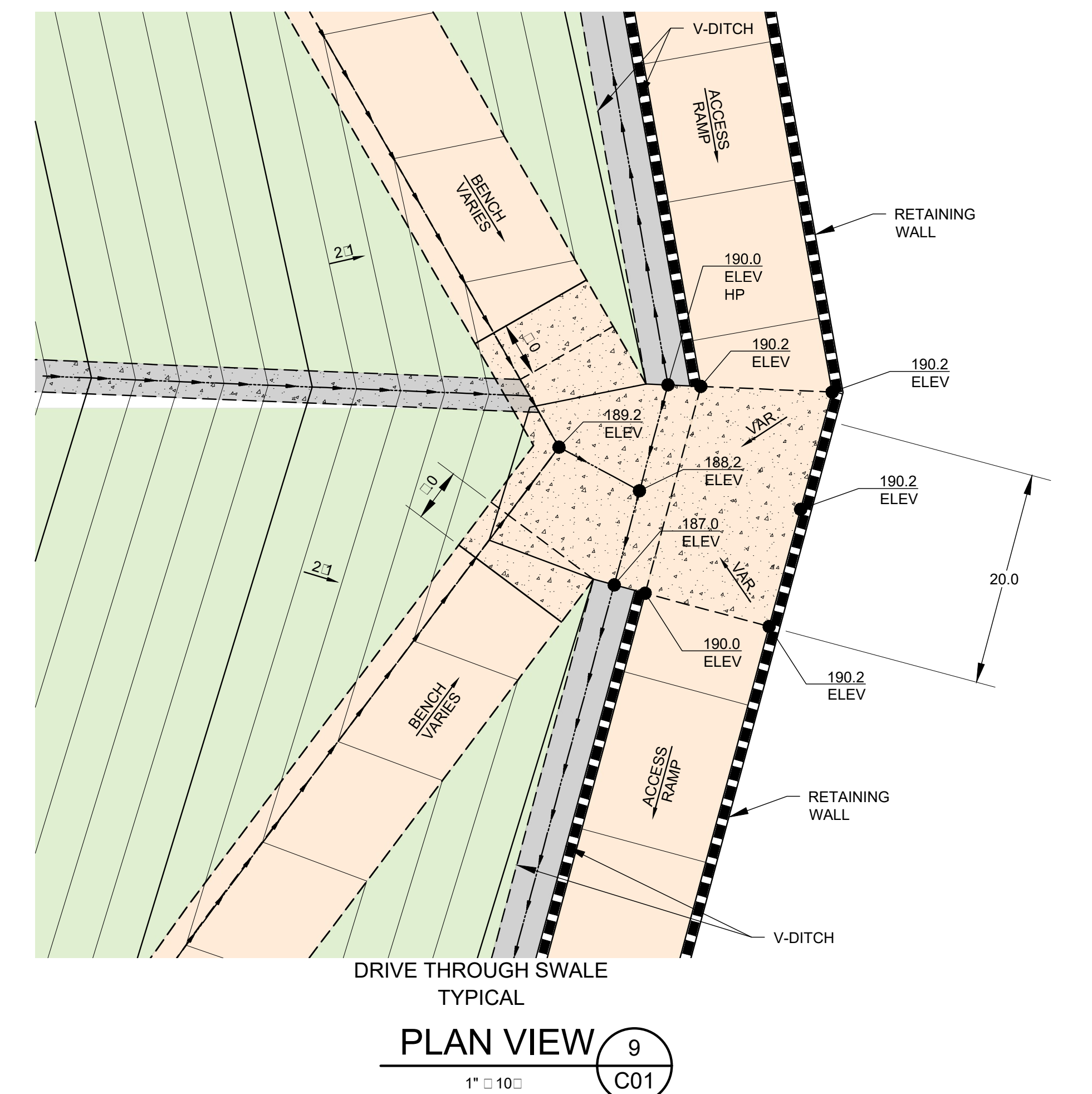
BENCH SECTION 3
1" = 4'
C01



ACCESS RAMP LONGITUDINAL TYPICAL SECTION 6
1" = 20'
C01



DOWN DRAIN SECTION 8
1" = 2'
C01

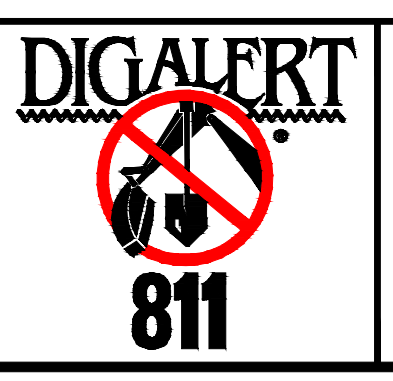


DRIVE THROUGH SWALE TYPICAL PLAN VIEW 9
1" = 10'
C01

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 CAD DESIGN BY: J. AMAYA
 CHECKED BY: D. HARICH
 APPROVED BY: D. HARICH



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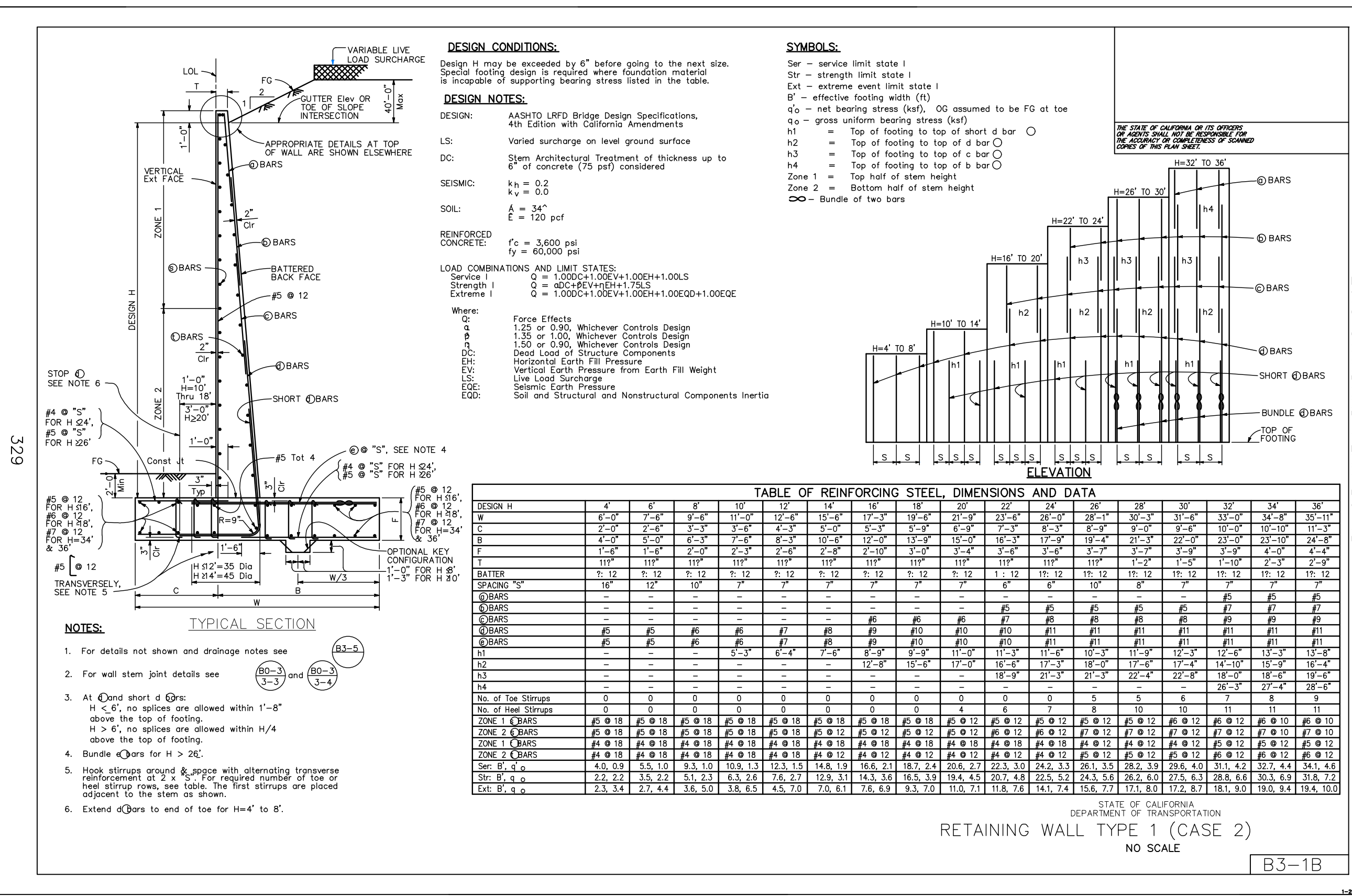
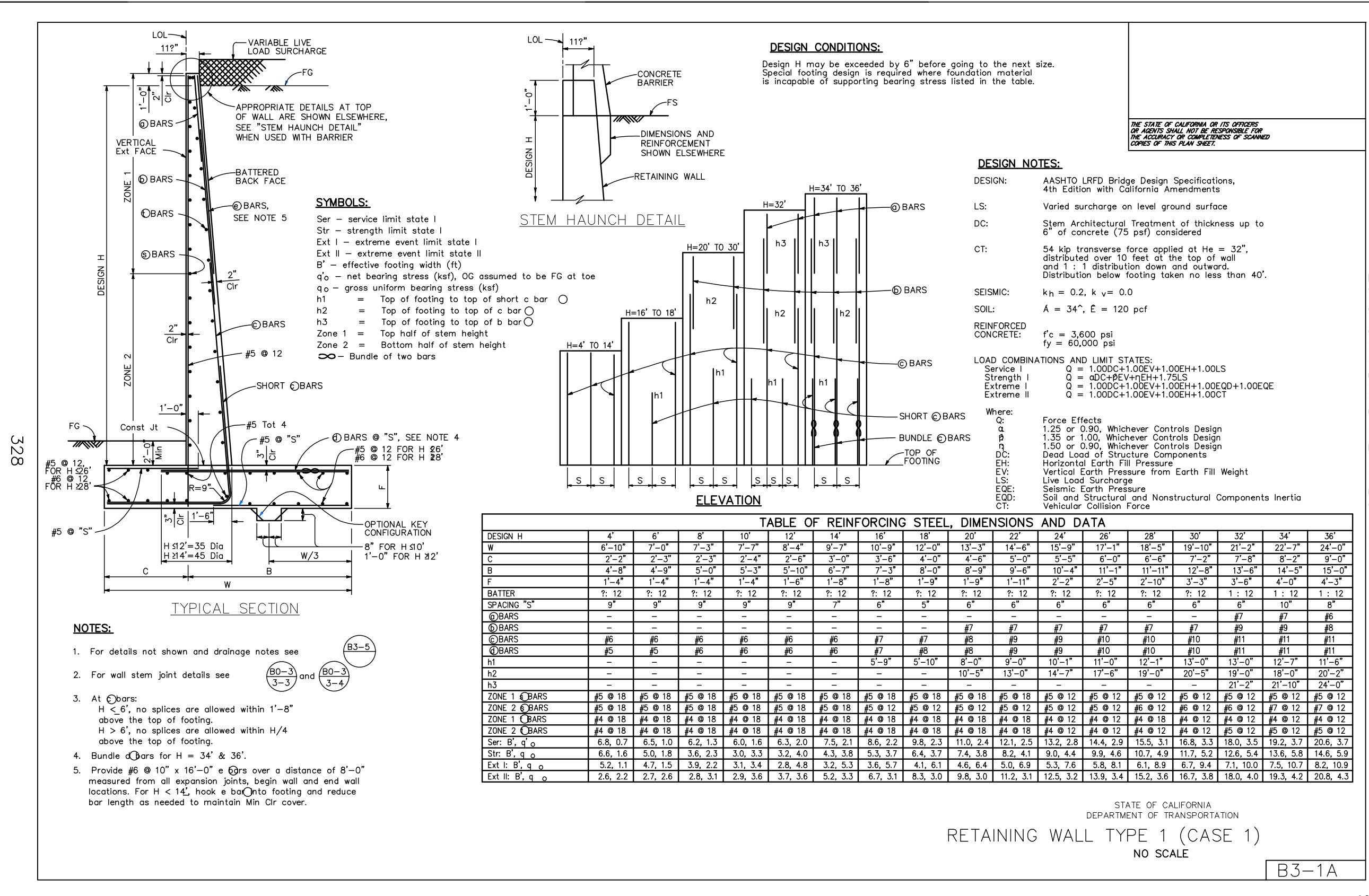
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PEBBLY BEACH LANDFILL, AVALON, CA
 SITE LIFE OPTIMIZATION
 DESIGN REPORT DRAWINGS
 GRADING DETAILS

DWG. NO. C04
 PROJECT NO. S021.1196

ISSUED FOR PERMIT
 REFERENCE AERIAL TOPO BASED ON SEPTEMBER 29, 2021 AERIAL SURVEY BY DON READ CORPORATION

P:\SITES\PEBBLY BEACH\FEASIBILITY 2021\DWG SETS\S021.1196-ACS-06-DETAILS.DWG March 8, 2022 - 12:40 PM BY: JORGE AMAYA



P:\SITES\PEBBLY BEACH\FEASIBILITY 2021\DWG SETS\SO21.1196-ACS-07-DETAILS.DWG March 8, 2022 - 12:40 PM BY: JORGE AMAYA

2018 STANDARD PLAN B3-1A

2018 STANDARD PLAN B3-1B

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 DESIGNED BY: D HARICH
 CAD DESIGN BY: J AMAYA
 CHECKED BY: D HARICH
 APPROVED BY: D HARICH

DIGALERT

811

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STATE OF CALIFORNIA
 DEPARTMENT OF TRANSPORTATION
RETAINING WALL TYPE 1 (CASE 2)
 NO SCALE
 B3-1B

CR&R
 INCORPORATED
 environmental services

1 DUMP ROAD
 AVALON, CALIFORNIA 90704

PEBBLY BEACH LANDFILL, AVALON, CA
 SITE LIFE OPTIMIZATION
 DESIGN REPORT DRAWINGS

GRADING DETAILS

DWG NO.
C0
 PROJECT NO.
 SO21.1196

ISSUED FOR PERMIT
 REFERENCE AERIAL TOPO BASED ON SEPTEMBER 29, 2021 AERIAL SURVEY BY DON READ CORPORATION

Appendix B

Seismic Hazard and Refuse Slope Stability Analysis

Appendix B.1

Echo of Input Data for the EZ- FRISK Program

```
1 *****
2 ***** EZ-FRISK *****
3 ***** SEISMIC HAZARD ANALYSIS DEFINITION *****
4 ***** FUGRO CONSULTANTS, INC. *****
5 ***** WALNUT CREEK, CA USA *****
6 *****
7
8 PROGRAM VERSION
9 EZ-FRISK 8.07 Build 044
10
11 ANALYSIS TITLE:
12 Seismic Hazard Analysis 2 (Vs = 505 m/s)
13
14 ANALYSIS TYPE:
15 Single Site Analysis
16
17 SITE COORDINATES
18 Latitude 33.3298
19 Longitude -118.31
20
21 INTENSITY TYPE: Spectral Response @ 5% Damping
22
23 HAZARD DEAGGREGATION
24 Status: OFF
25
26 SOIL AMPLIFICATION
27 Method: Do not use soil amplification
28
29 ATTENUATION EQUATION SITE PARAMETERS
30 Depth[Vs=1000m/s] (m): 40
31 Estimate Z1 from Vs30 for CY NGA: 1
32 Vs30 (m/s): 505
33 Vs30 Is Measured: 0
34 Z25 (km): 2
35
36 AMPLITUDES - Acceleration (g)
37 0.0001
38 0.001
39 0.01
40 0.02
41 0.05
42 0.07
43 0.1
44 0.2
45 0.3
46 0.4
47 0.5
48 0.7
49 1
50 2
51 3
52
53 PERIODS (s)
54 PGA
55 0.05
56 0.1
57 0.2
58 0.3
59 0.4
60 0.5
61 0.75
62 1
63 2
64 3
```

```
65 4
66
67 DETERMINISTIC FRACTILES
68 0.5
69 0.84
70
71 PLOTTING PARAMETERS
72 Period at which to plot PGA: 0.005
73
74 CALCULATIONAL PARAMETERS
75 Fault Seismic Sources -
76 Maximum inclusion distance : 200 km
77 Down dip integration increment : 1 km
78 Horizontal integration increment : 1 km
79 Number rupture length per earthquake : 1
80 Subduction Interface Seismic Sources -
81 Maximum inclusion distance : 1000 km
82 Down dip integration increment : 5 km
83 Horizontal integration increment : 20 km
84 Number rupture length per earthquake : 1
85 Subduction Slab Seismic Sources -
86 Maximum inclusion distance : 1000 km
87 Down dip integration increment : 5 km
88 Horizontal integration increment : 20 km
89 Number rupture length per earthquake : 1
90 Area Seismic Sources -
91 Maximum inclusion distance : 200 km
92 Vertical integration increment : 3 km
93 Number of rupture azimuths : 3
94 Minimum epicentral distance step : 0.5 km
95 Maximum epicentral distance step : 10 km
96 Gridded Seismic Sources -
97 Maximum inclusion distance : 200 km
98 Default number of rupture azimuths : 20
99 Maximum distance for default azimuths : 40 km
100 Minimum distance for one azimuth : 150
101 Use binned calculations if possible : true
102 Bins per decade in distance (km) : 20
103 All Seismic Sources -
104 Magnitude integration step : 0.1 M
105 Apply magnitude scaling : NO
106 Include near-source directivity : NO
107
108 ATTENUATION EQUATIONS
109
110 Name: Atkinson-Boore (2003) Cascadia Subduction USGS 2008
111 Database: C:\Program Files (x86)\EZ-FRISK 8.07\Files\standard.bin-attendb
112 Base: Atkinson-Boore 2003-3
113 Truncation Type: USGS 2008 NSHM Truncation
114 Truncation Value: 3
115 Magnitude Scale: Moment Magnitude
116 Distance Type: Distance To Rupture
117
118 Name: Atkinson-Boore (2003) Worldwide Subduction USGS 2008
119 Database: C:\Program Files (x86)\EZ-FRISK 8.07\Files\standard.bin-attendb
120 Base: Atkinson-Boore 2003-3
121 Truncation Type: USGS 2008 NSHM Truncation
122 Truncation Value: 3
123 Magnitude Scale: Moment Magnitude
124 Distance Type: Distance To Rupture
125
126 Name: Boore-Atkinson (2008) NGA USGS 2008
127 Database: C:\Program Files (x86)\EZ-FRISK 8.07\Files\standard.bin-attendb
128 Base: Boore-Atkinson 2007 NGA
```

129 Truncation Type: Trunc Sigma*Value
 130 Truncation Value: 3
 131 Magnitude Scale: Moment Magnitude
 132 Distance Type: Horizontal Distance To Rupture
 133
 134 Name: Campbell-Bozorgnia (2008) NGA USGS 2008
 135 Database: C:\Program Files (x86)\EZ-FRISK 8.07\Files\standard.bin-attendb
 136 Base: Campbell-Bozorgnia 2008 NGA
 137 Truncation Type: Trunc Sigma*Value
 138 Truncation Value: 3
 139 Magnitude Scale: Moment Magnitude
 140 Distance Type: Distance To Rupture
 141
 142 Name: Chiou-Youngs (2007) NGA USGS 2008
 143 Database: C:\Program Files (x86)\EZ-FRISK 8.07\Files\standard.bin-attendb
 144 Base: Chiou-Youngs 2008 NGA
 145 Truncation Type: Trunc Sigma*Value
 146 Truncation Value: 3
 147 Magnitude Scale: Moment Magnitude
 148 Distance Type: Distance To Rupture
 149
 150 Name: Youngs (1997) Subduction USGS 2008
 151 Database: C:\Program Files (x86)\EZ-FRISK 8.07\Files\standard.bin-attendb
 152 Base: Vs30Mixer - 2 Inputs
 153 Truncation Type: USGS 2008 NSHM Truncation
 154 Truncation Value: 3
 155 Magnitude Scale: Moment Magnitude
 156 Distance Type: Distance To Rupture
 157

158 SEISMIC SOURCE SUMMARY TABLE

Source	Region	Closest Distance	Determinis Magnit
162 1003 Dutchman Draw fault	USGS 2008 Arizona	591.63	6.4
163 944 Algodones fault zone	USGS 2008 Arizona	360.69	6.5
164 951 Big Chino fault	USGS 2008 Arizona	530.95	7.0
165 955 Aubrey fault zone	USGS 2008 Arizona	523.53	7.1
166 997bcd Sevier-Toroweap fault zone (southern)	USGS 2008 Arizona	527.01	7.5
167 998cd Hurricane fault zone (central)	USGS 2008 Arizona	554.44	7.4
168 998ef Hurricane fault zone (southern)	USGS 2008 Arizona	494.79	7.4
169 Anacapa-Dume	USGS 2008 California	79.84	7.2
170 Bartlett Springs	USGS 2008 California	727.51	7.3
171 Battle Creek	USGS 2008 California	850.76	6.7
172 Big Lagoon-Bald Mtn	USGS 2008 California	989.86	7.5
173 Birch Creek	USGS 2008 California	405.26	6.6
174 Blackwater	USGS 2008 California	220.77	7.1
175 Brawley Gridded, Strike Slip	USGS 2008 California	214.46	6.5
176 Brawley Gridded,Normal	USGS 2008 California	214.46	6.5
177 Burnt Mtn	USGS 2008 California	184.18	6.8
178 Calaveras	USGS 2008 California	479.23	7.0
179 Calico-Hidalgo	USGS 2008 California	220.08	7.4
180 California Gridded	USGS 2008 California	0.00	7.0
181 California Gridded Deep	USGS 2008 California	586.52	7.2
182 Casmalia (Orcutt Frontal)	USGS 2008 California	251.88	6.7
183 Cedar Mtn-Mahogany Mtn	USGS 2008 California	962.91	7.1
184 Channel Islands Thrust	USGS 2008 California	117.39	7.3
185 Chino	USGS 2008 California	80.70	6.8
186 Clamshell-Sawpit	USGS 2008 California	98.50	6.7
187 Cleghorn	USGS 2008 California	134.12	6.8
188 Collayomi	USGS 2008 California	721.61	6.7
189 Coronado Bank	USGS 2008 California	36.90	7.4
190 Cucamonga	USGS 2008 California	103.44	6.7
191 Death Valley	USGS 2008 California	305.50	7.9
192 Deep Springs	USGS 2008 California	434.11	6.8

193	Earthquake Valley	USGS 2008	California	161.06	6.8
194	Elmore Ranch	USGS 2008	California	230.80	6.7
195	Elsinore	USGS 2008	California	77.10	7.8
196	Elysian Park (Upper)	USGS 2008	California	82.33	6.7
197	Eureka Peak	USGS 2008	California	195.07	6.7
198	Fickle Hill	USGS 2008	California	961.77	7.1
199	Fish Slough	USGS 2008	California	448.11	6.8
200	Garlock	USGS 2008	California	174.39	7.7
201	Gillem-Big Crack	USGS 2008	California	968.00	6.8
202	Gravel Hills-Harper Lk	USGS 2008	California	213.10	7.1
203	Great Valley 1	USGS 2008	California	751.24	6.8
204	Great Valley 10	USGS 2008	California	423.79	6.5
205	Great Valley 11	USGS 2008	California	399.20	6.6
206	Great Valley 12	USGS 2008	California	382.81	6.4
207	Great Valley 13 (Coalinga)	USGS 2008	California	348.87	7.1
208	Great Valley 14 (Kettleman Hills)	USGS 2008	California	317.81	7.2
209	Great Valley 2	USGS 2008	California	732.78	6.5
210	Great Valley 3, Mysterious Ridge	USGS 2008	California	683.89	7.1
211	Great Valley 4a, Trout Creek	USGS 2008	California	666.56	6.6
212	Great Valley 4b, Gordon Valley	USGS 2008	California	640.94	6.8
213	Great Valley 5, Pittsburg Kirby Hills	USGS 2008	California	610.78	6.7
214	Great Valley 7	USGS 2008	California	523.73	6.9
215	Great Valley 8	USGS 2008	California	483.79	6.8
216	Great Valley 9	USGS 2008	California	445.53	6.8
217	Green Valley Connected	USGS 2008	California	606.96	6.8
218	Greenville Connected	USGS 2008	California	547.98	7.0
219	Greenville Connected U	USGS 2008	California	547.98	7.0
220	Hartley Springs	USGS 2008	California	481.83	6.8
221	Hat Creek-McArthur-Mayfield	USGS 2008	California	854.13	7.2
222	Hayward-Rodgers Creek	USGS 2008	California	559.27	7.3
223	Helendale-So Lockhart	USGS 2008	California	176.34	7.4
224	Hilton Creek	USGS 2008	California	460.76	6.9
225	Hollywood	USGS 2008	California	84.29	6.7
226	Holser, alt 1	USGS 2008	California	114.71	6.8
227	Honey Lake	USGS 2008	California	756.53	7.0
228	Hosgri	USGS 2008	California	280.53	7.3
229	Hunter Mountain Connected	USGS 2008	California	284.44	7.6
230	Hunter Mountain-Saline Valley	USGS 2008	California	358.60	7.2
231	Hunting Creek-Berryessa	USGS 2008	California	668.17	7.1
232	Imp Extensional Gridded, Char, Normal	USGS 2008	California	56.59	7.0
233	Imp Extensional Gridded, Char, Strike Slip	USGS 2008	California	56.59	7.0
234	Imp Extensional Gridded, GR, Normal	USGS 2008	California	56.59	7.0
235	Imp Extensional Gridded, GR, Strike Slip	USGS 2008	California	56.59	7.0
236	Imperial	USGS 2008	California	260.51	7.0
237	Independence	USGS 2008	California	353.05	7.2
238	Johnson Valley (No)	USGS 2008	California	200.02	6.9
239	Laguna Salada	USGS 2008	California	235.82	7.3
240	Landers	USGS 2008	California	197.84	7.4
241	Lenwood-Lockhart-Old Woman Springs	USGS 2008	California	191.10	7.5
242	Likely	USGS 2008	California	857.58	7.0
243	Lions Head	USGS 2008	California	233.59	6.8
244	Little Lake	USGS 2008	California	260.05	6.9
245	Little Salmon (Offshore)	USGS 2008	California	978.45	7.3
246	Little Salmon (Onshore)	USGS 2008	California	945.89	7.1
247	Little Salmon Connected	USGS 2008	California	945.89	7.5
248	Los Alamos-West Baseline	USGS 2008	California	214.99	6.9
249	Los Osos	USGS 2008	California	275.07	7.0
250	Maacama-Garberville	USGS 2008	California	703.62	7.4
251	Mad River	USGS 2008	California	959.64	7.2
252	Malibu Coast	USGS 2008	California	80.18	7.0
253	McKinleyville	USGS 2008	California	960.79	7.2
254	Mendocino Gridded, Reverse	USGS 2008	California	864.79	7.3
255	Mendocino Gridded, Strike Slip	USGS 2008	California	864.79	7.3
256	Mission Ridge-Arroyo Parida-Santa Ana	USGS 2008	California	146.85	6.9

257	Mojave Shear Gridded	USGS 2008	California	174.24	7.6
258	Mono Lake	USGS 2008	California	516.75	6.8
259	Monte Vista-Shannon	USGS 2008	California	534.76	6.5
260	Monterey Bay-Tularcitos	USGS 2008	California	447.20	7.3
261	Mount Diablo Thrust	USGS 2008	California	583.13	6.7
262	Northern San Andreas	USGS 2008	California	482.10	8.0
263	Newport-Inglewood	USGS 2008	California	46.75	7.5
264	North Channel	USGS 2008	California	150.62	6.8
265	North Frontal (East)	USGS 2008	California	167.36	7.0
266	North Frontal (West)	USGS 2008	California	144.96	7.2
267	North Tahoe	USGS 2008	California	658.71	6.7
268	Northridge	USGS 2008	California	99.27	6.9
269	Oak Ridge (Offshore)	USGS 2008	California	124.43	7.0
270	Oak Ridge (Onshore)	USGS 2008	California	118.47	7.2
271	Oak Ridge Connected	USGS 2008	California	115.75	7.4
272	Ortugalita	USGS 2008	California	445.31	7.1
273	Owens Valley	USGS 2008	California	353.83	7.3
274	Owl Lake	USGS 2008	California	285.02	6.7
275	Palos Verdes	USGS 2008	California	26.36	7.3
276	Palos Verdes Connected	USGS 2008	California	26.36	7.7
277	Panamint Valley	USGS 2008	California	284.44	7.4
278	Pinto Mtn	USGS 2008	California	167.61	7.3
279	Pisgah-Bullion Mtn-Mesquite Lk	USGS 2008	California	227.93	7.3
280	Pitas Point (Lower)-Montalvo	USGS 2008	California	142.82	7.3
281	Pitas Point (Lower, West)	USGS 2008	California	168.62	7.3
282	Pitas Point (Upper)	USGS 2008	California	159.37	6.9
283	Pitas Point Connected	USGS 2008	California	136.14	7.3
284	Pleito	USGS 2008	California	174.65	7.1
285	Point Reyes	USGS 2008	California	655.44	6.9
286	Puente Hills	USGS 2008	California	71.17	7.1
287	Puente Hills (Coyote Hills)	USGS 2008	California	67.30	6.9
288	Puente Hills (LA)	USGS 2008	California	70.56	7.0
289	Puente Hills (Santa Fe Springs)	USGS 2008	California	66.15	6.7
290	Quien Sabe	USGS 2008	California	463.70	6.6
291	Raymond	USGS 2008	California	88.37	6.8
292	Red Mountain	USGS 2008	California	144.89	7.4
293	Rinconada	USGS 2008	California	299.21	7.5
294	Robinson Creek	USGS 2008	California	544.47	6.7
295	Rose Canyon	USGS 2008	California	85.56	6.9
296	Round Valley	USGS 2008	California	435.63	7.1
297	Southern San Andreas	USGS 2008	California	129.57	8.2
298	SAF - creeping segment	USGS 2008	California	361.27	6.7
299	San Andreas Creeping Section Gridded	USGS 2008	California	324.47	6.0
300	San Cayetano	USGS 2008	California	128.78	7.2
301	San Gabriel	USGS 2008	California	109.84	7.3
302	San Gorgonio Shear Gridded	USGS 2008	California	140.81	7.6
303	San Gregorio Connected	USGS 2008	California	463.40	7.5
304	San Jacinto	USGS 2008	California	124.96	7.8
305	San Joaquin Hills	USGS 2008	California	38.75	7.1
306	San Jose	USGS 2008	California	88.34	6.7
307	San Juan	USGS 2008	California	252.62	7.1
308	San Luis Range (So Margin)	USGS 2008	California	244.96	7.2
309	Santa Cruz Island	USGS 2008	California	113.99	7.2
310	Santa Monica	USGS 2008	California	79.35	7.4
311	Santa Rosa Island	USGS 2008	California	165.93	6.9
312	Santa Susana, alt 1	USGS 2008	California	110.04	6.9
313	Santa Ynez (East)	USGS 2008	California	145.57	7.2
314	Santa Ynez (West)	USGS 2008	California	176.93	7.0
315	Santa Ynez Connected	USGS 2008	California	146.09	7.4
316	Shear 1 Gridded	USGS 2008	California	474.72	7.6
317	Shear 2 Gridded	USGS 2008	California	784.67	7.6
318	Shear 3 Gridded	USGS 2008	California	715.59	7.6
319	Shear 4 Gridded	USGS 2008	California	476.64	7.6
320	Sierra Madre	USGS 2008	California	96.64	7.2

Appendix B.2

De-Aggregation of Seismic Hazard at Zero Period Computed Using USGS Unified Hazard Tool

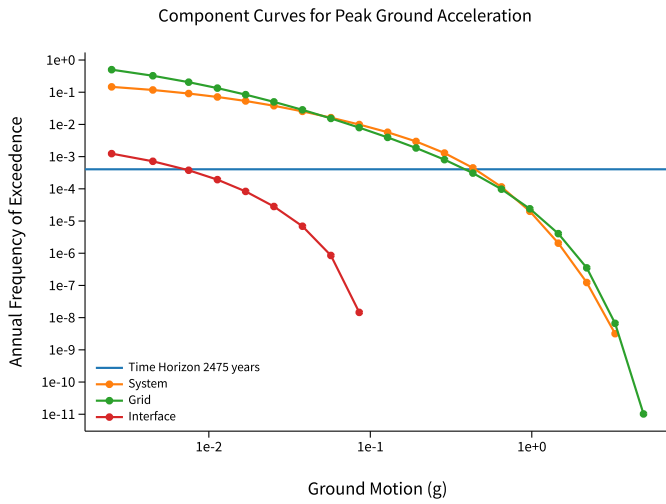
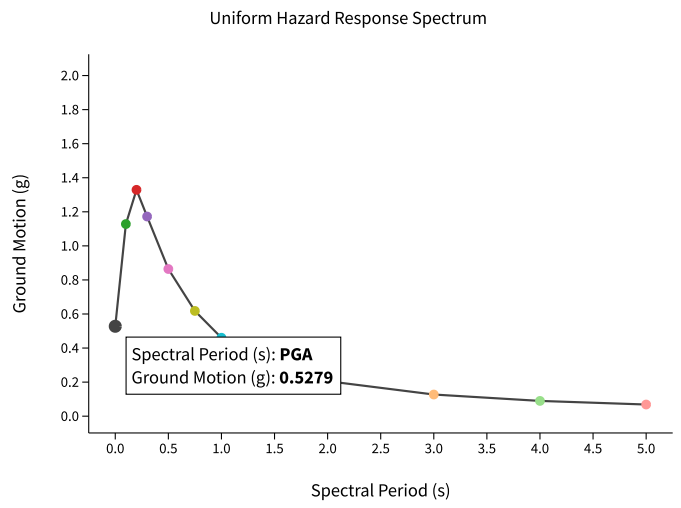
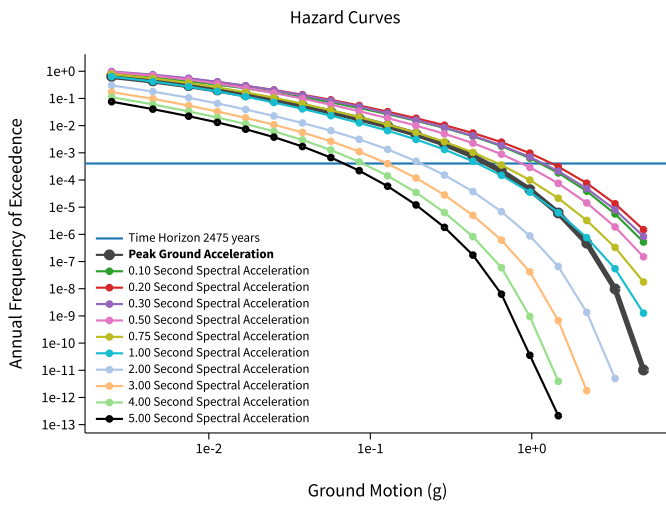
Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the [U.S. Seismic Design Maps web tools](#) (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

^ Input

Edition Dynamic: Conterminous U.S. 2014 (update) (v4.2.0) ▼	Spectral Period Peak Ground Acceleration ▼
Latitude Decimal degrees 33.3298	Time Horizon Return period in years 2475
Longitude Decimal degrees, negative values for western longitudes -118.31	
Site Class 537 m/s (Site class C) ▼	

^ Hazard Curve

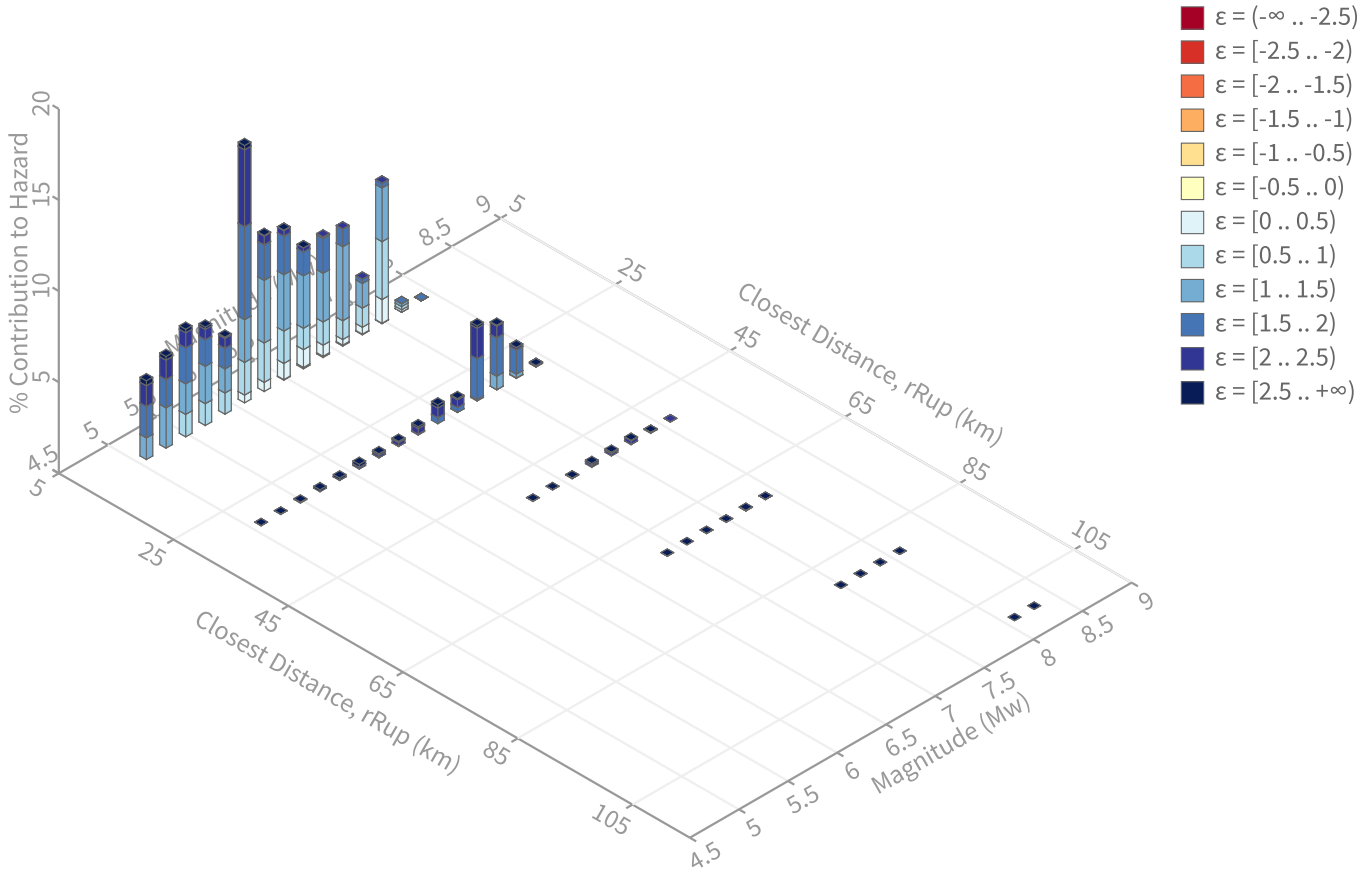


[View Raw Data](#)

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr⁻¹
PGA ground motion: 0.52786206 g

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.04 %

Mode (largest m-r bin)

m: 6.11
r: 11.66 km
ε₀: 1.62 σ
Contribution: 14.11 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Recovered targets

Return period: 2732.3247 yrs
Exceedance rate: 0.00036598871 yr⁻¹

Mean (over all sources)

m: 6.46
r: 13.27 km
ε₀: 1.42 σ

Mode (largest m-r-ε₀ bin)

m: 6.12
r: 11.45 km
ε₀: 1.68 σ
Contribution: 5.1 %

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set	Source	Type	r	m	ϵ_0	lon	lat	az	%
UC33brAvg_FM32		System							29.38
	Santa Cruz Catalina Ridge alt2 [0]		9.65	6.66	1.12	118.349°W	33.250°N	202.22	14.28
	San Pedro Basin [0]		16.09	6.74	1.70	118.334°W	33.472°N	351.92	4.44
	Palos Verdes [4]		26.44	7.46	1.84	118.060°W	33.443°N	61.32	2.74
	San Diego Trough north alt2 [28]		19.78	6.90	1.91	118.164°W	33.201°N	136.43	2.74
	Santa Cruz Catalina Ridge alt2 [1]		10.87	7.29	0.81	118.395°W	33.264°N	227.20	1.39
UC33brAvg_FM31		System							26.57
	San Diego Trough north alt1 [33]		12.79	6.64	1.53	118.241°W	33.429°N	30.04	10.49
	San Pedro Basin [0]		16.09	6.77	1.69	118.334°W	33.472°N	351.92	5.62
	Palos Verdes [4]		26.44	7.29	1.97	118.060°W	33.443°N	61.32	2.52
	Santa Cruz Catalina Ridge alt1 [0]		20.75	7.09	1.73	118.523°W	33.384°N	287.11	1.82
	San Diego Trough north alt1 [32]		13.01	7.27	1.16	118.212°W	33.413°N	44.75	1.10
UC33brAvg_FM32 (opt)		Grid							22.03
	PointSourceFinite: -118.310, 33.361		6.12	5.67	0.97	118.310°W	33.361°N	0.00	3.54
	PointSourceFinite: -118.310, 33.361		6.12	5.67	0.97	118.310°W	33.361°N	0.00	3.54
	PointSourceFinite: -118.310, 33.397		8.18	5.96	1.15	118.310°W	33.397°N	0.00	2.53
	PointSourceFinite: -118.310, 33.397		8.18	5.96	1.15	118.310°W	33.397°N	0.00	2.53
	PointSourceFinite: -118.310, 33.424		9.97	6.11	1.32	118.310°W	33.424°N	0.00	1.27
	PointSourceFinite: -118.310, 33.424		9.97	6.11	1.32	118.310°W	33.424°N	0.00	1.27
UC33brAvg_FM31 (opt)		Grid							22.01
	PointSourceFinite: -118.310, 33.361		6.02	5.76	0.90	118.310°W	33.361°N	0.00	4.27
	PointSourceFinite: -118.310, 33.361		6.02	5.76	0.90	118.310°W	33.361°N	0.00	4.27
	PointSourceFinite: -118.310, 33.397		8.51	5.81	1.31	118.310°W	33.397°N	0.00	1.84
	PointSourceFinite: -118.310, 33.397		8.51	5.81	1.31	118.310°W	33.397°N	0.00	1.84
	PointSourceFinite: -118.310, 33.406		8.76	6.01	1.21	118.310°W	33.406°N	0.00	1.58
	PointSourceFinite: -118.310, 33.406		8.76	6.01	1.21	118.310°W	33.406°N	0.00	1.58

Appendix B.3

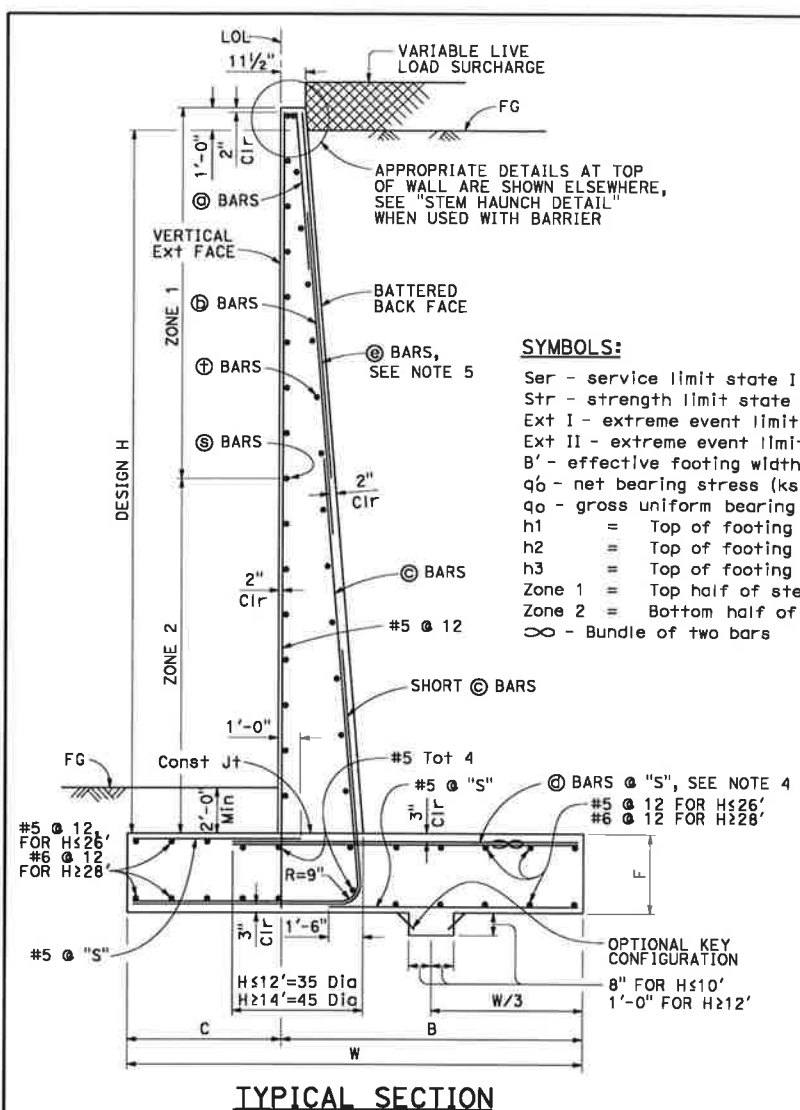
Probabilistic Seismic Hazard Analysis Response Spectrum, the Output of the EZ-FRISK

1 Probabilistic Spectra results for EZ-FRISK 8.07 Build 044
 2
 3 ANNUAL FREQUENCY OF EXCEEDANCE: 4.041e-004
 4 RETURN PERIOD: 2474.9
 5 PROBABILITY OF EXCEEDENCE: 2.0% IN 50.0 YEARS
 6 Column 1: Spectral Period
 7 Column 2: Acceleration (g) for: Mean
 8 Column 3: Acceleration (g) for: Boore-Atkinson (2008) NGA USGS 2008
 9 Column 4: Acceleration (g) for: Campbell-Bozorgnia (2008) NGA USGS 2008
 10 Column 5: Acceleration (g) for: Chiou-Youngs (2007) NGA USGS 2008
 11 Column 6: Acceleration (g) for: Atkinson-Boore (2003) Worldwide Subduction USGS 2008
 12 Column 7: Acceleration (g) for: Atkinson-Boore (2003) Cascadia Subduction USGS 2008
 13 Column 8: Acceleration (g) for: Youngs (1997) Subduction USGS 2008
 14
 15 1 2 3 4 5 6 7
 16 PGA 3.714e-001 3.521e-001 3.561e-001 4.082e-001 * 0.000e+000 * 0.000e+000 * 0.0
 17 0.05 4.968e-001 4.462e-001 4.899e-001 5.514e-001 * 0.000e+000 * 0.000e+000 * 0.0
 18 0.1 7.512e-001 6.625e-001 7.735e-001 8.104e-001 * 0.000e+000 * 0.000e+000 * 0.0
 19 0.2 8.786e-001 7.768e-001 9.042e-001 9.562e-001 * 0.000e+000 * 0.000e+000 * 0.0
 20 0.3 7.693e-001 7.096e-001 7.585e-001 8.449e-001 * 0.000e+000 * 0.000e+000 * 0.0
 21 0.4 6.602e-001 6.196e-001 6.550e-001 7.089e-001 * 0.000e+000 * 0.000e+000 * 0.0
 22 0.5 5.640e-001 5.391e-001 5.647e-001 5.903e-001 * 0.000e+000 * 0.000e+000 * 0.0
 23 0.75 4.107e-001 4.144e-001 4.063e-001 4.114e-001 * 0.000e+000 * 0.000e+000 * 0.0
 24 1 3.192e-001 3.252e-001 3.175e-001 3.140e-001 * 0.000e+000 * 0.000e+000 * 0.0
 25 2 1.612e-001 1.804e-001 1.594e-001 1.384e-001 * 0.000e+000 * 0.000e+000 * 0.0
 26 3 1.048e-001 1.182e-001 1.041e-001 8.449e-002 * 0.000e+000 * 0.000e+000 * 0.0
 27 4 7.520e-002 8.390e-002 7.800e-002 5.846e-002 * 0.000e+000 * 0.000e+000 * 0.0
 28
 29
 30

Appendix B.4

Standard Plans, California State Transportation Agency

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- NOTES:**
- For details not shown and drainage notes see B3-5
 - For wall stem joint details see B0-3 and B0-3
 - At \textcircled{C} bars:
 $H \leq 6'$, no splices are allowed within 1'-8" above the top of footing.
 $H > 6'$, no splices are allowed within H/4 above the top of footing.
 - Bundle \textcircled{C} bars for $H = 34'$ & $36'$.
 - Provide #6 @ 10" x 16'-0" \textcircled{C} bars over a distance of 8'-0" measured from all expansion joints, begin wall and end wall locations. For $H \leq 14'$, hook \textcircled{C} bar into footing and reduce bar length as needed to maintain Min CIR cover.

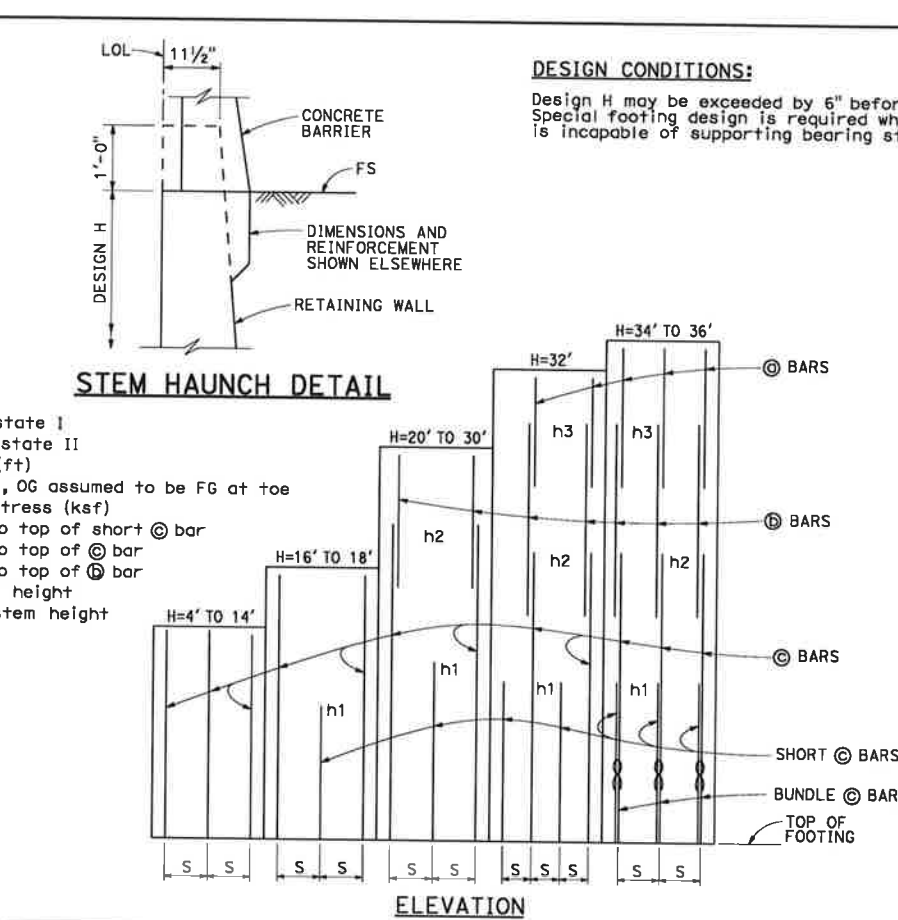


TABLE OF REINFORCING STEEL, DIMENSIONS AND DATA

DESIGN H	4'	6'	8'	10'	12'	14'	16'	18'	20'	22'	24'	26'	28'	30'	32'	34'	36'
W	6'-10"	7'-0"	7'-3"	7'-7"	8'-4"	9'-7"	10'-9"	12'-0"	13'-3"	14'-6"	15'-9"	17'-1"	18'-5"	19'-10"	21'-2"	22'-7"	24'-0"
C	2'-2"	2'-3"	2'-3"	2'-4"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-5"	6'-0"	6'-6"	7'-2"	7'-8"	8'-2"	9'-0"
B	4'-8"	4'-9"	5'-0"	5'-3"	5'-10"	6'-7"	7'-3"	8'-0"	8'-9"	9'-6"	10'-4"	11'-1"	11'-11"	12'-8"	13'-6"	14'-5"	15'-0"
F	1'-4"	1'-4"	1'-4"	1'-4"	1'-6"	1'-8"	1'-8"	1'-9"	1'-9"	1'-11"	2'-2"	2'-5"	2'-10"	3'-3"	3'-6"	4'-0"	4'-3"
BATTER	1/2: 12	1/2: 12	1/2: 12	1/2: 12	1/2: 12	1/2: 12	1/2: 12	1/2: 12	1/2: 12	1/2: 12	3/8: 12	3/8: 12	3/8: 12	3/8: 12	1: 12	1: 12	1: 12
SPACING "S"	9"	9"	9"	9"	9"	7"	6"	5"	6"	6"	6"	6"	6"	6"	6"	10"	8"
\textcircled{A} BARS	-	-	-	-	-	-	-	-	#7	#7	#7	#7	#7	#7	#7	#7	#6
\textcircled{B} BARS	-	-	-	-	-	-	-	-	#8	#8	#8	#8	#8	#8	#8	#8	#8
\textcircled{C} BARS	#6	#6	#6	#6	#6	#6	#7	#7	#8	#8	#9	#9	#10	#10	#10	#11	#11
\textcircled{D} BARS	#5	#5	#6	#6	#6	#6	#7	#8	#8	#9	#9	#10	#10	#10	#11	#11	#11
h1	-	-	-	-	-	-	5'-9"	5'-10"	8'-0"	9'-0"	10'-1"	11'-0"	12'-1"	13'-0"	13'-0"	12'-7"	11'-6"
h2	-	-	-	-	-	-	-	-	10'-5"	13'-0"	14'-7"	17'-6"	19'-0"	20'-5"	19'-0"	18'-0"	20'-2"
h3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	21'-2"	21'-10"	24'-0"
ZONE 1 \textcircled{C} BARS	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12
ZONE 2 \textcircled{C} BARS	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12
ZONE 1 \textcircled{D} BARS	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12
ZONE 2 \textcircled{D} BARS	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12
Ser: B', q _o	6.8, 0.7	6.5, 1.0	6.2, 1.3	6.0, 1.6	6.3, 2.0	7.5, 2.1	8.6, 2.2	9.8, 2.3	11.0, 2.4	12.1, 2.5	13.2, 2.8	14.4, 2.9	15.5, 3.1	16.8, 3.3	18.0, 3.5	19.2, 3.7	20.6, 3.7
Str: B', q _o	6.6, 1.6	5.0, 1.8	3.6, 2.3	3.0, 3.3	3.2, 4.0	4.3, 3.8	5.3, 3.7	6.4, 3.7	7.4, 3.8	8.2, 4.1	9.0, 4.4	9.9, 4.6	10.7, 4.9	11.7, 5.2	12.6, 5.4	13.6, 5.8	14.6, 5.9
Ext I: B', q _o	5.2, 1.1	4.7, 1.5	3.9, 2.2	3.1, 3.4	2.8, 4.8	3.2, 5.3	3.6, 5.7	4.1, 6.1	4.6, 6.4	5.0, 6.9	5.3, 7.6	5.8, 8.1	6.1, 8.9	6.7, 9.4	7.1, 10.0	7.5, 10.7	8.2, 10.9
Ext II: B', q _o	2.6, 2.2	2.7, 2.6	2.8, 3.1	2.9, 3.6	3.7, 3.6	5.2, 3.3	6.7, 3.1	8.3, 3.0	9.8, 3.0	11.2, 3.1	12.5, 3.2	13.9, 3.4	15.2, 3.6	16.7, 3.8	18.0, 4.0	19.3, 4.2	20.8, 4.3

DESIGN CONDITIONS:
 Design H may be exceeded by 6" before going to the next size. Special footing design is required where foundation material is incapable of supporting bearing stress listed in the table.

Dist COUNTY ROUTE POST MILES SHEET TOTAL
 TOTAL PROJECT SHEETS

Jerry Wong
 REGISTERED CIVIL ENGINEER

May 31, 2018
 PLANS APPROVAL DATE

Gary Wong
 REGISTERED PROFESSIONAL ENGINEER
 No. CS8298
 Exp. 6-30-18
 CIVIL
 STATE OF CALIFORNIA

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

- DESIGN NOTES:**
- DESIGN: AASHTO LRFD Bridge Design Specifications, 4th Edition with California Amendments
- LS: Varied surcharge on level ground surface
- DC: Stem Architectural Treatment of thickness up to 6" of concrete (75 psf) considered
- CT: 54 kip transverse force applied at $H_e = 32'$, distributed over 10 feet at the top of wall and 1:1 distribution down and outward. Distribution below footing taken no less than 40'.
- SEISMIC: $k_h = 0.2, k_v = 0.0$
- SOIL: $\phi = 34^\circ, \gamma = 120 \text{ pcf}$
- REINFORCED CONCRETE: $f'_c = 3,600 \text{ psi}$
 $f_y = 60,000 \text{ psi}$
- LOAD COMBINATIONS AND LIMIT STATES:
 Service I $Q = 1.00DC + 1.00EV + 1.00EH + 1.00LS$
 Strength I $Q = 1.25DC + 1.00EV + 1.00EH + 1.75LS$
 Extreme I $Q = 1.00DC + 1.00EV + 1.00EH + 1.00EQD + 1.00EQE$
 Extreme II $Q = 1.00DC + 1.00EV + 1.00EH + 1.00CT$
- Where:
 Q: Force Effects
 a: 1.25 or 0.90, Whichever Controls Design
 b: 1.35 or 1.00, Whichever Controls Design
 n: 1.50 or 0.90, Whichever Controls Design
 DC: Dead Load of Structure Components
 EH: Horizontal Earth Fill Pressure
 EV: Vertical Earth Pressure from Earth Fill Weight
 LS: Live Load Surcharge
 EQE: Seismic Earth Pressure
 EQD: Soil and Structural and Nonstructural Components Inertia
 CT: Vehicular Collision Force

STATE OF CALIFORNIA
 DEPARTMENT OF TRANSPORTATION

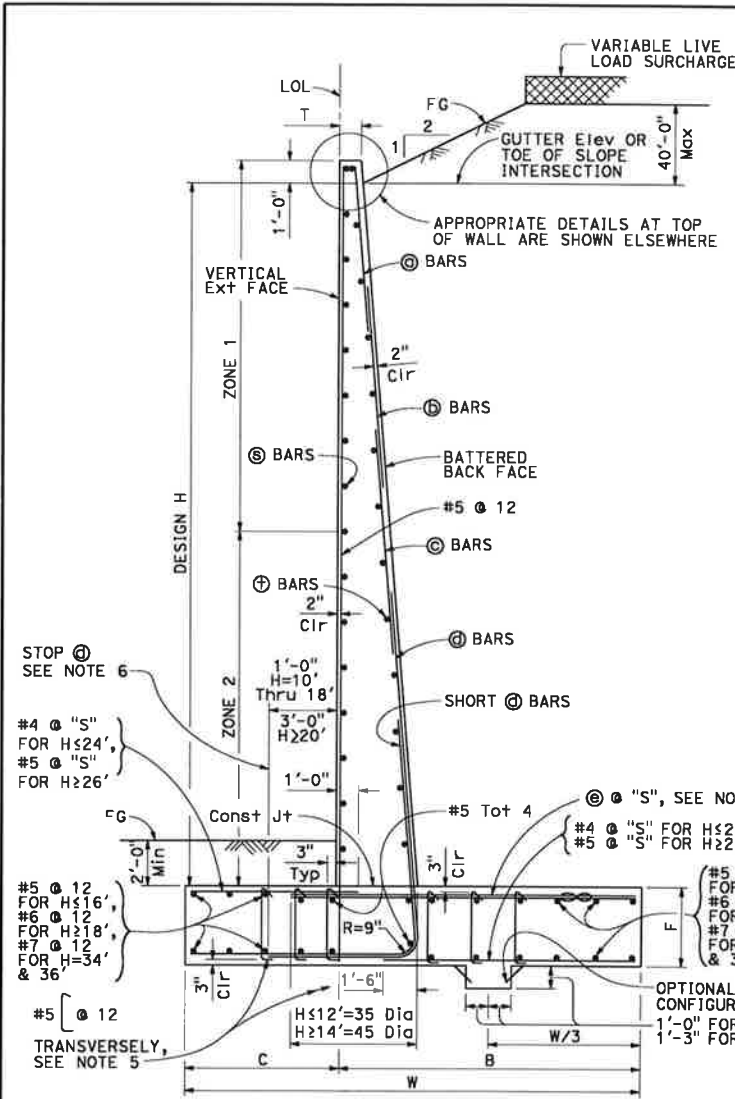
RETAINING WALL TYPE 1 (CASE 1)
 NO SCALE

B3-1A

2018 STANDARD PLAN B3-1A

Return to Table of Contents

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- NOTES:**
- For details not shown and drainage notes see B3-5
 - For wall stem joint details see B0-3 and B0-3
 - At \textcircled{a} and short \textcircled{a} bars:
 $H \leq 6'$, no splices are allowed within 1'-8" above the top of footing.
 $H > 6'$, no splices are allowed within $H/4$ above the top of footing.
 - Bundle \textcircled{a} bars for $H \geq 26'$.
 - Hook stirrups around & space with alternating transverse reinforcement at $2 \times "s"$. For required number of toe or heel stirrup rows, see table. The first stirrups are placed adjacent to the stem as shown.
 - Extend \textcircled{a} bars to end of toe for $H=4'$ to $8'$.

DESIGN CONDITIONS:

Design H may be exceeded by 6" before going to the next size. Special footing design is required where foundation material is incapable of supporting bearing stress listed in the table.

DESIGN NOTES:

DESIGN: AASHTO LRFD Bridge Design Specifications, 4th Edition with California Amendments
 LS: Varied surcharge on level ground surface
 DC: Stem Architectural Treatment of thickness up to 6" of concrete (75 psf) considered
 SEISMIC: $k_h = 0.2$
 $k_v = 0.0$
 SOIL: $\phi = 34^\circ$
 $\gamma = 120$ pcf
 REINFORCED CONCRETE: $f'_c = 3,600$ psi
 $f_y = 60,000$ psi

LOAD COMBINATIONS AND LIMIT STATES:
 Service I $Q = 1.00DC + 1.00EV + 1.00EH + 1.00LS$
 Strength I $Q = aDC + \rho EV + \eta EH + 1.75LS$
 Extreme I $Q = 1.00DC + 1.00EV + 1.00EH + 1.00EOD + 1.00EQE$

Where:
 Q: Force Effects
 a: 1.25 or 0.90, Whichever Controls Design
 ρ : 1.35 or 1.00, Whichever Controls Design
 η : 1.50 or 0.90, Whichever Controls Design
 DC: Dead Load of Structure Components
 EV: Vertical Earth Pressure
 EH: Horizontal Earth Fill Pressure
 LS: Live Load Surcharge
 EOE: Seismic Earth Pressure
 EOD: Soil and Structural and Nonstructural Components Inertia

SYMBOLS:

Ser - service limit state I
 Str - strength limit state I
 Ext - extreme event limit state I
 B' - effective footing width (ft)
 q_0 - net bearing stress (ksf), OG assumed to be FG at toe
 q_0 - gross uniform bearing stress (ksf)
 h1 = Top of footing to top of short \textcircled{a} bar
 h2 = Top of footing to top of \textcircled{b} bar
 h3 = Top of footing to top of \textcircled{c} bar
 h4 = Top of footing to top of \textcircled{d} bar
 Zone 1 = Top half of stem height
 Zone 2 = Bottom half of stem height
 ∞ - Bundle of two bars

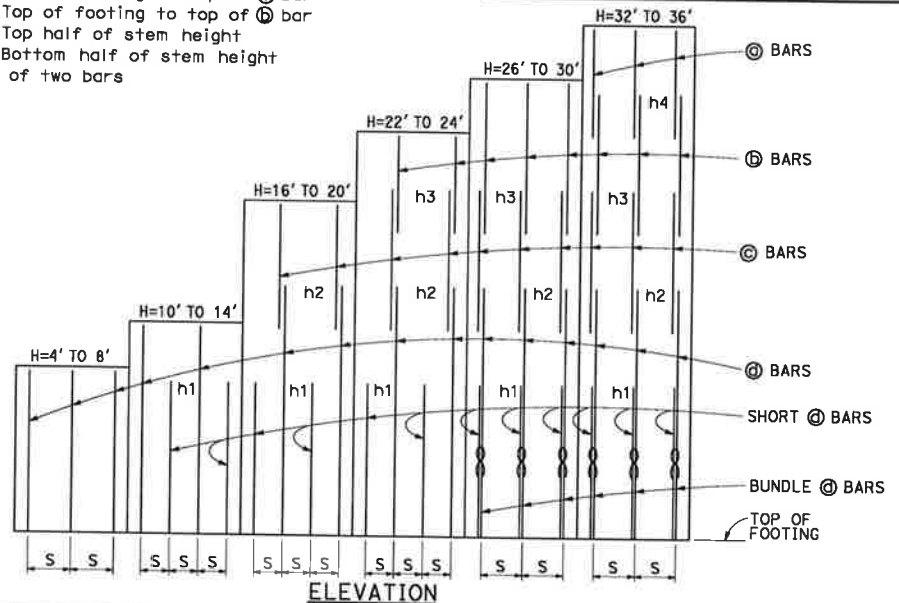


TABLE OF REINFORCING STEEL, DIMENSIONS AND DATA

DESIGN H	4'	6'	8'	10'	12'	14'	16'	18'	20'	22'	24'	26'	28'	30'	32'	34'	36'
W	6'-0"	7'-6"	9'-6"	11'-0"	12'-6"	15'-6"	17'-3"	19'-6"	21'-9"	23'-6"	26'-0"	28'-1"	30'-3"	31'-6"	33'-0"	34'-8"	35'-11"
C	2'-0"	2'-6"	3'-3"	3'-6"	4'-3"	5'-0"	5'-3"	5'-9"	6'-9"	7'-3"	8'-3"	8'-9"	9'-0"	9'-6"	10'-0"	10'-10"	11'-3"
B	4'-0"	5'-0"	6'-3"	7'-6"	8'-3"	10'-6"	12'-0"	13'-9"	15'-0"	16'-3"	17'-9"	19'-4"	21'-3"	22'-0"	23'-0"	23'-10"	24'-8"
F	1'-6"	1'-6"	2'-0"	2'-3"	2'-6"	2'-8"	2'-10"	3'-0"	3'-4"	3'-6"	3'-6"	3'-7"	3'-7"	3'-9"	3'-9"	4'-0"	4'-4"
T	11 1/2"	11 1/2"	11 1/2"	11 1/2"	11 1/2"	11 1/2"	11 1/2"	11 1/2"	11 1/2"	11 1/2"	11 1/2"	11 1/2"	11 1/2"	11 1/2"	11 1/2"	11 1/2"	11 1/2"
BATTER	1/2: 12	1/2: 12	1/2: 12	1/2: 12	1/2: 12	5/8: 12	5/8: 12	5/8: 12	5/8: 12	1: 12	1 1/8: 12	1 1/8: 12	1 1/8: 12	1 1/8: 12	1 1/8: 12	1 1/8: 12	1 1/8: 12
SPACING "s"	16"	12"	10"	7"	7"	7"	7"	7"	7"	6"	6"	10"	8"	8"	7"	7"	7"
\textcircled{a} BARS	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
\textcircled{b} BARS	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
\textcircled{c} BARS	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
\textcircled{d} BARS	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
\textcircled{a} BARS	#5	#5	#6	#6	#7	#8	#9	#10	#10	#10	#11	#11	#11	#11	#11	#11	#11
h1	-	-	-	5'-3"	6'-4"	7'-6"	8'-9"	9'-9"	11'-0"	11'-3"	11'-6"	10'-3"	11'-9"	12'-3"	12'-6"	13'-3"	13'-8"
h2	-	-	-	-	-	-	12'-8"	15'-6"	17'-0"	16'-6"	17'-3"	18'-0"	17'-6"	17'-4"	14'-10"	15'-9"	16'-4"
h3	-	-	-	-	-	-	-	-	-	18'-9"	21'-3"	21'-3"	22'-4"	22'-8"	18'-0"	18'-6"	19'-6"
h4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	26'-3"	27'-4"	28'-6"
No. of Toe Stirrups	0	0	0	0	0	0	0	0	0	0	0	5	5	6	7	8	9
No. of Heel Stirrups	0	0	0	0	0	0	0	0	0	4	6	7	8	10	10	11	11
ZONE 1 \textcircled{a} BARS	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#6 @ 12	#6 @ 10	#6 @ 10
ZONE 2 \textcircled{b} BARS	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#6 @ 12	#6 @ 12	#7 @ 12	#7 @ 12	#7 @ 12	#7 @ 12	#7 @ 10	#7 @ 10
ZONE 1 \textcircled{c} BARS	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#5 @ 12	#5 @ 12	#5 @ 12
ZONE 2 \textcircled{d} BARS	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12
Ser: B', q_0	4.0, 0.9	5.5, 1.0	9.3, 1.0	10.9, 1.3	12.3, 1.5	14.8, 1.9	16.6, 2.1	18.7, 2.4	20.6, 2.7	22.3, 3.0	24.2, 3.3	26.1, 3.5	28.2, 3.9	29.6, 4.0	31.1, 4.2	32.7, 4.4	34.1, 4.6
Str: B', q_0	2.2, 2.2	3.5, 2.2	5.1, 2.3	6.3, 2.6	7.6, 2.7	12.9, 3.1	14.3, 3.6	16.5, 3.9	19.4, 4.5	20.7, 4.8	22.5, 5.2	24.3, 5.6	26.2, 6.0	27.5, 6.3	28.8, 6.6	30.3, 6.9	31.8, 7.2
Ext: B', q_0	2.3, 3.4	2.7, 4.4	3.6, 5.0	3.8, 6.5	4.5, 7.0	7.0, 6.1	7.6, 6.9	9.3, 7.0	11.0, 7.1	11.8, 7.6	14.1, 7.4	15.6, 7.7	17.1, 8.0	17.2, 8.7	18.1, 9.0	19.0, 9.4	19.4, 10.0

STATE OF CALIFORNIA
 DEPARTMENT OF TRANSPORTATION
RETAINING WALL TYPE 1 (CASE 2)
 NO SCALE

B3-1B

2018 STANDARD PLAN B3-1B

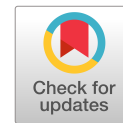
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO.	TOTAL SHEETS

May 31, 2018
 PLANS APPROVAL DATE

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Appendix B.5

Case Study Written by
Kavazanjian Et Al. (2013), 11th
Peck Lecture: Predesign
Geotechnical Investigation for the
OII Superfund Site Landfill



11th Peck Lecture: Predesign Geotechnical Investigation for the OII Superfund Site Landfill

Edward Kavazanjian Jr., F.ASCE¹; Neven Matasovic, F.ASCE²; and Robert C. Bachus, M.ASCE³

Abstract: The predesign geotechnical investigation for closure of the Operating Industries, Inc. (OII) Superfund site landfill significantly advanced the state of the art for solid waste landfill engineering. Contributions to solid waste landfill engineering from the OII predesign geotechnical investigation include a solid waste classification system, a method for in situ measurement of waste unit weight, an enhanced understanding of the mechanical properties of solid waste, including unit weight, compressibility, shear strength, shear wave velocity, and equivalent linear shear modulus and equivalent viscous damping, and a methodology for seismic stability and deformation analysis of the waste mass. Many of the procedures and waste property relationships developed for the OII predesign geotechnical investigation remain the state of practice today for solid waste landfill engineering. DOI: [10.1061/\(ASCE\)GT.1943-5606.0000923](https://doi.org/10.1061/(ASCE)GT.1943-5606.0000923). © 2013 American Society of Civil Engineers.

CE Database subject headings: Solid wastes; Material properties; Case studies; Seismic design; Landfills.

Author keywords: Solid waste; Material properties; Case history; Seismic design; Closure design.

Introduction

The Operating Industries, Inc. (OII) Superfund site is located in Monterey Park, California, approximately 16 km east of downtown Los Angeles. The OII site is divided into the relatively small and level north parcel and the 58-ha south parcel. The south parcel landfill site was placed on the National Priorities List, i.e., named a Superfund site, in 1985. The landfill site was formerly a sand and gravel quarry that was excavated to depths in excess of 30 m below grade. Over a 40-year period after cessation of sand and gravel mining activities, municipal, commercial, and industrial solid waste and liquid hazardous waste were disposed at the site. When waste placement ceased in 1984, the landfill rose between 21 and 76 m above the adjacent grades. The maximum waste thickness is over 100 m at the site.

Fig. 1 shows an aerial view of the OII site looking from the northwest at around the time of the predesign geotechnical investigation (circa 1995). Characteristics of the site that led the EPA to deem the OII landfill a unique urban hazard included:

- The development of the landfill in an abandoned sand and gravel pit without an engineered containment system at its base;
- The codisposal of liquid hazardous waste with municipal and industrial solid waste in the southwest corner of the site;
- The steep grades on the north and east sides of the landfill that rise approximately 60 m above grade at an average inclination of 1.5

horizontal (H):1 vertical (V) and as steeply as 1.3H:1V immediately adjacent to State Route 60 (the Pomona Freeway);

- The proximity of homes to the south side of the landfill, where a mechanically stabilized earth (MSE) toe buttress was constructed in the 1980s because of stability concerns; and
- The severe seismic exposure of the site.

Concerns over stability of the landfill were exacerbated by the performance of the site in the 1987 Whittier Narrows moment magnitude (M_w) 5.9 earthquake when severe cracking was observed on the benches on the steep north slope. Fig. 2 shows similar cracking on one of the north slope benches following the 1994 Northridge M_w 6.6 earthquake. Concerns over landfill stability following the Whittier Narrows event led to the EPA's development of an instrumentation and monitoring program that included installation of four inclinometers on the steep north slope and two strong motion stations: one on top of the landfill and one on what was believed to be natural ground at the base of the landfill. The EPA negotiations with the potentially responsible parties (PRPs) for remediation of the site resulted in a consent decree that included a predesign geotechnical investigation program to assess the stability and long-term deformation potential of the landfill (GeoSyntec Consultants 1996).

Scope of the Predesign Geotechnical Investigation

In 1995, as part of the consent decree negotiated with the EPA, the predesign geotechnical investigation described herein was commissioned by New Cure, Inc., the site management organization representing the PRPs. The scope of work for the predesign geotechnical investigation for the landfill included the following:

- Review and synthesis of available geotechnical information;
- A field investigation;
- A laboratory testing program;
- Limit equilibrium stability analyses;
- Seismic hazard analyses, including development of design ground motions;
- Seismic response and deformation analyses;
- Static deformation analysis;

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Fig. 1. OII Superfund site landfill circa 1995 (Photograph courtesy of New Cure, Inc.)



Fig. 2. Cracking on a north slope bench following the 1994 Northridge earthquake (Photograph courtesy of Raymond B. Seed)

- Assessment of the stability of the MSE toe buttress for the south slope; and
- Development of findings and recommendations for closure design.

The predesign geotechnical investigation was overseen by two technical review panels: one representing EPA and another representing New Cure, Inc. Both review panels were composed of eminent geotechnical engineers and engineering geologists experienced in landfill engineering and seismic analysis and design. Both review panels actively participated in the development of the scope of work and interpretation of the results of the investigation described herein.

Review and synthesis of available information included collecting information regarding:

- Bottom contours of the quarry prior to the start of landfilling;
- The history of waste disposal at the site, including codisposal of liquid hazardous waste in the southwest corner of the site;
- An interim soil cover that was placed on the top deck at the east end of the landfill prior to reaching final grade during a temporary halt in landfilling;
- Design, construction, and performance of the south slope MSE toe buttress;

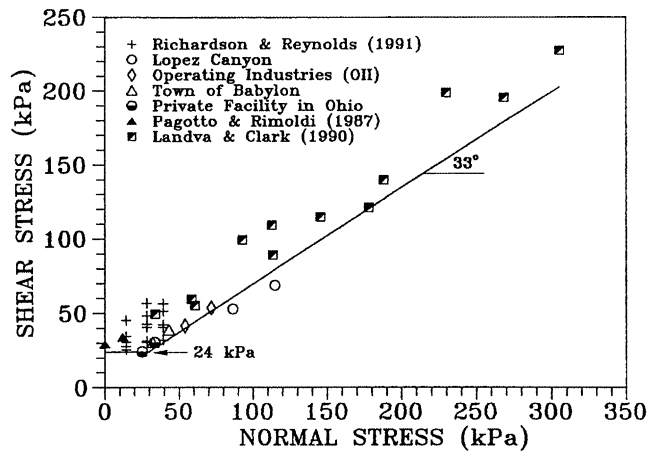


Fig. 3. Bilinear shear strength envelope for MSW (Kavazanjian et al. 1995, © ASCE)

- Postlandfilling waste mass deformation, including data on settlement of the top deck and lateral deformation of the steep north slope;
- Previous geotechnical studies at the site;
- Records captured by the strong motion stations installed at the site after the 1987 Whittier Narrows earthquake;
- A veneer cover failure on the north slope following a rainstorm in 1992; and
- The mechanical properties of municipal solid waste (MSW).

The information regarding waste disposal at the site indicated that although industrial wastes including hazardous liquids were codisposed with MSW, the landfill was composed predominantly of MSW. Furthermore, interviews with former site operations personnel indicated that most of the hazardous liquids were disposed of in the southwest corner of the site where slopes were on the order of 3H:1V. Available information also indicated that the east end of the landfill, where the steepest slopes and the MSE toe buttress were located, was relatively dry and composed almost entirely of MSW.

Review and synthesis of available information in the technical literature indicated that MSW shear strength was typically represented by a cohesion on the order of 5 kPa and a friction angle on the order of 20°. However, shear strength values of that order of magnitude were contradicted by the observed stability of the steep north slope as well as several other sources of available information. Based upon available information on MSW shear strength, the bilinear waste shear strength envelope shown in Fig. 3, first presented by Kavazanjian et al. (1995), was developed for use in preliminary stability analyses. Information used to develop this strength envelope included large-scale testing on MSW, back analysis of the steep waste slopes at OII and several other landfills using an assumed static factor of safety of 1.1, and observations from landfill operators that they could make vertical cuts in waste with heights in excess of 12 m that would remain stable indefinitely. This bilinear shear strength envelope remains one of the most commonly used MSW shear strength envelopes in landfill practice today.

Field Investigation

Field work at the site for the predesign geotechnical investigation included noninvasive spectral analysis of surface wave (SASW)

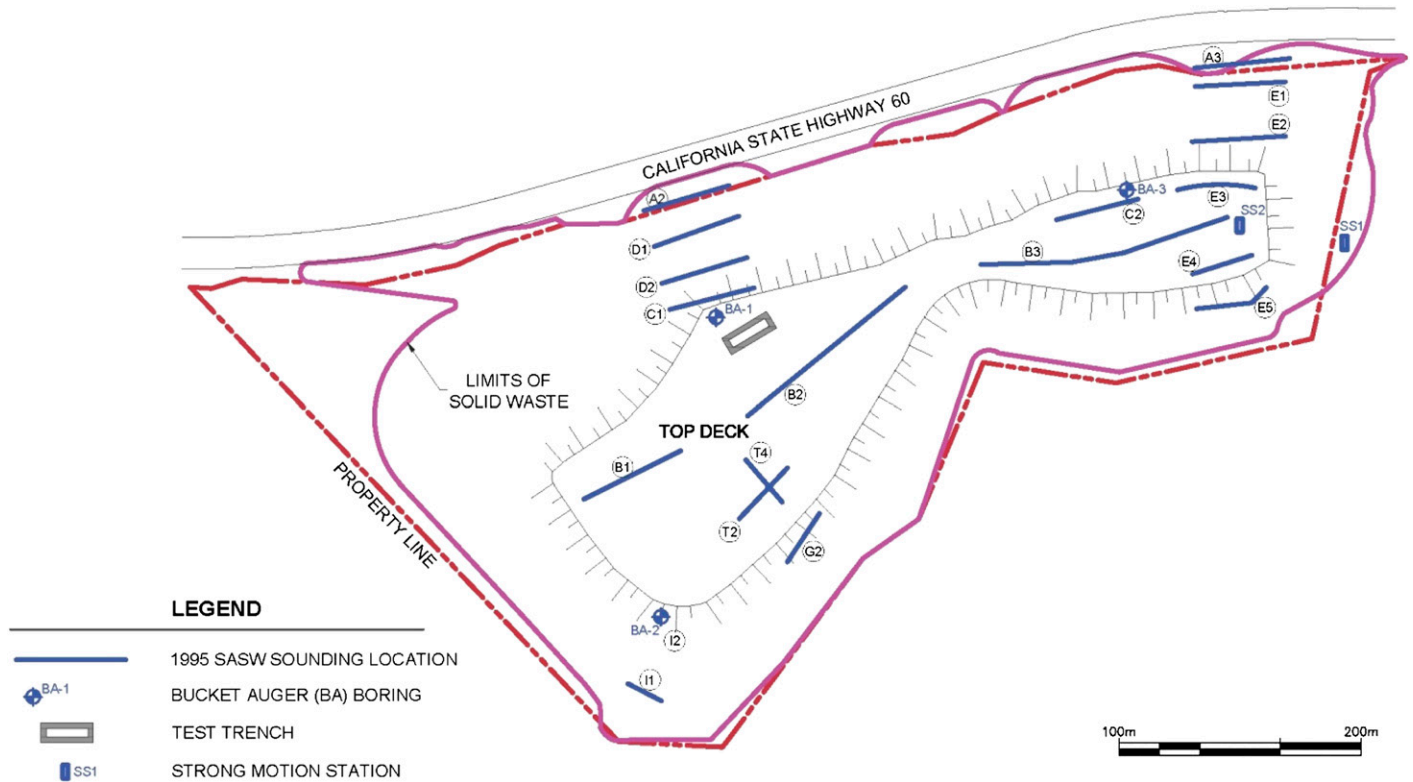


Fig. 4. Predesign geotechnical investigation field exploration plan

testing to develop shear wave velocity profiles, three 864-mm-diameter bucket auger borings to depths of up to 46 m, a 6-m-deep test trench on the top deck of the landfill, a condition survey of the MSE toe buttress, and an assessment of the thickness and condition of the cover soil. Fig. 4 shows the field exploration plan, including the SASW survey lines and locations of the bucket auger borings and test trench.

Fig. 4 shows the locations of the 27 shear wave velocity profiles developed in waste across the site. Fig. 5 shows the 27 individual shear wave velocity profiles captured in waste at the site along with the statistically derived mean and plus and minus one standard deviation profiles for waste shear wave velocity at OIL. Auger boring sampling and testing intervals were selected based upon these statistical profiles and the individual SASW profile closest to the boring location. The sampling and testing intervals in the borings were chosen based upon the hypothesis that the composition and mechanical behavior of the waste at the boring location governed the shear wave velocity in the vicinity of the borings. There were six sampling and testing intervals in each of the three large diameter bucket auger borings: two sampling intervals were where the shear wave velocity in the vicinity boring was close to the mean shear wave velocity for the site, two sampling intervals were where the shear wave velocity in the vicinity boring was near the mean plus one standard deviation shear wave velocity for the site, and two sampling intervals were where the shear wave velocity in the vicinity the boring was near the mean minus one standard deviation shear wave velocity for the site. This approach was assumed to provide a good distribution of representative waste samples and representative intervals for the in-hole unit weight measurements. Bulk samples were recovered from the selected intervals for laboratory testing. In situ unit weight tests were conducted in the intervals from which the bulk samples were recovered.

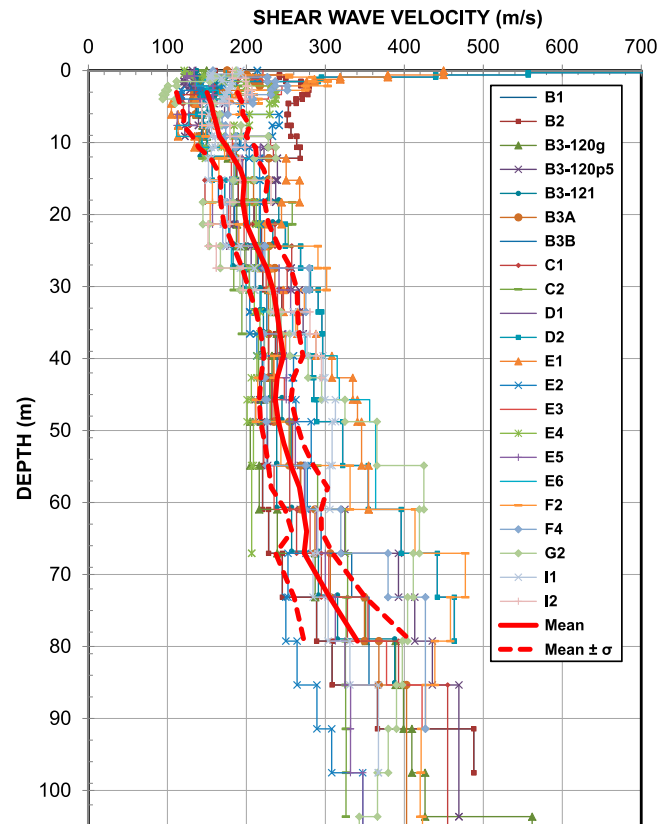


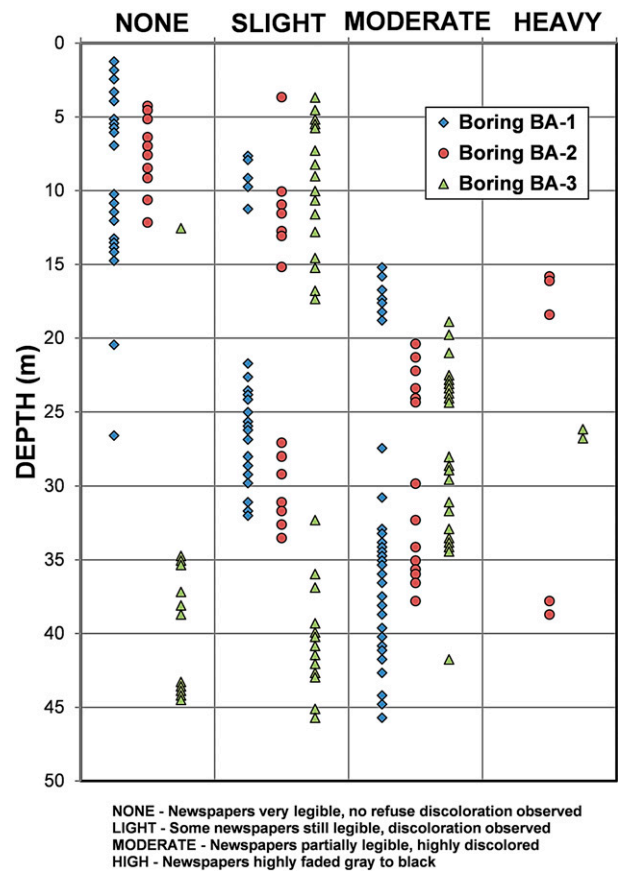
Fig. 5. Shear wave velocity profiles from SASW testing

Table 1. Field Classification System for OII Solid Waste

Moisture content	
1	Dry-damp moisture levels
2	Wet moisture levels
3	Standing water
Compaction	
1	Slight: refuse easily falls out of bucket auger
2	Moderate: refuse falls out of bucket auger upon impact
3	Heavy: refuse falls out of bucket auger only after being struck multiple times
Degradation	
1	None: newspaper very legible, no refuse discoloration
2	Slight: some newspapers still legible, discoloration
3	Moderate: newspaper partly legible, highly discolored
4	High: newspaper highly faded gray to black
Composition	
1	Household paper and plastics
2	Putrescible organics
3	Concrete, bricks
4	Wiring
5	Metal
6	Nonferrous metal
7	Tires
8	Asphalt
9	Soil
10	Medical
11	Indistinguishable
12	Glass
13	Other (specify)
Structure	
1	Layered
2	Encapsulated
3	Fibrous
4	Interlocked
5	Indistinguishable

As part of the field investigation, a solid waste field classification and logging system was developed. Table 1 presents the field classification scheme developed for this logging system. Field classification categories for the waste included moisture content, e.g., dry or wet; degree of compaction, e.g., slight or heavy; state of degradation, e.g., none or high; structure, e.g., layered, encapsulated, or indistinguishable; and physical composition, e.g., paper and plastics, putrescible waste, concrete, asphalt, or soil. Field logging also included measurement of the temperature of the waste as it was discharged from the bucket auger. Information from the field logs was summarized to provide information on the characteristics of the waste versus depth, including the state of waste degradation, moisture content, and waste temperature. For instance, Fig. 6 shows a plot developed from the field logs of waste degradation versus depth, indicating a general increase in degradation with depth. The waste classification scheme in Table 1 formed the basis for the MSW field classification scheme recently proposed by Zekkos et al. (2010).

At the time of the predesign geotechnical investigation, one of the glaring deficiencies in the available information on solid waste properties was the sparse and contradictory data regarding the in situ unit weight of solid waste. Therefore, a method of evaluating the in

**Fig. 6.** Observed waste degradation versus depth at the OII landfill

situ unit weight of waste within the large diameter auger borings was developed for the project. In this method, in situ unit weight was evaluated within the bucket auger borings using a gravel replacement method similar to the method used by Landva and Clark (1986) to evaluate in situ unit weight in shallow test trenches. The volume of the borehole interval from which a measured weight of waste was removed was evaluated by backfilling the borehole with calibrated gravel, much in the same way that sand is used to backfill the excavated hole in the sand cone test (ASTM 2013). The in situ total unit weight over that borehole interval was then calculated based upon the calculated volume of the sampling interval and the measured weight of the waste removed from that interval. The waste recovered from each sampling interval was also subject to compositional analysis and testing for moisture content.

Fig. 7 presents a summary of the in situ total unit weight tests conducted for the predesign geotechnical investigation. It should be noted that the highest in situ values shown in this figure were measured in zones where the waste appeared to be saturated, i.e., was very wet. Fig. 7 also shows the average unit weight within a 6-m-deep and 6-m-wide test trench excavated on the top deck of the landfill. The volume of the excavated trench, and thus the calculated unit weight, was evaluated using the gravel replacement technique and substantiated by independent survey measurements. Zekkos et al. (2006) describes the in situ unit weight measurement technique developed at OII in detail. Unit weight measurements made using this technique at OII and several MSW landfills were employed in the development of the typical MSW unit weight profiles presented by Zekkos et al. (2006).

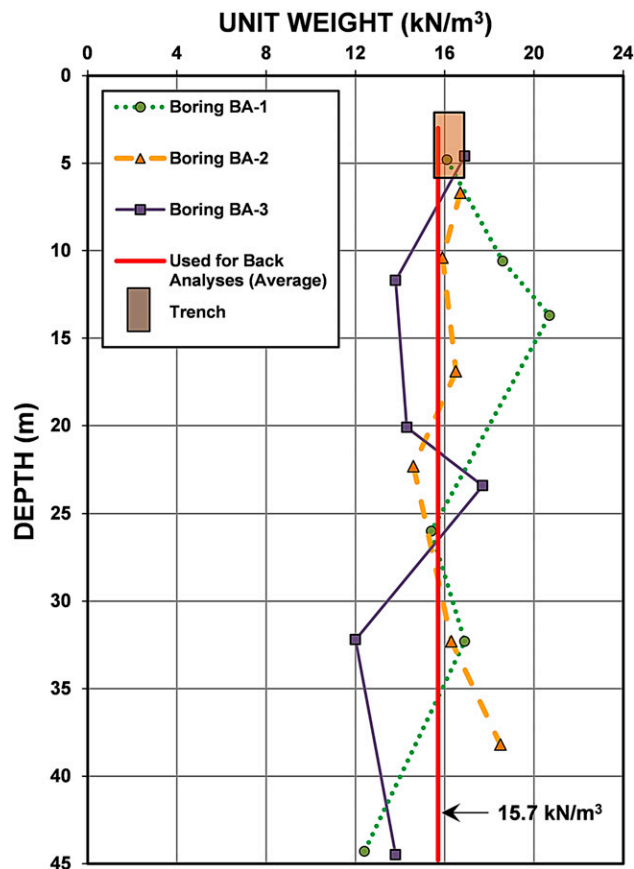


Fig. 7. Oil waste unit weight versus depth

To further support the waste characterization activities, down-hole video logging was conducted in the bucket auger borings. The video logs indicated a preferential horizontal orientation for the large particles in the waste mass. This preferential structure, also observed in the test trench on the top deck, was attributed to the method of waste placement, wherein waste dumped at the working face was spread horizontally and then compacted. This preferential structure observed in the field investigation suggested that the properties of the waste were likely to be anisotropic.

Laboratory Testing

Matasovic et al. (1998) and Kavazanjian et al. (1999) describe the static and cyclic laboratory testing programs conducted as part of the OII landfill predesign geotechnical investigation in detail. An on-site laboratory was constructed for the waste testing. The laboratory included 457-mm-diameter direct shear, one-dimensional compression and cyclic simple shear testing devices. The laboratory equipment was fabricated, assembled at the site, and commissioned for use in less than 1 year after commencement of equipment design. All waste testing was conducted on reconstituted specimens at the field moisture content and the field constituent compositional ratio (by weight). Initially, attempts were made to reconstitute, i.e., to compact, the waste to a unit weight as close as possible to the in situ unit weight of the specimen as measured in the bucket auger boring. While in general it was not possible to initially compact the waste to its field-measured in situ unit weight, the calculated unit weight in the laboratory testing device usually approached the measured in situ value once the calculated field overburden stress was applied to a

Table 2. Summary of Composition of OII Waste Specimens

Test	Depth (m)	Age	Percentage of soil and soil-sized material	Percentage passing #200 sieve	Unified Soil Classification System classification
CONDLV	3–6	1984	83.6	35	SM
CONDMV	3–6	1980	76.1	75	CH
CONDHV	9–12	—	97.1	90	CH
CONMLV	4.5–7.5	1984	94.3	80	CH
CONMMV	9–12	—	99	85	SP
CONMHV	15–18	1983	90	80	ML
SWCONDLV	15–18	—	74.5	58	CL
SWCONDMV	12–15	—	99	85	CL
SWCONMHV	9–12	—	97.1	90	CH
DSDLVLS	3–6	—	83.7	37	SM
DSDHVLS	9–12	—	99	95	CL
DSDHVMS	21–24	1968	76.1	80	CL
DSDLVHS	30–33	1964	10	60	—
DSDMVHS	42–45	1960	76.2	80	ML
DSDHVHS	24–27	1965	50	—	MH
DSMHVLS	15–18	1983	77	80	ML
DSMLVHS	21–24	1982	74.7	90	CH
DSMHVHS	36–39	—	92.2	65	ML
SSDLVLS	3–6	—	84.8	35	SM
SSDMVLS	3–6	1980	94.6	75	CH
SSDHVLS	9–12	—	64.6	90	CH
SSDLVMS	15–18	1984	77.5	52	CL
SSDLVHS	30–33	1964	85.4	—	—
SSMHVLS	15–18	1983	84.8	55	ML

Note: Test designations in order of precedence: SW = shear wave velocity; CON = consolidation; DS = direct shear; SS = simple shear; D = dry; M = moist; LV = low velocity; MV = mean velocity; HV = high velocity; LS = low stress; MS = medium stress; HS = high stress.

waste specimen. The field and laboratory classification and compositional data were compiled in an attempt to correlate waste properties with waste composition. Analysis of the data suggested that the percentage of soil and soil-sized material, evaluated as the percentage of the material by weight that passed a 19-mm sieve, was a key compositional parameter for the waste.

Specimens used in the laboratory testing program were subject to detailed compositional characterization in the laboratory. Table 2 summarizes the compositional data for 24 454-mm-diameter one-dimensional compression tests on OII solid waste. The table includes results on specimens with a range of soil and soil-sized materials. The table also presents the Unified Soil Classification System classification on the portion of the soil and soil-sized materials that passed the #40 sieve. Some of these tests represent the consolidation phase of the direct shear (DS) and simple shear (SS) tests, and in some of the consolidation (CON) tests, shear wave (SW) velocity measurements were made. The test designation also indicated whether the tested specimen was relatively dry (D) or moist (M) based upon visual classification and whether the waste was recovered from a low (LV), medium (MV), or high (HV) shear wave velocity interval of the borehole based upon the mean shear wave velocity from the SASW testing and the adjacent SASW shear wave velocity profile. The test designation also reflects the relative overburden stress of the borehole interval from which the sample was recovered, with LS referring to samples recovered from an interval with relative low overburden (relatively shallow depth of typically less than 12 m), MS referring to samples recovered from an interval with an intermediate overburden stress (typically from a depth between 12 and 24 m), and HS referring to samples recovered from an interval with

a relatively high overburden stress (typically from a depth greater than 24 m).

Fig. 8 presents a graphical summary of the one-dimensional compression test results. The results of these tests yielded values of the virgin modified compression index, C_{ce} , i.e., for virgin compressibility on a volumetric strain basis, between 0.12 and 0.25. The value of the modified recompression index C_{re} in these tests varied from 0.003 to 0.017 and was typically less than 10% of C_{ce} . The more compressible material tended to correspond to less degraded waste and/or specimens with a smaller amount of soil, i.e., the specimens with the smallest amount of soil and soil-sized material. However, no further correlation with waste properties was readily discernible. Laboratory testing on the OII waste resulted in a measured secondary compression rate, C_{α} , on the order of 0.01 (1%

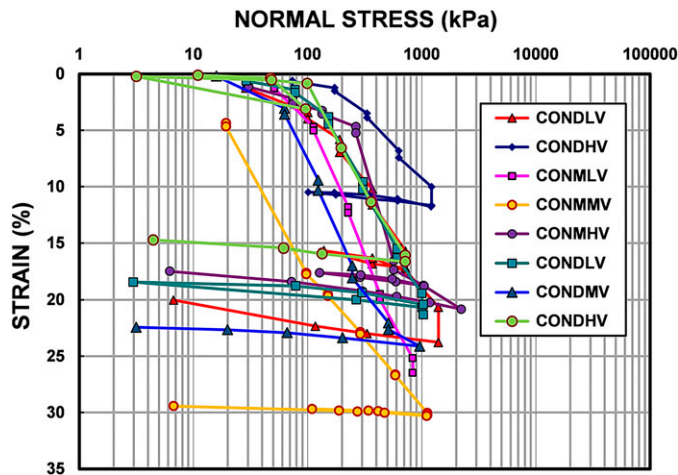


Fig. 8. One-dimensional compression test results on reconstituted specimens of OII waste

vertical strain per log cycle of time). However, secondary compression values back-calculated from field settlement data were on the order of 0.10–0.18. This was not surprising because the laboratory values reflect only mechanical secondary compression and hence greatly underestimate the total secondary compression due to combined mechanical compression and waste degradation effects.

Fig. 9 shows results of the direct shear tests conducted on reconstituted specimens of waste. Waste composition in the specimens used in these tests was split into two broad categories: material retained on a screen with 19-mm openings, termed refuse, and material passing through the 19-mm screen, termed soil-sized material. The numerical values in the boxes on Fig. 9 represent the percentage of refuse, by weight, in each specimen. The data in Fig. 9 suggest a general relationship between waste shear strength and the percentage of refuse or soil-sized material in the specimen, with the weakest specimens being the specimens with the highest soil-sized material content or the lowest refuse content. Based on Fig. 9, a cohesion of 43 kPa and a friction angle of 31° corresponding to the best estimate line in Fig. 9 were used to characterize the site-specific waste shear strength for nonhorizontal shear planes in the limit equilibrium analyses described subsequently. However, based upon discussions with the two technical review panels, horizontal shear planes in the waste mass were characterized by zero cohesion and a friction angle of 30° , corresponding to the lower bound in Fig. 9 over the stress range of interest if cohesion is set equal to zero, to account for the preferred horizontal orientation of the large particles as well as the potential for continuous horizontal planes of daily cover soil in the waste mass.

Matasovic and Kavazanjian (1998) discuss the cyclic simple shear testing program in detail. Table 3 summarizes the cyclic simple shear testing program, including information on specimen composition, applied normal stress, and shear wave velocity. Rows in this table with two test numbers indicate replicate tests. Fig. 10 presents the equivalent linear shear modulus reduction and damping (as the fraction of critical damping) data points developed from the cyclic simple shear tests for cyclic shear strains that ranged from approximately 0.1 to 5%. Fig. 10 also shows upper bound, average,

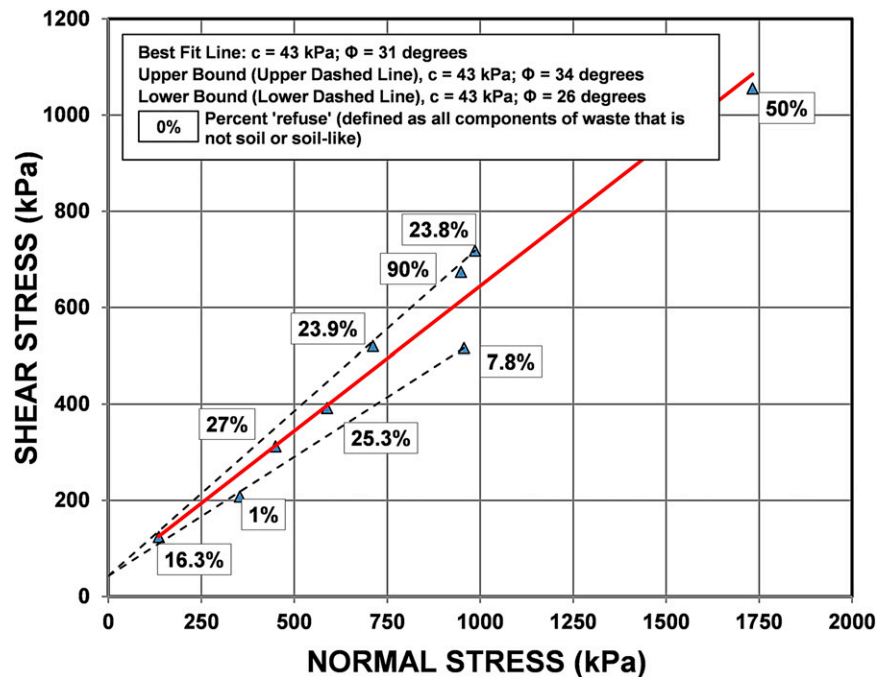


Fig. 9. Direct shear test results on reconstituted specimens of OII waste

Table 3. Cyclic Simple Shear Testing Program for OII Landfill Solid Waste

Test number	Boring number	Sampling depth (m)	Age of sample ^a	Applied normal stress (kPa)	Moisture content ^b (%)	Shear wave velocity ^c (m/s)	Unit weight ^d (kN/m ³)
1, 2	BA-1	3.4–6.1	—	95.8; 94.1	15.3	122	16.0
3	BA-3	3.4–6.1	1980	97.4	16.6	162	16.9
4, 5	BA-1	9.2–12.2	—	176.2; 176.4	33.5	231	18.5
6	BA-3	30.5–33.5	1964	511.4	24.7	195	12.0
7, 8	BA-2	15.2–18.3	1983	292.1; 292.3	41.2	231	16.5
9	BA-1	15.2–18.3	1984	315.1	25.5	148	18.1
Waste composition ^e							
Test number	Paper, cardboard (%)	Plastics, rubber (%)	Wood (%)	Metal (%)	Glass (%)	Textile, miscellaneous (%)	Soil and soil-sized materials composition ^f (%)
1, 2	1.6	7.0	2.3	0.6	2.5	1.2	84.8 (SM: 10% gravel, 50–60% sand)
3	1.7	1.1	0.4	0.8	0.4	0.9	94.6 (CH: 10% gravel, 15% sand)
4, 5	6.9	11.8	4.0	0.8	5.6	5.3	64.6 (CH: 5% gravel, 5% sand)
6	10.3	0.4	2.9	0.9	0.1	0.1	85.4 (fine grained)
7, 8	1.2	3.7	3.4	4.9	0.4	1.5	84.8 (ML: 10% gravel, 30–40% sand)
9	9.5	3.2	3.7	2.4	1.6	2.2	77.5 (CL: 10–15% gravel, 30–40% sand)
Average	5.2	4.5	2.8	1.7	1.8	2.0	82.0

^aAge of the solid waste part of the sample estimated, where possible, from dated newspapers contained in the solid waste.

^bMoisture content of soil and soil-like materials measured prior to the test.

^cMeasured in the 460-mm consolidation apparatus for the same initial density and overburden pressure.

^dIn situ unit weight; unit weight of the reconstituted sample in the CyDSS apparatus was $\pm 2\%$ of this value.

^ePercentage developed on weight basis.

^fUnified Soil Classification System by visual classification.

and lower bound normalized shear modulus and damping curves fit to the laboratory test data. Note that if Masing behavior is assumed, the upper bound normalized shear modulus curve is consistent with the lower bound damping curve and vice versa.

Seismic Analysis

The seismic analysis for the predesign geotechnical investigation included a back analysis of the strong motion records captured at the base and top deck of the landfill to help establish waste properties, a seismic hazard analysis to establish ground motions for use in design, and a forward analysis to evaluate the response of the waste mass and landfill cover to the design ground motions. The back analyses of strong motion records captured at the site are documented in detail by Matasovic and Kavazanjian (1998) and summarized herein. Fig. 11 shows a cross section through the east end of the landfill where the two strong motion stations at the site were located. Note that strong motion station number one (SS-1) is located on a wedge of fill at the toe of the landfill. It was initially believed that SS-1 was founded on bedrock. However, the unusual characteristics of some of the ground motions recorded by SS-1 led to a supplemental investigation of the subsurface conditions at the east end of the site. This supplemental investigation indicated that, in fact, SS-1 was founded upon a wedge of fill that had been placed to remediate a landslide at the site during quarry operations. The location of SS-1 on fill greatly complicated the seismic analysis because strong motion records captured by SS-1 had to be deconvolved to the bedrock beneath SS-1 to provide input motions for the back analysis of waste properties conducted using the records obtained at strong motion station number two (SS-2). This deconvolution added additional levels of complexity and uncertainty to the seismic analyses.

One important observation from the deconvolution and back analyses was the amplification of strong ground motions by the waste fill. Some previous investigators had suggested that amplification was not likely at waste fills. Fig. 12 compares the peak horizontal ground acceleration (PHGA) recorded by SS-2 at the top of the landfill to the PHGA measured at SS-1 and as estimated for a hypothetical bedrock outcrop at the site based upon the deconvolved ground motion from SS-1. Comparison of peak accelerations measured at SS-1 and SS-2 showed no evidence of amplification. However, comparison of the deconvolved PHGA to the PHGA measured at SS-2 clearly shows amplification of the bedrock ground motions by the OII waste fill, with a pattern of amplification similar to that observed by Harder (1991) for the transverse acceleration at the crest of earth dams, at least at low acceleration levels.

To establish the equivalent linear shear modulus reduction and damping curves for the OII waste fill, deconvolved bedrock ground motions from SS-1 were input along with shear wave velocity and unit weight values from the site investigation to *QUAD4M*, a finite-element program for two-dimensional equivalent linear seismic site response analyses (Hudson et al. 1994). The maximum strains generated in the waste by the available earthquake records from the site were somewhat less than 0.1%. Therefore, the field data had to be supplemented by the laboratory test data to establish the complete equivalent linear shear modulus reduction and damping curves for the waste. The best-fit equivalent linear curves were established by varying the curves within the constraints provided by the cyclic simple shear test results for strains greater than 0.1% and the Masing criteria to achieve the best fit between the seismic response predicted at the top of the landfill and the ground motions recorded at SS-2, the strong motion station at the top of the landfill. Matasovic and Kavazanjian (1998) provide additional details on the back analysis.

Two FEM models were developed for cross sections passing through SS-2, the strong motion station at the top of the landfill, for

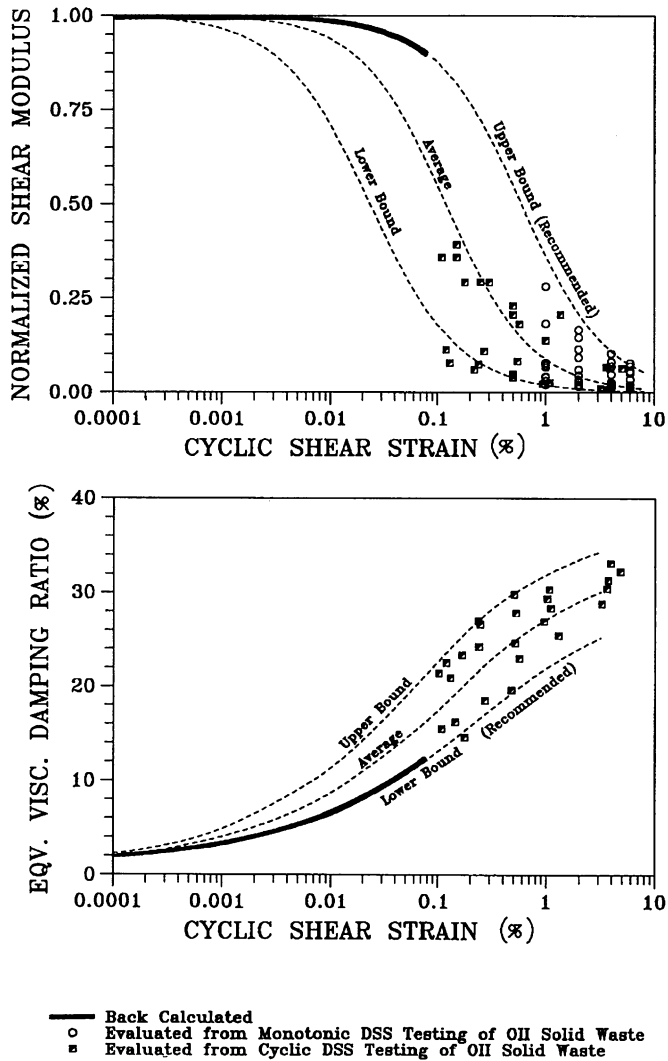


Fig. 10. OII waste shear modulus reduction and damping curves (Matasovic and Kavazanjian 1998, © ASCE)

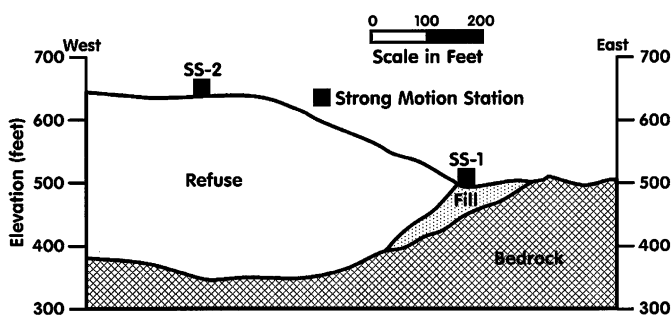


Fig. 11. Cross section through east end of the OII landfill (data from Hushmand Associates 1994)

the purpose of establishing the equivalent linear shear modulus reduction and damping curves: one model was developed in the longitudinal (east-west) direction and the other model was developed in the transverse (north-south) direction. Both models were employed to develop the best fit curves for the equivalent linear soil model. As discussed by Matasovic and Kavazanjian

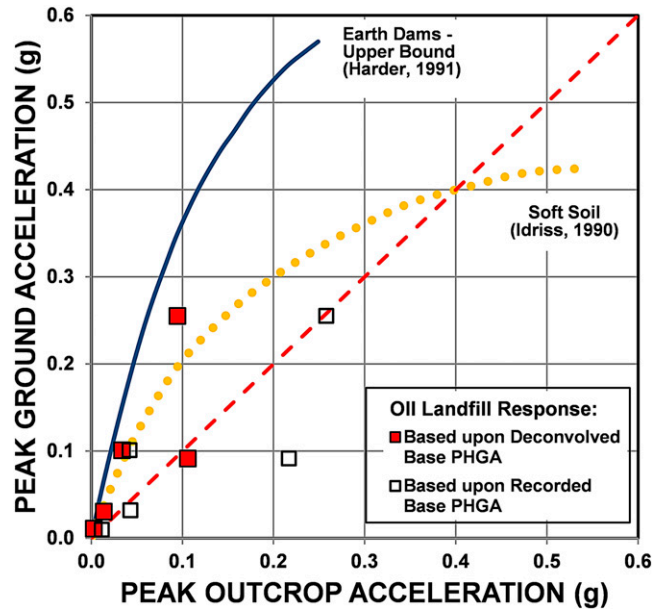


Fig. 12. Comparison of base and top deck PHGA at the OII landfill

(1998) and shown in Fig. 10, the best-fit normalized shear modulus and damping curves corresponded to the upper bound shear modulus and lower bound damping values developed from the laboratory simple shear testing program. Therefore, as indicated on Fig. 10, the upper bound normalized shear modulus and lower bound damping curves developed from the field and laboratory data were considered to be the recommended curves for the OII landfill based upon the back analysis of landfill seismic response. The recommended normalized shear modulus reduction and damping curves shown in Fig. 10 remain widely used for seismic analysis of MSW landfills in practice today.

The ground motions for closure design of the OII landfill were based upon the maximum credible earthquake (MCE), defined as the most damaging earthquake expected to occur within the currently known tectonic framework. The seismic hazard analysis identified two potential MCE events that had to be considered in the seismic response analysis: a nearby M_w 6.9 event generating a free-field PHGA of 0.61 g in a hypothetical bedrock outcrop at the site and a distant M_w 7.8 event generating a free-field PHGA of 0.23 g in a hypothetical bedrock outcrop at the site. A suite of six time histories was established to represent these two design events. The M_w 6.9 design event was represented by the Corralitos-Eureka Canyon Road (0°) record from the M_w 6.9 Loma Prieta earthquake, the Rio Dell (270°) record from the M_w 7.0 Cape Mendocino/Petrolia earthquake, and the C4 synthetic record developed by Silva (W. J. Silva, personal communication, 1995) to model near-field strong ground motions from the M_w 6.7 Northridge earthquake. The M_w 7.8 design event was represented by the Joshua Tree Fire Station (90°) record from the M_w 7.3 Landers earthquake, the Tabas, Iran, accelerogram from the M_w 7.4 Tabas-e-Golshan earthquake, and the A1 synthetic accelerogram developed by Jennings et al. (1968) to represent a large magnitude earthquake on the San Andreas fault, a particularly long duration record. Table 4 presents the characteristics of the design strong ground motion records. Fig. 13 compares the acceleration response spectra from these time histories to target spectra developed using the Abrahamson and Silva (1997) attenuation relationship for shallow crustal earthquakes. The seismic response analysis was conducted using the same shear wave velocity and unit weight profiles

Table 4. Characteristics of OII Design Ground Motions

Design event	Record	PHGA		
		(g)	D_s (s)	T_p (s)
Near field	Corralitos-Eureka Canyon Road (0°)	0.61	6.9	0.29
	Rio Dell (270°)	0.61	15.4	0.24
	C4 synthetic (W. J. Silva, personal communication, 1995)	0.61	6.7	0.20
Far field	Joshua Tree Fire Station (90°)	0.23	28.2	0.68
	Tabas, Iran (90°)	0.23	16.3	0.16
	A1 synthetic (Jennings et al. 1968)	0.23	41.5	0.46

Note: D_s = significant duration of strong ground shaking evaluated using the Trifunac and Brady (1975) duration model; PHGA = design peak horizontal ground acceleration in hypothetical bedrock outcrop at the geometric center of the site; T_p = predominant period of the record.

employed in the back analysis and the range of shear modulus reduction and damping curves shown in Fig. 10, although the best-fit recommended parameters in Fig. 10 invariably produced the maximum response of the waste mass.

Limit Equilibrium Analyses

Static and pseudostatic limit equilibrium analyses were conducted to evaluate static stability of the waste mass and cover system and to provide input for seismic deformation analyses, i.e., to provide the yield acceleration of different potential sliding masses. These analyses included both planar (sliding block) failure surfaces for analyses that considered horizontal planes of weakness and circular failure surfaces for the waste mass and veneer failure surfaces for the cover. No pore pressures were considered in the analyses using circular surfaces. However, the sliding block analyses considered the potential for horizontal perched water tables, with 2.4 m of hydraulic head perched on the basal horizontal surface corresponding to the sliding block failure plane based upon the maximum observed liquid level in the borings. Waste mass failure surfaces were propagated from the waste face at the heel of each bench back into the waste mass. In the static stability assessment, these failure surfaces were progressively enlarged until the factor of safety in the static analysis reached a minimum value. All of the static factors of safety in the waste mass stability analyses were comfortably above the design criterion of 1.5, even when the perched water table was considered.

In the pseudostatic analyses, the failure surfaces were progressively enlarged until the yield acceleration in the pseudostatic analysis dropped to an asymptotic value. These yield accelerations were employed in the Newmark-type seismic deformation analyses described subsequently. Fig. 14 shows the results of the waste mass pseudostatic stability analyses for sliding block surfaces for one of the assumed horizontal failure planes. In this figure, the calculated yield acceleration is plotted directly above the point where the back slope of the failure surface intersects the landfill surface, with the scale for the yield acceleration in gravitational acceleration on the right-hand side of the figure. Fig. 15 summarizes the results of one set of the pseudostatic stability analyses for circular surfaces using a similar format to Fig. 14.

Veneer stability analyses for the existing soil cover indicated unacceptably low factors of safety for the north slope for both static and extreme event conditions, e.g., seepage parallel to the slope and seismic loading. These low factors of safety indicated that some type of remedial action would be required for cover soil veneer stability. Ultimately, the soil cover on the north slope was completely removed and replaced with horizontal layers of geogrid reinforcement anchored back into the waste.

Seismic Deformation Analyses

The results of the seismic response analyses and pseudostatic limit equilibrium analyses were combined to conduct a Newmark-type seismic deformation analysis of the waste mass. A mass-weighted average acceleration time history was calculated for representative blocks of the waste mass using the seismic coefficient option in *QUAD4M* (Hudson et al. 1994) for each of the six design ground motion time histories. These average acceleration time histories were used in a Newmark analysis to develop curves of calculated permanent seismic displacement versus yield acceleration for each of the six design ground motions for each representative block. The yield accelerations calculated in the pseudostatic limit equilibrium analysis were then compared with the permanent seismic displacement curves to estimate the range of permanent seismic displacements anticipated in the design event. Fig. 16 shows the representative blocks of waste considered in the Newmark analysis along with the maximum peak average acceleration calculated for each block from the six records employed in the seismic response analysis.

Fig. 17 shows a set of calculated permanent seismic displacement curves from the Newmark analysis for the block sliding failure mode. Calculated permanent seismic displacement was sensitive to the strong motion time history as well as to the particular block for which it was calculated. However, in all cases, the calculated displacements for the far-field time histories were relatively small. Furthermore, the Rio Dell near-field input motion generally resulted in the largest seismic displacement, although the Corralitos-Eureka Canyon Road motion occasionally produced a larger seismic displacement. The calculated permanent seismic displacements from the Newmark analyses for all input motions and all waste blocks satisfied the preestablished design criterion for the waste mass of a maximum calculated permanent seismic displacement of 0.9 m and were typically less than 0.3 m.

Static Deformation Analysis

The predesign geotechnical analyses also included a static deformation analysis to help evaluate the performance of an engineered final cover over a 30-year period following engineered final cover placement. Performance concerns over the 30-year period following placement of the final cover included reversal of drainage grades, excessive tensile loading of geosynthetic elements of the cover system, and cracking of cover soil layers. The static deformation analysis employed the time-dependent double yield surface Cam Clay plasticity model developed by Hsieh et al. (1990), as coded into the finite-element computer program *PISA* (Chan and Morgenstern 1992). This model is capable of accounting for both volumetric creep and deviatoric creep within the waste mass. In using this model, it was assumed that increasing the coefficient of secondary compression beyond the value measured in the laboratory testing program could account for volume loss due to waste degradation. Waste properties were initially based upon the laboratory testing data and then adjusted in an attempt to match the results of a 1980s load test performed at OII, measurements of lateral deformation along the steep north slope from the inclinometers installed by the EPA after the Whittier earthquake, and measurements of vertical and lateral surface displacements subsequent to cessation of waste placement in 1985–1994, a period over which it was reported that the landfill had been settling at approximately 0.3 m per year (Measurement Science 1996).

Back analysis of the field data proved to be a difficult task. When parameters were chosen to match the vertical displacements

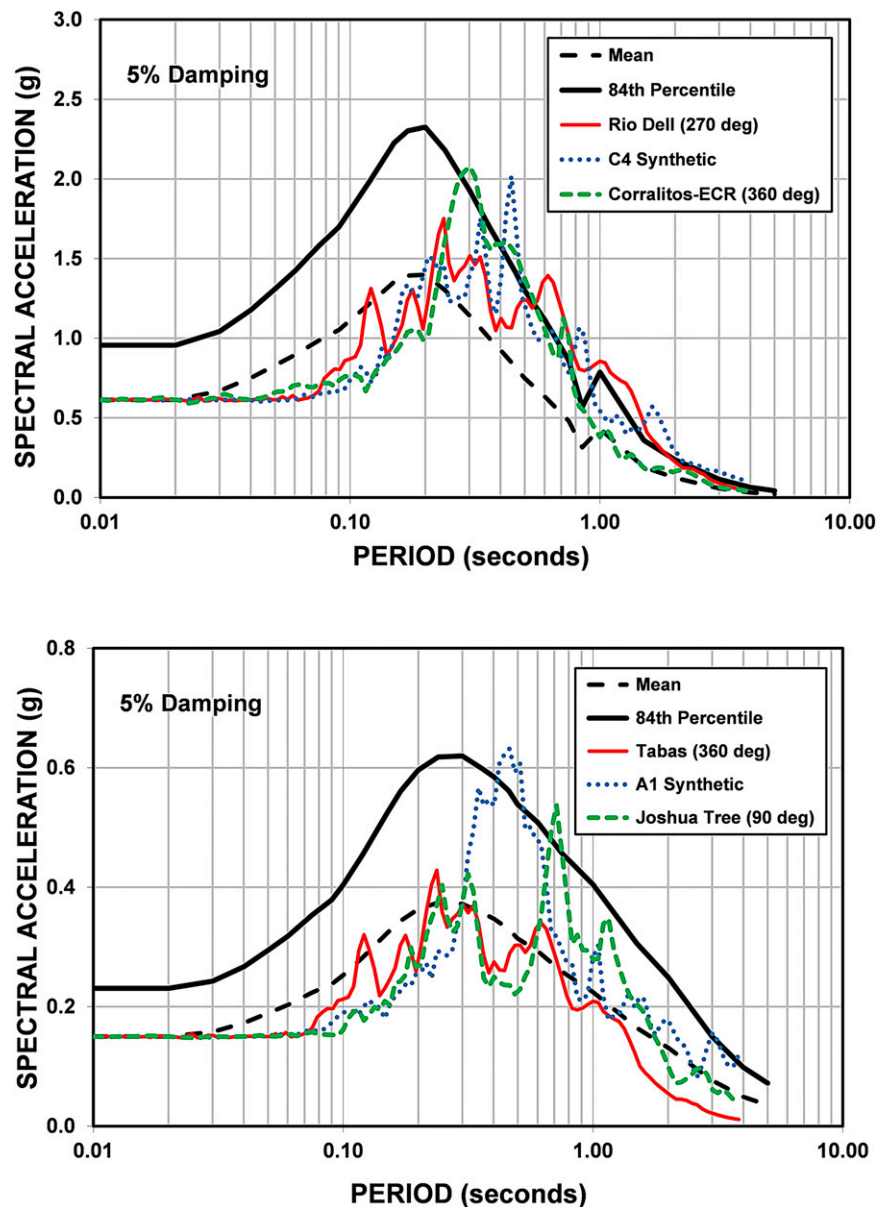


Fig. 13. Acceleration response spectra of OII design earthquakes

of the landfill, the predicted lateral displacements did not match the field values. When parameters were adjusted to match the observed lateral displacements, the predicted vertical displacements did not match field values. Therefore, a set of parameters that enveloped both the vertical and lateral displacements observed at the site were used to predict the 30-year deformation response of the waste mass.

Table 5 presents the Cam Clay properties used to model the waste in the finite-element analysis of long-term deformation of the waste mass. In this table, C_c is the virgin compression ratio, C_r is the recompression ratio, e_{cs} is the critical state void ratio at a unit normal stress, p_o is the apparent preconsolidation pressure of the waste at the end of construction, and C_α is the coefficient of secondary compression. The primary conclusions drawn from the static deformation analysis were that large deformations, and thus constant maintenance, should be expected over the 30-year postclosure period considered in the analysis and that uncontrolled lateral deformation due to deviatoric creep rupture was unlikely.

Toe Buttress Analysis

Because of the proximity of the waste mass to homes on the south side of the landfill, special attention was paid to the stability of the MSE toe buttress constructed to restrain waste mass lateral deformations in this area. Of particular concern was a section of the toe buttress constructed where the waste slope terminated at the property line for the adjacent homes. This section of the toe buttress, shown in the cross section in Fig. 18, was founded upon a single row of 0.9-m-diameter cast-in-place concrete drilled piers founded in bedrock at a center-to-center spacing of 2.4 m. The toe buttress analysis included consideration of both global stability and internal stability. Limit equilibrium analyses of global stability yielded a static factor of safety of 2.6 and a yield acceleration of 0.45g. The seismic response analysis yielded a maximum value of 0.48g for the peak average horizontal acceleration of the toe buttress from the six design time histories. As the yield acceleration was only slightly less than the peak average acceleration, the calculated permanent seismic

deformation of the toe buttress was minimal, i.e., less than 25 mm, and the global, i.e., external, seismic stability of the toe buttress was considered satisfactory.

Zornberg and Kavazanjian (2001) describe the analysis of the internal integrity of the MSE toe buttress in detail. The internal stability analysis was conducted using finite-element analysis that modeled the individual geogrid layers in the toe buttress. In the analysis, the toe buttress was first subjected to the projected 30-year static deformation of the underlying waste mass and then subject to a pseudostatic seismic load. The 30-year static settlement was projected based upon settlements measured at the V ditch shown in Fig. 18 at the back of the top deck of the buttress. Settlements measured at eight sections along the V ditch over the 9-year period after the end of toe buttress construction were linearly extrapolated out to 30 years on the plot of settlement versus log time presented in Fig. 19.

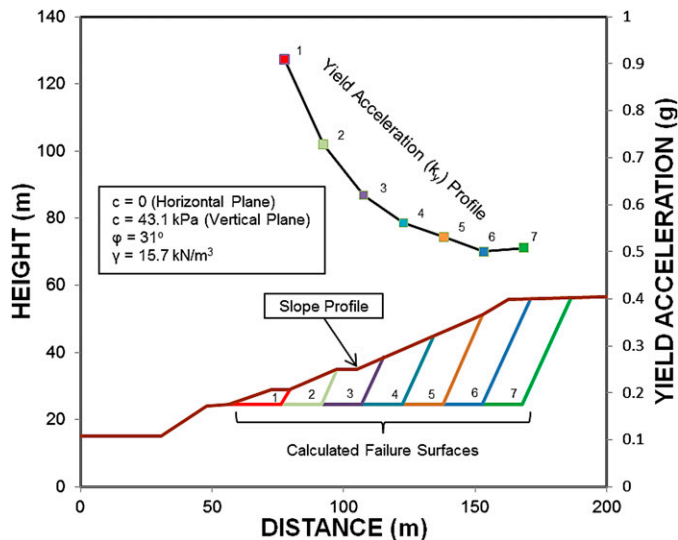


Fig. 14. Summary plot for pseudostatic limit equilibrium analyses for selected block-type failure surfaces at OII

To provide a conservative basis for the internal stability assessment, the extrapolated settlement of 1.95 m shown in Fig. 19 for Section 3, the maximum extrapolated value, was used in the design.

In the finite-element analysis, the bottom of the finite-element mesh used to model the toe buttress was subjected to a linear distribution of settlement that varied from zero at the drilled pier location to a maximum value at the back edge of the mesh, as illustrated in Fig. 18. The magnitude of settlement at the back edge of the mesh was incrementally increased until the settlement at the V ditch equaled the value of 1.95 m extrapolated from the field measurements at Section 3. Once the settlement at the V ditch equaled the target value of 1.95 m, a progressively increasing pseudostatic horizontal load, as defined by a seismic coefficient, was applied to the centroid of each element in the finite-element mesh up to a value corresponding to a seismic coefficient, i.e., an average horizontal acceleration, of 1.0g. The tensile strains in the geogrid were monitored at every stage of this process. Fig. 20 shows the calculated tensile strains in the geogrid at 6 of the 11 reinforcement layers for four cases: at the end of construction; following 9 years of settlement termed the current condition; following imposition of the extrapolated 30-year settlement of 1.95 m; and for a seismic coefficient of 1.0g after 30 years of settlement. The maximum geogrid tensile strain was calculated for reinforcement level 9 and was slightly more than 8% after 30 years of settlement and imposition of a seismic coefficient of 1.0g.

Numerical analysis of the toe buttress was accompanied by testing of archived samples of the actual geogrid used to construct the MSE buttress. One sample of the archived geogrid was subjected to creep loading at a stress level representative of the projected stress level in geogrid layer 9 after 30 years of settlement and then after a sustained period of creep under an elevated temperature, subjected to renewed loading to see if the creep loading induced any degradation in the geogrid tensile strain capacity or tensile strength. The stress strain behavior of the geogrid subsequent to loading following a sustained period of creep showed an apparent stiffening but then quickly returned to the stress strain curve that characterized monotonic loading of the geogrid. Therefore, it was concluded that the potential for long-term degradation of the geogrid tensile strength and strain capacity due to creep was relatively small, even after 30 years of settlement.

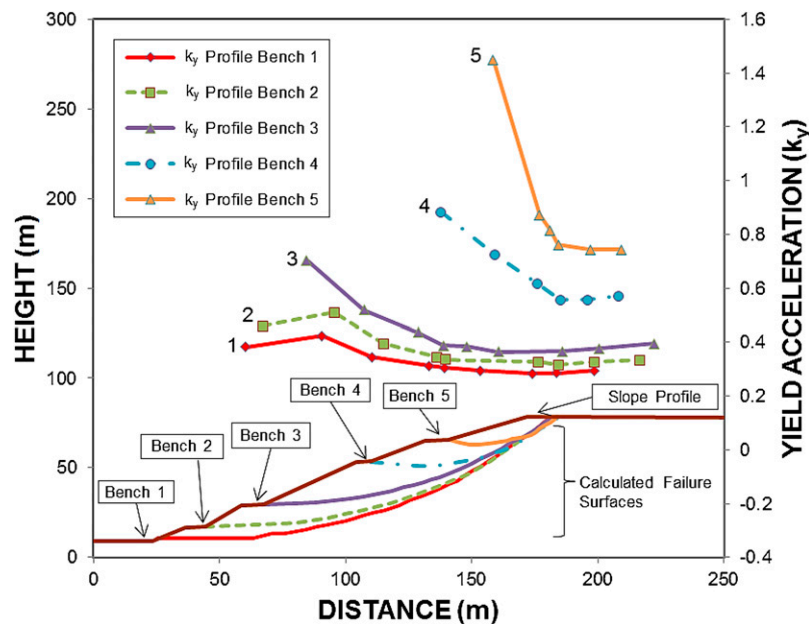


Fig. 15. Summary plot for pseudostatic limit equilibrium analyses for circular failure surfaces at OII

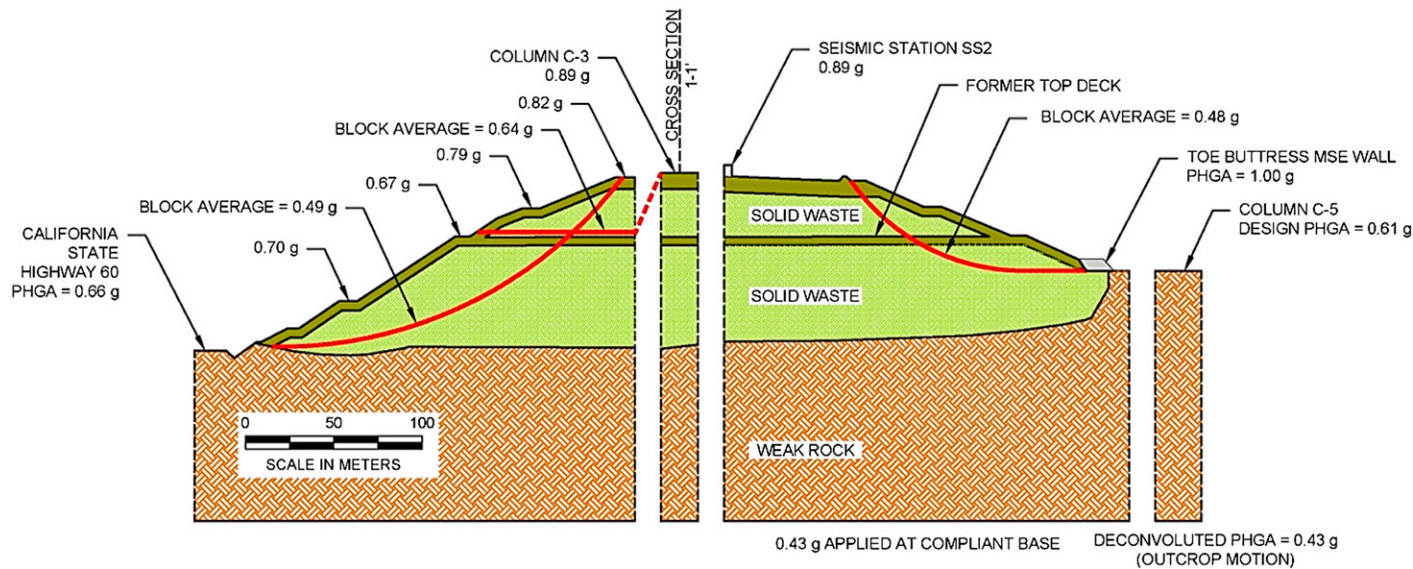


Fig. 16. Representative blocks of the OII waste mass used in the Newmark seismic deformation analysis

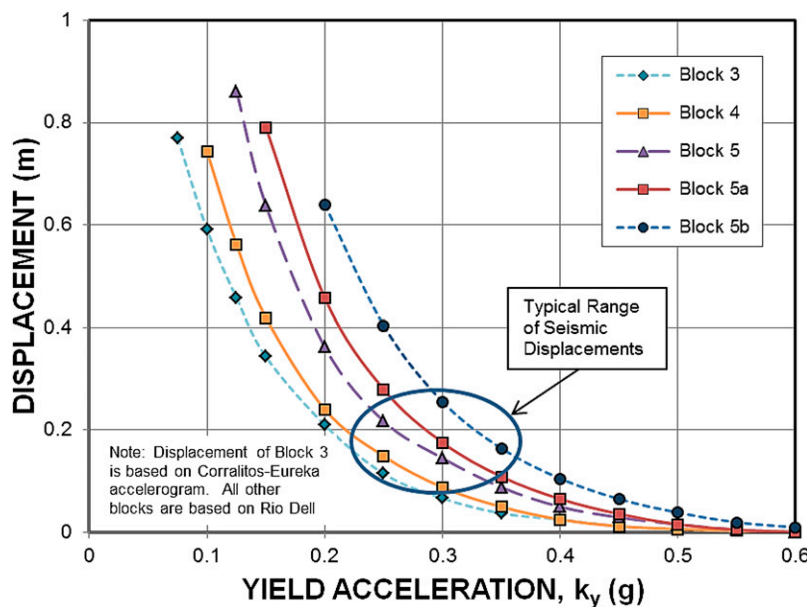


Fig. 17. Selected seismic displacement curves for block failure mode from Newmark analysis for the OII waste mass

Table 5. Best-Fit Cam Clay Parameters for OII Waste from Finite-Element Back Analysis

Parameter	Value
C_c	0.42
C_r	0.042
Φ	32°
e_{cs}	2.60
p_o	6,000 psf
C_α	0.14

The laboratory testing on archived samples indicated the strain capacity of the geogrid in uniaxial tension is somewhat in excess of 18%. However, the strain capacity of a geosynthetic element subject to plane strain loading will be less than that measured in a uniaxial tension test. Giroud (2005) presents a relationship between the yield

strain of a geosynthetic element in uniaxial tension and the yield strain of a geosynthetic element in plane strain that depends only on Poisson's ratio. The maximum difference in yield strain occurs for a Poisson's ratio of 0.5, in which case the plane strain yield strain is 0.866 times the uniaxial yield strain. Even after applying a correction factor of 0.866 to the laboratory test data to account for plane strain conditions in the field, the maximum strains induced in the geogrid by the design earthquake seismic loading after 30 years of settlement remain well within the tensile strain capacity of the geogrid.

Subsequent Advances in Understanding of MSW Properties

The predesign investigation for the OII Superfund site landfill established a new standard for solid waste landfill engineering.

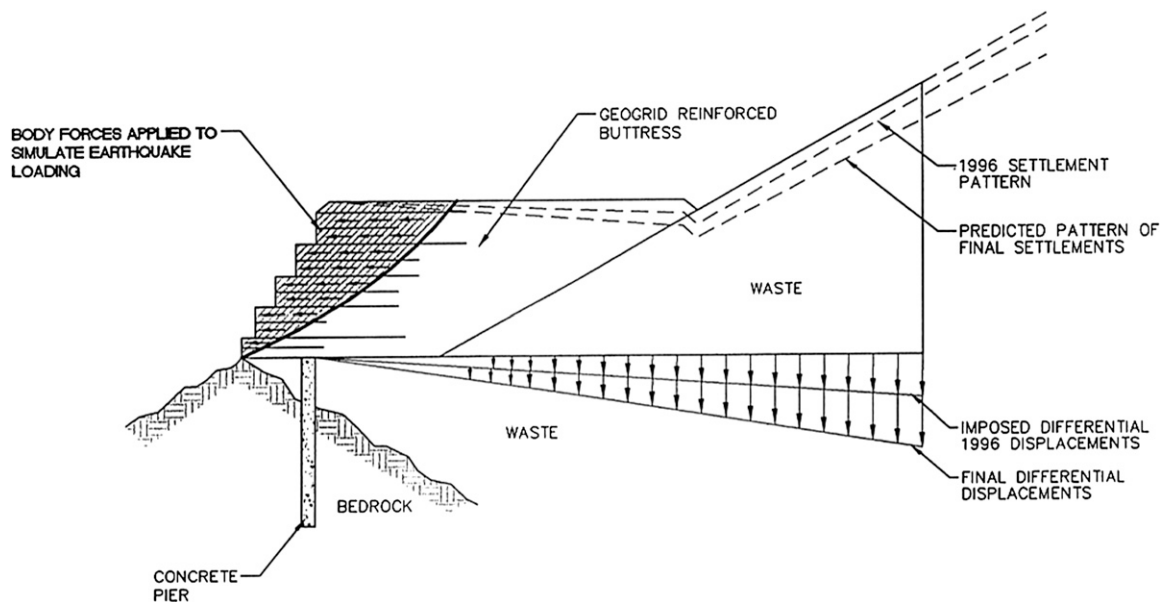


Fig. 18. OII MSE toe buttress cross section

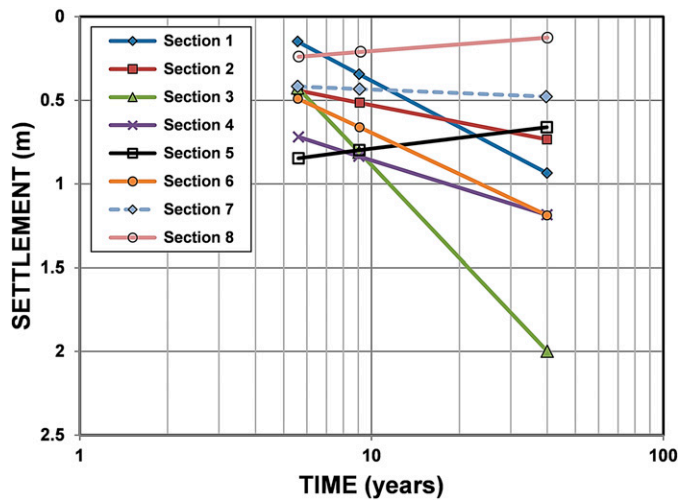


Fig. 19. Measured and extrapolated settlement at the OII MSE toe buttress V ditch

However, in the 15 years since its completion, understanding of the mechanical properties of MSW has continued to advance, in many cases along the same lines as established on the OII project. The method of in situ evaluation of MSW unit weight has since been used on a number of other MSW landfills, including the Azusa landfill in southern California, the Tri-Cities landfill in the San Francisco Bay area, and the Cherry Island landfill in Delaware. Zekkos et al. (2006) have employed the data from these studies along with data from other large-scale in situ unit weight evaluations at MSW landfills to develop a family of representative unit weight profiles for MSW landfills that depend upon initial compaction effort and/or soil-sized material content. Zekkos et al. (2010) presented a MSW classification system that was based upon the classification system developed for the OII project and then refined at the Tri-Cities landfill.

Employing some of the same data employed by Kavazanjian et al. (1995) and some additional data, Eid et al. (2000) proposed that the average MSW shear strength envelope be characterized by

a cohesion of 25 kPa and a friction angle of 35°, with a lower bound envelope characterized by zero cohesion and a friction angle of 35°. More recently, Bray et al. (2009) proposed a curved shear strength envelope for MSW characterized by a cohesion of 15 kPa and a friction angle of 36° at a normal stress of 1 atm (96 kPa) and a decrease in friction angle of 5° per log cycle of normal stress. The difference between these shear strength envelopes and the bilinear envelope proposed by Kavazanjian et al. (1995) are relatively minor over the range of normal stress generally encountered in practice.

Zekkos et al. (2008) and Yuan et al. (2011) present shear modulus reduction and damping curves from cyclic laboratory tests on reconstituted MSW from the Tri-Cities landfill. Zekkos et al. (2008) employed large diameter triaxial tests, while Yuan et al. (2011) employed the OII simple shear apparatus. Both studies looked at the effect of waste composition and waste unit weight on shear modulus and damping. The modulus reduction and damping curves developed in both studies fell within the range of upper and lower bound curves developed in the OII project. Thus, while there have been incremental advances in the understanding of MSW properties in the 15 years since the OII predesign investigation was completed, the OII study remains the primary basis for understanding of the mechanical properties of MSW today.

Summary and Conclusions

The findings from the predesign geotechnical investigation of the OII Superfund site landfill may be summarized as follows:

- The waste mass met both static (factor of safety ≥ 1.0) and seismic (maximum calculated permanent seismic displacement ≤ 0.9 m) stability criteria;
- Large static deformations should be expected over the 30 years following final cover construction, requiring continuous maintenance of the final cover;
- The MSE toe buttress should maintain its integrity when subject to the maximum credible earthquakes used in design even after 30 years of settlement; and
- The veneer stability of the soil cover is a major concern, particularly when subject to seismic loading, and some sort of

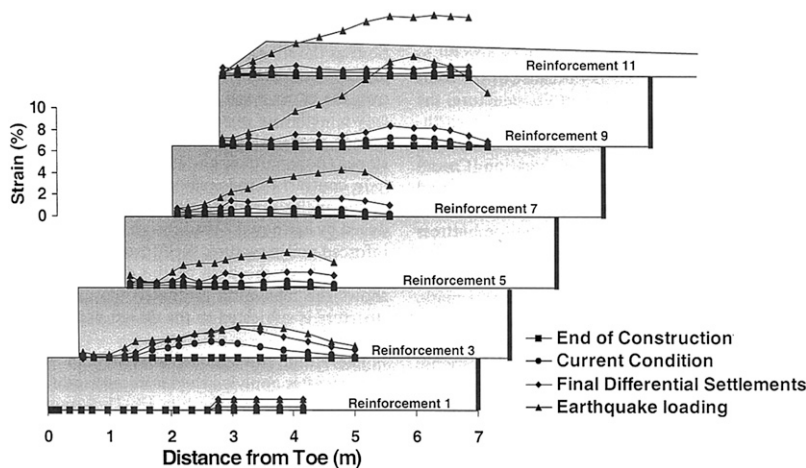


Fig. 20. Calculated tensile strains in the OII MSE toe buttress geogrid after 30 years of settlement and for a seismic coefficient of 1.0g (Zornberg and Kavazanjian 2001)

remedial action regarding cover soil stability appeared to be warranted.

Lessons about MSW properties learned from the predesign geotechnical investigation of the OII Superfund site landfill included the following:

- Municipal solid waste is a relatively strong material, stronger than was often assumed in practice at that time, and can be characterized by a friction angle equal to or greater than 30° ;
- The unit weight of municipal solid waste at the OII landfill is greater than what was typically assumed in practice at that time, with an average value of 15.7 kN/m^3 and reaching values as high as 19.6 kN/m^3 at depth in zones where the waste was saturated;
- The strain-dependent shear modulus degradation (reduction) and increase in damping of the OII waste is relatively slow, and therefore there is a significant potential for amplification of seismic motions within the waste mass;
- The OII waste tends to be anisotropic in structure and properties due to a preferred horizontal orientation for the large waste particles; and
- The deformation of the OII landfill is very nonhomogeneous, and it is very difficult to predict deformations at specific locations, either analytically or by extrapolation of field measurements.

Contributions of the OII landfill predesign geotechnical investigation to landfill engineering practice that were described in this paper included the following:

- An in situ method for evaluating the unit waste of municipal solid waste;
- A field classification system for municipal solid waste;
- A bilinear municipal solid waste shear strength used extensively in practice today;
- Data on compositional influences on the strength and compressibility of municipal solid waste;
- Data on the magnitude and variability of the shear wave velocity within a municipal solid waste landfill; and
- Shear modulus reduction and damping curves for municipal solid waste that are used extensively in practice today.

Acknowledgments

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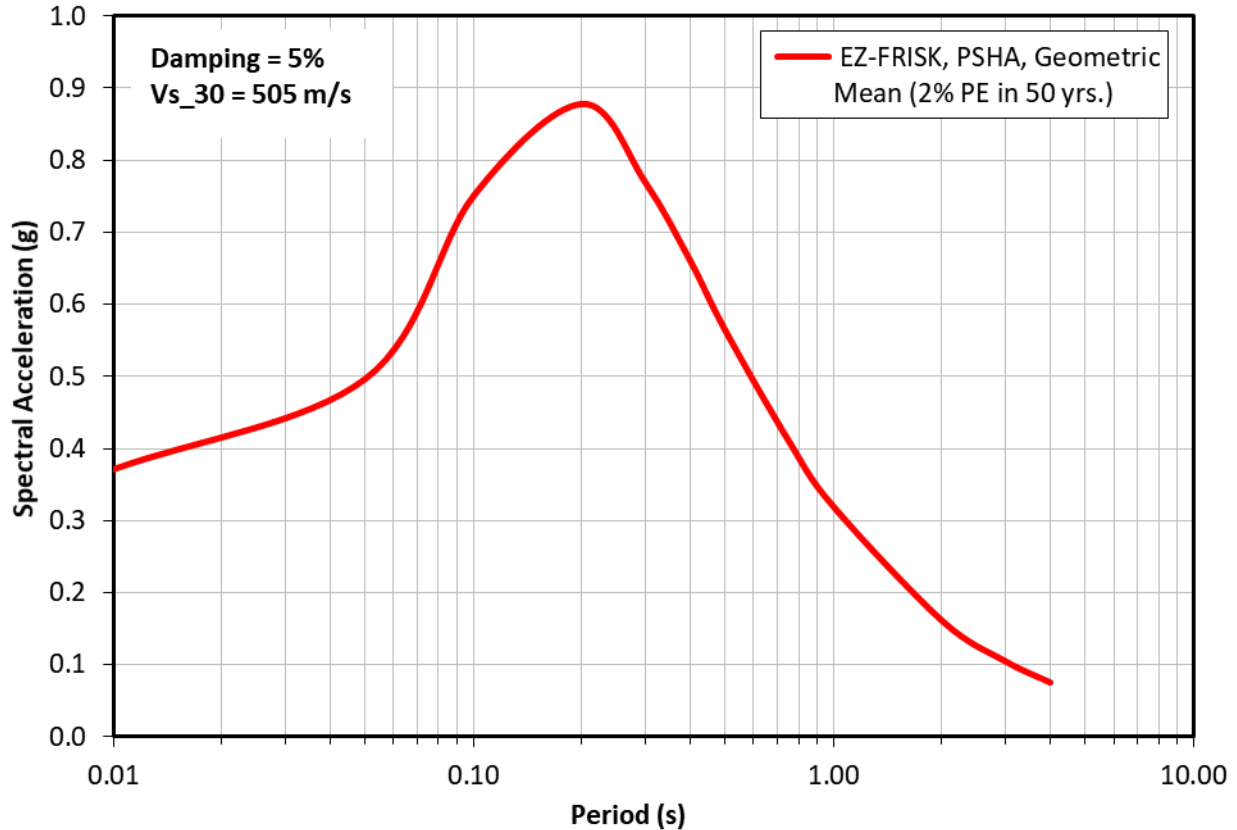
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Appendix B.6

Figures



Notes:

- The Probabilistic Seismic Hazard Analysis (**PSHA**) corresponds to a 2% probability of exceedance (PE) in 50 years, which equals to 2,475-year return period.
- EZ-FRISK program was used to calculate the probabilistic seismic hazard analyses.
- PSHA response spectra are the geometric mean spectra for the soil layer with an average shear wave velocity of about 505 m/s (1,657 ft/s).
- The spectrum shown above is representative of the Maximum Credible Earthquake (MCE).
- The scaling factors for adjustment for the maximum rotated component were not applied.
- The response spectrum was developed for the following conditions:
 - Free-field conditions with no structures and level ground;
 - 5% spectral damping;
 - the shear wave velocity of the soil layer is 505 m/s (1,657 ft/s); and
 - horizontal components of ground motion.

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ACCELERATION RESPONSE SPECTRUM
PEBBLY BEACH LANDFILL
AVALON, CALIFORNIA

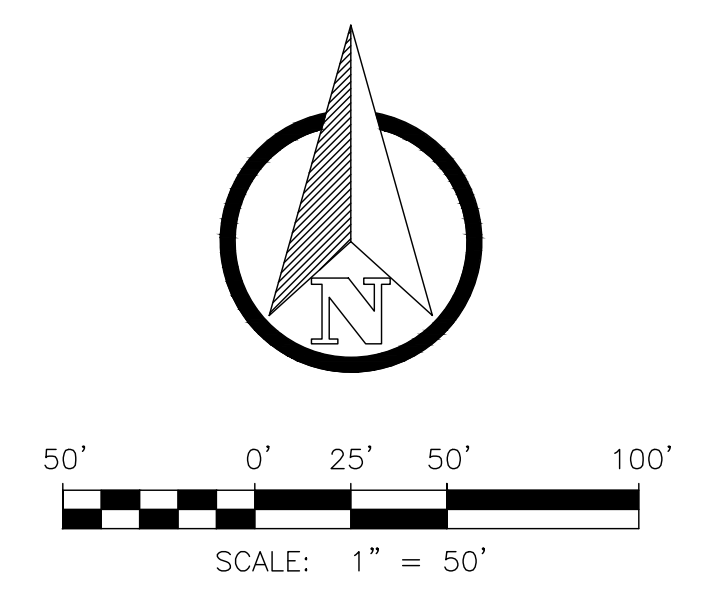
DATE:	FEB 2022	FIGURE NO.	1
PROJECT NO.	SO21.1196.00		



- LEGEND**
- PROPERTY BOUNDARY
 - LIMIT OF WASTE
 - (1400) E.G. GROUND CONTOUR, FEET
 - 1400 DESIGN GROUND CONTOUR, FEET
 - E.G. UNPAVED ROAD
 - E.G. FENCE
 - PROJECT GRADING LIMITS
 - GRADING HINGE
 - RETAINING WALL
 - DESIGN GRADE DIRECTION
 - DETAIL OR SECTION NUMBER
 - SHEET NUMBER FOR DETAIL LOCATION

- FILL PATTERN**
- GRADING WASTE LIMITS
 - PROPOSED ACCESS AND BENCHES
 - CONCRETE CHANNEL

N 3,868,750
 N 3,868,500
 N 3,868,250
 N 3,868,000



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 CAD DESIGN BY: J. AMAYA
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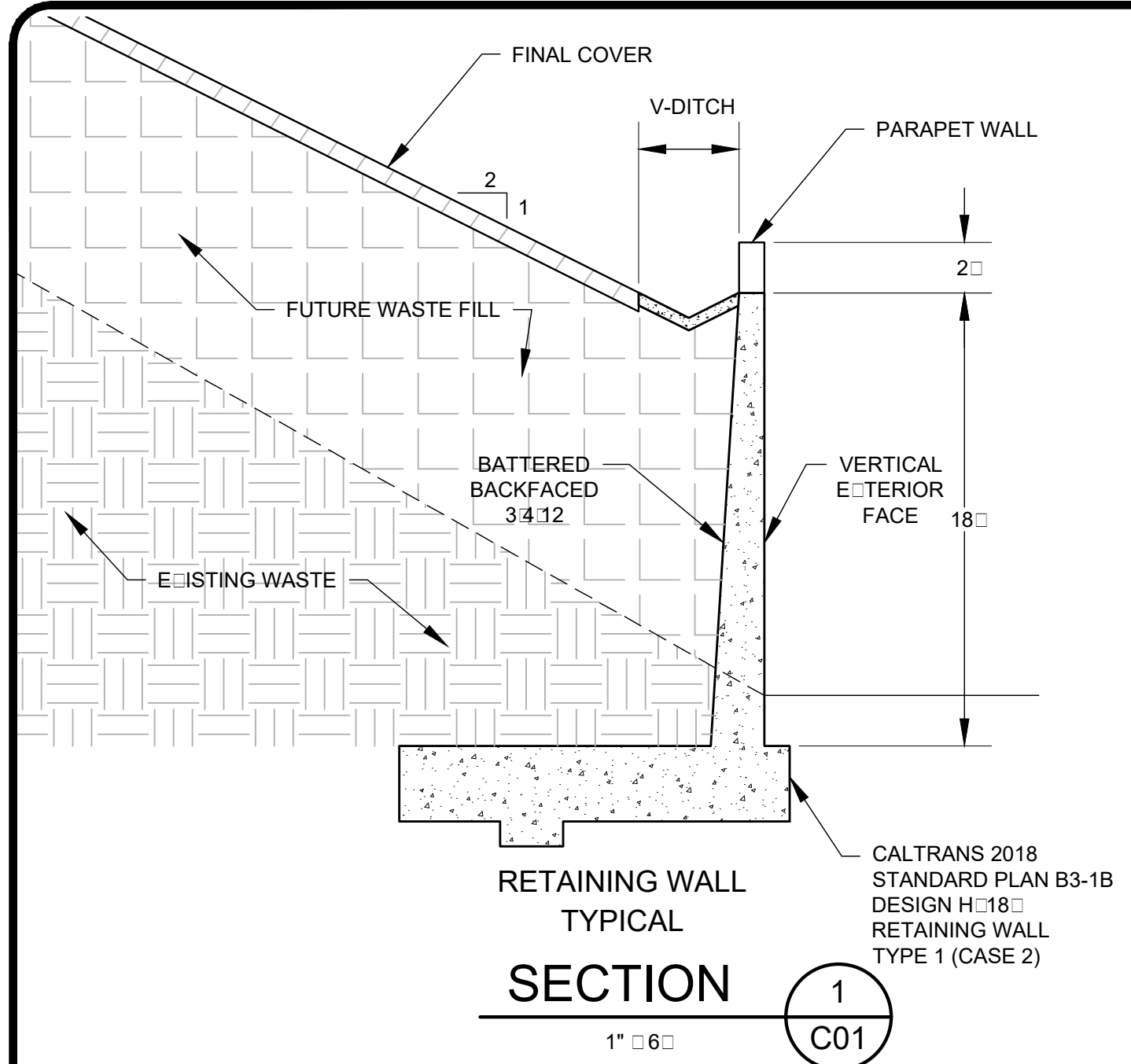
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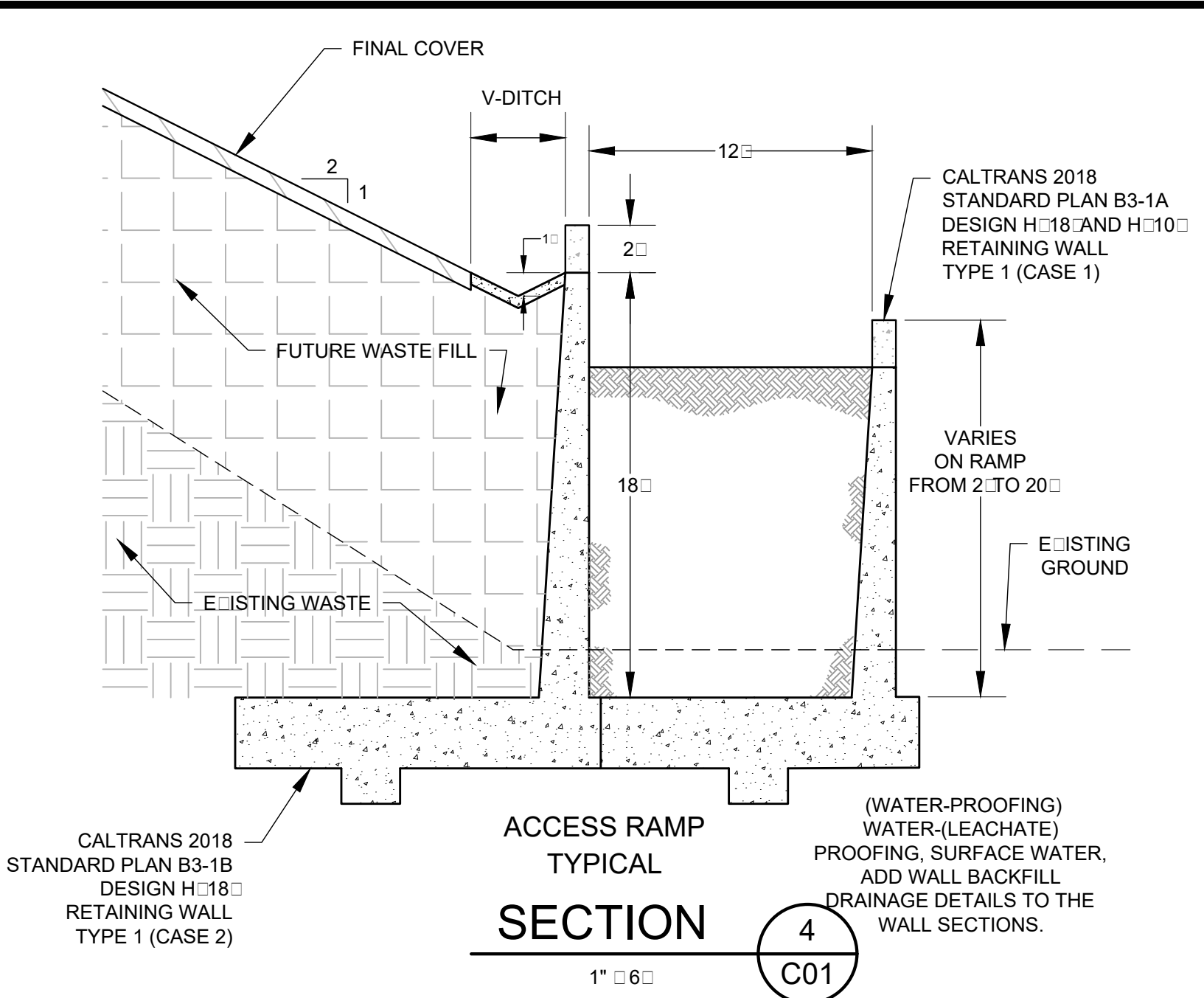
PEBBLY BEACH LANDFILL, AVALON, CA
 SITE LIFE OPTIMIZATION
 DESIGN REPORT DRAWINGS
 PROPOSED FINAL GRADING PLAN

DWG. NO. 2
 PROJECT NO. S021.1196

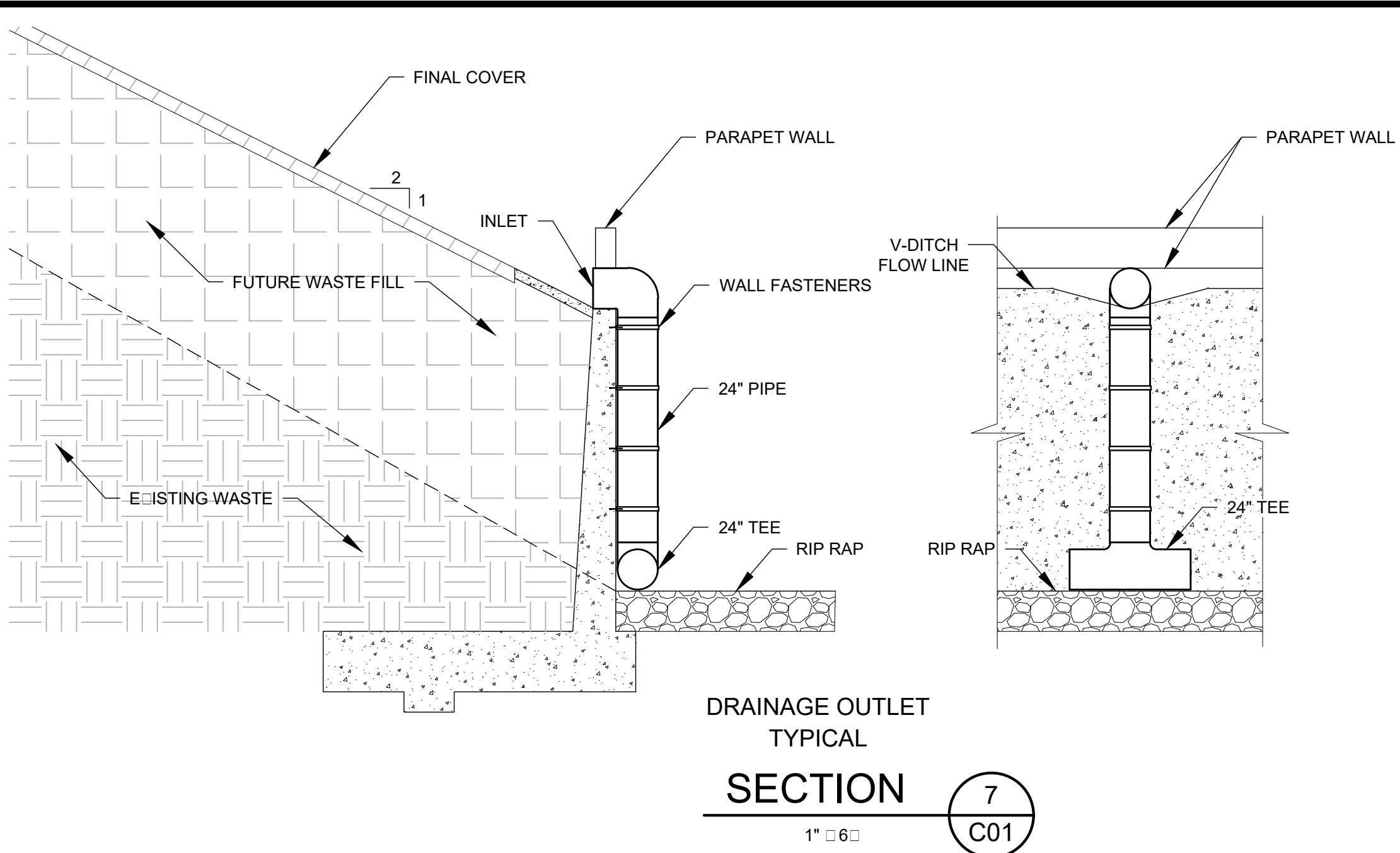
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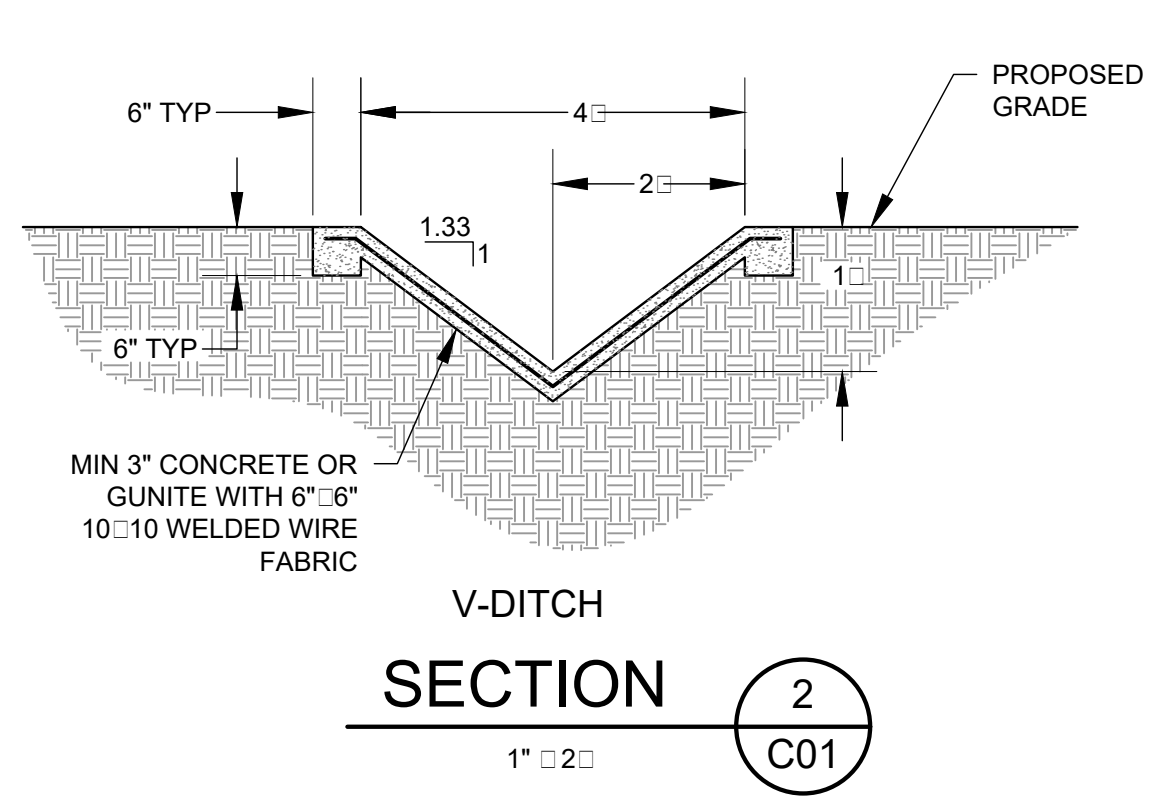
RETAINING WALL
TYPICAL
SECTION 1
1" = 6" C01



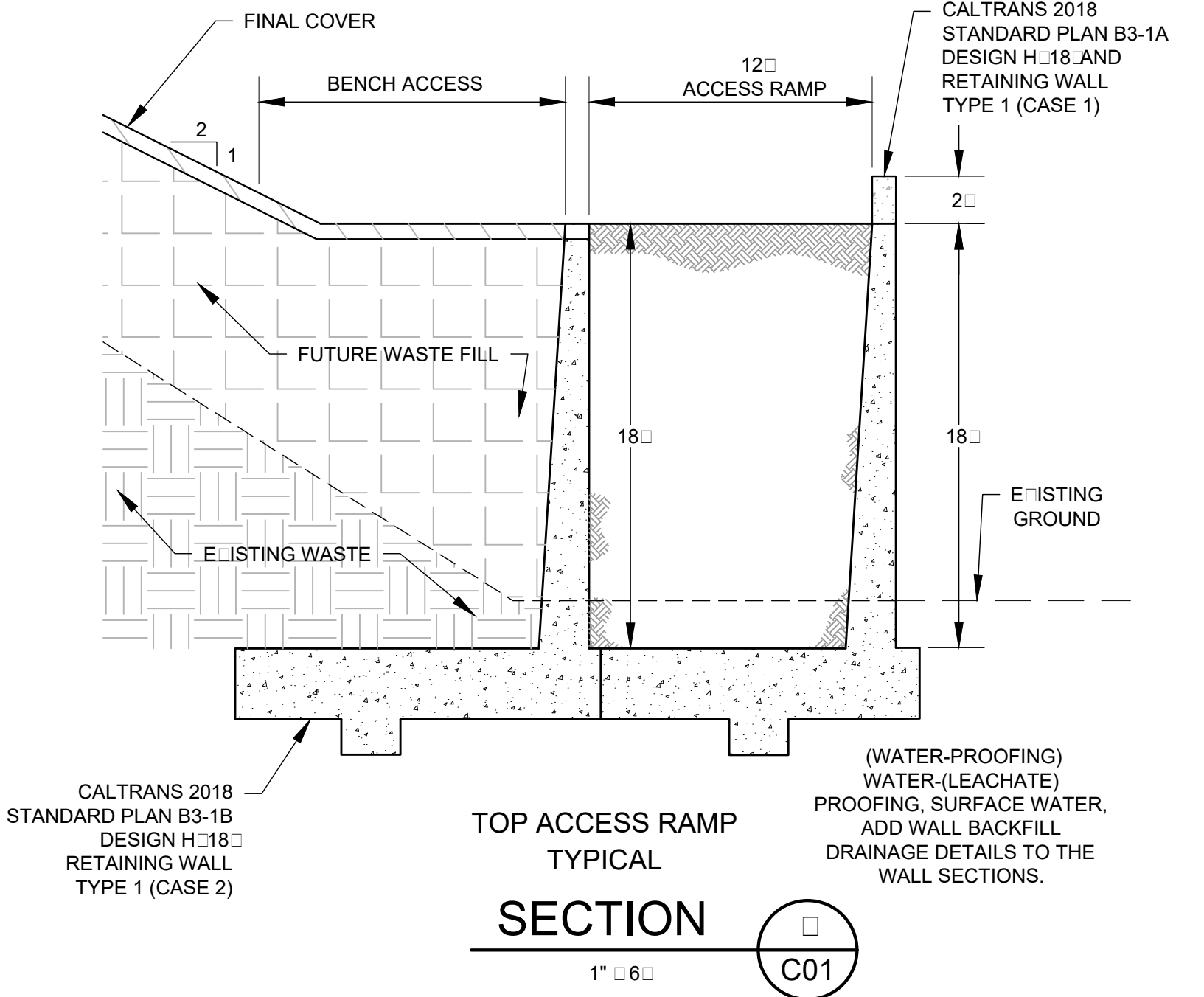
ACCESS RAMP
TYPICAL
SECTION 4
1" = 6" C01



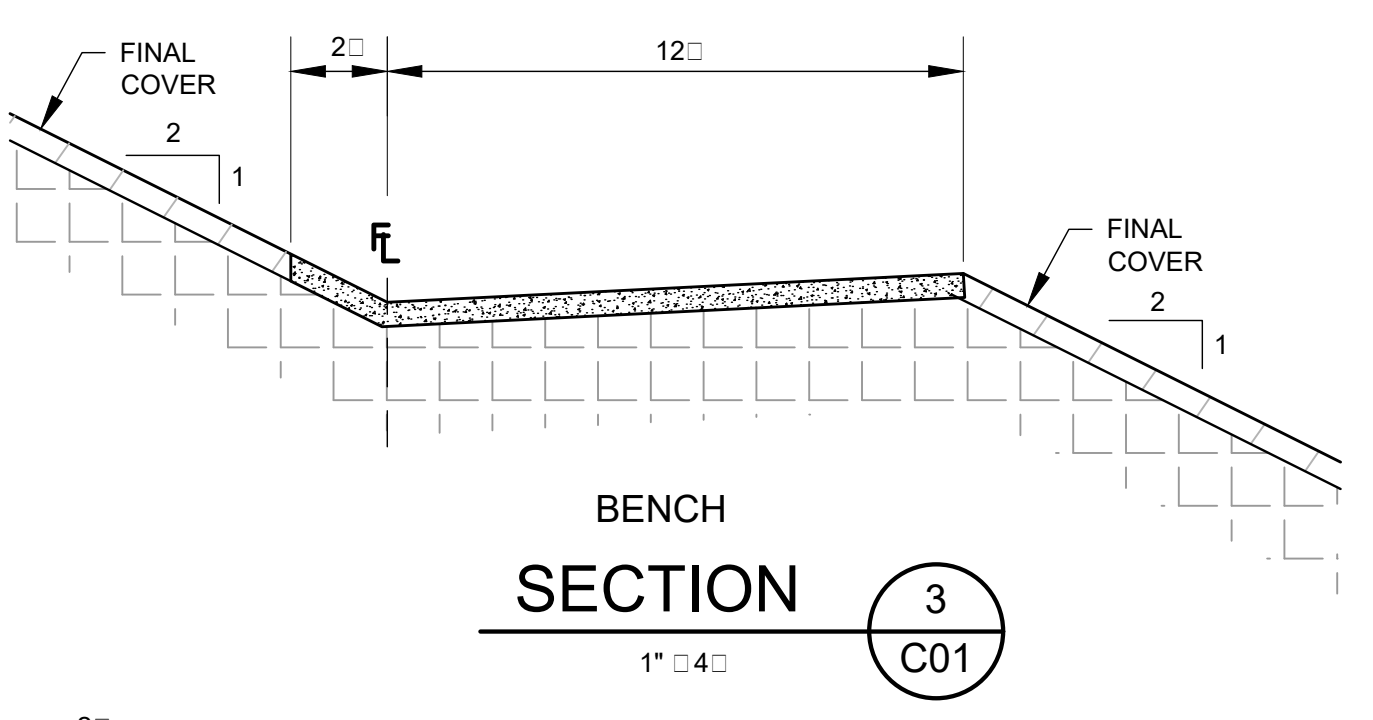
DRAINAGE OUTLET
TYPICAL
SECTION 7
1" = 6" C01



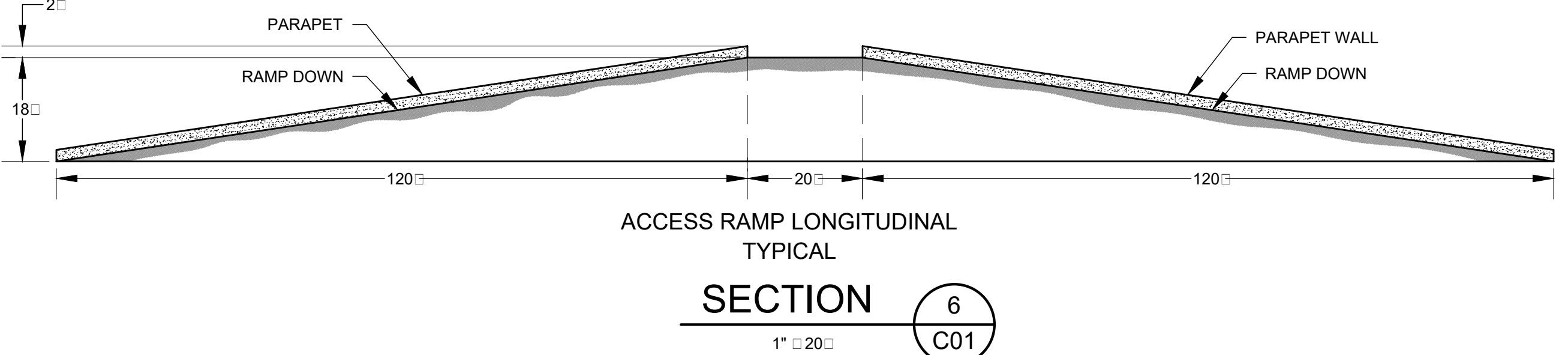
V-DITCH
SECTION 2
1" = 2" C01



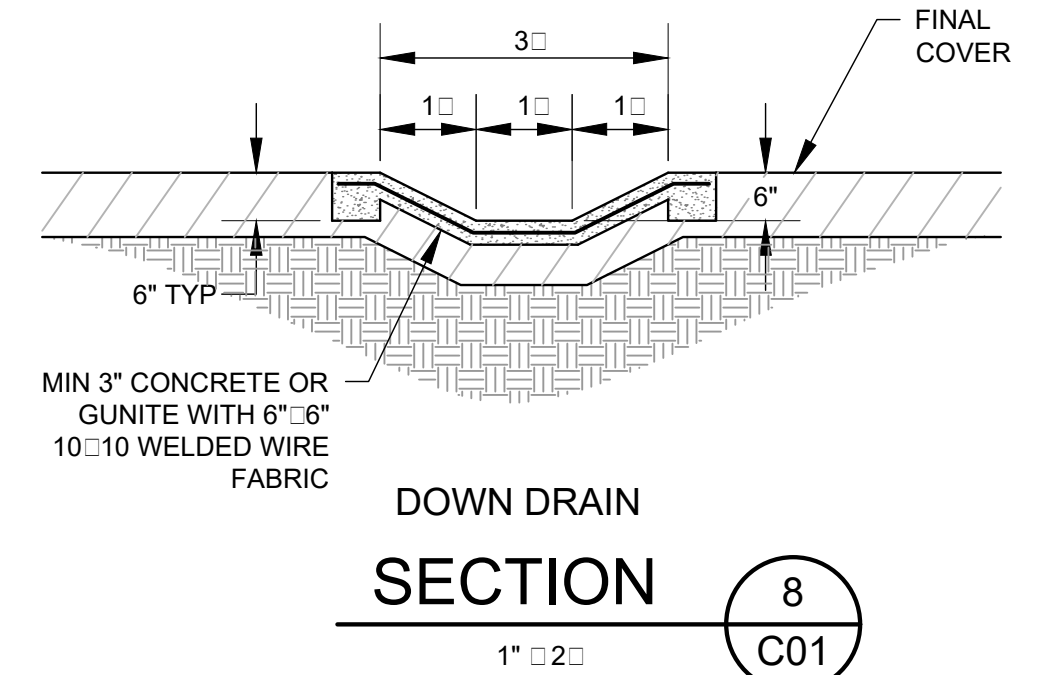
TOP ACCESS RAMP
TYPICAL
SECTION 5
1" = 6" C01



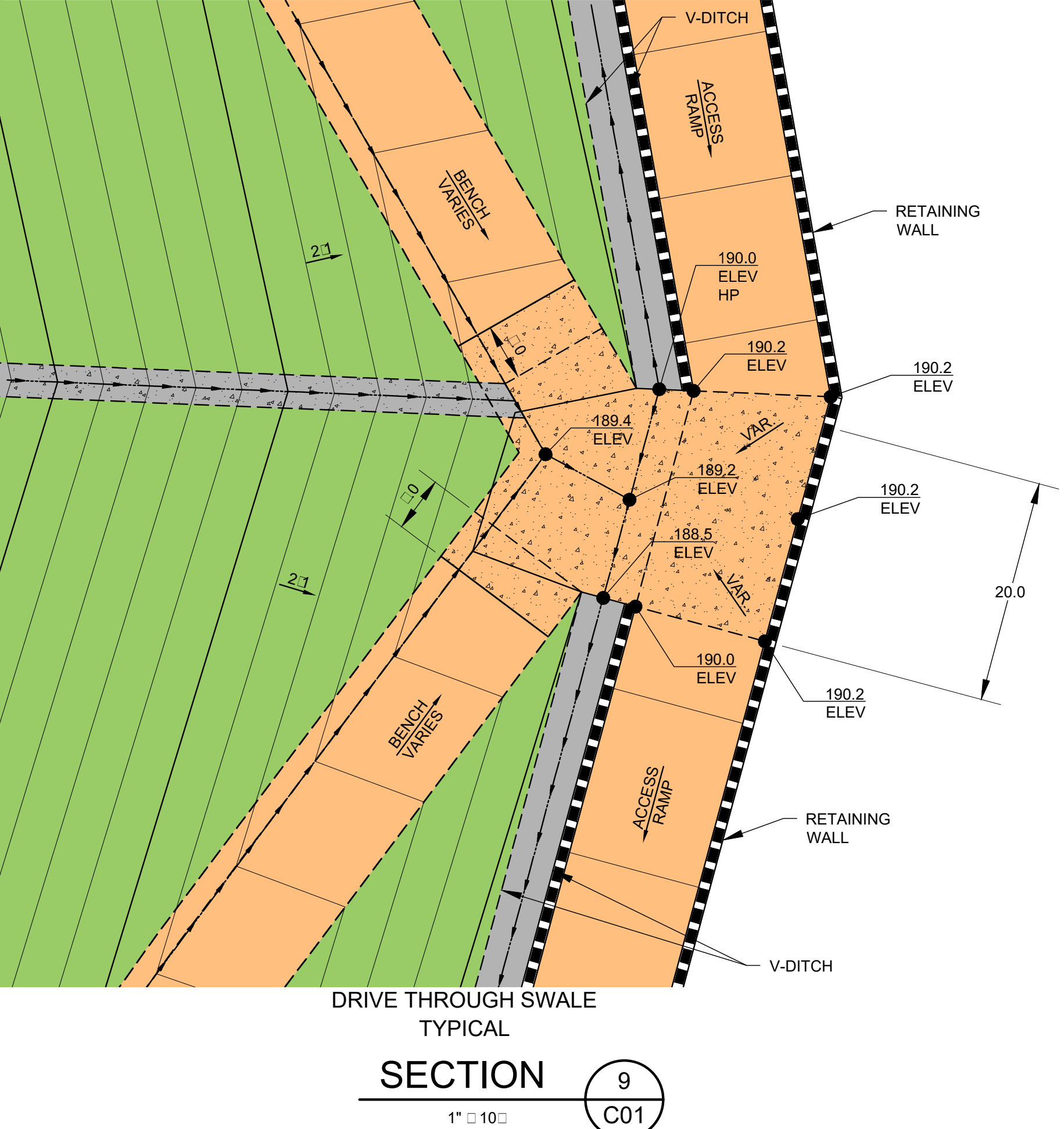
BENCH
SECTION 3
1" = 4" C01



ACCESS RAMP LONGITUDINAL
TYPICAL
SECTION 6
1" = 20" C01



DOWN DRAIN
SECTION 8
1" = 2" C01



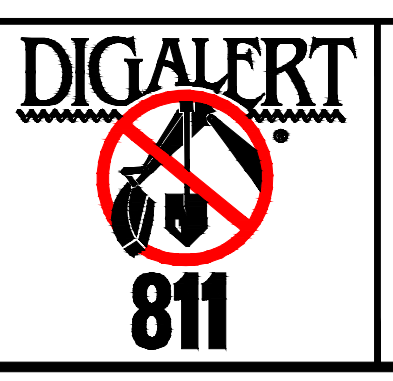
DRIVE THROUGH SWALE
TYPICAL
SECTION 9
1" = 10" C01

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REV. NO.	DATE	DESCRIPTION	APPROVED BY

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 APPROVED BY: D. HARICH

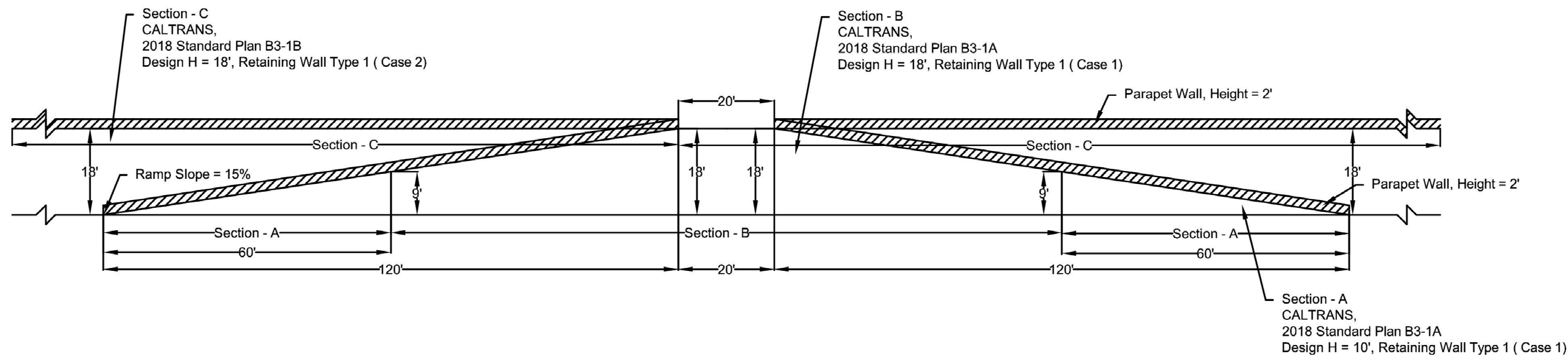


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 SITE LIFE OPTIMIZATION
 DESIGN REPORT DRAWINGS
 GRADING DETAILS

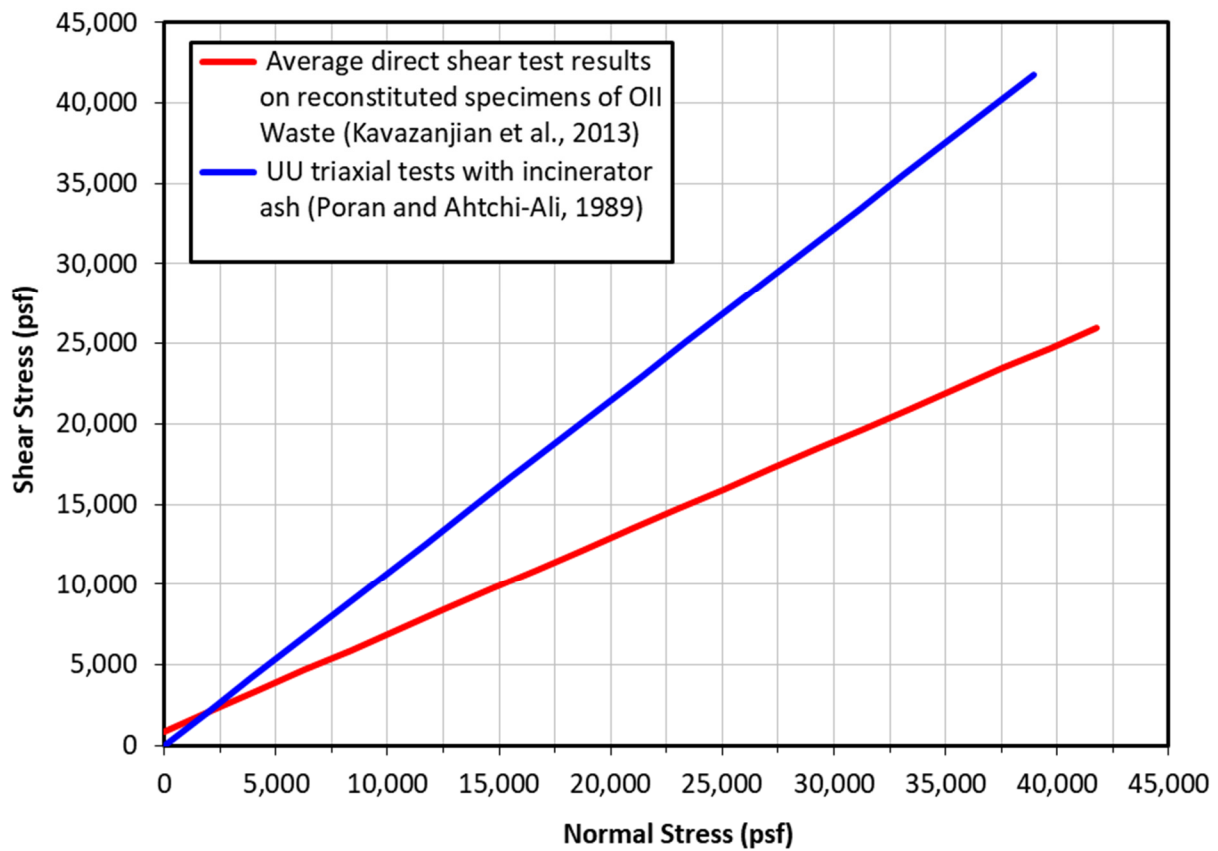
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 DWG. NO. 3
 PROJECT NO. S021.1196



Notes:

- The schematic view shows the retaining wall system along the access ramp of the landfill, and it also depicts the retaining wall for the proposed expansion of the landfill.
- See Appendix D for details about Sections- A, B, and C.
- Section – A: CALTRANS, 2018 Standard Plan, Page: B3-1A, Design Height: H = 10', Name: Retaining Wall Type 1 (Case 1)
- Section – B: CALTRANS, 2018 Standard Plan, Page: B3-1A, Design Height: H = 18', Name: Retaining Wall Type 1 (Case 1)
- Section – C: CALTRANS, 2018 Standard Plan, Page: B3-1B, Design Height: H = 18', Name: Retaining Wall Type 1 (Case 2)

Geo-Logic ASSOCIATES			
SCHEMATIC VIEW OF RETAINING WALL PEBBLY BEACH LANDFILL AVALON, CALIFORNIA			
DATE:	FEB 2022	FIGURE NO.	5
PROJECT NO.	SO21.1196.00		



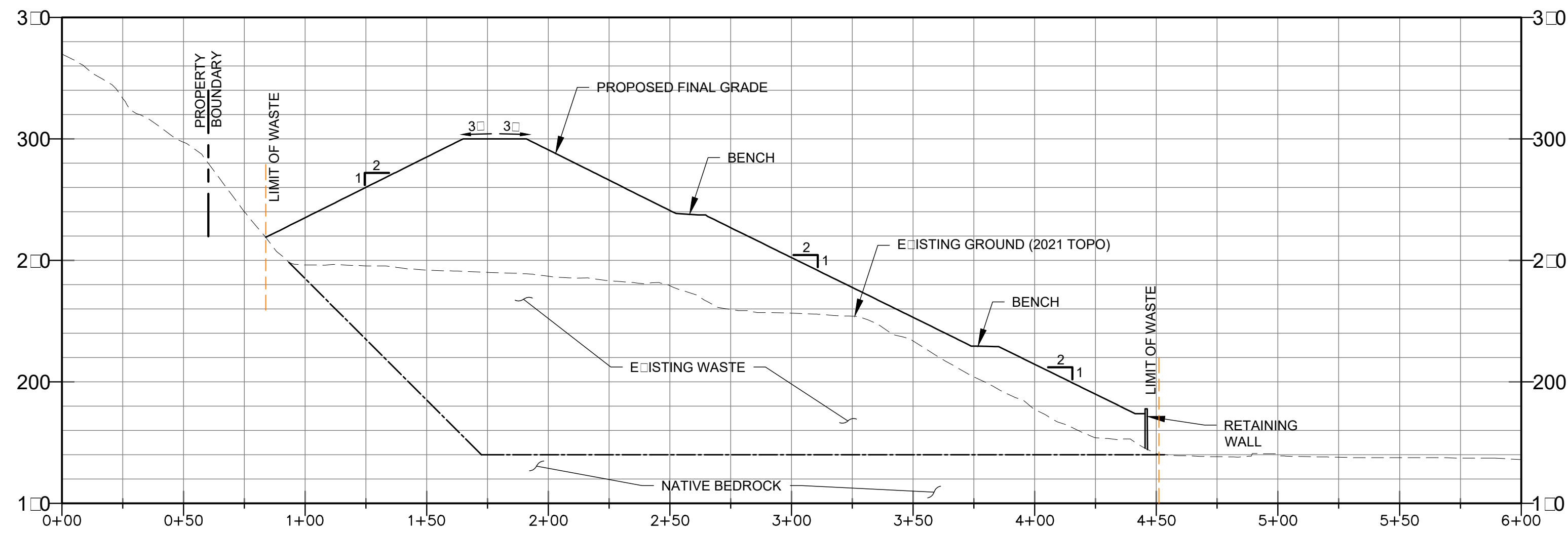
Notes:

- The shear strength envelope of the reconstituted specimens of Oil landfill is developed based on the friction angle $\phi = 31^\circ$ and cohesion of 900 psf (Kavazanjian et al., 2013).
- The shear strength envelope of incinerator ash is computed based on the friction angle of the friction angle $\phi = 43^\circ$ and cohesion of 0 psf. The shear strength envelope shows the lower limit shear strength of incinerator ash (Poran and Ahtchi-Ali, 1989).
- UU triaxial test stands for unconsolidated undrain triaxial test.

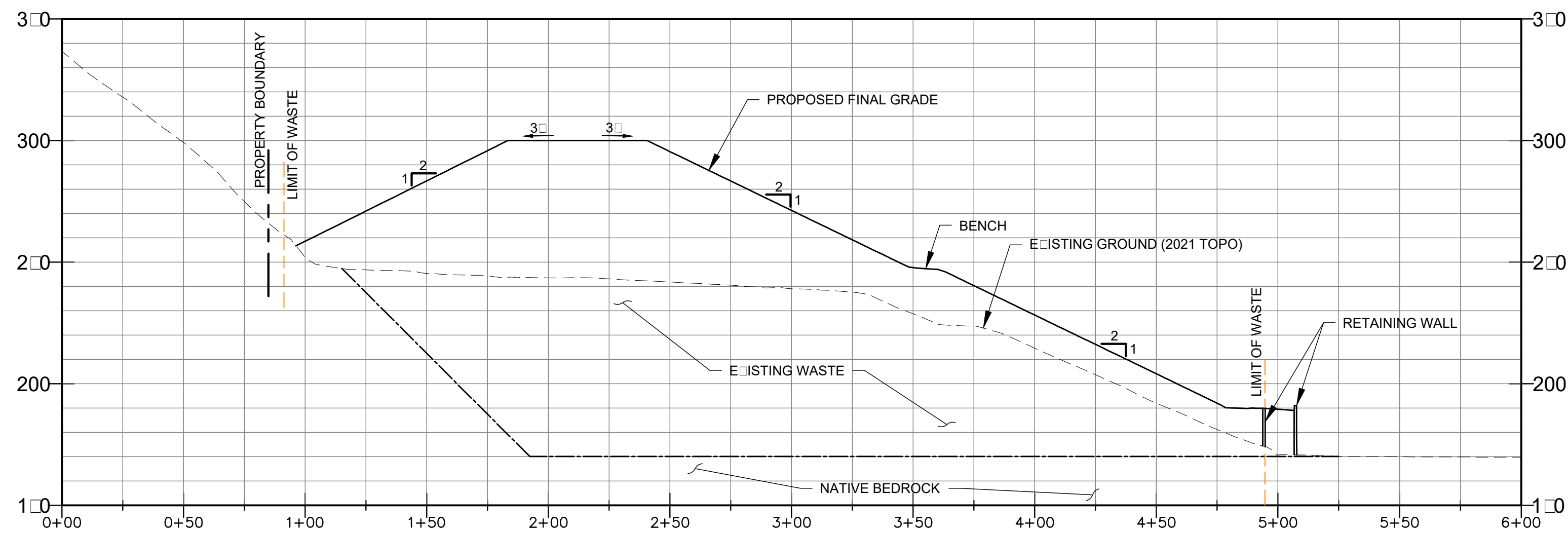
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SHEAR STRENGTH ENVELOPES
PEBBLY BEACH LANDFILL
AVALON, CALIFORNIA

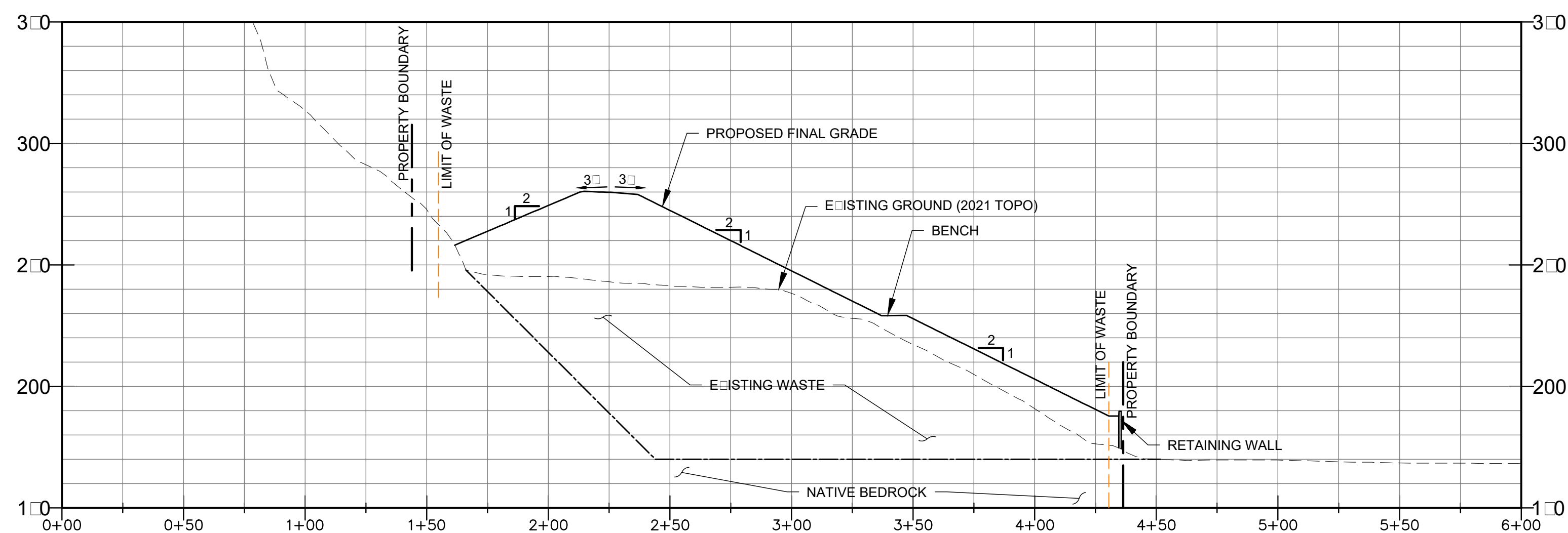
DATE:	FEB 2022	FIGURE NO.	6
PROJECT NO.	SO21.1196.00		



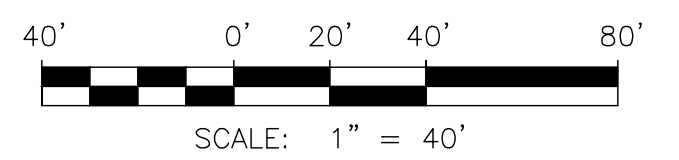
A SECTION



B SECTION



C SECTION



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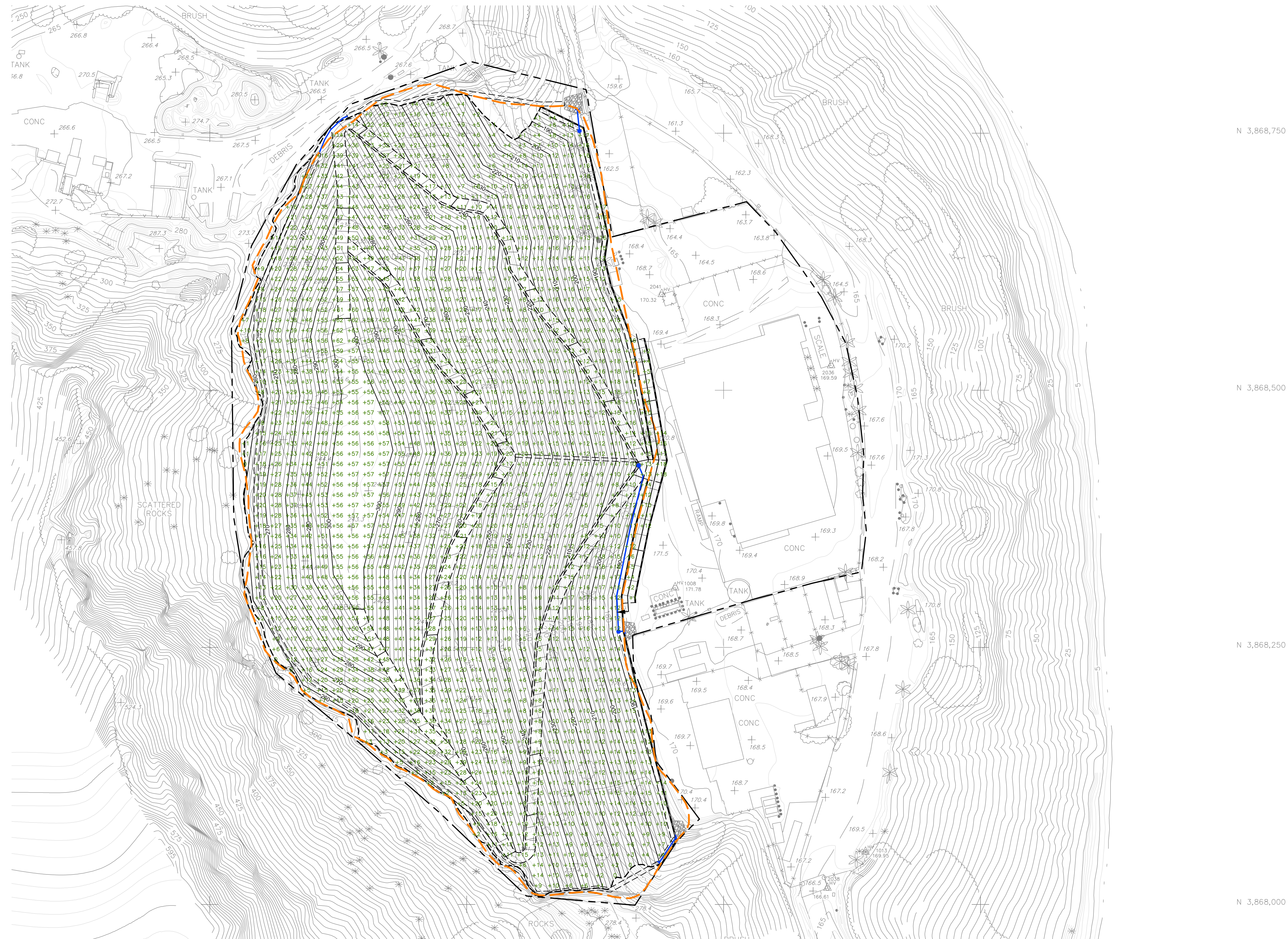


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 SITE LIFE OPTIMIZATION
 DESIGN REPORT DRAWINGS
 REPRESENTATIVE CROSS SECTION

DWG NO. 7
 PROJECT NO. S021.1196



LEGEND	
	PROPERTY BOUNDARY
	E.G. GROUND CONTOUR, FEET
	DESIGN GROUND CONTOUR, FEET
	E.G. UNPAVED ROAD
	E.G. FENCE
	PROJECT GRADING LIMITS
	GRADING HINGE
	RETAINING WALL
	FILL DEPTH IN FEET (16' TO 10' GRID)
	+1

P:\SITES\PEBBLY BEACH\EXPANSION 2021\DWG SETS\2021.1196-ACS-04-PROPOSED FINAL GRADING PLAN ISOPACH.DWG January 29, 2022 - 3:57 PM BY: JORGE AMAYA

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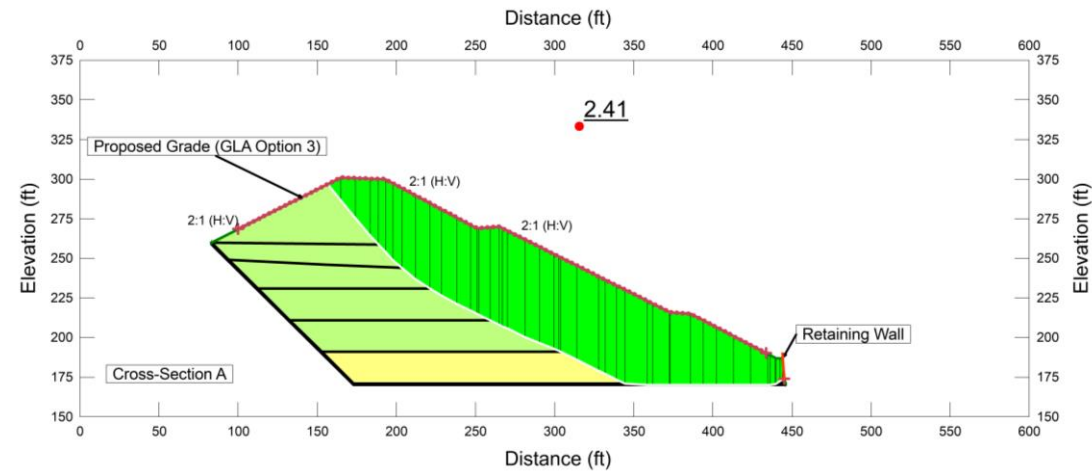
PEBBLY BEACH LANDFILL, AVALON, CA
 SITE LIFE OPTIMIZATION
 PROPOSED FINAL GRADING PLAN ISOPACH

DWG NO. 8
 PROJECT NO. S021.1196

ISSUED FOR REVIEW

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Light Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

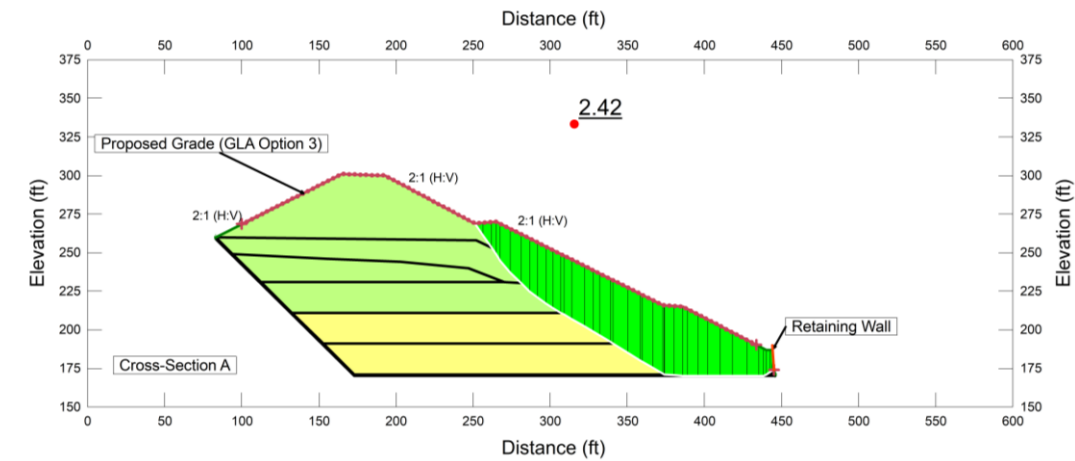
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 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 2.41
 Horz Seismic Coef.: 0



(a) Scenario 1 - The thickness of the incinerator ash is about 20 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Light Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

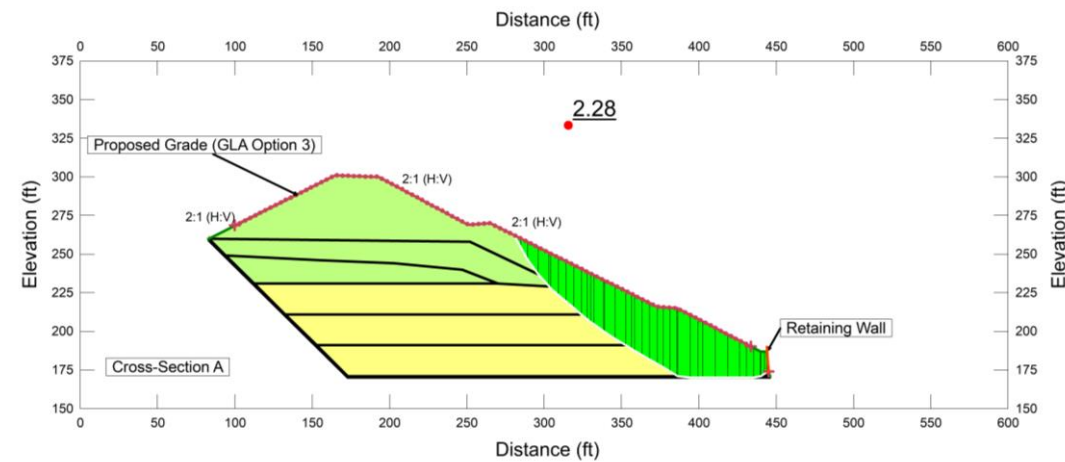
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 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 2.42
 Horz Seismic Coef.: 0



(b) Scenario 2 - The thickness of the incinerator ash is about 40 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Light Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

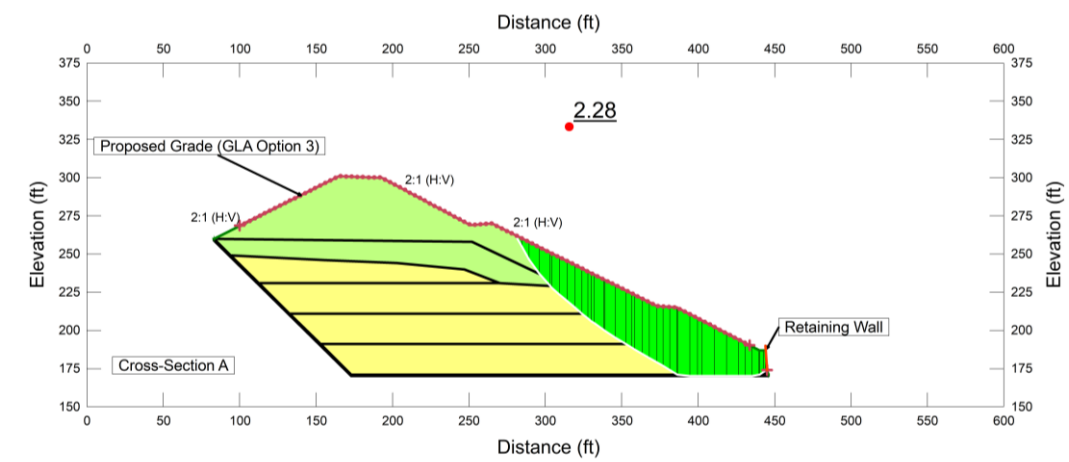
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 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 2.28
 Horz Seismic Coef.: 0



(c) Scenario 3 - The thickness of the incinerator ash is about 60 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Light Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

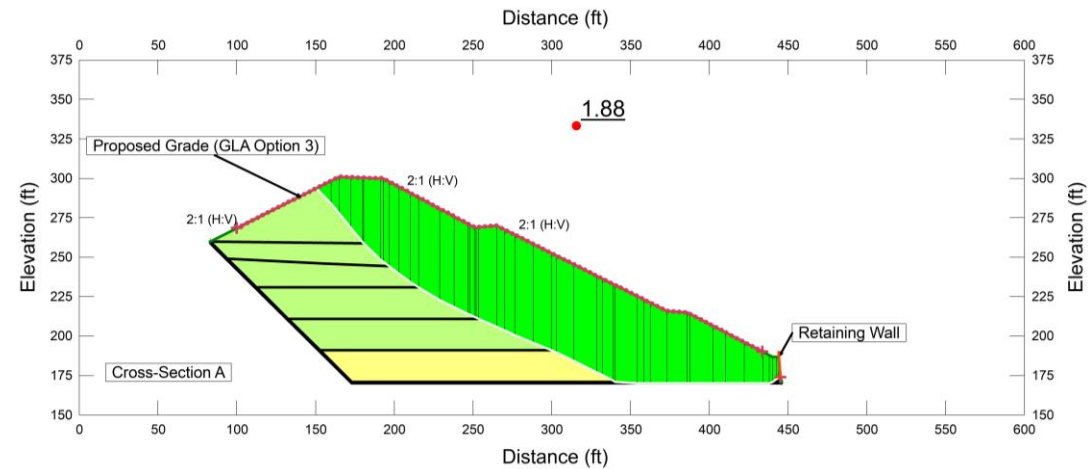
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 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 2.28
 Horz Seismic Coef.: 0



(d) Scenario 4 - The thickness of the incinerator ash is to the elevation of the existing ground.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Light Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

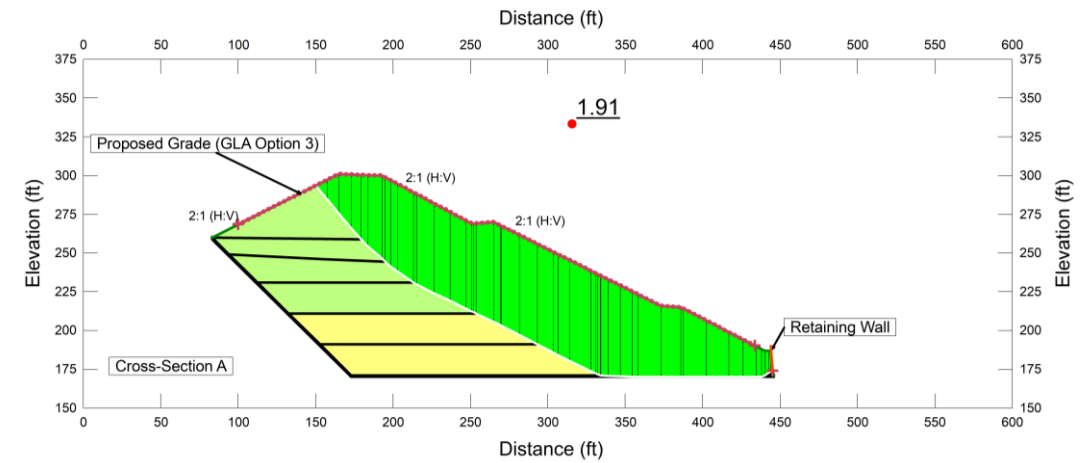
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 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 1.88
 Horz Seismic Coef.: 0.1



(a) Scenario 1 - The thickness of the incinerator ash is about 20 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Light Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

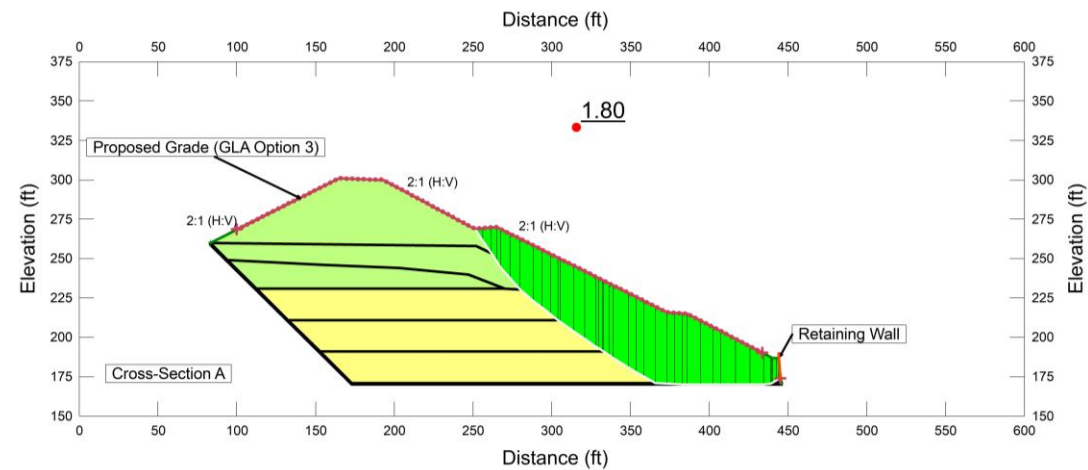
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 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 1.91
 Horz Seismic Coef.: 0.1



(b) Scenario 2 - The thickness of the incinerator ash is about 40 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Light Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

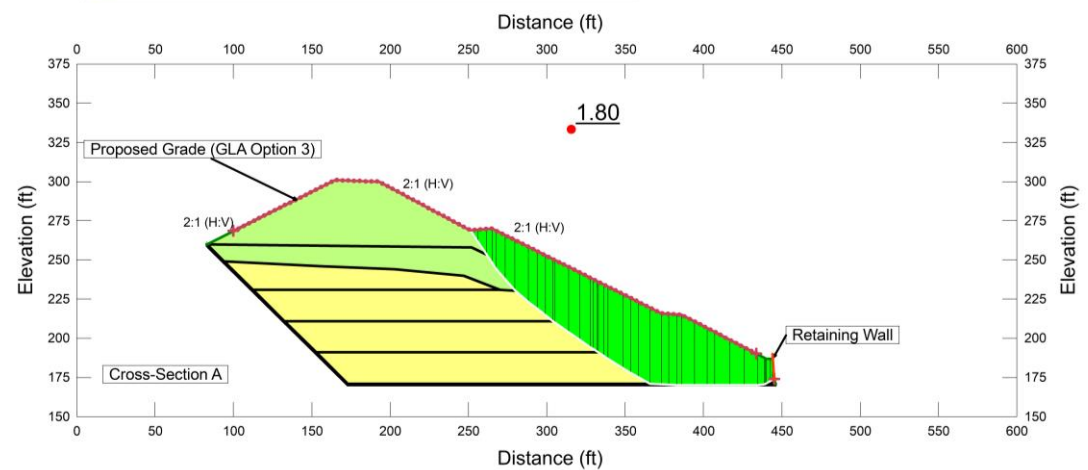
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 Method: Morgenstern-Price
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 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 1.80
 Horz Seismic Coef.: 0.1



(c) Scenario 3 - The thickness of the incinerator ash is about 60 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Light Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

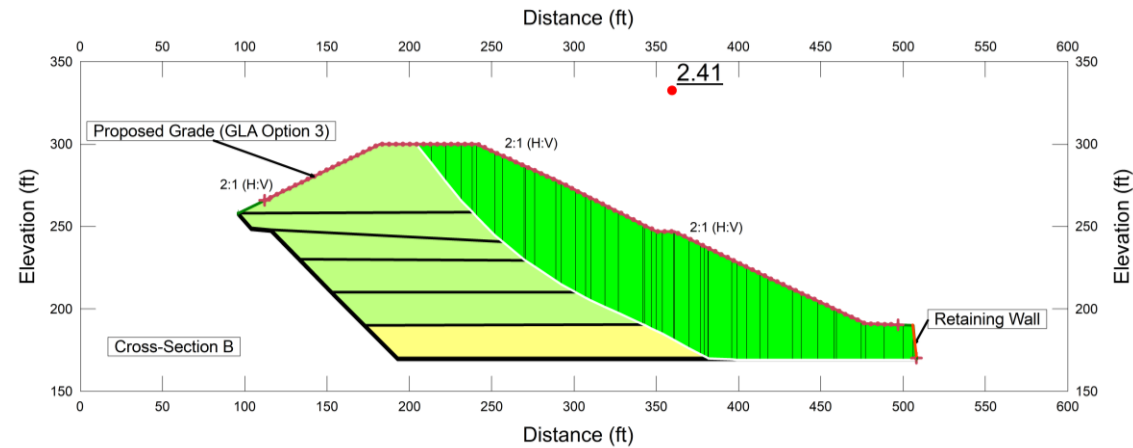
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 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 1.80
 Horz Seismic Coef.: 0.1



(d) Scenario 4 - The thickness of the incinerator ash is to the elevation of the existing ground.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Light Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

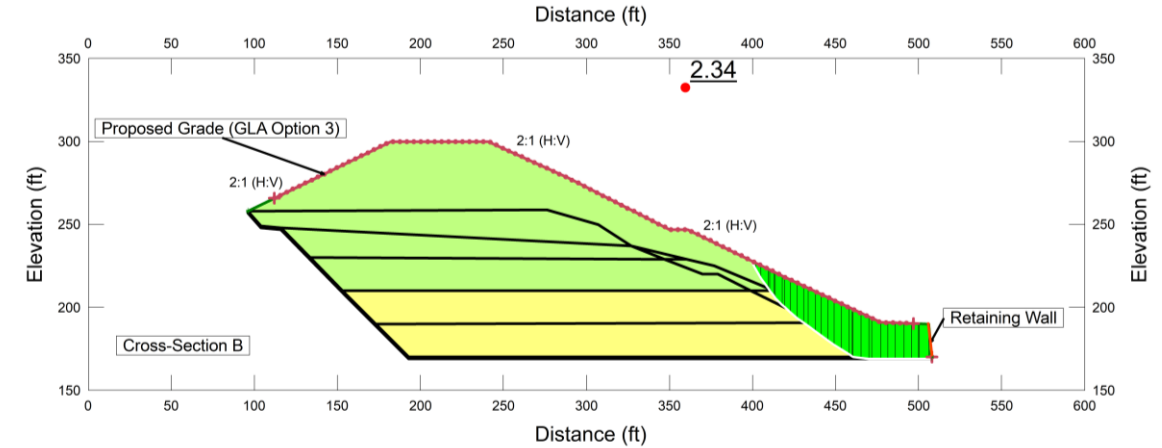
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 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 2.41
 Horz Seismic Coef.: 0



(a) Scenario 1 - The thickness of the incinerator ash is about 20 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Light Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

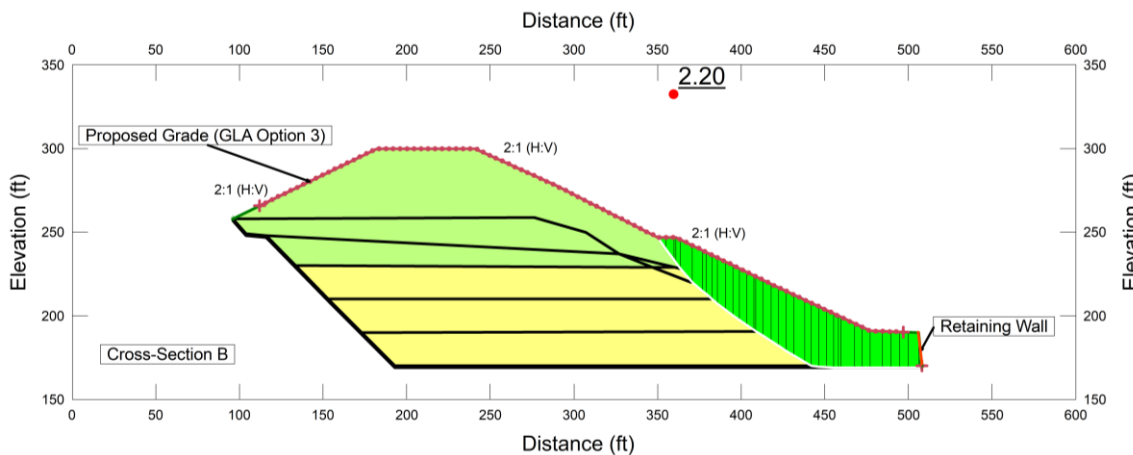
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 Name: 2 - 1 - Scenario 2 - Ash 1st to 2nd - Static
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 2.34
 Horz Seismic Coef.: 0



(b) Scenario 2 - The thickness of the incinerator ash is about 40 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Light Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

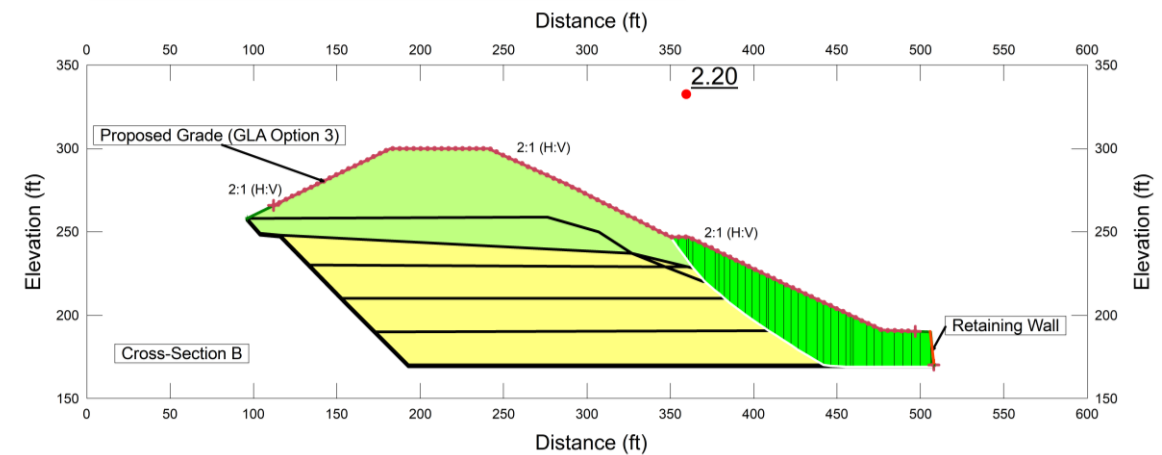
Title: Cross-Section B - Pebbly Beach Landfill
 Name: 3 - 1 - Scenario 3 - Ash 1st to 3rd - Static
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 2.20
 Horz Seismic Coef.: 0



(c) Scenario 3 - The thickness of the incinerator ash is about 60 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Light Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

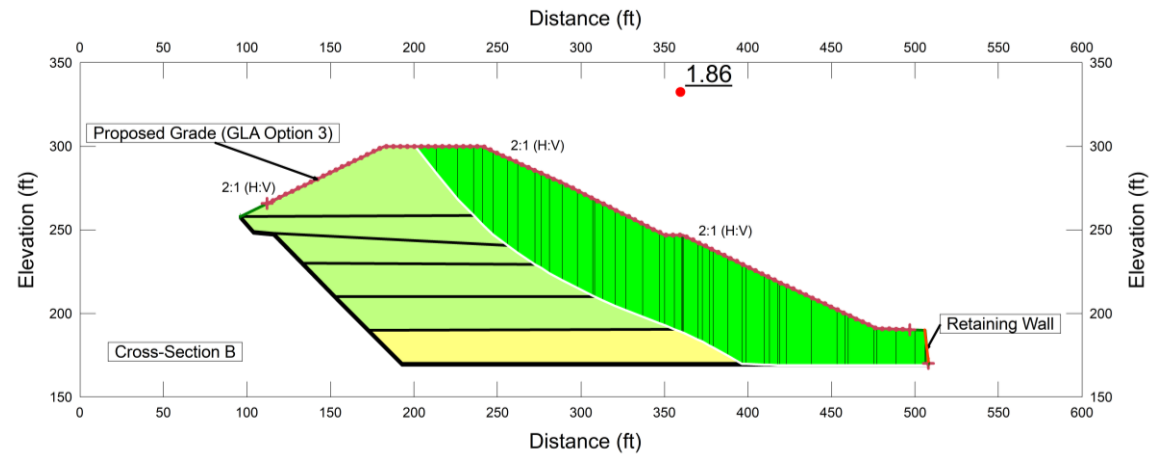
Title: Cross-Section B - Pebbly Beach Landfill
 Name: 4 - 1 - Scenario 4 - Ash 1st to 4th - Static
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 2.20
 Horz Seismic Coef.: 0



(d) Scenario 4 - The thickness of the incinerator ash is to the elevation of the existing ground.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

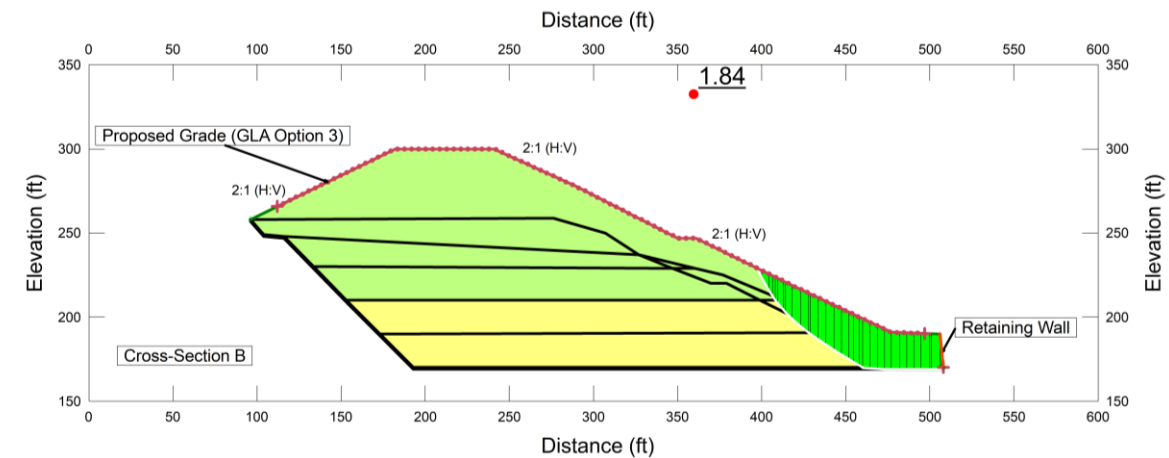
Title: Cross-Section B - Pebbly Beach Landfill
 Name: 1 - 3 - Scenario 1 - Ash 1st - Seismic 2
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 1.86
 Horz Seismic Coef.: 0.1



(a) Scenario 1 - The thickness of the incinerator ash is about 20 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

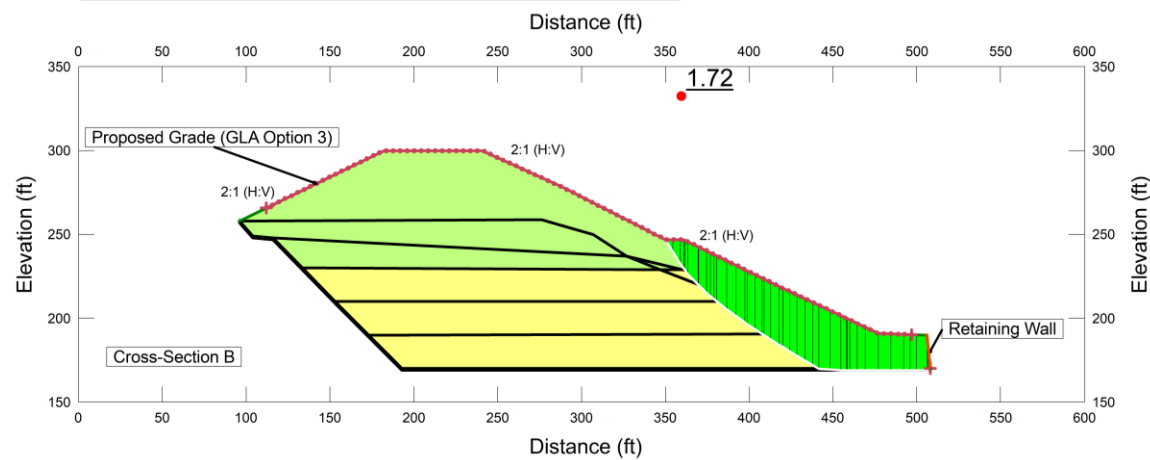
Title: Cross-Section B - Pebbly Beach Landfill
 Name: 2 - 3 - Scenario 2 - Ash 1st to 2nd - Seismic 2
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 1.84
 Horz Seismic Coef.: 0.1



(b) Scenario 2 - The thickness of the incinerator ash is about 40 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

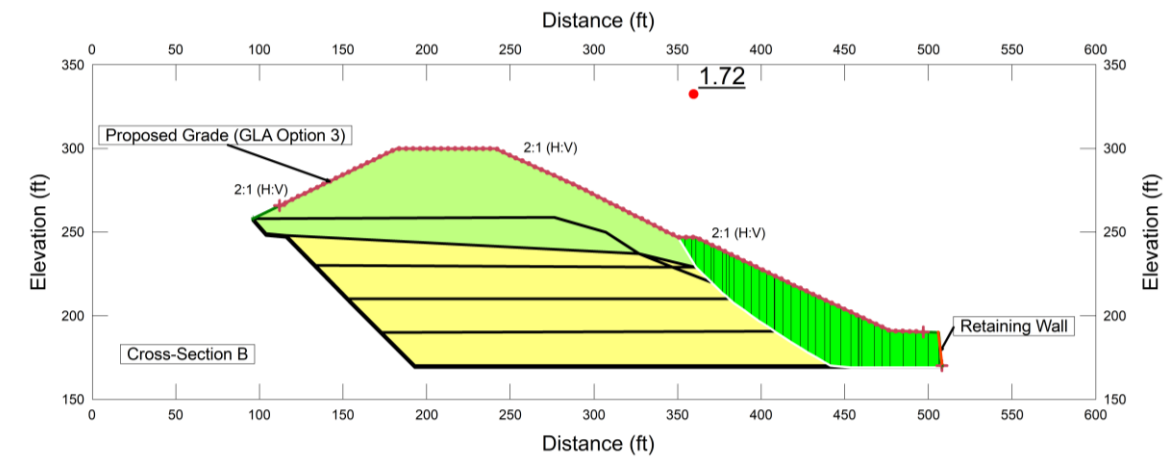
Title: Cross-Section B - Pebbly Beach Landfill
 Name: 3 - 3 - Scenario 3 - Ash 1st to 3rd - Seismic 2
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 1.72
 Horz Seismic Coef.: 0.1



(c) Scenario 3 - The thickness of the incinerator ash is about 60 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

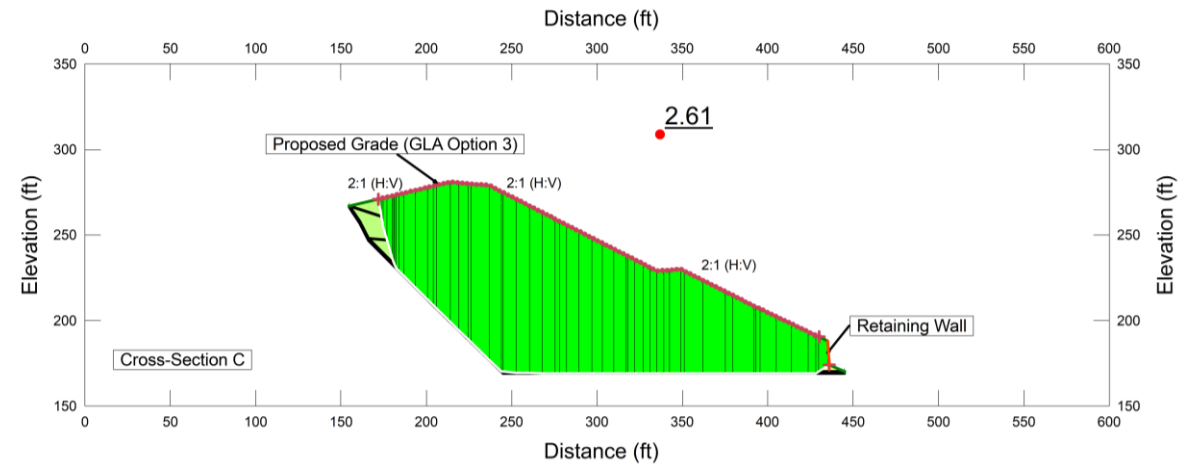
Title: Cross-Section B - Pebbly Beach Landfill
 Name: 4 - 3 - Scenario 4 - Ash 1st to 4th - Seismic 2
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 1.72
 Horz Seismic Coef.: 0.1



(d) Scenario 4 - The thickness of the incinerator ash is to the elevation of the existing ground.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

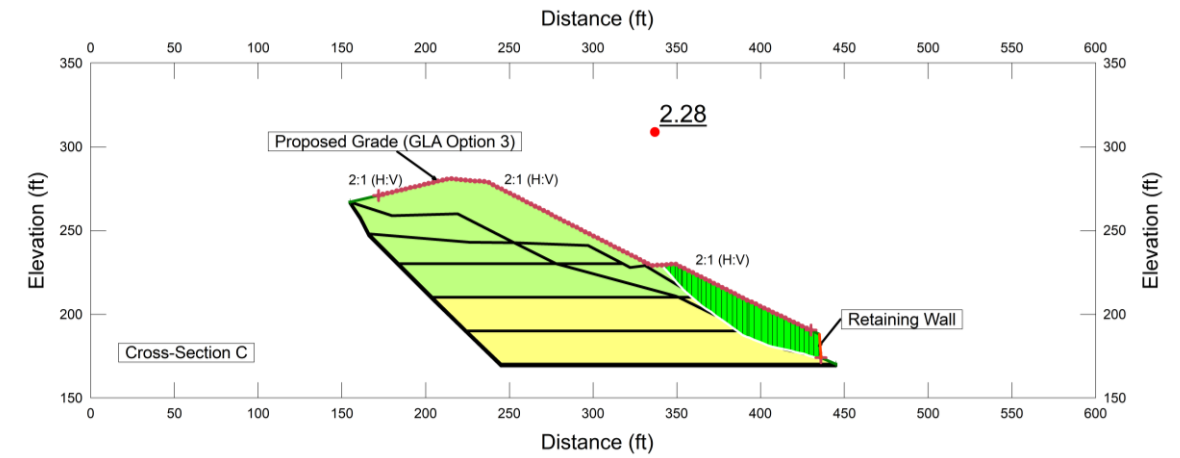
Title: Cross-Section C - Pebbly Beach Landfill
 Name: 1 - 1 - Scenario 1 - Ash 1st Layer - Static
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 2.61
 Horz Seismic Coef.: 0



(a) Scenario 1 - The thickness of the incinerator ash is about 20 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

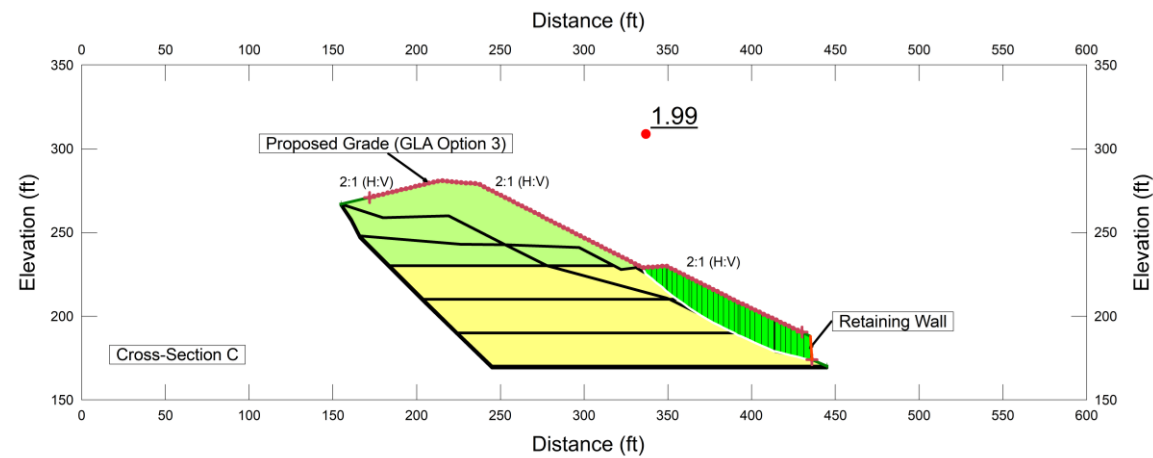
Title: Cross-Section C - Pebbly Beach Landfill
 Name: 2 - 1 - Scenario 2 - Ash 1st and 2nd Layers - Static
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 2.28
 Horz Seismic Coef.: 0



(b) Scenario 2 - The thickness of the incinerator ash is about 40 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

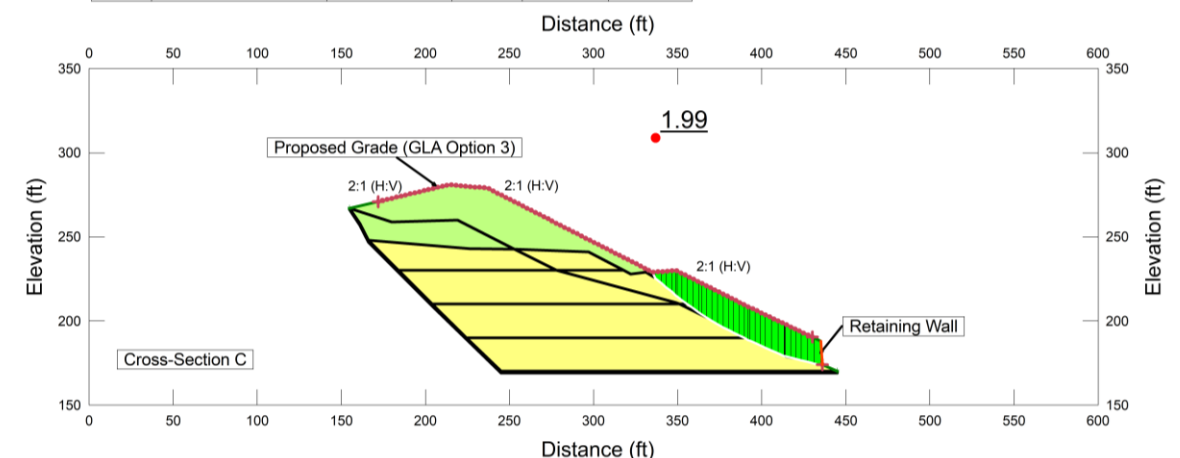
Title: Cross-Section C - Pebbly Beach Landfill
 Name: 3 - 1 - Scenario 3 - Ash 1st to 3rd Layers - Static
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 1.99
 Horz Seismic Coef.: 0



(c) Scenario 3 - The thickness of the incinerator ash is about 60 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

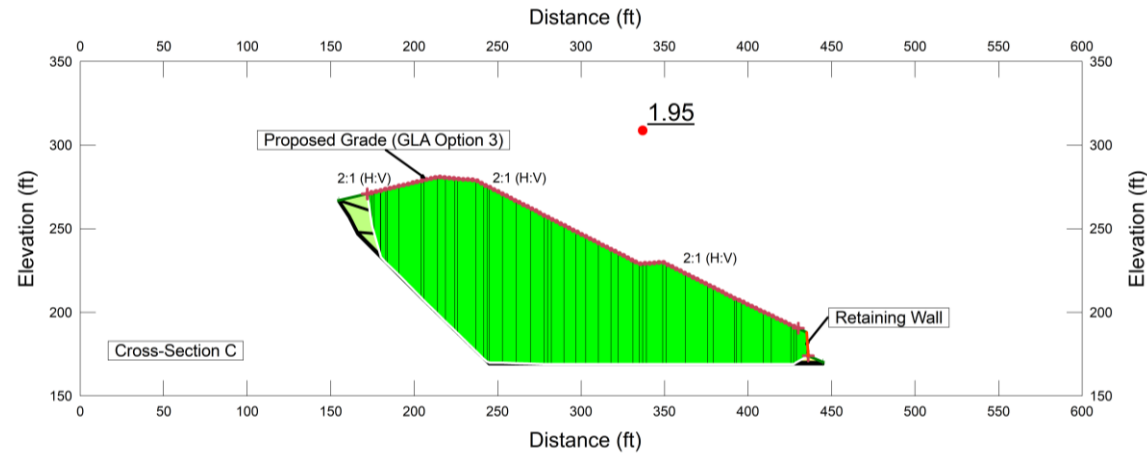
Title: Cross-Section C - Pebbly Beach Landfill
 Name: 4 - 1 - Scenario 4 - Ash 1st to 4th Layers - Static
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 1.99
 Horz Seismic Coef.: 0



(d) Scenario 4 - The thickness of the incinerator ash is to the elevation of the existing ground.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

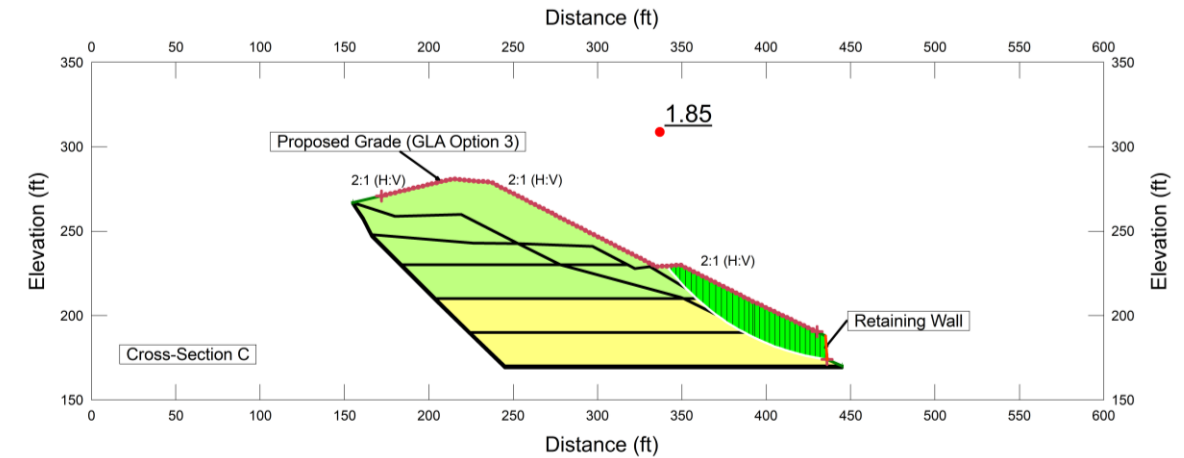
Title: Cross-Section C - Pebbly Beach Landfill
 Name: 1 - 3 - Scenario 1 - Ash 1st Layers - Seismic 2
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 1.95
 Horz Seismic Coef.: 0.1



(a) Scenario 1 - The thickness of the incinerator ash is about 20 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

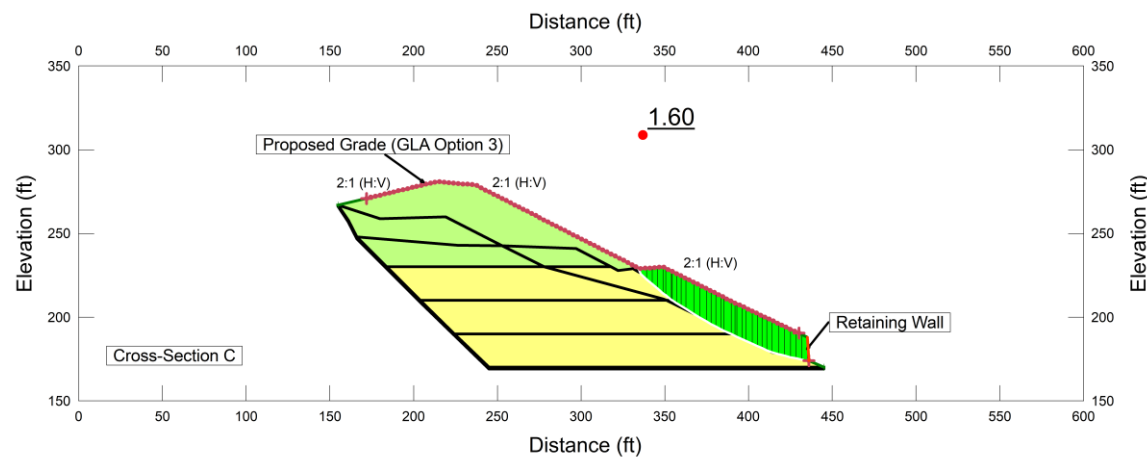
Title: Cross-Section C - Pebbly Beach Landfill
 Name: 2 - 3 - Scenario 2 - Ash 1st and 2nd Layers - Seismic 2
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 1.85
 Horz Seismic Coef.: 0.1



(b) Scenario 2 - The thickness of the incinerator ash is about 40 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

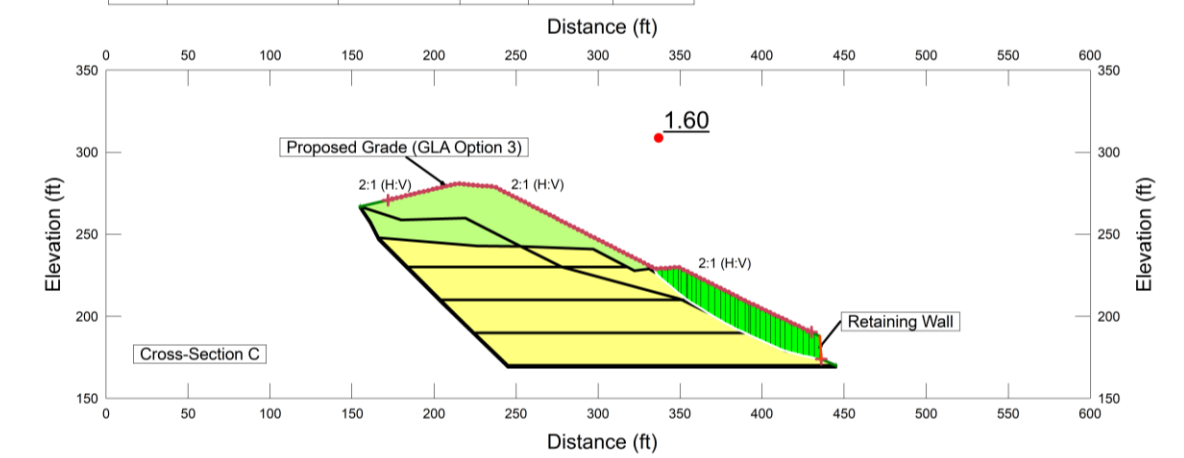
Title: Cross-Section C - Pebbly Beach Landfill
 Name: 3 - 3 - Scenario 3 - Ash 1st to 3rd Layers - Seismic 2
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 1.60
 Horz Seismic Coef.: 0.1



(c) Scenario 3 - The thickness of the incinerator ash is about 60 feet.

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Green	Baled Waste	Mohr-Coulomb	65	900	31
Yellow	Incinerator Ash	Mohr-Coulomb	105.7	0	43
Red	Interface - Base	Mohr-Coulomb	105.7	0	28.6
Blue	Interface - Side Slope	Mohr-Coulomb	65	0	20.7

Title: Cross-Section C - Pebbly Beach Landfill
 Name: 4 - 3 - Scenario 4 - Ash 1st to 4th Layers - Seismic 2
 Date: 02/03/2022
 Method: Morgenstern-Price
 Slip Surface Option: Entry and Exit
 Optimize Critical Slip Surface Location: Yes
 Factor of Safety: 1.60
 Horz Seismic Coef.: 0.1

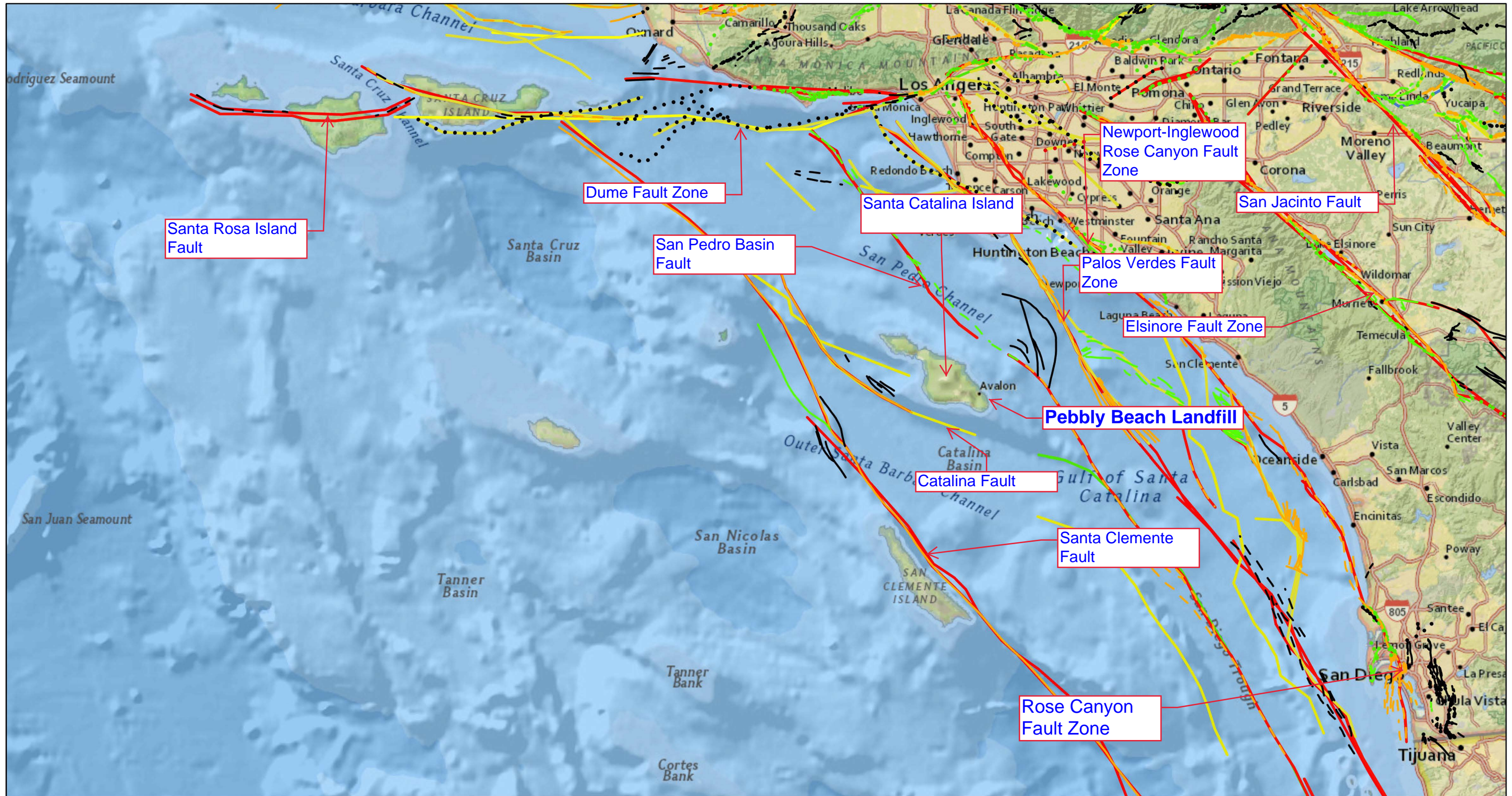


(d) Scenario 4 - The thickness of the incinerator ash is to the elevation of the existing ground.

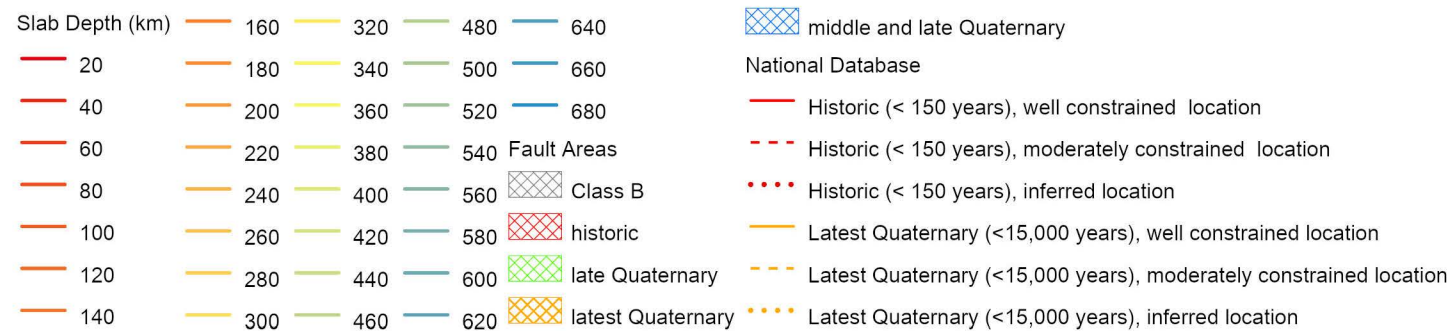
Appendix B.7

Fault Map

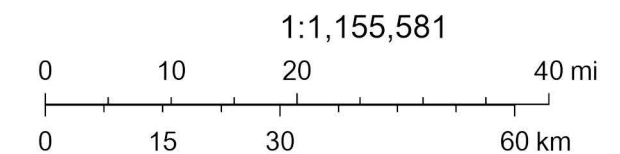
U.S. Geological Survey Quaternary Faults



3/7/2022, 11:54:43 AM



Note:
The map shows major potentially active faults within 60 miles radius of the site.



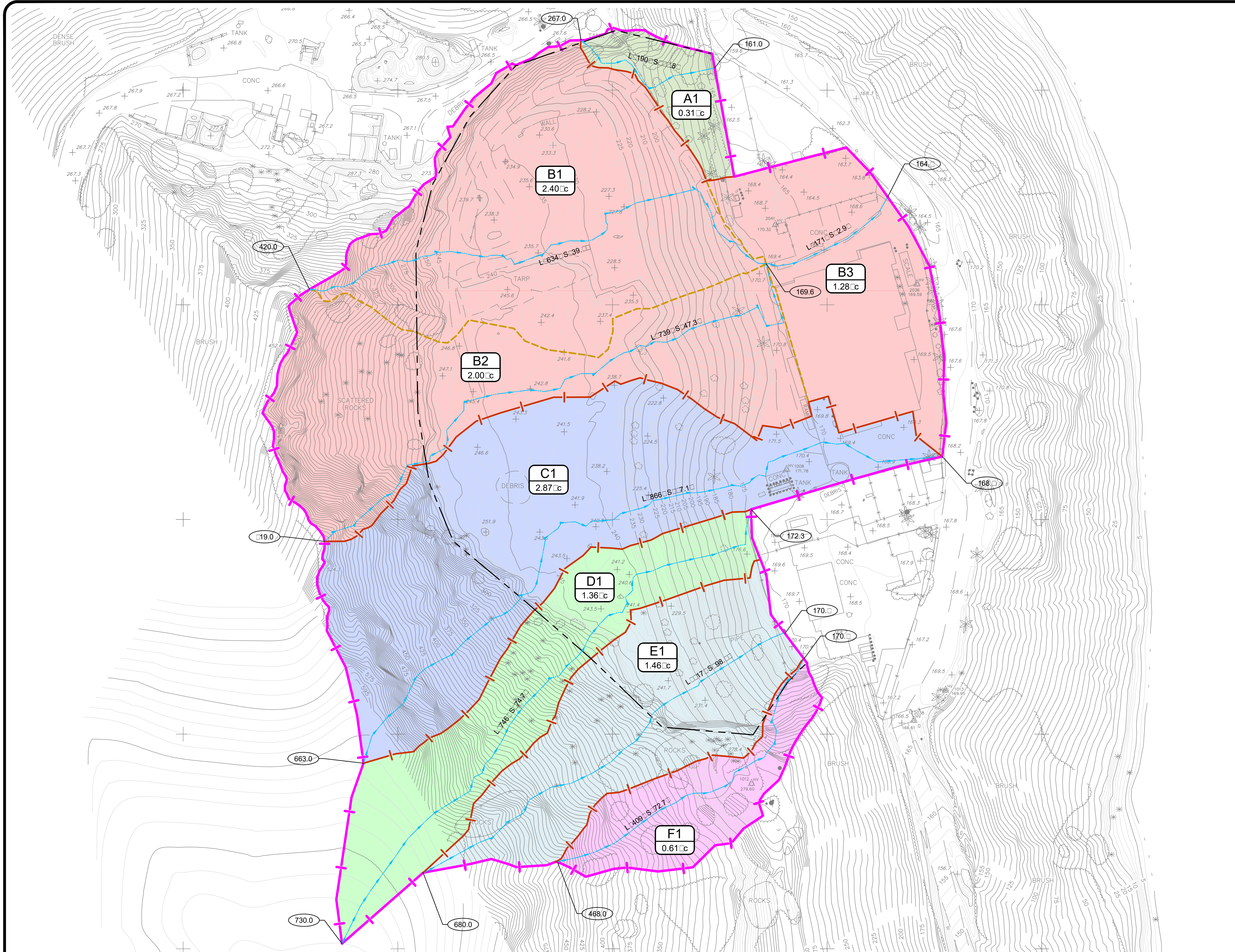
USGS, National Geographic, Esri, Garmin, HERE, UNEP-WCMC, USGS, NASA, ESA, METI, NRCAN, GEBCO, NOAA, increment P Corp.

Appendix C

Surface Water Analysis

Appendix C.1

Figures



LEGEND

- OVERALL HYDROLOGY DRAINAGE BOUNDARY
- HYDROLOGY SUB-DRAINAGE BOUNDARY
- HYDROLOGY SUB-AREA
- PROPERTY BOUNDARY
- EXISTING GROUND SURFACE CONTOUR EL, FEET
- PROPOSED GROUND SURFACE CONTOUR EL, FEET
- DRAINAGE COURSE
- DRAINAGE ELEVATION
- SUB-AREA IDENTIFICATION
- AREA IN ACRES

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 N 3,868,500
 N 3,868,250
 N 3,868,000
 N 3,867,750

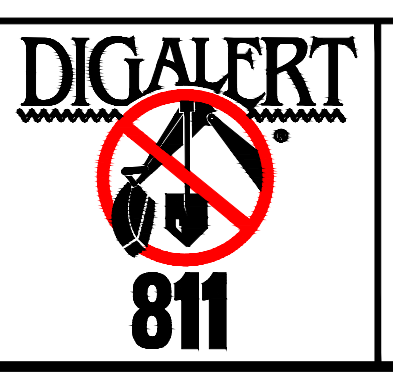
SCALE: 1" = 60'

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REFERENCE AERIAL TOPO BASED ON SEPTEMBER 29, 2021 AERIAL SURVEY BY DON READ CORPORATION

REV. NO.	DATE	DESCRIPTION	APPROVED BY

DATE OF ISSUE: MARCH 2022
 DESIGNED BY: D. HARICH
 CAD DESIGN BY: J. AMAYA
 CHECKED BY: D. HARICH
 APPROVED BY: D. HARICH



Geo-Logic ASSOCIATES

2777 EAST GUASTI ROAD
 SUITE 1
 ONTARIO, CA 91761
 (909) 626-2282
 www.geo-logic.com

CR&R INCORPORATED
 environmental services

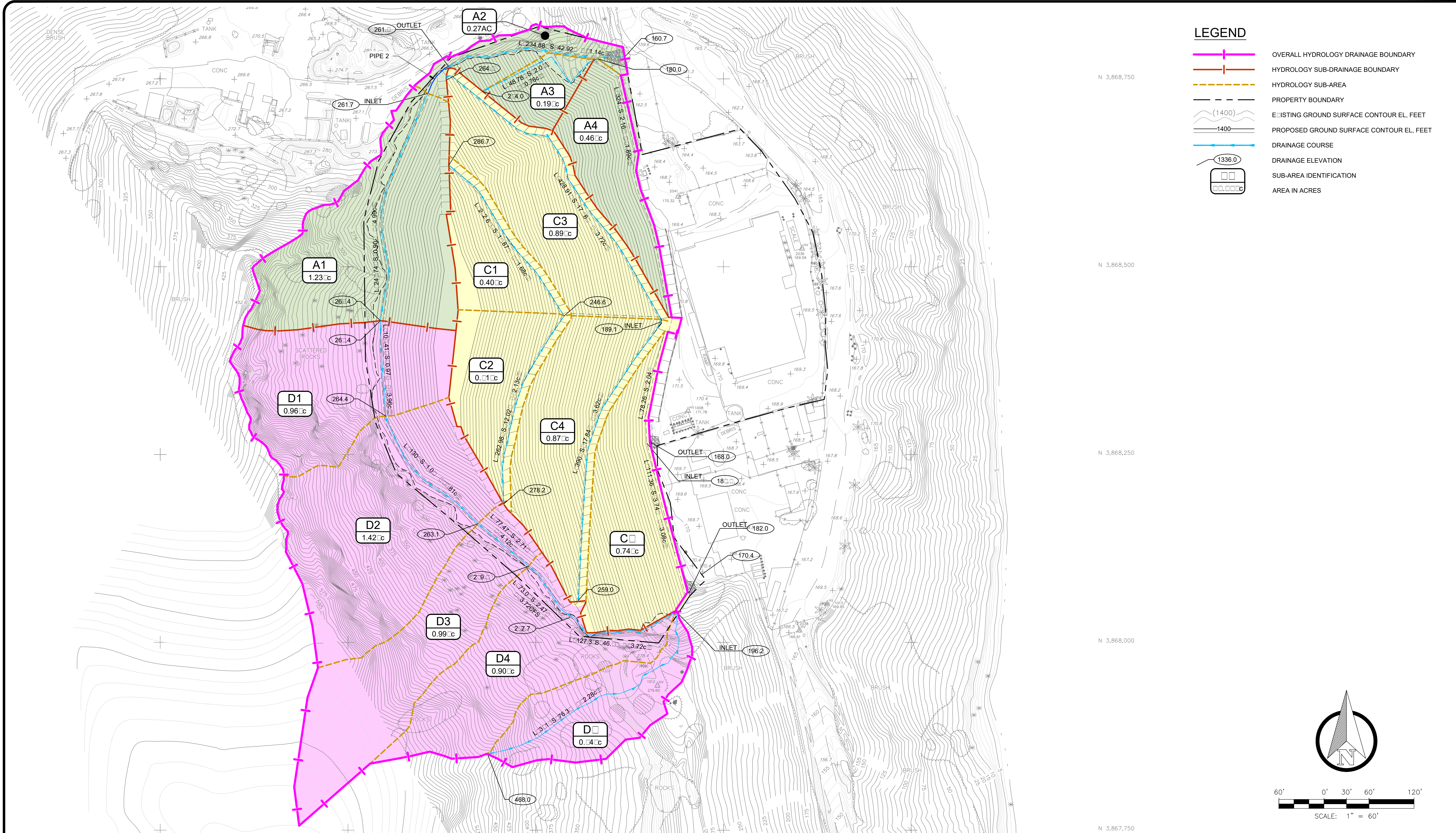
1 DUMP ROAD
 AVALON, CALIFORNIA 90704

ISSUED FOR PERMIT

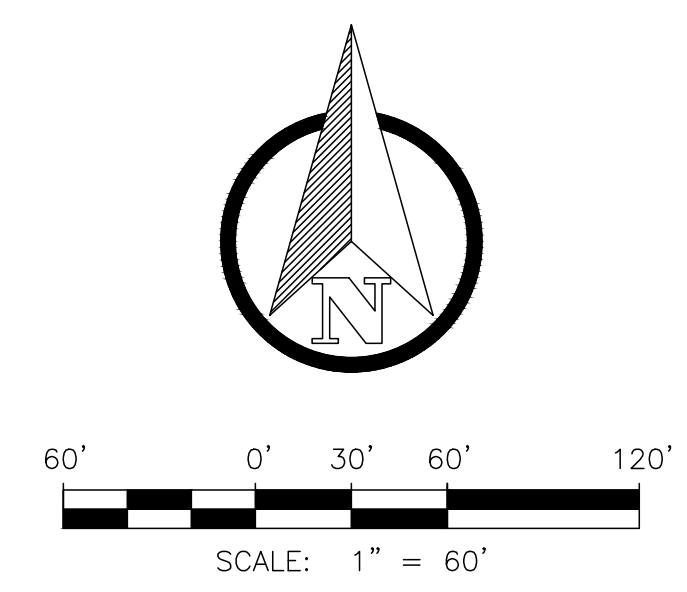
PEBBLY BEACH LANDFILL, AVALON, CA
 SITE LIFE OPTIMIZATION
 DESIGN REPORT DRAWINGS
 EXISTING HYDROLOGY

DWG. NO.
FIG-1
 PROJECT NO.
 S021.1196

P:\SITES\PEBBLY BEACH\FEASIBILITY 2021\DWG SETS\S021.1196-ACS-08-HYDROLOGY EXISTING CONDITION.DWG March 9, 2022 - 1:21 PM BY: JORGE AMAYA



- LEGEND**
- OVERALL HYDROLOGY DRAINAGE BOUNDARY
 - HYDROLOGY SUB-DRAINAGE BOUNDARY
 - HYDROLOGY SUB-AREA
 - PROPERTY BOUNDARY
 - EXISTING GROUND SURFACE CONTOUR EL, FEET
 - PROPOSED GROUND SURFACE CONTOUR EL, FEET
 - DRAINAGE COURSE
 - DRAINAGE ELEVATION
 - SUB-AREA IDENTIFICATION
 - AREA IN ACRES



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PEBBLY BEACH LANDFILL, AVALON, CA

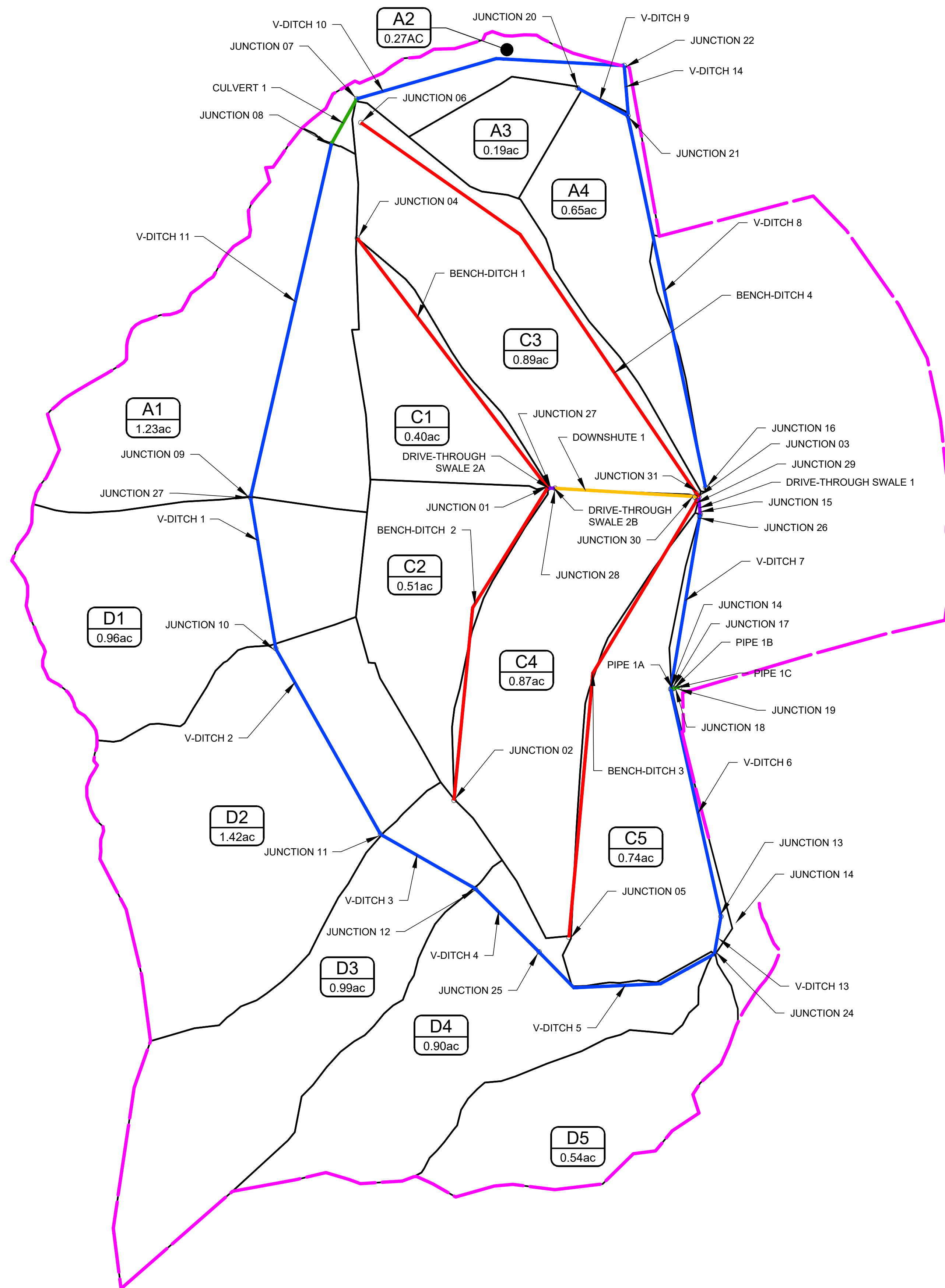
SITE LIFE OPTIMIZATION
 DESIGN REPORT DRAWINGS

HYDROLOGY PROPOSED CONDITION

DWG. NO.
FIG-2
 PROJECT NO.
 S021.1196

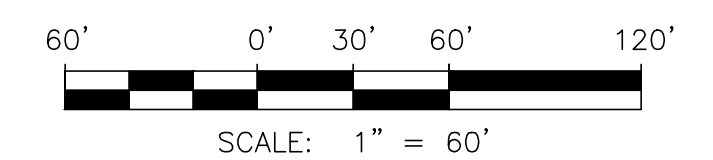
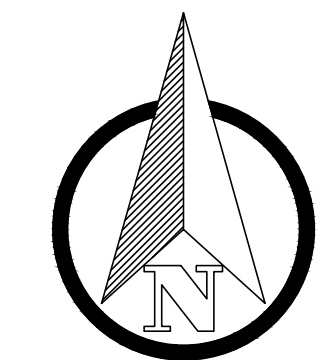
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P:\SITES\PEBBLY BEACH\FEASIBILITY 2021\DWG SETS\S021.1196-ACS-10-STORM & SANITARY ANALYSIS NETWORK.DWG March 9, 2022 - 1:33 PM BY: JORGE AMAYA



LEGEND

- OVERALL HYDROLOGY DRAINAGE BOUNDARY
- HYDROLOGY SUB-AREA
- PROPOSED V-DITCH
- PROPOSED BENCH
- PROPOSED CULVERT
- PROPOSED DOWNSHUTE
- PROPOSED DRIVE THROUGH SWALE
- SUB-AREA IDENTIFICATION
- AREA IN ACRES



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ASSOCIATES

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ONTARIO, CA 91761
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PEBBLY BEACH LANDFILL, AVALON, CA
 SITE LIFE OPTIMIZATION
 DESIGN REPORT DRAWINGS
 STORM & SANITARY ANALYSIS NETWORK

DWG NO.
FIG-3
PROJECT NO.
S021.1196

ISSUED FOR PERMIT

Appendix C.2
Design Storm Data



NOAA Atlas 14, Volume 6, Version 2
Location name: Avalon, California, USA*
Latitude: 33.3314°, Longitude: -118.3111°
Elevation: 96.77 ft**
 * source: ESRI Maps
 ** source: USGS



POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitania, Deborah Martin,
 Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao,
 Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

NOAA, National Weather Service, Silver Spring, Maryland

[PF_tabular](#) | [PF_graphical](#) | [Maps & aerals](#)

PF tabular

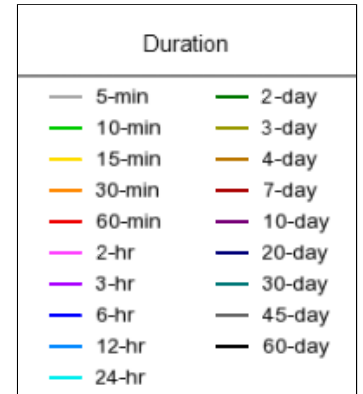
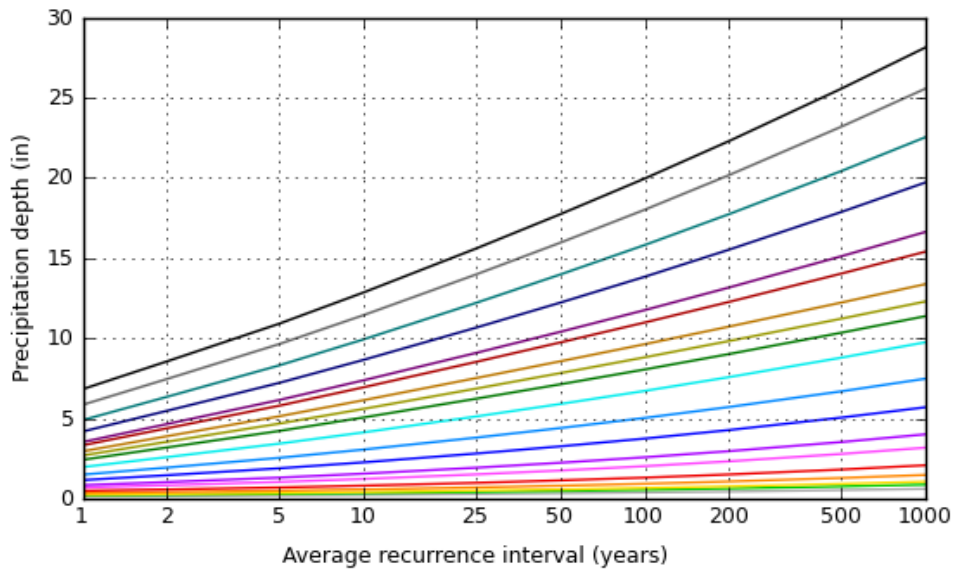
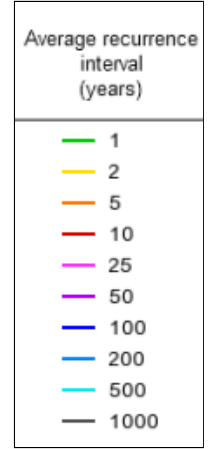
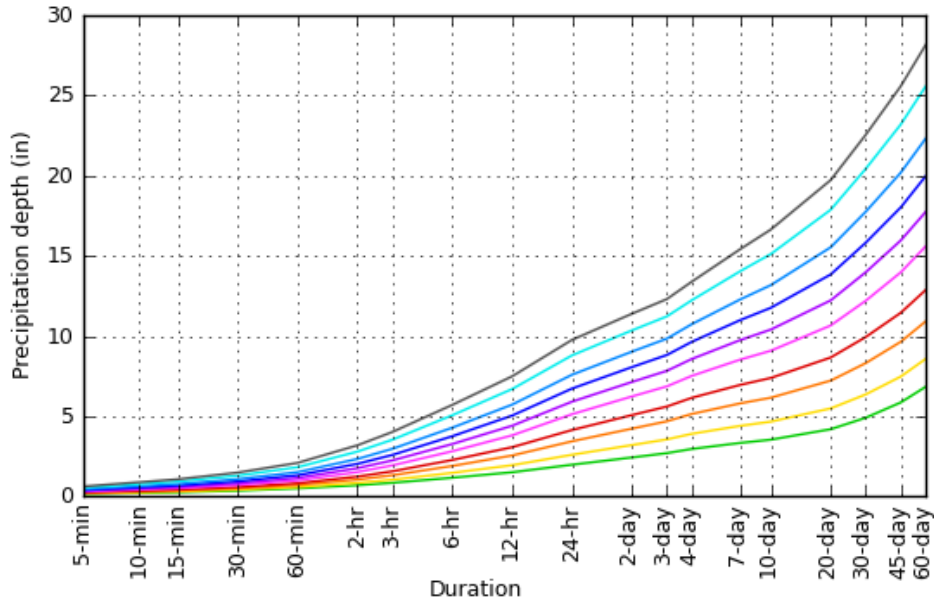
PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)¹										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.146 (0.122-0.176)	0.170 (0.142-0.206)	0.206 (0.172-0.250)	0.240 (0.198-0.293)	0.291 (0.232-0.369)	0.336 (0.262-0.435)	0.387 (0.294-0.514)	0.445 (0.328-0.610)	0.535 (0.378-0.766)	0.614 (0.418-0.912)
10-min	0.209 (0.175-0.252)	0.244 (0.204-0.295)	0.296 (0.246-0.359)	0.344 (0.284-0.420)	0.417 (0.332-0.529)	0.482 (0.375-0.624)	0.554 (0.421-0.737)	0.638 (0.470-0.874)	0.767 (0.541-1.10)	0.881 (0.599-1.31)
15-min	0.252 (0.211-0.305)	0.295 (0.246-0.356)	0.358 (0.298-0.434)	0.415 (0.343-0.508)	0.505 (0.402-0.639)	0.582 (0.454-0.755)	0.670 (0.509-0.892)	0.772 (0.569-1.06)	0.928 (0.654-1.33)	1.07 (0.725-1.58)
30-min	0.351 (0.294-0.424)	0.410 (0.343-0.496)	0.498 (0.415-0.604)	0.578 (0.477-0.707)	0.702 (0.560-0.890)	0.810 (0.632-1.05)	0.933 (0.709-1.24)	1.07 (0.792-1.47)	1.29 (0.911-1.85)	1.48 (1.01-2.20)
60-min	0.495 (0.414-0.597)	0.578 (0.482-0.698)	0.701 (0.584-0.850)	0.814 (0.672-0.996)	0.988 (0.788-1.25)	1.14 (0.889-1.48)	1.31 (0.998-1.75)	1.51 (1.12-2.07)	1.82 (1.28-2.60)	2.09 (1.42-3.10)
2-hr	0.693 (0.580-0.837)	0.836 (0.699-1.01)	1.05 (0.870-1.27)	1.23 (1.02-1.51)	1.52 (1.21-1.92)	1.76 (1.37-2.28)	2.03 (1.54-2.70)	2.33 (1.72-3.19)	2.79 (1.97-3.99)	3.18 (2.17-4.72)
3-hr	0.844 (0.706-1.02)	1.04 (0.866-1.25)	1.32 (1.10-1.60)	1.56 (1.29-1.91)	1.93 (1.54-2.45)	2.24 (1.75-2.90)	2.58 (1.96-3.44)	2.97 (2.19-4.06)	3.53 (2.49-5.06)	4.02 (2.73-5.96)
6-hr	1.16 (0.970-1.40)	1.47 (1.22-1.77)	1.89 (1.58-2.30)	2.27 (1.87-2.77)	2.81 (2.24-3.57)	3.26 (2.54-4.23)	3.75 (2.85-4.99)	4.28 (3.16-5.87)	5.06 (3.57-7.24)	5.70 (3.88-8.47)
12-hr	1.50 (1.25-1.81)	1.94 (1.62-2.35)	2.55 (2.12-3.09)	3.07 (2.53-3.75)	3.80 (3.03-4.82)	4.40 (3.43-5.71)	5.04 (3.83-6.70)	5.72 (4.22-7.84)	6.69 (4.72-9.58)	7.49 (5.09-11.1)
24-hr	1.97 (1.74-2.29)	2.59 (2.28-3.01)	3.43 (3.01-3.99)	4.14 (3.60-4.85)	5.12 (4.32-6.20)	5.90 (4.88-7.29)	6.72 (5.43-8.49)	7.58 (5.96-9.84)	8.79 (6.64-11.9)	9.76 (7.14-13.6)
2-day	2.42 (2.13-2.81)	3.20 (2.81-3.71)	4.22 (3.70-4.91)	5.07 (4.41-5.94)	6.23 (5.25-7.54)	7.13 (5.89-8.81)	8.06 (6.51-10.2)	9.02 (7.10-11.7)	10.3 (7.82-14.0)	11.4 (8.33-15.9)
3-day	2.69 (2.37-3.12)	3.55 (3.12-4.12)	4.68 (4.10-5.45)	5.60 (4.87-6.57)	6.86 (5.78-8.30)	7.83 (6.47-9.66)	8.82 (7.12-11.1)	9.84 (7.73-12.8)	11.2 (8.48-15.2)	12.3 (9.00-17.2)
4-day	2.96 (2.60-3.43)	3.90 (3.43-4.53)	5.14 (4.50-5.98)	6.14 (5.35-7.21)	7.51 (6.33-9.09)	8.57 (7.08-10.6)	9.63 (7.78-12.2)	10.7 (8.44-13.9)	12.2 (9.24-16.5)	13.4 (9.79-18.7)
7-day	3.33 (2.93-3.86)	4.39 (3.86-5.10)	5.80 (5.08-6.75)	6.95 (6.05-8.15)	8.52 (7.19-10.3)	9.74 (8.05-12.0)	11.0 (8.87-13.9)	12.3 (9.65-15.9)	14.0 (10.6-18.9)	15.4 (11.3-21.5)
10-day	3.53 (3.11-4.10)	4.66 (4.10-5.41)	6.15 (5.39-7.16)	7.38 (6.43-8.66)	9.08 (7.66-11.0)	10.4 (8.60-12.8)	11.8 (9.50-14.9)	13.2 (10.4-17.1)	15.1 (11.4-20.4)	16.6 (12.2-23.2)
20-day	4.19 (3.69-4.85)	5.48 (4.82-6.37)	7.22 (6.33-8.40)	8.66 (7.53-10.2)	10.7 (8.98-12.9)	12.2 (10.1-15.1)	13.8 (11.2-17.5)	15.5 (12.2-20.2)	17.9 (13.5-24.1)	19.7 (14.4-27.5)
30-day	4.90 (4.31-5.68)	6.36 (5.59-7.38)	8.31 (7.29-9.67)	9.94 (8.65-11.7)	12.2 (10.3-14.8)	14.0 (11.6-17.3)	15.8 (12.8-20.0)	17.8 (14.0-23.0)	20.4 (15.4-27.6)	22.5 (16.5-31.5)
45-day	5.86 (5.15-6.79)	7.47 (6.57-8.67)	9.64 (8.45-11.2)	11.4 (9.96-13.4)	14.0 (11.8-16.9)	16.0 (13.2-19.7)	18.0 (14.6-22.8)	20.2 (15.9-26.2)	23.2 (17.5-31.3)	25.6 (18.7-35.7)
60-day	6.83 (6.01-7.92)	8.57 (7.53-9.95)	10.9 (9.57-12.7)	12.9 (11.2-15.1)	15.6 (13.1-18.9)	17.7 (14.7-21.9)	20.0 (16.1-25.2)	22.3 (17.5-28.9)	25.6 (19.3-34.5)	28.1 (20.6-39.3)

¹ Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).
 Numbers in parenthesis are PF estimates at low er and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the low er bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.
 Please refer to NOAA Atlas 14 document for more information.

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PF graphical

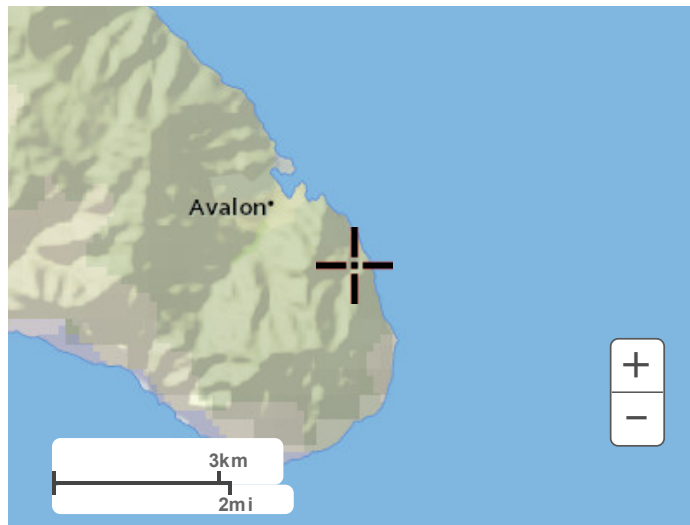
PDS-based depth-duration-frequency (DDF) curves
Latitude: 33.3314°, Longitude: -118.3111°



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Maps & aerials

Small scale terrain



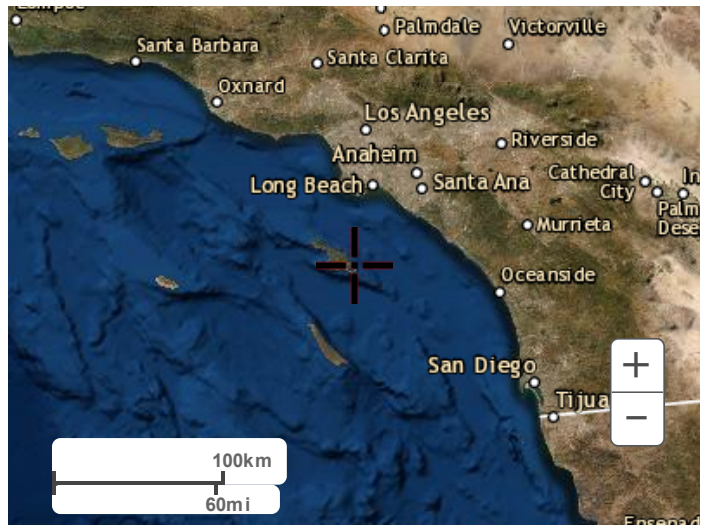
Large scale terrain



Large scale map



Large scale aerial



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Silver Spring, MD 20910
Questions?: HDSC.Questions@noaa.gov

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Project Name: Pebbly Beach Landfill Surface Water Analysis
Client: CR&R
Job No.: SO21.1196
Date: 02/02/2022
Calculated By: AV
Checked By: JMG

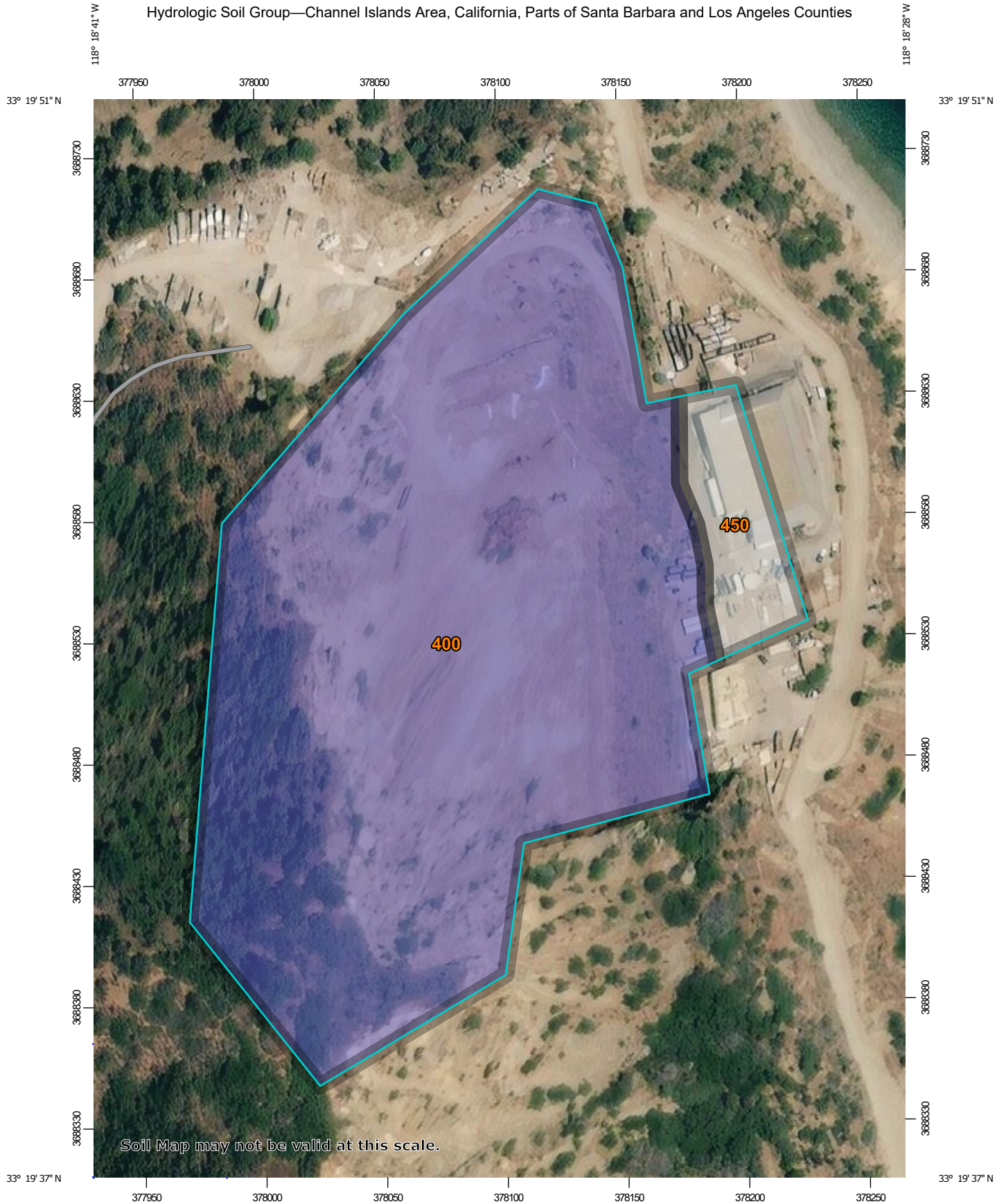
TABLE 4
RAIN GAUGE SUMMARY

Element ID	Data Source	Data Source ID	Rainfall Type	Rain Units	State	County	Return Period (years)	Rainfall Depth (inches)	Rainfall Distribution
Rain Gage-01	Time Series	TS-01	Cumulative	inches	California	Los Angeles (Long Beach)	100	6.72	SCS Type I 24-hr

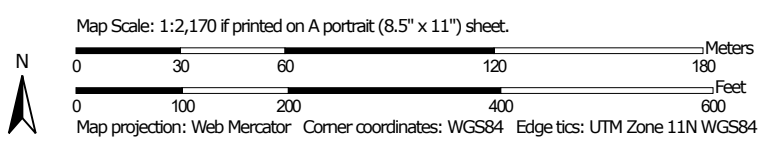
Appendix C.3

Hydrologic Soil Group Data

Hydrologic Soil Group—Channel Islands Area, California, Parts of Santa Barbara and Los Angeles Counties




Soil Map may not be valid at this scale.



MAP LEGEND

Area of Interest (AOI)









 Area of Interest (AOI)

Soils

Soil Rating Polygons





 A
 A/D
 B
 B/D
 C
 C/D
 D
 Not rated or not available

Soil Rating Lines

 A
 A/D
 B
 B/D
 C
 C/D
 D
 Not rated or not available

Soil Rating Points





 A
 A/D
 B
 B/D

 C
 C/D
 D
 Not rated or not available


Water Features

 Streams and Canals

Transportation

 Rails
 Interstate Highways
 US Routes
 Major Roads
 Local Roads

Background

 Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
 Web Soil Survey URL:
 Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Channel Islands Area, California, Parts of Santa Barbara and Los Angeles Counties
 Survey Area Data: Version 16, Sep 13, 2021

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Dec 31, 2009—Oct 25, 2017

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
400	Oboship-Nauti-Bosun complex, 50 to 75 percent slopes	B	12.8	93.6%
450	Urban land-Xerorthents, landscaped, association 0 to 8 percent slopes		0.9	6.4%
Totals for Area of Interest			13.7	100.0%

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

Appendix C.4

Storm and Sanitary Output

Autodesk® Storm and Sanitary Analysis 2016 - Version 13.4.133 (Build 0)

Project Description

File Name 2022-01-26 Pebbly Beach.SPF
Description N:\Pebbly Beach\EXHIBITS\Watersheds.dwg
N:\Pebbly Beach\EXHIBITS\Watersheds.dwg

Analysis Options

Flow Units cfs
Subbasin Hydrograph Method. SCS TR-55
Time of Concentration..... SCS TR-55
Link Routing Method Kinematic Wave
Storage Node Exfiltration.. None
Starting Date JAN-26-2022 00:00:00
Ending Date JAN-27-2022 00:00:00
Report Time Step 00:05:00

Element Count

Number of rain gages 1
Number of subbasins 14
Number of nodes 31
Number of links 29

Raingage Summary

Gage ID	Data Source	Data Type	Recording Interval	min
Rain Gage-01	TS-01	CUMULATIVE	6.00	

Subbasin Summary

Subbasin ID	Total Area acres	Peak Rate Factor
{Site 1}.A1	1.23	484.00
{Site 1}.A2	0.27	484.00
{Site 1}.A3	0.19	484.00
{Site 1}.A4	0.46	484.00
{Site 1}.C1	0.40	484.00
{Site 1}.C2	0.51	484.00
{Site 1}.C3	0.89	484.00
{Site 1}.C4	0.87	484.00
{Site 1}.C5	0.74	484.00
{Site 1}.D1	0.96	484.00
{Site 1}.D2	1.42	484.00
{Site 1}.D3	1.00	484.00
{Site 1}.D4	0.90	484.00
{Site 1}.D5	0.55	484.00

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Node Summary

Node ID	Element Type	Invert Elevation ft	Maximum Elev. ft	Ponded Area ft ²	External Inflow
Jun-01	JUNCTION	246.60	247.60	0.00	
Jun-02	JUNCTION	278.20	279.20	0.00	
Jun-03	JUNCTION	189.17	190.17	0.00	
Jun-04	JUNCTION	286.70	287.70	0.00	
Jun-05	JUNCTION	259.00	260.00	0.00	
Jun-06	JUNCTION	264.50	265.50	0.00	
Jun-07	JUNCTION	261.50	263.00	0.00	
Jun-08	JUNCTION	261.70	263.20	0.00	
Jun-09	JUNCTION	263.92	265.42	0.00	
Jun-10	JUNCTION	262.90	264.40	0.00	
Jun-11	JUNCTION	261.60	263.10	0.00	
Jun-12	JUNCTION	259.50	261.00	0.00	
Jun-13	JUNCTION	189.67	191.17	0.00	
Jun-14	JUNCTION	185.50	187.50	0.00	
Jun-15	JUNCTION	187.60	189.10	0.00	
Jun-16	JUNCTION	188.50	190.00	0.00	
Jun-17	JUNCTION	185.20	187.20	0.00	
Jun-18	JUNCTION	168.50	186.70	0.00	
Jun-19	JUNCTION	168.00	170.00	0.00	
Jun-20	JUNCTION	182.50	184.00	0.00	
Jun-21	JUNCTION	181.50	183.00	0.00	
Jun-22	JUNCTION	160.70	162.20	0.00	
Jun-23	JUNCTION	263.92	265.42	0.00	
Jun-24	JUNCTION	198.50	200.00	0.00	
Jun-25	JUNCTION	257.70	259.20	0.00	
Jun-26	JUNCTION	187.10	188.60	0.00	
Jun-27	JUNCTION	246.00	247.00	0.00	
Jun-28	JUNCTION	245.50	246.50	0.00	
Jun-29	JUNCTION	189.67	190.42	0.00	
Jun-30	JUNCTION	189.67	190.42	0.00	
Jun-31	JUNCTION	189.67	190.42	0.00	

Link Summary

Link ID	From Node	To Node	Element Type	Length ft	Slope %	Manning's Roughness
BENCH-DITCH-1	Jun-04	Jun-01	CHANNEL	252.7	15.8718	0.0190
BENCH-DITCH-2	Jun-02	Jun-01	CHANNEL	263.0	12.0161	0.0190
BENCH-DITCH-3	Jun-05	Jun-29	CHANNEL	390.0	17.7769	0.0190
BENCH-DITCH-4	Jun-06	Jun-31	CHANNEL	428.9	17.4466	0.0190
BENCH-DITCH-5	Jun-31	Jun-03	CHANNEL	3.2	7.7882	0.0190
BENCH-DITCH-6	Jun-29	Jun-03	CHANNEL	3.0	8.3612	0.0190
CULVERT-1	Jun-08	Jun-07	CONDUIT	45.2	0.4424	0.0100
DOWNCHUTE-1A	Jun-28	Jun-30	CHANNEL	107.9	51.7519	0.0320
DOWNCHUTE-1B	Jun-30	Jun-03	CHANNEL	2.4	10.5485	0.0320
DRIVE-THROUGH-SWALE-1	Jun-03	Jun-15	CHANNEL	20.2	7.7608	0.0190
DRIVE-THROUGH-SWALE-2A	Jun-01	Jun-27	CHANNEL	9.7	6.1920	0.0320
DRIVE-THROUGH-SWALE-2B	Jun-27	Jun-28	CHANNEL	2.8	18.1818	0.0320
PIPE-1A	Jun-14	Jun-17	CONDUIT	8.7	3.4602	0.0100
PIPE-1B	Jun-17	Jun-18	CONDUIT	0.0835	500.0000	0.0100
PIPE-1C	Jun-18	Jun-19	CONDUIT	1.9	25.6410	0.0100
V-DITCH-1	Jun-23	Jun-10	CHANNEL	105.4	0.9677	0.0190
V-DITCH-10	Jun-07	Jun-22	CHANNEL	234.9	42.9155	0.0190
V-DITCH-11	Jun-09	Jun-08	CHANNEL	245.7	0.9035	0.0190

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V-DITCH-12	Jun-15	Jun-26	CHANNEL	5.0	10.0000	0.0190
V-DITCH-13	Jun-24	Jun-13	CHANNEL	30.4	29.0939	0.0190
V-DITCH-14	Jun-21	Jun-22	CHANNEL	42.7	48.7005	0.0190
V-DITCH-2	Jun-10	Jun-11	CHANNEL	130.0	1.0000	0.0190
V-DITCH-3	Jun-11	Jun-12	CHANNEL	77.5	2.7107	0.0190
V-DITCH-4	Jun-12	Jun-25	CHANNEL	73.0	2.4658	0.0190
V-DITCH-5	Jun-25	Jun-24	CHANNEL	127.3	46.5043	0.0190
V-DITCH-6	Jun-13	Jun-14	CHANNEL	111.4	3.7446	0.0190
V-DITCH-7	Jun-26	Jun-14	CHANNEL	78.3	2.0445	0.0190
V-DITCH-8	Jun-16	Jun-21	CHANNEL	324.0	2.1605	0.0190
V-DITCH-9	Jun-20	Jun-21	CHANNEL	48.8	2.0500	0.0190

Cross Section Summary

Link Design ID Flow Capacity	Shape	Depth/ Diameter ft	Width ft	No. of Barrels	Cross Sectional Area ft ²	Full Flow Hydraulic Radius ft	
36.73	BENCH-DITCH-1	IRREGULAR	0.50	12.00	1	3.00	0.25
31.96	BENCH-DITCH-2	IRREGULAR	0.50	12.00	1	3.00	0.25
75.80	BENCH-DITCH-3	IRREGULAR	0.75	12.00	1	4.50	0.37
38.51	BENCH-DITCH-4	IRREGULAR	0.50	12.00	1	3.00	0.25
50.17	BENCH-DITCH-5	IRREGULAR	0.75	12.00	1	4.50	0.37
51.99	BENCH-DITCH-6	IRREGULAR	0.75	12.00	1	4.50	0.37
9.08	CULVERT-1	CIRCULAR	1.50	1.50	1	1.77	0.38
15.27	DOWNCHUTE-1A	TRAPEZOIDAL	0.50	3.00	1	1.00	0.31
16.13	DOWNCHUTE-1B	TRAPEZOIDAL	0.75	4.00	1	1.88	0.43
0.60	DRIVE-THROUGH-SWALE-1	TRAPEZOIDAL	1.00	25.00	1	15.00	
0.60	DRIVE-THROUGH-SWALE-2A	TRAPEZOIDAL	1.00	25.00	1	15.00	
0.60	DRIVE-THROUGH-SWALE-2B	TRAPEZOIDAL	1.00	25.00	1	15.00	
54.71	PIPE-1A	CIRCULAR	2.00	2.00	1	3.14	0.50
8498.17	PIPE-1B	CIRCULAR	2.00	2.00	1	3.14	0.50
148.92	PIPE-1C	CIRCULAR	2.00	2.00	1	3.14	0.50
16.42	V-DITCH-1	TRIANGULAR	1.50	4.00	1	3.00	0.60
59.93	V-DITCH-10	TRIANGULAR	1.00	4.00	1	2.00	0.45
15.87	V-DITCH-11	TRIANGULAR	1.50	4.00	1	3.00	0.60
52.78	V-DITCH-12	TRIANGULAR	1.50	4.00	1	3.00	0.60
90.03	V-DITCH-13	TRIANGULAR	1.50	4.00	1	3.00	0.60

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V-DITCH-14	TRIANGULAR	1.50	4.00	1	3.00	0.60
116.48						
V-DITCH-2	TRIANGULAR	1.50	4.00	1	3.00	0.60
16.69						
V-DITCH-3	TRIANGULAR	1.50	4.00	1	3.00	0.60
27.48						
V-DITCH-4	TRIANGULAR	1.50	4.00	1	3.00	0.60
26.21						
V-DITCH-5	TRIANGULAR	1.50	4.00	1	3.00	0.60
113.82						
V-DITCH-6	TRIANGULAR	1.50	4.00	1	3.00	0.60
32.30						
V-DITCH-7	TRIANGULAR	1.50	4.00	1	3.00	0.60
23.87						
V-DITCH-8	TRIANGULAR	1.50	4.00	1	3.00	0.60
24.53						
V-DITCH-9	TRIANGULAR	1.50	4.00	1	3.00	0.60
23.90						

Transect Summary

Transect BENCH DITCH
Area:

0.0004	0.0016	0.0036	0.0064	0.0100
0.0144	0.0196	0.0256	0.0324	0.0400
0.0484	0.0576	0.0676	0.0784	0.0900
0.1024	0.1156	0.1296	0.1444	0.1600
0.1764	0.1936	0.2116	0.2304	0.2500
0.2704	0.2916	0.3136	0.3364	0.3600
0.3844	0.4096	0.4356	0.4624	0.4900
0.5184	0.5476	0.5776	0.6084	0.6400
0.6724	0.7056	0.7396	0.7744	0.8100
0.8464	0.8836	0.9216	0.9604	1.0000

Hrad:

0.0200	0.0400	0.0600	0.0800	0.1000
0.1200	0.1400	0.1600	0.1800	0.2000
0.2200	0.2400	0.2600	0.2800	0.3000
0.3200	0.3400	0.3600	0.3800	0.4000
0.4200	0.4400	0.4600	0.4800	0.5000
0.5200	0.5400	0.5600	0.5800	0.6000
0.6200	0.6400	0.6600	0.6800	0.7000
0.7200	0.7400	0.7600	0.7800	0.8000
0.8200	0.8400	0.8600	0.8800	0.9000
0.9200	0.9400	0.9600	0.9800	1.0000

Width:

0.0200	0.0400	0.0600	0.0800	0.1000
0.1200	0.1400	0.1600	0.1800	0.2000
0.2200	0.2400	0.2600	0.2800	0.3000
0.3200	0.3400	0.3600	0.3800	0.4000
0.4200	0.4400	0.4600	0.4800	0.5000
0.5200	0.5400	0.5600	0.5800	0.6000
0.6200	0.6400	0.6600	0.6800	0.7000
0.7200	0.7400	0.7600	0.7800	0.8000
0.8200	0.8400	0.8600	0.8800	0.9000
0.9200	0.9400	0.9600	0.9800	1.0000

Transect BENCH DITCH 0.75

Area:

0.0004	0.0016	0.0036	0.0064	0.0100
0.0144	0.0196	0.0256	0.0324	0.0400
0.0484	0.0576	0.0676	0.0784	0.0900
0.1024	0.1156	0.1296	0.1444	0.1600
0.1764	0.1936	0.2116	0.2304	0.2500
0.2704	0.2916	0.3136	0.3364	0.3600

	0.3844	0.4096	0.4356	0.4624	0.4900
	0.5184	0.5476	0.5776	0.6084	0.6400
	0.6724	0.7056	0.7396	0.7744	0.8100
	0.8464	0.8836	0.9216	0.9604	1.0000
Hrad:					
	0.0200	0.0400	0.0600	0.0800	0.1000
	0.1200	0.1400	0.1600	0.1800	0.2000
	0.2200	0.2400	0.2600	0.2800	0.3000
	0.3200	0.3400	0.3600	0.3800	0.4000
	0.4200	0.4400	0.4600	0.4800	0.5000
	0.5200	0.5400	0.5600	0.5800	0.6000
	0.6200	0.6400	0.6600	0.6800	0.7000
	0.7200	0.7400	0.7600	0.7800	0.8000
	0.8200	0.8400	0.8600	0.8800	0.9000
	0.9200	0.9400	0.9600	0.9800	1.0000
Width:					
	0.0200	0.0400	0.0600	0.0800	0.1000
	0.1200	0.1400	0.1600	0.1800	0.2000
	0.2200	0.2400	0.2600	0.2800	0.3000
	0.3200	0.3400	0.3600	0.3800	0.4000
	0.4200	0.4400	0.4600	0.4800	0.5000
	0.5200	0.5400	0.5600	0.5800	0.6000
	0.6200	0.6400	0.6600	0.6800	0.7000
	0.7200	0.7400	0.7600	0.7800	0.8000
	0.8200	0.8400	0.8600	0.8800	0.9000
	0.9200	0.9400	0.9600	0.9800	1.0000

```

*****
Runoff Quantity Continuity      Volume      Depth
*****                          acre-ft      inches
-----                          -
Total Precipitation .....      5.841      6.750
Surface Runoff .....           0.440      0.508
Continuity Error (%) .....     -0.002
    
```

```

*****
Flow Routing Continuity      Volume      Volume
*****                          acre-ft      Mgallons
-----                          -
External Inflow .....        0.000      0.000
External Outflow .....       4.394      1.432
Initial Stored Volume ...     0.000      0.000
Final Stored Volume .....     0.003      0.001
Continuity Error (%) .....     -0.000
    
```

 Composite Curve Number Computations Report

 Subbasin {Site 1}.A1

Soil/Surface Description	Area (acres)	Soil Group	CN
-	0.31	C	86.00
-	0.92	C	86.00
Composite Area & Weighted CN	1.23		86.00

 Subbasin {Site 1}.A2

Soil/Surface Description	Area (acres)	Soil Group	CN
-	0.07	C	86.00

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-	0.20	C	86.00
Composite Area & Weighted CN	0.27		86.00

 Subbasin {Site 1}.A3

Soil/Surface Description	Area (acres)	Soil Group	CN
-	0.19	C	86.00
Composite Area & Weighted CN	0.19		86.00

 Subbasin {Site 1}.A4

Soil/Surface Description	Area (acres)	Soil Group	CN
-	0.46	C	86.00
Composite Area & Weighted CN	0.46		86.00

 Subbasin {Site 1}.C1

Soil/Surface Description	Area (acres)	Soil Group	CN
-	0.10	C	86.00
-	0.30	C	86.00
Composite Area & Weighted CN	0.40		86.00

 Subbasin {Site 1}.C2

Soil/Surface Description	Area (acres)	Soil Group	CN
-	0.51	C	86.00
Composite Area & Weighted CN	0.51		86.00

 Subbasin {Site 1}.C3

Soil/Surface Description	Area (acres)	Soil Group	CN
-	0.22	C	86.00
-	0.67	C	86.00
Composite Area & Weighted CN	0.89		86.00

 Subbasin {Site 1}.C4

Soil/Surface Description	Area (acres)	Soil Group	CN
-	0.22	C	86.00
-	0.65	C	86.00
Composite Area & Weighted CN	0.87		86.00

 Subbasin {Site 1}.C5

Soil/Surface Description	Area (acres)	Soil Group	CN
-	0.18	C	86.00

-	0.55	C	86.00
Composite Area & Weighted CN	0.74		86.00

 Subbasin {Site 1}.D1

Soil/Surface Description	Area (acres)	Soil Group	CN
-	0.24	C	86.00
-	0.72	C	86.00
Composite Area & Weighted CN	0.96		86.00

 Subbasin {Site 1}.D2

Soil/Surface Description	Area (acres)	Soil Group	CN
-	0.36	C	86.00
-	1.07	C	86.00
Composite Area & Weighted CN	1.42		86.00

 Subbasin {Site 1}.D3

Soil/Surface Description	Area (acres)	Soil Group	CN
-	0.25	C	86.00
-	0.75	C	86.00
Composite Area & Weighted CN	1.00		86.00

 Subbasin {Site 1}.D4

Soil/Surface Description	Area (acres)	Soil Group	CN
-	0.22	C	86.00
-	0.67	C	86.00
Composite Area & Weighted CN	0.90		86.00

 Subbasin {Site 1}.D5

Soil/Surface Description	Area (acres)	Soil Group	CN
-	0.14	C	86.00
-	0.41	C	86.00
Composite Area & Weighted CN	0.55		86.00

 SCS TR-55 Time of Concentration Computations Report

Sheet Flow Equation

$$T_c = (0.007 * ((n * L_f)^{0.8})) / ((P^{0.5}) * (S_f^{0.4}))$$

Where:

Tc = Time of Concentration (hrs)
 n = Manning's Roughness

Lf = Flow Length (ft)
 P = 2 yr, 24 hr Rainfall (inches)
 Sf = Slope (ft/ft)

Shallow Concentrated Flow Equation

V = 16.1345 * (Sf^0.5) (unpaved surface)
 V = 20.3282 * (Sf^0.5) (paved surface)
 V = 15.0 * (Sf^0.5) (grassed waterway surface)
 V = 10.0 * (Sf^0.5) (nearly bare & untilled surface)
 V = 9.0 * (Sf^0.5) (cultivated straight rows surface)
 V = 7.0 * (Sf^0.5) (short grass pasture surface)
 V = 5.0 * (Sf^0.5) (woodland surface)
 V = 2.5 * (Sf^0.5) (forest w/heavy litter surface)
 Tc = (Lf / V) / (3600 sec/hr)

Where:

Tc = Time of Concentration (hrs)
 Lf = Flow Length (ft)
 V = Velocity (ft/sec)
 Sf = Slope (ft/ft)

Channel Flow Equation

V = (1.49 * (R^(2/3)) * (Sf^0.5)) / n
 R = Aq / Wp
 Tc = (Lf / V) / (3600 sec/hr)

Where:

Tc = Time of Concentration (hrs)
 Lf = Flow Length (ft)
 R = Hydraulic Radius (ft)
 Aq = Flow Area (ft²)
 Wp = Wetted Perimeter (ft)
 V = Velocity (ft/sec)
 Sf = Slope (ft/ft)
 n = Manning's Roughness

Subbasin {Site 1}.A1

Sheet Flow Computations

	Subarea A	Subarea B	Subarea
C			
0.00	Manning's Roughness:	0.30	0.00
0.00	Flow Length (ft):	100.00	0.00
0.00	Slope (%):	38.10	0.00
0.00	2 yr, 24 hr Rainfall (in):	2.59	0.00
0.00	Velocity (ft/sec):	0.29	0.00
0.00	Computed Flow Time (minutes):	5.83	0.00
=====			
	Total TOC (minutes):	5.83	
=====			

 Subbasin (Site 1).A2

Sheet Flow Computations

		Subarea A	Subarea B	Subarea
C				
	Manning's Roughness:	0.30	0.00	
0.00				
	Flow Length (ft):	100.00	0.00	
0.00				
	Slope (%):	62.80	0.00	
0.00				
	2 yr, 24 hr Rainfall (in):	2.59	1.00	
1.00				
	Velocity (ft/sec):	0.35	0.00	
0.00				
	Computed Flow Time (minutes):	4.78	0.00	
0.00				
=====				
	Total TOC (minutes):	4.78		
=====				

 Subbasin (Site 1).A3

Sheet Flow Computations

		Subarea A	Subarea B	Subarea
C				
	Manning's Roughness:	0.30	0.00	
0.00				
	Flow Length (ft):	100.00	0.00	
0.00				
	Slope (%):	31.30	0.00	
0.00				
	2 yr, 24 hr Rainfall (in):	2.59	0.00	
0.00				
	Velocity (ft/sec):	0.26	0.00	
0.00				
	Computed Flow Time (minutes):	6.31	0.00	
0.00				

Channel Flow Computations

		Subarea A	Subarea B	Subarea
C				
	Manning's Roughness:	0.02	0.00	
0.00				
	Flow Length (ft):	127.00	0.00	
0.00				
	Channel Slope (%):	31.30	0.00	
0.00				
	Cross Section Area (ft ²):	3.00	0.00	
0.00				
	Wetted Perimeter (ft):	5.00	0.00	
0.00				
	Velocity (ft/sec):	31.21	0.00	
0.00				
	Computed Flow Time (minutes):	0.07	0.00	

0.00

```

=====
Total TOC (minutes):                6.38
=====
    
```

```

-----
Subbasin {Site 1}.A4
-----
    
```

Sheet Flow Computations

		Subarea A	Subarea B	Subarea
C				
0.00	Manning's Roughness:	0.30	0.00	
0.00	Flow Length (ft):	100.00	0.00	
0.00	Slope (%):	62.80	0.00	
0.00	2 yr, 24 hr Rainfall (in):	2.59	0.00	
0.00	Velocity (ft/sec):	0.35	0.00	
0.00	Computed Flow Time (minutes):	4.78	0.00	

```

=====
Total TOC (minutes):                4.78
=====
    
```

```

-----
Subbasin {Site 1}.C1
-----
    
```

Sheet Flow Computations

		Subarea A	Subarea B	Subarea
C				
0.00	Manning's Roughness:	0.30	0.00	
0.00	Flow Length (ft):	100.00	0.00	
0.00	Slope (%):	62.80	0.00	
1.00	2 yr, 24 hr Rainfall (in):	2.59	1.00	
0.00	Velocity (ft/sec):	0.35	0.00	
0.00	Computed Flow Time (minutes):	4.78	0.00	

```

=====
Total TOC (minutes):                4.78
=====
    
```

```

-----
Subbasin {Site 1}.C2
-----
    
```

Sheet Flow Computations

```

-----
C                               Subarea A           Subarea B           Subarea
0.00   Manning's Roughness:           0.30             0.00
0.00   Flow Length (ft):             100.00           0.00
0.00   Slope (%):                    62.80            0.00
0.00   2 yr, 24 hr Rainfall (in):    2.59             0.00
0.00   Velocity (ft/sec):             0.35             0.00
0.00   Computed Flow Time (minutes):  4.78             0.00

=====
Total TOC (minutes):                4.78
=====

```

Subbasin {Site 1}.C3

Sheet Flow Computations

```

-----
C                               Subarea A           Subarea B           Subarea
0.00   Manning's Roughness:           0.30             0.00
0.00   Flow Length (ft):             100.00           0.00
0.00   Slope (%):                    62.80            0.00
0.00   2 yr, 24 hr Rainfall (in):    2.59             1.00
1.00   Velocity (ft/sec):             0.35             0.00
0.00   Computed Flow Time (minutes):  4.78             0.00

=====
Total TOC (minutes):                4.78
=====

```

Subbasin {Site 1}.C4

Sheet Flow Computations

```

-----
C                               Subarea A           Subarea B           Subarea
0.00   Manning's Roughness:           0.30             0.00
0.00   Flow Length (ft):             100.00           0.00
0.00   Slope (%):                    62.80            0.00
0.00   2 yr, 24 hr Rainfall (in):    2.59             0.00
0.00   Velocity (ft/sec):             0.35             0.00
0.00

```

0.00 Computed Flow Time (minutes): 4.78 0.00

=====
 Total TOC (minutes): 4.78
 =====

 Subbasin {Site 1}.C5

Sheet Flow Computations

	Subarea A	Subarea B	Subarea
C			
0.00	Manning's Roughness: 0.30	0.00	
0.00	Flow Length (ft): 100.00	0.00	
0.00	Slope (%): 72.80	0.00	
0.00	2 yr, 24 hr Rainfall (in): 2.59	0.00	
0.00	Velocity (ft/sec): 0.37	0.00	
0.00	Computed Flow Time (minutes): 4.50	0.00	

Shallow Concentrated Flow Computations

	Subarea A	Subarea B	Subarea
C			
0.00	Flow Length (ft): 30.00	0.00	
0.00	Slope (%): 72.90	0.00	
Unpaved	Surface Type: Unpaved	Unpaved	
0.00	Velocity (ft/sec): 13.78	0.00	
0.00	Computed Flow Time (minutes): 0.04	0.00	

=====
 Total TOC (minutes): 4.54
 =====

 Subbasin {Site 1}.D1

Sheet Flow Computations

	Subarea A	Subarea B	Subarea
C			
0.00	Manning's Roughness: 0.30	0.00	
0.00	Flow Length (ft): 100.00	0.00	
0.00	Slope (%): 62.80	0.00	
0.00	2 yr, 24 hr Rainfall (in): 2.59	0.00	

0.00	Velocity (ft/sec):	0.35	0.00
0.00	Computed Flow Time (minutes):	4.78	0.00

=====
 Total TOC (minutes): 4.78
 =====

 Subbasin {Site 1}.D2

Sheet Flow Computations

		Subarea A	Subarea B	Subarea
C				
0.00	Manning's Roughness:	0.30	0.00	
0.00	Flow Length (ft):	100.00	0.00	
0.00	Slope (%):	62.80	0.00	
1.00	2 yr, 24 hr Rainfall (in):	2.59	0.00	
0.00	Velocity (ft/sec):	0.35	0.00	
0.00	Computed Flow Time (minutes):	4.78	0.00	

Channel Flow Computations

		Subarea A	Subarea B	Subarea
C				
0.00	Manning's Roughness:	0.02	0.00	
0.00	Flow Length (ft):	194.00	0.00	
0.00	Channel Slope (%):	1.00	0.00	
0.00	Cross Section Area (ft ²):	3.00	0.00	
0.00	Wetted Perimeter (ft):	5.00	0.00	
0.00	Velocity (ft/sec):	5.58	0.00	
0.00	Computed Flow Time (minutes):	0.58	0.00	

=====
 Total TOC (minutes): 5.36
 =====

 Subbasin {Site 1}.D3

Sheet Flow Computations

		Subarea A	Subarea B	Subarea
C				
0.00	Manning's Roughness:	0.30	0.00	

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0.00	Flow Length (ft):	100.00	0.00
0.00	Slope (%):	62.80	0.00
0.00	2 yr, 24 hr Rainfall (in):	2.59	0.00
0.00	Velocity (ft/sec):	0.35	0.00
0.00	Computed Flow Time (minutes):	4.78	0.00

Channel Flow Computations

		Subarea A	Subarea B	Subarea
C	Manning's Roughness:	0.02	0.00	
0.00	Flow Length (ft):	88.00	0.00	
0.00	Channel Slope (%):	1.00	0.00	
0.00	Cross Section Area (ft ²):	3.00	0.00	
0.00	Wetted Perimeter (ft):	5.00	0.00	
0.00	Velocity (ft/sec):	5.58	0.00	
0.00	Computed Flow Time (minutes):	0.26	0.00	
Total TOC (minutes):		5.04		

Subbasin {Site 1}.D4

Sheet Flow Computations

		Subarea A	Subarea B	Subarea
C	Manning's Roughness:	0.30	0.00	
0.00	Flow Length (ft):	100.00	0.00	
0.00	Slope (%):	62.80	0.00	
0.00	2 yr, 24 hr Rainfall (in):	2.59	0.00	
0.00	Velocity (ft/sec):	0.35	0.00	
0.00	Computed Flow Time (minutes):	4.78	0.00	
Total TOC (minutes):		4.78		

Subbasin {Site 1}.D5

Sheet Flow Computations

	Subarea A	Subarea B	Subarea
C			
0.00	Manning's Roughness:	0.30	0.00
0.00	Flow Length (ft):	100.00	0.00
0.00	Slope (%):	76.30	0.00
0.00	2 yr, 24 hr Rainfall (in):	2.59	0.00
0.00	Velocity (ft/sec):	0.38	0.00
0.00	Computed Flow Time (minutes):	4.42	0.00
=====			
	Total TOC (minutes):	4.42	
=====			

Subbasin Runoff Summary

Subbasin ID	Total Precip in	Total Runoff in	Peak Runoff cfs	Weighted Curve Number	Time of Concentration days hh:mm:ss
{Site 1}.A1	6.72	5.10	4.99	86.000	0 00:05:49
{Site 1}.A2	6.72	5.10	1.14	86.000	0 00:04:46
{Site 1}.A3	6.72	5.10	0.76	86.000	0 00:06:22
{Site 1}.A4	6.72	5.10	1.89	86.000	0 00:04:46
{Site 1}.C1	6.72	5.10	1.68	86.000	0 00:04:46
{Site 1}.C2	6.72	5.10	2.13	86.000	0 00:04:46
{Site 1}.C3	6.72	5.10	3.72	86.000	0 00:04:46
{Site 1}.C4	6.72	5.10	3.62	86.000	0 00:04:46
{Site 1}.C5	6.72	5.10	3.08	86.000	0 00:04:32
{Site 1}.D1	6.72	5.10	3.96	86.000	0 00:04:46
{Site 1}.D2	6.72	5.10	5.81	86.000	0 00:05:21
{Site 1}.D3	6.72	5.10	4.12	86.000	0 00:05:02
{Site 1}.D4	6.72	5.10	3.72	86.000	0 00:04:46
{Site 1}.D5	6.72	5.10	2.28	86.000	0 00:04:25

Node Depth Summary

Node ID	Average Depth Attained ft	Maximum Depth Attained ft	Maximum HGL Attained ft	Time of Max Occurrence days hh:mm	Total Flooded Volume acre-in	Total Time Flooded minutes	Retention Time hh:mm:ss
Jun-01	0.05	0.18	246.78	0 10:00	0	0	0:00:00
Jun-02	0.05	0.18	278.38	0 10:00	0	0	0:00:00
Jun-03	0.33	0.62	189.79	0 10:01	0	0	0:00:00
Jun-04	0.04	0.16	286.86	0 10:00	0	0	0:00:00
Jun-05	0.06	0.24	259.24	0 10:00	0	0	0:00:00
Jun-06	0.06	0.21	264.71	0 10:00	0	0	0:00:00
Jun-07	0.14	0.78	262.28	0 10:01	0	0	0:00:00
Jun-08	0.26	0.96	262.66	0 10:01	0	0	0:00:00

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Jun-09	0.26	0.97	264.89	0	10:00	0	0	0:00:00
Jun-10	0.24	0.88	263.78	0	10:00	0	0	0:00:00
Jun-11	0.28	1.01	262.61	0	10:00	0	0	0:00:00
Jun-12	0.35	1.28	260.78	0	10:00	0	0	0:00:00
Jun-13	0.36	1.31	190.98	0	10:00	0	0	0:00:00
Jun-14	0.36	1.31	186.81	0	10:00	0	0	0:00:00
Jun-15	0.23	0.83	188.43	0	10:01	0	0	0:00:00
Jun-16	0.15	0.57	189.07	0	10:00	0	0	0:00:00
Jun-17	0.20	1.13	186.33	0	10:00	0	0	0:00:00
Jun-18	0.13	0.64	169.14	0	10:00	0	0	0:00:00
Jun-19	1.75	2.00	170.00	0	03:00	0	0	0:00:00
Jun-20	0.11	0.41	182.91	0	10:00	0	0	0:00:00
Jun-21	0.15	0.57	182.07	0	10:01	0	0	0:00:00
Jun-22	1.31	1.50	162.20	0	03:01	0	0	0:00:00
Jun-23	0.24	0.88	264.80	0	10:00	0	0	0:00:00
Jun-24	0.23	0.85	199.35	0	10:00	0	0	0:00:00
Jun-25	0.35	1.28	258.98	0	10:00	0	0	0:00:00
Jun-26	0.30	1.12	188.22	0	10:01	0	0	0:00:00
Jun-27	0.03	0.18	246.18	0	10:00	0	0	0:00:00
Jun-28	0.04	0.24	245.74	0	10:00	0	0	0:00:00
Jun-29	0.07	0.27	189.94	0	10:00	0	0	0:00:00
Jun-30	0.06	0.37	190.04	0	10:01	0	0	0:00:00
Jun-31	0.08	0.28	189.95	0	10:00	0	0	0:00:00

Node Flow Summary

Node ID	Element Type	Maximum Lateral Inflow cfs	Peak Inflow cfs	Time of Peak Inflow Occurrence days hh:mm	Maximum Flooding Overflow cfs	Time of Peak Flooding Occurrence days hh:mm
Jun-01	JUNCTION	0.00	3.75	0 10:00	0.00	
Jun-02	JUNCTION	2.13	2.13	0 10:00	0.00	
Jun-03	JUNCTION	0.00	10.93	0 10:00	0.00	
Jun-04	JUNCTION	1.68	1.68	0 10:00	0.00	
Jun-05	JUNCTION	3.62	3.62	0 10:00	0.00	
Jun-06	JUNCTION	3.72	3.72	0 10:00	0.00	
Jun-07	JUNCTION	1.14	5.97	0 10:01	0.00	
Jun-08	JUNCTION	0.00	4.87	0 10:01	0.00	
Jun-09	JUNCTION	4.96	4.96	0 10:00	0.00	
Jun-10	JUNCTION	0.00	3.93	0 10:00	0.00	
Jun-11	JUNCTION	5.79	9.54	0 10:00	0.00	
Jun-12	JUNCTION	7.84	17.26	0 10:00	0.00	
Jun-13	JUNCTION	3.08	22.54	0 10:00	0.00	
Jun-14	JUNCTION	0.00	33.33	0 10:00	0.00	
Jun-15	JUNCTION	0.00	10.94	0 10:01	0.00	
Jun-16	JUNCTION	1.89	1.89	0 10:00	0.00	
Jun-17	JUNCTION	0.00	33.32	0 10:00	0.00	
Jun-18	JUNCTION	0.00	33.32	0 10:00	0.00	
Jun-19	JUNCTION	0.00	33.35	0 10:01	0.00	
Jun-20	JUNCTION	0.75	0.75	0 10:00	0.00	
Jun-21	JUNCTION	0.00	2.58	0 10:00	0.00	
Jun-22	JUNCTION	0.00	8.55	0 10:01	0.00	
Jun-23	JUNCTION	3.96	3.96	0 10:00	0.00	
Jun-24	JUNCTION	2.28	19.53	0 10:00	0.00	
Jun-25	JUNCTION	0.00	17.30	0 10:00	0.00	
Jun-26	JUNCTION	0.00	10.94	0 10:01	0.00	
Jun-27	JUNCTION	0.00	3.75	0 10:00	0.00	
Jun-28	JUNCTION	0.00	3.75	0 10:00	0.00	
Jun-29	JUNCTION	0.00	3.57	0 10:00	0.00	
Jun-30	JUNCTION	0.00	3.74	0 10:01	0.00	
Jun-31	JUNCTION	0.00	3.65	0 10:00	0.00	

 Link Flow Summary

Link ID	Element	Time of	Maximum	Length	Peak Flow	Design	Ratio of
Ratio of	Total	Reported	Peak Flow	Velocity	during	Flow	Maximum
Maximum	Time	Type	Occurrence	Attained	Analysis	Capacity	/Design
Flow	Surcharged	Condition	days	ft/sec	cfs	cfs	Flow
Depth	minutes		hh:mm				
BENCH-DITCH-1	CHANNEL	0 10:00	5.67	1.00	1.66	36.73	0.05
0.31	0 Calculated						
BENCH-DITCH-2	CHANNEL	0 10:00	5.41	1.00	2.10	31.96	0.07
0.36	0 Calculated						
BENCH-DITCH-3	CHANNEL	0 10:00	7.90	1.00	3.57	75.80	0.05
0.32	0 Calculated						
BENCH-DITCH-4	CHANNEL	0 10:00	7.17	1.00	3.65	38.51	0.09
0.41	0 Calculated						
BENCH-DITCH-5	CHANNEL	0 10:00	5.79	1.00	3.65	50.17	0.07
0.37	0 Calculated						
BENCH-DITCH-6	CHANNEL	0 10:00	5.91	1.00	3.56	51.99	0.07
0.37	0 Calculated						
CULVERT-1	CONDUIT	0 10:01	5.24	1.00	4.88	9.08	0.54
0.52	0 Calculated						
DOWNCHUTE-1A	CHANNEL	0 10:01	10.39	1.00	3.74	15.27	0.25
0.49	0 Calculated						
DOWNCHUTE-1B	CHANNEL	0 10:01	5.85	1.00	3.74	16.13	0.23
0.49	0 Calculated						
DRIVE-THROUGH-SWALE-1	CHANNEL	0 10:01	6.72	1.00	10.94	231.88	0.05
0.22	0 Calculated						
DRIVE-THROUGH-SWALE-2A	CHANNEL	0 10:00	3.12	1.00	3.75	122.98	
0.03	0.18 0 Calculated						
DRIVE-THROUGH-SWALE-2B	CHANNEL	0 10:00	4.51	1.00	3.75	210.73	
0.02	0.13 0 Calculated						
PIPE-1A	CONDUIT	0 10:00	18.25	1.00	33.32	54.71	0.61
0.56	0 Calculated						
PIPE-1B	CONDUIT	0 10:00	>50.00	1.00	33.32	8498.17	0.00
0.05	0 Calculated						
PIPE-1C	CONDUIT	0 10:01	38.23	1.00	33.35	148.92	0.22
0.32	0 Calculated						
V-DITCH-1	CHANNEL	0 10:00	3.85	1.00	3.93	16.42	0.24
0.58	0 Calculated						
V-DITCH-10	CHANNEL	0 10:01	16.84	1.00	5.97	59.93	0.10
0.42	0 Calculated						
V-DITCH-11	CHANNEL	0 10:01	3.96	1.00	4.87	15.87	0.31
0.64	0 Calculated						
V-DITCH-12	CHANNEL	0 10:01	11.87	1.00	10.94	52.78	0.21
0.55	0 Calculated						
V-DITCH-13	CHANNEL	0 10:00	20.48	1.00	19.53	90.03	0.22
0.56	0 Calculated						
V-DITCH-14	CHANNEL	0 10:01	14.99	1.00	2.58	116.48	0.02
0.24	0 Calculated						
V-DITCH-2	CHANNEL	0 10:01	3.89	1.00	3.90	16.69	0.23
0.58	0 Calculated						
V-DITCH-3	CHANNEL	0 10:00	7.04	1.00	9.57	27.48	0.35
0.67	0 Calculated						
V-DITCH-4	CHANNEL	0 10:00	7.89	1.00	17.30	26.21	0.66
0.86	0 Calculated						

N:\Pebbly Beach\EXHIBITS\Watersheds.dwg

V-DITCH-5		CHANNEL	0	10:00	23.69	1.00	17.30	113.82	0.15
0.49	0	Calculated							
V-DITCH-6		CHANNEL	0	10:00	9.84	1.00	22.54	32.30	0.70
0.87	0	Calculated							
V-DITCH-7		CHANNEL	0	10:01	6.55	1.00	10.96	23.87	0.46
0.75	0	Calculated							
V-DITCH-8		CHANNEL	0	10:01	4.33	1.00	1.84	24.53	0.08
0.38	0	Calculated							
V-DITCH-9		CHANNEL	0	10:00	3.36	1.00	0.75	23.90	0.03
0.27	0	Calculated							

Highest Flow Instability Indexes

Link PIPE-1C (4)
Link PIPE-1B (4)
Link BENCH-DITCH-5 (4)
Link BENCH-DITCH-6 (3)
Link DRIVE-THROUGH-SWALE-1 (1)

WARNING 107 : Initial water surface elevation defined for Junction Jun-08 is below junction invert elevation.

Assumed initial water surface elevation equal to invert elevation.

WARNING 117 : Conduit outlet invert elevation defined for Conduit BENCH-DITCH-3 is below downstream node invert elevation.

Assumed conduit outlet invert elevation equal to downstream node invert elevation.

WARNING 117 : Conduit outlet invert elevation defined for Conduit BENCH-DITCH-4 is below downstream node invert elevation.

Assumed conduit outlet invert elevation equal to downstream node invert elevation.

WARNING 008 : Elevation drop exceeds length for Conduit PIPE-1B.

WARNING 002 : Max/rim elevation (depth) increased to account for connecting conduit height dimensions for Node Jun-14.

WARNING 002 : Max/rim elevation (depth) increased to account for connecting conduit height dimensions for Node Jun-15.

WARNING 002 : Max/rim elevation (depth) increased to account for connecting conduit height dimensions for Node Jun-28.

WARNING 002 : Max/rim elevation (depth) increased to account for connecting conduit height dimensions for Node Jun-29.

WARNING 002 : Max/rim elevation (depth) increased to account for connecting conduit height dimensions for Node Jun-30.

WARNING 002 : Max/rim elevation (depth) increased to account for connecting conduit height dimensions for Node Jun-31.

Analysis began on: Thu Feb 3 13:13:21 2022

Analysis ended on: Thu Feb 3 13:13:24 2022

Total elapsed time: 00:00:03

Appendix C.5

Peak Discharge Summary

Project Name: Pebbly Beach Landfill Surface Water Analysis
Client: CR&R
Job No.: SO21.1196
Date: 02/02/2022
Calculated By: AV
Checked By: JMG

TABLE 5
PEAK DISCHARGE SUMMARY

Sub-Area Name	Area (acres)	Weighted Curve Number	Rain Gage ID	Total Precipitation (inches)	Total Runoff (inches)	Peak Runoff (cfs)	Time of Concentration (days hh:mm:ss)
{Site 1}.A1	1.23	86.00	Rain Gage-01	6.72	5.10	4.99	0 00:05:49
{Site 1}.A2	0.27	86.00	Rain Gage-01	6.72	5.10	1.14	0 00:04:46
{Site 1}.A3	0.19	86.00	Rain Gage-01	6.72	5.10	0.76	0 00:06:22
{Site 1}.A4	0.46	86.00	Rain Gage-01	6.72	5.10	1.89	0 00:04:46
{Site 1}.C1	0.40	86.00	Rain Gage-01	6.72	5.10	1.68	0 00:04:46
{Site 1}.C2	0.51	86.00	Rain Gage-01	6.72	5.10	2.13	0 00:04:46
{Site 1}.C3	0.89	86.00	Rain Gage-01	6.72	5.10	3.72	0 00:04:46
{Site 1}.C4	0.87	86.00	Rain Gage-01	6.72	5.10	3.62	0 00:04:46
{Site 1}.C5	0.74	86.00	Rain Gage-01	6.72	5.10	3.08	0 00:04:32
{Site 1}.D1	0.96	86.00	Rain Gage-01	6.72	5.10	3.96	0 00:04:46
{Site 1}.D2	1.42	86.00	Rain Gage-01	6.72	5.10	5.81	0 00:05:21
{Site 1}.D3	1.00	86.00	Rain Gage-01	6.72	5.10	4.12	0 00:05:02
{Site 1}.D4	0.90	86.00	Rain Gage-01	6.72	5.10	3.72	0 00:04:46
{Site 1}.D5	0.55	86.00	Rain Gage-01	6.72	5.10	2.28	0 00:04:25

Appendix C.6

Open Channel Design

Channel Report

Downshute

Triangular

Side Slopes (z:1) = 2.00, 23.98
Total Depth (ft) = 0.50

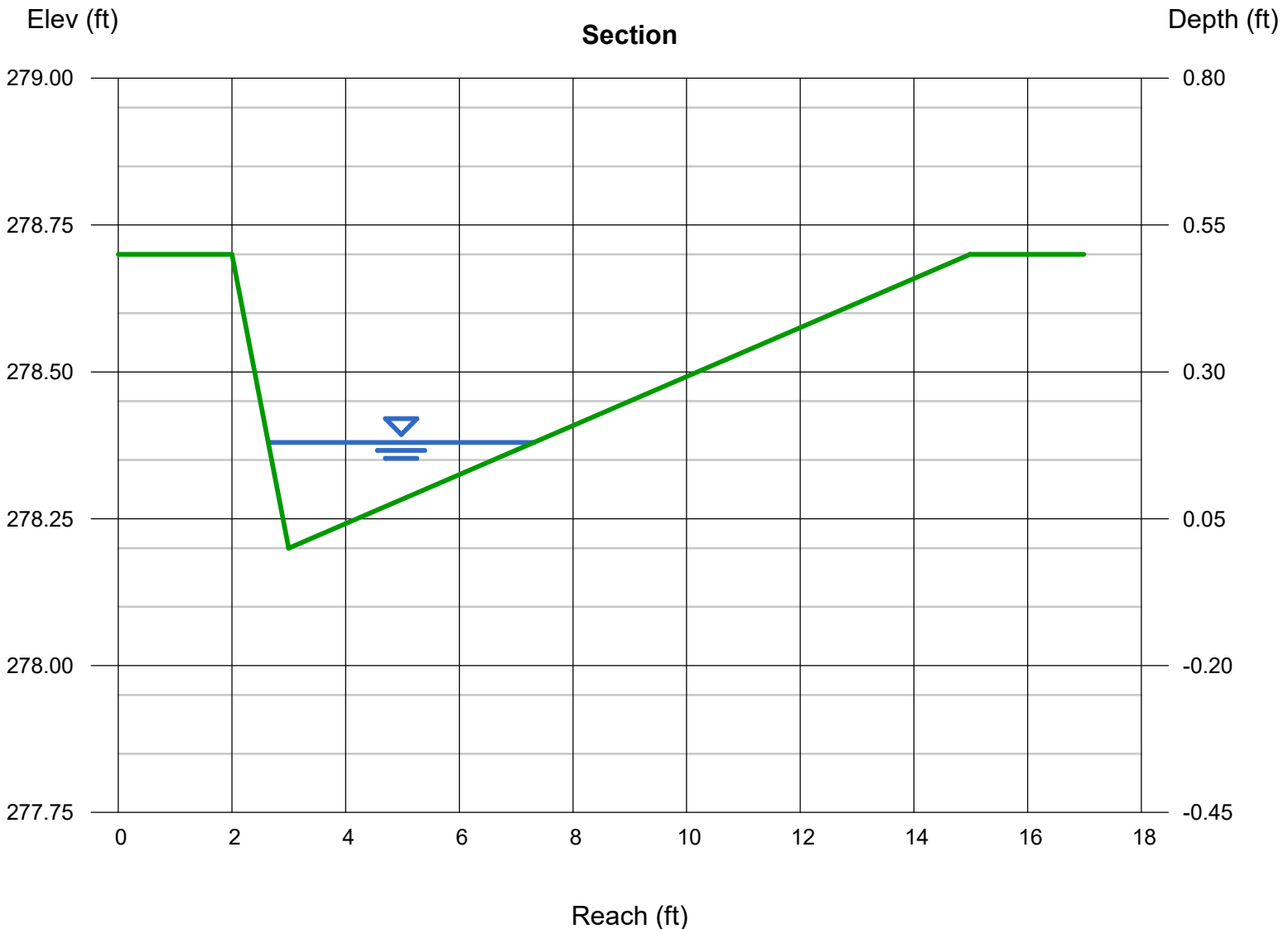
Invert Elev (ft) = 278.20
Slope (%) = 12.00
N-Value = 0.019

Calculations

Compute by: Known Q
Known Q (cfs) = 2.10

Highlighted

Depth (ft) = 0.18
Q (cfs) = 2.100
Area (sqft) = 0.42
Velocity (ft/s) = 4.99
Wetted Perim (ft) = 4.72
Crit Depth, Yc (ft) = 0.28
Top Width (ft) = 4.68
EGL (ft) = 0.57



Channel Report

Culvert 18 in

Circular

Diameter (ft) = 1.50

Invert Elev (ft) = 261.70

Slope (%) = 0.44

N-Value = 0.010

Calculations

Compute by: Known Q

Known Q (cfs) = 4.88

Highlighted

Depth (ft) = 0.79

Q (cfs) = 4.880

Area (sqft) = 0.94

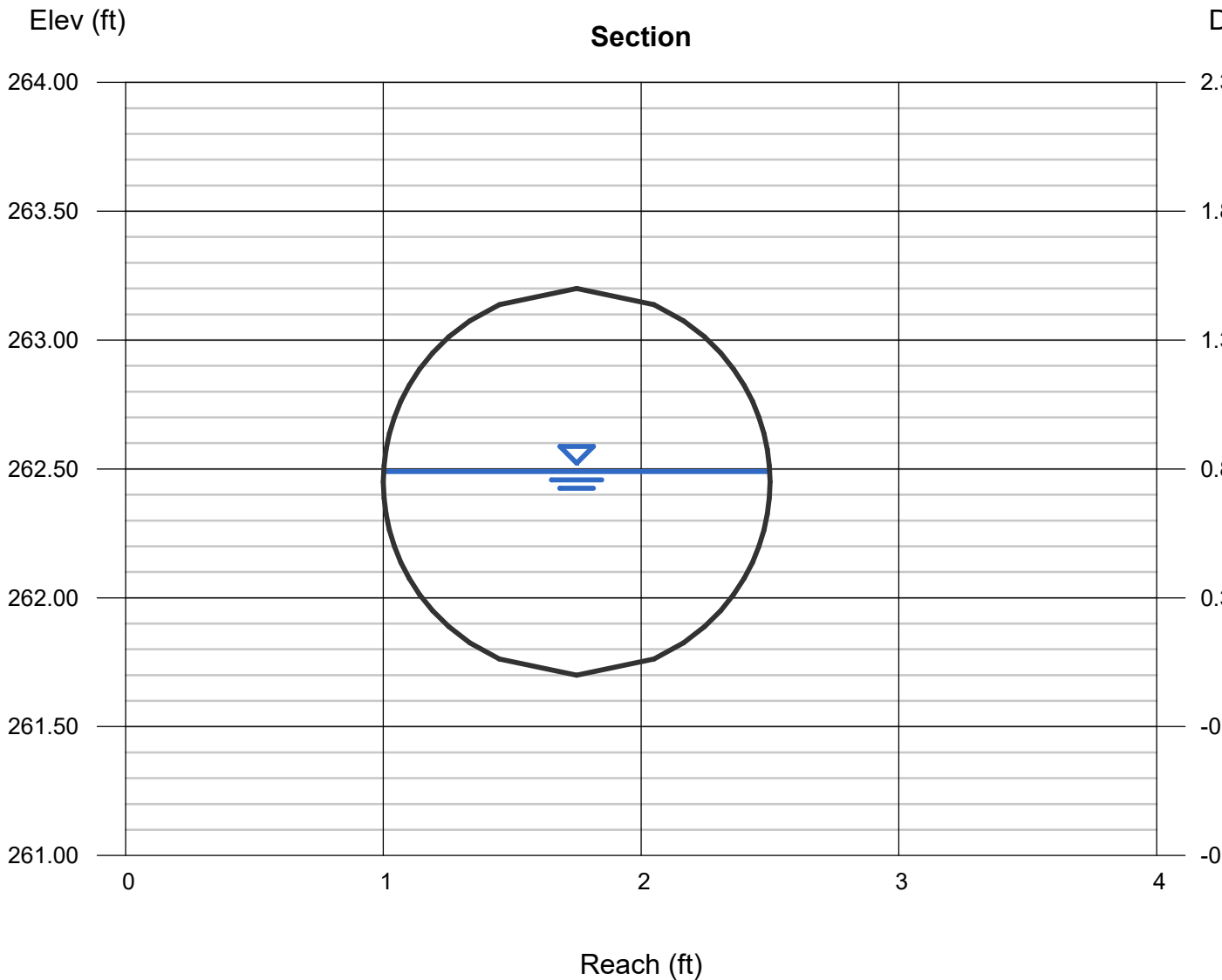
Velocity (ft/s) = 5.17

Wetted Perim (ft) = 2.44

Crit Depth, Y_c (ft) = 0.85

Top Width (ft) = 1.50

EGL (ft) = 1.21



Channel Report

Downchute

Trapezoidal

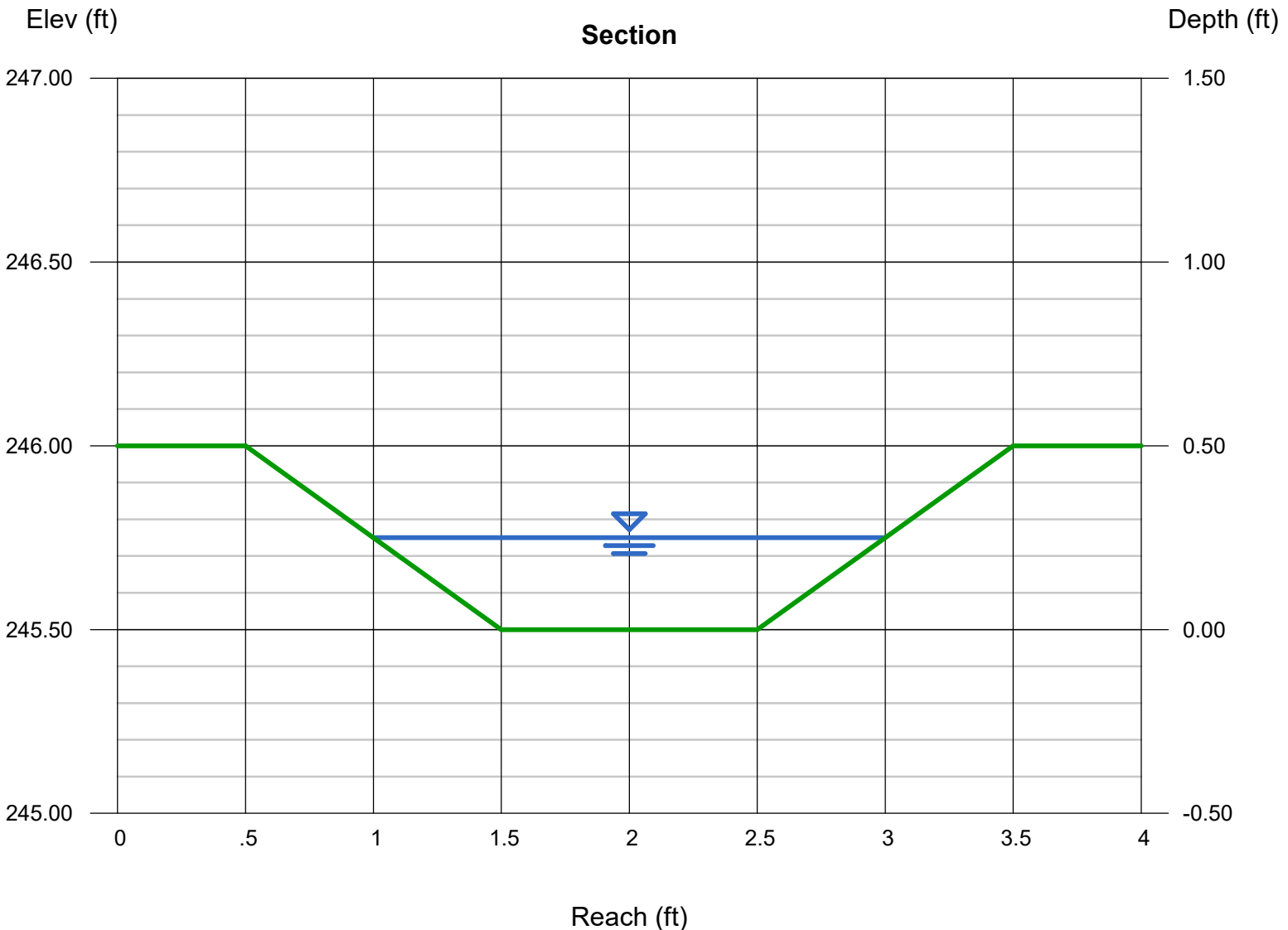
Bottom Width (ft) = 1.00
Side Slopes (z:1) = 2.00, 2.00
Total Depth (ft) = 0.50
Invert Elev (ft) = 245.50
Slope (%) = 51.75
N-Value = 0.032

Highlighted

Depth (ft) = 0.25
Q (cfs) = 3.740
Area (sqft) = 0.38
Velocity (ft/s) = 9.97
Wetted Perim (ft) = 2.12
Crit Depth, Yc (ft) = 0.50
Top Width (ft) = 2.00
EGL (ft) = 1.80

Calculations

Compute by: Known Q
Known Q (cfs) = 3.74



Channel Report

Drive-Through Swale

Trapezoidal

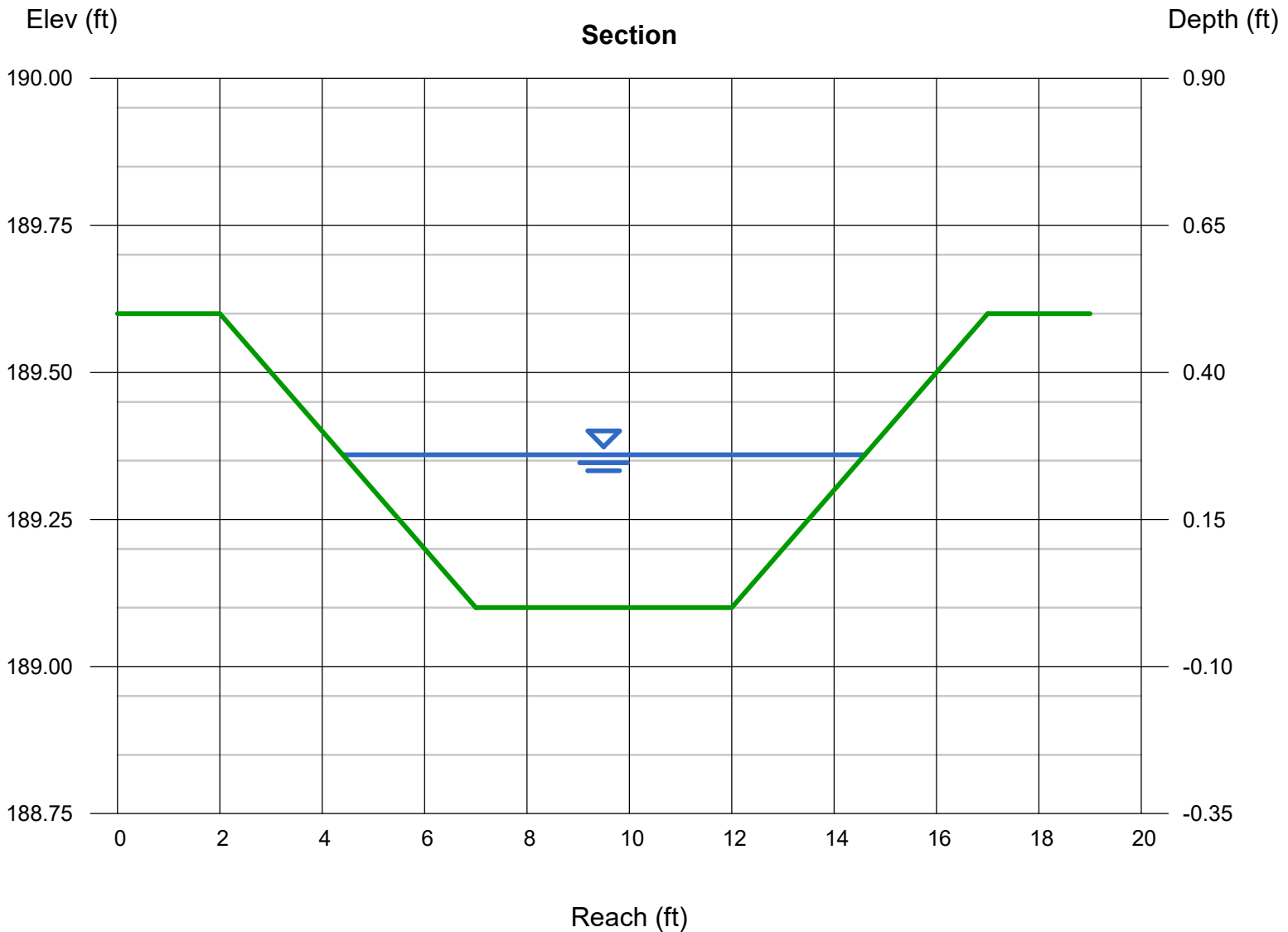
Bottom Width (ft) = 5.00
Side Slopes (z:1) = 10.00, 10.00
Total Depth (ft) = 0.50
Invert Elev (ft) = 189.10
Slope (%) = 4.90
N-Value = 0.019

Highlighted

Depth (ft) = 0.26
Q (cfs) = 10.94
Area (sqft) = 1.98
Velocity (ft/s) = 5.54
Wetted Perim (ft) = 10.23
Crit Depth, Yc (ft) = 0.41
Top Width (ft) = 10.20
EGL (ft) = 0.74

Calculations

Compute by: Known Q
Known Q (cfs) = 10.94



Project Name: Pebbly Beach Landfill Surface Water Analysis
Client: CR&R
Job No.: SO21.1196
Date: 02/02/2022
Calculated By: AV
Checked By: JMG

TABLE 6
OPEN CHANNEL SUMMARY

Element ID	Length (ft)	Inlet Invert Elevation (ft)	Outlet Invert Elevation (ft)	Average Slope (%)	Channel Type	Channel Height (ft)	Channel Width (ft)	Channel Manning's Roughness	Peak Flow (cfs)	Time of Peak Flow Occurrence (days hh:mm)	Max Flow Velocity (ft/sec)	Design Flow Capacity (cfs)	Total Time Surcharged (min)	Max Flow Depth (ft)
BENCH-DITCH-1	252.65	286.70	246.60	15.87	User-Defined	0.50	12.00	0.02	1.66	0 10:00	5.67	36.73	0.00	0.16
BENCH-DITCH-2	262.98	278.20	246.60	12.02	User-Defined	0.50	12.00	0.02	2.10	0 10:00	5.41	31.96	0.00	0.18
BENCH-DITCH-3	390.00	259.00	189.42	17.84	User-Defined	0.75	12.00	0.02	3.57	0 10:00	7.90	75.80	0.00	0.24
BENCH-DITCH-4	428.91	264.50	189.10	17.58	User-Defined	0.50	12.00	0.02	3.65	0 10:00	7.17	38.51	0.00	0.21
BENCH-DITCH-5	3.21	189.67	189.42	7.79	User-Defined	0.75	12.00	0.02	3.65	0 10:00	5.79	50.17	0.00	0.28
BENCH-DITCH-6	2.99	189.67	189.42	8.36	User-Defined	0.75	12.00	0.02	3.56	0 10:00	5.91	51.99	0.00	0.27
DOWNCHUTE-1A	107.88	245.50	189.67	51.75	Trapezoidal	0.50	3.00	0.03	3.74	0 10:01	10.39	15.27	0.00	0.24
DOWNCHUTE-1B	2.37	189.67	189.42	10.55	Trapezoidal	0.75	4.00	0.03	3.74	0 10:01	5.85	16.13	0.00	0.36
DRIVE-THROUGH-SWALE-1	20.23	189.17	187.60	7.76	Trapezoidal	1.00	25.00	0.02	10.94	0 10:01	6.72	231.88	0.00	0.22
DRIVE-THROUGH-SWALE-2A	9.69	246.60	246.00	6.19	Trapezoidal	1.00	25.00	0.03	3.75	0 10:00	3.12	122.98	0.00	0.17
DRIVE-THROUGH-SWALE-2B	2.75	246.00	245.50	18.18	Trapezoidal	1.00	25.00	0.03	3.75	0 10:00	4.51	210.73	0.00	0.13
V-DITCH-1	105.41	263.92	262.90	0.97	Triangular	1.50	4.00	0.02	3.93	0 10:00	3.85	16.42	0.00	0.88
V-DITCH-2	130.00	262.90	261.60	1.00	Triangular	1.50	4.00	0.02	3.90	0 10:01	3.89	16.69	0.00	0.86
V-DITCH-3	77.47	261.60	259.50	2.71	Triangular	1.50	4.00	0.02	9.57	0 10:00	7.04	27.48	0.00	1.01
V-DITCH-4	73.00	259.50	257.70	2.47	Triangular	1.50	4.00	0.02	17.30	0 10:00	7.89	26.21	0.00	1.28
V-DITCH-5	127.30	257.70	198.50	46.50	Triangular	1.50	4.00	0.02	17.30	0 10:00	23.69	113.82	0.00	0.74
V-DITCH-6	111.36	189.67	185.50	3.74	Triangular	1.50	4.00	0.02	22.54	0 10:00	9.84	32.30	0.00	1.30
V-DITCH-7	78.26	187.10	185.50	2.04	Triangular	1.50	4.00	0.02	10.96	0 10:01	6.55	23.87	0.00	1.10
V-DITCH-8	324.00	188.50	181.50	2.16	Triangular	1.50	4.00	0.02	1.84	0 10:01	4.33	24.53	0.00	0.57
V-DITCH-9	48.78	182.50	181.50	2.05	Triangular	1.50	4.00	0.02	0.75	0 10:00	3.36	23.90	0.00	0.41
V-DITCH-10	234.88	261.50	160.70	42.92	Triangular	1.00	4.00	0.02	5.97	0 10:01	16.84	59.93	0.00	0.41
V-DITCH-11	245.71	263.92	261.70	0.90	Triangular	1.50	4.00	0.02	4.87	0 10:01	3.96	15.87	0.00	0.96
V-DITCH-12	5.00	187.60	187.10	10.00	Triangular	1.50	4.00	0.02	10.94	0 10:01	11.87	52.78	0.00	0.82
V-DITCH-13	30.35	198.50	189.67	29.09	Triangular	1.50	4.00	0.02	19.53	0 10:00	20.48	90.03	0.00	0.84
V-DITCH-14	42.71	181.50	160.70	48.70	Triangular	1.50	4.00	0.02	2.58	0 10:01	14.99	116.48	0.00	0.35

Channel Report

Pipe 24 in

Circular

Diameter (ft) = 2.00

Invert Elev (ft) = 185.50

Slope (%) = 3.46

N-Value = 0.010

Calculations

Compute by: Known Q

Known Q (cfs) = 33.32

Highlighted

Depth (ft) = 1.13

Q (cfs) = 33.32

Area (sqft) = 1.84

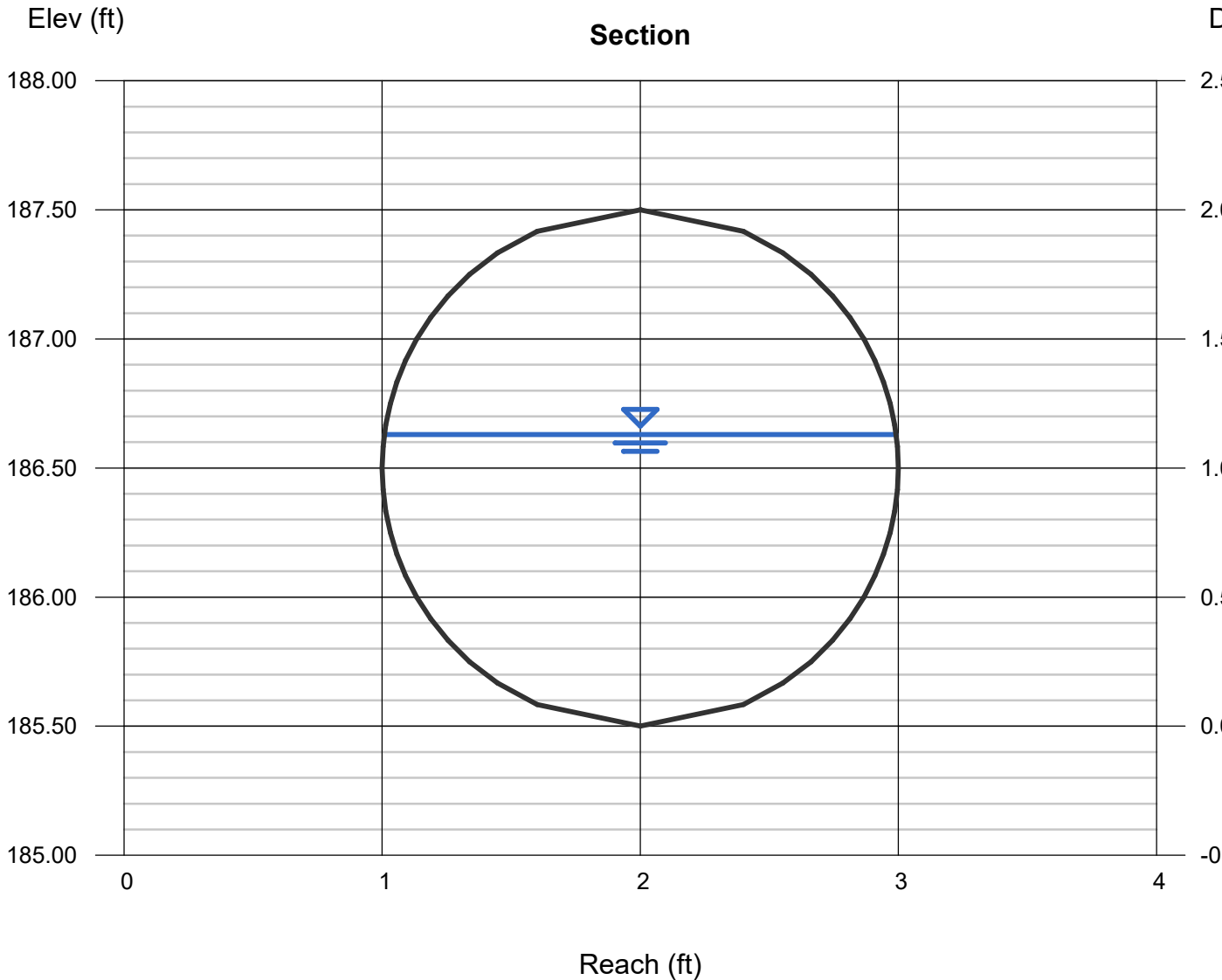
Velocity (ft/s) = 18.13

Wetted Perim (ft) = 3.41

Crit Depth, Y_c (ft) = 1.91

Top Width (ft) = 1.98

EGL (ft) = 6.24



Channel Report

V-Ditch

Triangular

Side Slopes (z:1) = 1.33, 1.33
Total Depth (ft) = 1.50

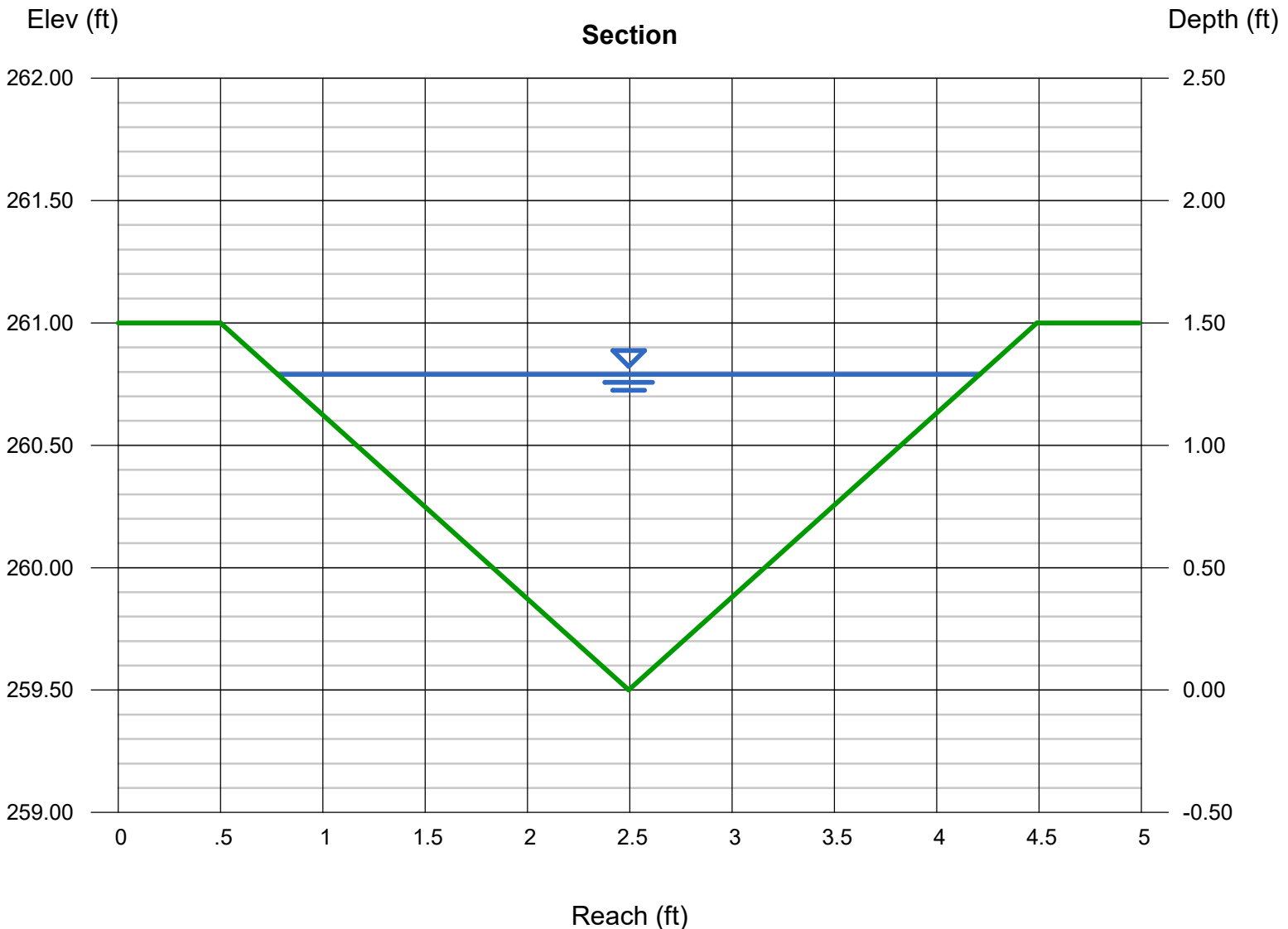
Invert Elev (ft) = 259.50
Slope (%) = 2.47
N-Value = 0.019

Calculations

Compute by: Known Q
Known Q (cfs) = 17.30

Highlighted

Depth (ft) = 1.29
Q (cfs) = 17.30
Area (sqft) = 2.21
Velocity (ft/s) = 7.82
Wetted Perim (ft) = 4.29
Crit Depth, Yc (ft) = 1.50
Top Width (ft) = 3.43
EGL (ft) = 2.24



Appendix C.7

Pipe and Culvert Design

Project Name: Pebbly Beach Landfill Surface Water Analysis
Client: CR&R
Job No.: SO21.1196
Date: 02/02/2022
Calculated By: AV
Checked By: JMG

TABLE 7
PIPE/CULVERT SUMMARY

Pipe/Culvert ID	Length (ft)	Inlet Invert Elevation (ft)	Outlet Invert Elevation (ft)	Average Slope (%)	Pipe Shape	Pipe Diameter or Height (inches)	Manning's Roughness	Peak Flow (cfs)	Time of Peak Flow Occurrence (days hh:mm)	Max Flow Velocity (ft/sec)	Design Flow Capacity (cfs)	Total Time Surcharged (min)	Max Flow Depth (ft)
CULVERT-1	45.21	261.70	261.50	0.44	CIRCULAR	18.00	0.01	4.88	0 10:01	5.24	9.08	0.00	0.76
PIPE-1A	8.67	185.50	185.20	3.46	CIRCULAR	24.00	0.01	33.32	0 10:00	18.25	54.71	0.00	1.11
PIPE-1B	0.02	185.20	168.50	83500.00	CIRCULAR	24.00	0.01	33.32	0 10:00	50.00	8498.17	0.00	0.09
PIPE-1C	1.95	168.50	168.00	25.64	CIRCULAR	24.00	0.01	33.35	0 10:01	38.23	148.92	0.00	0.63

Appendix C.8
Junction Summary

Project Name: Pebbly Beach Landfill Surface Water Analysis
Client: CR&R
Job No.: SO21.1196
Date: 02/02/2022
Calculated By: AV
Checked By: JMG

TABLE 8
JUNCTION SUMMARY

Junction Number	Invert Elevation (ft)	Ground/Rim (Max) Elevation (ft)	Initial Water Elevation (ft)	Surcharge Elevation (ft)	Peak Inflow (cfs)	Maximum HGL Elevation Attained (ft)	Maximum HGL Depth Attained (ft)	Minimum Freeboard Attained (ft)	Average HGL Elevation Attained (ft)	Average HGL Depth Attained (ft)	Time of Maximum HGL Occurrence (days hh:mm)	Time of Peak Flooding Occurrence (days hh:mm)	Total Flooded Volume (ac-inches)
1	246.60	247.60	246.60	247.60	3.75	246.78	0.18	0.82	246.65	0.05	0 10:00	0 00:00	0.00
2	278.20	279.20	278.20	279.20	2.13	278.38	0.18	0.82	278.25	0.05	0 10:00	0 00:00	0.00
3	189.17	190.17	189.42	190.17	10.93	189.79	0.62	0.38	189.50	0.33	0 10:01	0 00:00	0.00
4	286.70	287.70	286.70	287.70	1.68	286.86	0.16	0.84	286.74	0.04	0 10:00	0 00:00	0.00
5	259.00	260.00	259.00	260.00	3.62	259.24	0.24	0.76	259.06	0.06	0 10:00	0 00:00	0.00
6	264.50	265.50	264.50	265.50	3.72	264.71	0.21	0.79	264.56	0.06	0 10:00	0 00:00	0.00
7	261.50	263.00	261.50	263.00	5.97	262.28	0.78	0.72	261.64	0.14	0 10:01	0 00:00	0.00
8	261.70	263.20	261.20	263.20	4.87	262.66	0.96	0.54	261.96	0.26	0 10:01	0 00:00	0.00
9	263.92	265.42	263.92	265.42	4.96	264.89	0.97	0.53	264.18	0.26	0 10:00	0 00:00	0.00
10	262.90	264.40	262.90	264.40	3.93	263.78	0.88	0.62	263.14	0.24	0 10:00	0 00:00	0.00
11	261.60	263.10	261.60	263.10	9.54	262.61	1.01	0.49	261.88	0.28	0 10:00	0 00:00	0.00
12	259.50	261.00	259.50	261.00	17.26	260.78	1.28	0.22	259.85	0.35	0 10:00	0 00:00	0.00
13	189.67	191.17	189.67	191.17	22.54	190.98	1.31	0.19	190.03	0.36	0 10:00	0 00:00	0.00
14	185.50	187.00	185.50	187.00	33.33	186.81	1.31	0.69	185.86	0.36	0 10:00	0 00:00	0.00
15	187.60	188.60	187.60	188.60	10.94	188.43	0.83	0.67	187.83	0.23	0 10:01	0 00:00	0.00
16	188.50	190.00	188.50	190.00	1.89	189.07	0.57	0.93	188.65	0.15	0 10:00	0 00:00	0.00
17	185.20	187.20	185.20	187.20	33.32	186.33	1.13	0.87	185.40	0.20	0 10:00	0 00:00	0.00
18	168.50	186.70	168.50	186.70	33.32	169.14	0.64	17.56	168.63	0.13	0 10:00	0 00:00	0.00
19	168.00	170.00	168.00	170.00	33.35	170.00	2.00	0.00	169.75	1.75	0 03:00	0 00:00	0.00
20	182.50	184.00	182.50	184.00	0.75	182.91	0.41	1.09	182.61	0.11	0 10:00	0 00:00	0.00
21	181.50	183.00	181.50	183.00	2.58	182.07	0.57	0.93	181.65	0.15	0 10:01	0 00:00	0.00
22	160.70	162.20	160.70	162.20	8.55	162.20	1.50	0.00	162.01	1.31	0 03:01	0 00:00	0.00
23	263.92	265.42	263.92	265.42	3.96	264.80	0.88	0.62	264.16	0.24	0 10:00	0 00:00	0.00
24	198.50	200.00	198.50	200.00	19.53	199.35	0.85	0.65	198.73	0.23	0 10:00	0 00:00	0.00
25	257.70	259.20	257.70	259.20	17.30	258.98	1.28	0.22	258.05	0.35	0 10:00	0 00:00	0.00
26	187.10	188.60	187.10	188.60	10.94	188.22	1.12	0.38	187.40	0.30	0 10:01	0 00:00	0.00
27	246.00	247.00	246.00	247.00	3.75	246.18	0.18	0.82	246.03	0.03	0 10:00	0 00:00	0.00
28	245.50	246.00	245.50	246.00	3.75	245.74	0.24	0.76	245.54	0.04	0 10:00	0 00:00	0.00
29	189.67	190.17	189.67	190.17	3.57	189.94	0.27	0.48	189.74	0.07	0 10:00	0 00:00	0.00
30	189.67	190.17	189.67	190.17	3.74	190.04	0.37	0.38	189.73	0.06	0 10:01	0 00:00	0.00
31	189.67	190.17	189.67	190.17	3.65	189.95	0.28	0.47	189.75	0.08	0 10:00	0 00:00	0.00

Appendix C.9
Rip Rap Design

Project Name: Pebbly Beach Landfill Surface Water Analysis
Client: CR&R
Job No.: SO21.1196
Date: 02/02/2022
Calculated By: AV
Checked By: JMG

RIP-RAP APRON SIZING ¹								
Channel Section	Culvert Size/ Channel Width (inches)	Discharge (cfs)	Median Stone Diameter (inches)	Max Stone Diameter (inches)	Required Rip-Rap Thickness (inches)	Minimum Length of Apron (feet)	Minimum Width at Outlet (feet)	Minimum Width at Apron End (feet)
V-Ditch 10 Outlet at Juntion 22	48.0	59.9	9.0	13.5	20.3	20.0	12.0	24.0
Pipe 1C Outlet at Juntion 19	24.0	90.0	9.0	13.5	20.3	20.0	6.0	22.0

1) Rip-rap apron sized using Figure 7.45 or Figure 7.46 in the "Erosion and Sediment Control Handbook".