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HWD Manke CAP

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Memorandum

To: Russ McMillan (Washington Department of Ecology)

From: Clay Patmont and Greg Guannel

CC: John McBride (Manke Lumber Company)

Rebecca Desrosiers and Kimberly Magruder (Anchor)

Rob Gilmour (MCS Environmental)

Date: November 10, 2004

Re: Manke Under-Pier Cap Design Analysis

The purpose of this memo is to summarize the basis of remedial design for capping under the Manke pier, and to present the methods and material specifications to be used to construct the cap. All remedial design and construction activities would be performed under the terms of Consent Decree No. 01 2 04714 6, filed Jan. 17, 2001.

1 BACKGROUND

1.1 Site History

The Manke Lumber Company, Inc. (Manke), site is located at the head of the Hylebos Waterway in Commencement Bay. As part of the Hylebos Waterway Wood Debris Group, Manke entered into a Model Toxics Control Act (MTCA) Consent Decree (No. 01 2 04714 6) with the Washington State Department of Ecology (Ecology) to remediate certain contaminated sediments present in the head of the Hylebos Waterway. The primary goal of the MTCA cleanup action is to reduce wood debris and associated total volatile solids (TVS) content in sediments of the Hylebos Waterway Upper Turning Basin. Secondary contaminants of potential concern in this area, including beneath the Manke pier, also include arsenic and zinc.

1.2 Recent Remediation Work at the Site

Following Puget Sound Dredged Material Management Program (DMMP) guidelines, the Manke site was previously divided into 27 discrete dredged material management units

(DMMUs) and characterized accordingly for implementation of remedial design (originally performed by Floyd and Snider 2000). During late 2003 through early 2004, in accordance with the remedial action work plan approved by Ecology, Manke removed and reused approximately 13,300 cubic yards (cy) of logs and large woody debris from the site. Also during this period, approximately 19,900 cy of suitable sediment was disposed of at the Commencement Bay open-water site, and approximately 9,900 cy of unsuitable sediment was disposed of at the Blair Slip 1 nearshore confined disposal (NCD) facility. These remedial activities successfully accomplished sediment cleanup in open water areas near the face of the Manke pier, along with other areas of the site. Final sediment cleanup activities at the site are ongoing.

Beginning in late July 2004, and in accordance with the remedial action work plan approved by Ecology, Manke removed approximately 2 to 3 feet (ft) of arsenic- and zinc-contaminated sediments from beneath the Manke dock (between pile bents) using drag line methods. The materials are currently stockpiled underwater at the face of the Manke pier, awaiting final confirmation from Ecology that the under-pier removal action is complete. The under-pier material will then be dredged and disposed of at an approved upland disposal facility (e.g., the Slip 1 NCD or Subtitle D landfill).

Following initial indications in August 2004 that target dredge depths had been achieved beneath the dock, MCS Environmental performed dredge compliance monitoring under the dock, collecting surface sediments from five locations using diver cores (Figure 1). The five sampling stations were presented in the Letter of Addendum for the Supplemental Compliance Monitoring Plan, dated November 20, 2003 (Gilmour, 2003). Measured offsets from the end and the face of the pier were used to locate the sampling locations in the field. The actual sampling locations were moved near the middle of the bent (i.e., the space between piling rows) closest to the proposed location and approximately 30 ft shoreward of the dock face under the pier. The sediment coring apparatus consisted of a stainless steel "cookie cutter" style sampler that was worked into the bottom sediments by the divers using a rotating motion. The sampling device was 8-inch diameter with a sliding plate designed to retain the top 10 cm of sediment. The sampling device was decontaminated prior to use using a double Alconox wash with a distilled water final rinse.

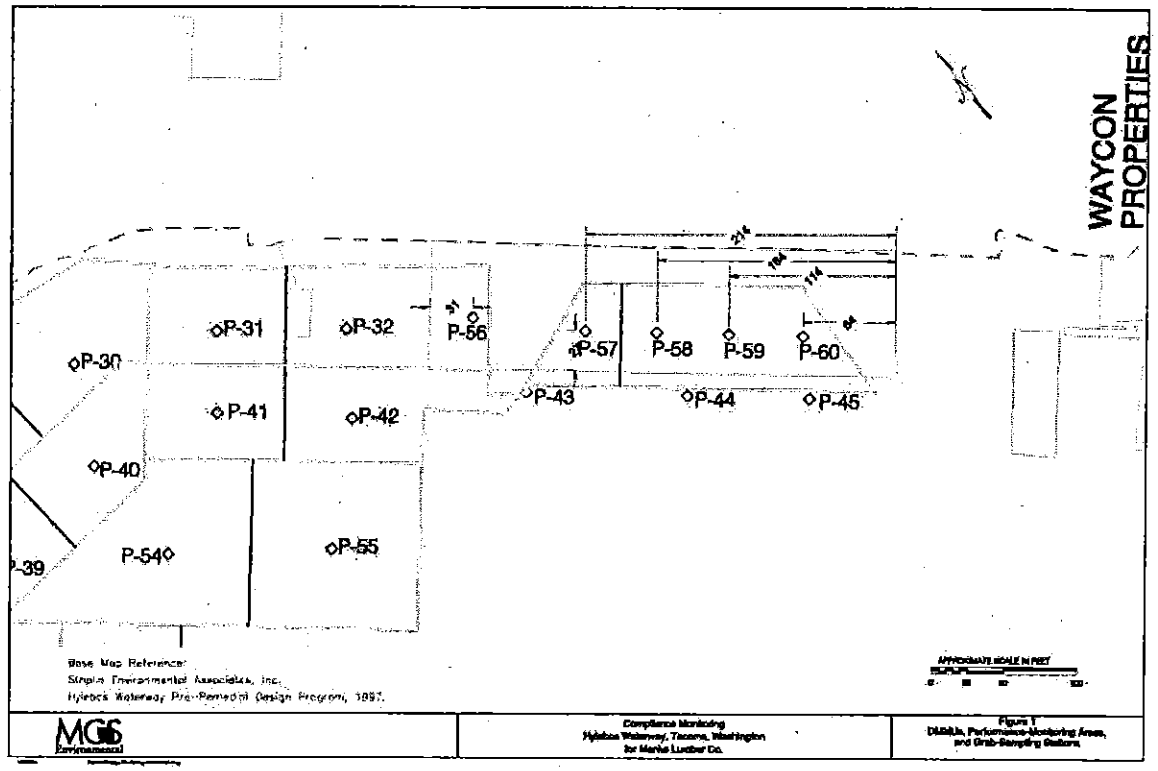

Figure 1: Under-pier Dredge Compliance Monitoring Core Locations (P-56 to P-60)

The initial round of under-dock sampling was conducted on August 12, 2004. The samples were homogenized and submitted to Analytical Resources, Inc., in Tukwila, Washington for analysis of arsenic and zinc. The results of the first round of sampling are presented in Table 1. The analytical results were compared to cleanup standards for these chemicals set forth in the Consent Decree (Sediment Quality Objective [SQO] chemical criteria). Arsenic and zinc at station P-56 were below SQOs. However, surface sediments at stations P-57, P-58, P-59, and P-60 exceeded SQOs, prompting Manke to conduct further sediment removal operations in the under-pier area.

A second round of core samples were collected at stations P-57, P-58, P-59, and P-60 following additional dredging under the pier, using the same methods described above. The results of the second round of sampling are also presented in Table 1. All of the sampled locations still had concentrations of arsenic and zinc that exceeded SQOs. While results for stations P-58, P-59, and P-60 were similar to the results from the first round, the sample collected at station P-57 (sample P-57R2) had substantially elevated levels of both arsenic and zinc. To determine if the presence of relatively coarse slag materials might have biased the results, the remaining sample material was sieved on a 2 millimeters mesh screen to remove larger pieces of sand blast aggregate, and the sample was re-homogenized and reanalyzed. The results of the reanalysis were similar to the initial results for Sample P-57R2.

Table 1
Results of Under-pier Sampling at the Manke Dock

Round 1 August 12, 2004							
Analyte	SQO (ppm)	Station Sample ID	P-56 P-56	P-57 P-57	P-58 P-58	P-59 P-59	P-60 P-60
Arsenic	57		30	80	150	110	190
Zinc	410		276	418	638	305	612
Round 2 September 16, 2004							
Analyte	SQO (ppm)	Station Sample ID	P-56 P-56	P-57 P-57R2	P-58 P-58R2	P-59 P-59R2	P-60 P-60R2
Arsenic	57		NS	950	140	140	155
Zinc	410		NS	2920	474	335	599
Round 2 Reanalysis							
Analyte	SQO (ppm)	Station Sample ID		P-57 P-57R2			
Arsenic	57			1060			
Zinc	410			3130			

NS: Not sampled

The dredge compliance monitoring data summarized above suggest that the dredging action appears to have achieved target dredge depths and also appears to have removed, to the extent practical, all soft sediment beneath the dock. However, the data also suggests that sediment residuals nevertheless remain on the post-dredge surface. The occurrence of sediment residuals in this setting was anticipated, owing to the difficulty in removing all contaminated sediments beneath the pier and near the numerous pilings that are in this area.

On October 27, 2004, Anchor Environmental conducted a lead line survey of the underpier area to evaluate post-dredging bathymetry at the site. During that survey effort, bed elevations at every other bent of the pier were measured. Results of this survey are presented in Figure 2. Appendix A presents different site cross-sections and comparisons between previous and current bathymetry. The data demonstrate that the current underpier slope is relatively uniform and consistent, averaging 2.4H:1V (horizontal:vertical). Also, it appeared that soft sediment are present up to approximately elevation -3 ft mean lower low water (MLLW); the remaining upper portion of the slope is armored with rocks varying in size from 0.5 to approximately 1.5 ft.

When compared with site bathymetric data available immediately prior to under-pier dredging, the recent lead line surveys suggest that approximately 2,300 cy of under-pier sediments are currently stockpiled underwater at the face of the Manke pier, awaiting final confirmation from Ecology that the under-pier removal action is complete. As discussed above, the under-pier sediments will be dredged and disposed at an approved upland disposal facility (e.g., the Slip 1 NCD or Subtitle D landfill).

2 UNDER-PIER SEDIMENT RESIDUALS CAPPING APPROACH

Following discussion with Ecology, and consistent with embankment cap designs developed for other sediment cleanup areas of the Hylebos Waterway under Superfund, a nominal 2-ft-thick cap/backfill of the under-pier dredge area containing sediment residuals is proposed for the Manke site. This cap thickness has been demonstrated to provide suitable isolation of contaminants from biological activity at the cap surface for a range of sediment contaminants, including arsenic and zinc. This section presents and analyzes different elements that need to be taken into account to ensure the long-term stability and performance of the cap, including the appropriate cap material gradation. Cap structural stability and constructability considerations are also addressed.

2.1 Cap Composition Determination

A remedial cap needs to effectively contain and isolate contaminated sediments and resist erosive forces. In order to determine appropriate cap thickness and sediment grain size, design analyses were conducted following United States Environmental Protection Agency (EPA) and United States Army Corps of Engineers cap design guidance documents (Palermo et al. 1998a and 1998b). The design analysis considered:

- Bioturbation
- Contaminant transport
- Erosion
- Consolidation

Each of these design components is discussed below, and an appropriate cap thickness and gradation to address these cumulative performance requirements is developed.

2.1.1 Chemical Isolation

As discussed above, a general performance specification for sediment caps within the Hylebos Waterway is that they should be at least two feet thick, with a minimum of two feet of fine grain materials to provide for effective chemical isolation.

2.1.2 Bioturbation

Marine and fresh water benthic organisms can modify sediment stability, erodibility, vertical distribution of dissolved and particulate matter, and transport of materials within the sediment and across the sediment water interface (Bosworth and Thibodeaux 1990). The majority of benthic organisms in Commencement Bay waterways are shallow burrowing infauna. Shallow burrowing infauna live within the top 10 cm (0.3 ft) of the sediment column and are dominated by amphipods, tubiculid and soft-bodied polychaete worms, and clams.

A deep burrowing species of thalassinid shrimp, commonly referred to as "ghost" shrimp (*Neotrypaea* [formerly *Callianassa*] spp.) is also present in the waterways. *Neotrypaea* spp. are tube builders that can burrow to depths of 75 cm (2.5 ft) into mud and fine sand sediments as they constantly rework and sift the sediments in search of food. However, ghost shrimp do not burrow in coarser sand or gravel, and a layer of 0.25 to 0.5 ft of gravel is sufficient to inhibit burrowing.

2.1.3 Erosion

There are a number of factors that might induce erosion of the cap, including wind-waves, vessel generated waves and currents, and tidal currents. This section examines the relative importance of all erosive forces at the site, and recommends a sediment size for the upper portion of the cap that can resist these forces.

2.1.3.1 Wind Waves

Based on the site configuration and location, most of the wind-waves that are generated in Commencement Bay are not likely to reach the site because of the constriction at the head of the waterway that acts as a filter; longer period waves and tidal waves will, however, penetrate the waterway. Furthermore, because bathymetric contours inside the waterway are parallel to the shore, and considering the limited width of the channel (less than a quarter of a mile), wind waves are not likely to form near the site. Hence, wind-waves are not a primary concern in this erosion analysis.

2.1.3.2 Vessel Wakes

Based on observations within the Hylebos Waterway, tugs, barges and other types of vessels regularly travel on the waterway, near the site. The wakes generated by the passage of these vessels create a force that can affect cap stability. Based on vessel studies by Sorensen (1993), maximum vessel-generated wave heights (measured near the vessel) generally range from 0.7 to 3.0 ft. PIE (2001) presented results of two national studies that looked in general at vessel wake height and period characteristics (Tables 2 and 3). Based on this data, it is likely that less than one percent of vessel wakes (including wakes generated by deep-draft vessels) are larger than 2 ft and less than six percent of the wakes are higher than approximately 1 ft. Consequently, in order to provide a conservative erosion protection design, a design wave height of 1.5 ft was considered in this analysis. A similarly conservative wave period of 7 seconds was also applied to that assumed wave height.

Table 2
Port Aransas Wake Data

Wake Height [ft]	Number of Occurrences	Percentage of Total	Origin
0.0-0.3	163	79.9	Deep-draft vessels
0.3-0.7	34	16.7	
0.7-1.0	7	3.4	
1.0-1.3	0	0.0	
> 1.3	0	0.0	
Subtotal	***204***		
0.0-0.3	226	55.4	Small/pleasure craft vessels
0.3-0.7	158	38.7	
0.7-1.0	13	3.2	
1.0-1.3	6	1.5	
> 1.3	5	1.2	
Subtotal	***408***		
0.0-0.3	389	63.6	All Vessels
0.3-0.7	192	31.4	
0.7-1.0	20	3.3	
1.0-1.3	6	1.0	
> 1.3	5	0.8	
Subtotal	***612***		

Source: PIE 2001

Table 3
Port of Oakland Wake Data

Wake Height [ft]	Number of Occurrences	Wave Period	Possible Wave Origin
< 0.25	2870	95% less than 3 sec, 92%less than 2 sec., and 5% larger than 3 sec.	Statistical noise, wind-wave, slow ferries, small crafts.
0.25-0.5	349		
Subtotal	***3219***		
0.5-0.75	198	2% less than 3 sec., 0% less than 2 sec., and 98% larger than 3 sec.	Ferries, small crafts, tugs, deep draft vessels.
0.75-1.0	309		
Subtotal	***507***		
1.0-1.25	151	0% less than 3 sec., 25% larger than 8sec., and 4% larger than 25 sec.	Fast moving ferries, pressure field form deep draft vessels with a period larger than 25 sec, statistical noise.
1.25-1.5	44		
1.5-1.75	24		
1.75-2.0	6		
Subtotal	***225***		
2.0-2.25	2	100% larger than 10 sec.	Pressure field from fast moving deep draft vessels.
2.25-2.5	2		
2.5-2.75	2		
2.75-3.0	0		
> 3.0	2		
Subtotal	***8***		
TOTAL	**3959**		

Source: PIE 2001

Vessel wakes lose some of their energy as they travel underneath the pier, largely because of the presence of the piles. However, in order to provide a conservative design, the effect of wake dissipation through the piles was not considered in this design analysis.

The design wave was shoaled from the middle of the navigation channel to the elevations of the capping areas, using the Linear Wave Theory/Snell's module of the ACES model (CERC 1984, 1992). The wave orientation was assumed to initially travel at a typical 45 degrees to the shoreline. Wave heights were computed at different water depths from elevation -30 ft MLLW to breaking depth. Results of this analysis are presented in Table 4. Based on this analysis, design waves will break in approximately 2.9 ft of water. Any smaller wave will break in shallower water depth.

Table 4
Design Wave Height at Different Water Depths

Water Depth [ft]	Wave Height [ft]
35	1.5
25	1.48
20	1.47
15	1.49
10	1.55
8	1.6
5	1.74
4	1.82
3	1.93

*Note: Design wave breaks in 2.92 ft of water or less with height higher than 3 ft

2.1.3.3 Currents

The main currents at the site are associated with tidal action. Different measurements of tidal currents are available for the general site area (Boateng 1995, Dames & Moore 1981, Norton and Barnard 1992). Based on these records, velocities within the head of the waterway can periodically reach speeds of up to approximately 40 cm/s (1.31 ft/s). Therefore, in order to provide a conservative erosion protection design, a design tidal current velocity of 1.5 ft/s was used in this analysis. In reality, as water depth decreases along the bank, bottom friction will reduce the tidal current velocity.

2.1.3.4 Propeller Wash

Propeller jets create currents that can sometimes resuspend bed sediments. Because the site is primarily used to moor barges, tugs boats are generally oriented along the pier when they operate. Consequently, propeller wash is unlikely to reach the under-pier capping area and will not be taken into account in this analysis.

2.1.3.5 Cumulative Erosional Forces

As summarized in the discussion above, the main forces that affect the potential under-pier capping area are vessel wakes and tidal currents. The wave and current interact to generate a shear stress that is different from that generated by the sum of the two components (WES, 1998). In this report, the Christoffersen and Jonsson

(1985) model was used to compute shear stress generated by waves and currents because of its simplicity, and also because shear stress predicted by this model compare well with experimental data (WES, 1998).

Because higher waves create more shear stress for a given wave period, and because of the presence of armoring in the nearshore region, stable sediment size for the cap's erosion layer was computed using wave height in 3 ft of water, before it breaks. Shear stress caused by the combination of that wave height and tidal current, as well as stable sediment size were computed using the Shield's diagram for initiation of bed material movement (Shield, 1936).

For sand and gravels, movement generally begins to occur when the dimensionless shear stress is greater than 0.047, hence stable sediment size was computed based on that assumption. The combination of current and waves at the site led to a design cap medium sediment size (D_{50}) of 6.1 mm, or 0.2 inches, which corresponds to coarse sand to fine gravel. This sediment size is predicted to be stable from -3 ft MLLW and deeper. This also corresponds generally to the upper elevation of under-pier dredging, and thus is the upper elevation limit of the proposed under-pier sediment residuals cap. As mentioned earlier, substrate above this depth is generally composed of coarser rip-rap materials, and is not targeted for capping.

2.1.4 Consolidation of the Cap and Cap Subgrade

As described above, the post-dredge sediment residual cap material will consist of 2 ft of well-graded fine gravel extending from elevation -30 ft to -3 ft MLLW. Based on observations of other similar caps constructed in the Puget Sound region, consolidation of the cap material is expected to occur rapidly following placement. The thickness of the cap will be monitored and corrected during placement to ensure adequate thickness is provided. The material underneath the cap is largely comprised of relatively dense native sands, which are not expected to consolidate significantly under the weight of the cap.

2.1.5 Recommendation

The erosion analysis indicated that coarse sand to fine gravel would be necessary to ensure that the cap will resist erosion forces at the site, given conservative design assumptions. The cap material specification will also control potential future bioturbation by deep burrowers such as ghost shrimp. We recommend that the cap be composed of a minimum of 2 ft of well-graded sandy gravel from elevation -30 ft up to -3 ft, MLLW. The sandy gravel should have a medium grain size of approximately 0.2 inches, and at least 30 percent of medium to fine sand to provide for effective long-term contaminant isolation. This material would also serve as an erosion protection layer.

An acceptable sandy gravel material gradation would follow the following requirements:

Sieve Size	Percent Passing
4″	90 – 100
0.75″	50 – 90
U.S. No. 4	35 – 65
U.S. No. 10	15 – 45
U.S. No. 40	2 – 10
U.S. No. 200	0 – 2

2.2 Cap Stability/Constructability

Placing sand and gravel underwater has been done many times in the Puget Sound region, using a range of hydraulic and mechanical methods. Because of the difficulty associated with under-pier placement, a hydraulic or conveyor placement system is recommended, with the selection of the specific method to be based on contractor preferences. Both hydraulic and conveyor systems can easily handle material size up to 2 inches, consistent with the cap specification requirements in this application.

During construction, sandy gravel will be placed on a slope inclined at angles ranging from 2.3H:1V to 2.5H:1V. This approach is consistent with, and more protective than, similar marine slope treatments commonly designed and constructed in the Puget Sound area. For example, in the Ports of Seattle and Tacoma, typical marine slopes are designed using fine sand and gravel materials with inclinations of 2H:1V, steeper than the slopes planned for this site.

The ability of a granular material to stand on an inclined slope is defined by the material internal friction angle. For the type of material currently proposed for this project (fine gravel), a typical internal friction angle is on the order of 35 to 45 degrees. A slope inclined at 2.5H:1V has an inclination angle of 22 degrees, which is well below the 35 to 45 degree internal friction angle of gravels and gravelly sands, indicating that the placed material will be stable.

3 CAP CONSTRUCTION

Construction of the slope coverage should proceed from the base of the slope working upward; material should not be dumped down the slope, because it would tend to "run" and assume a flatter angle of repose as it moves downward and entrains water. In order to provide more stability to the cap, a key trench should initially be excavated at the base of the slope and filled with gravel. This trench, which should be approximately 3 feet deep, with 1H:1V side slopes and a 5 ft wide base, would serve as a firm, stable base for the layers of gravel constructed on the slope. A typical cap cross-section is presented on Figure 3. Alternatively, the contractor could build a three feet high berm at the base of the slope.

During construction, the contractor should build the trench first, then fill it with cobbles (3 to 4 inches), or build the berm first. Placement of one 1 ft of fine gravel should ensue, from bottom up, all the way to the existing rip-rap wall. After that step, the contractor should proceed with placement of the second layer of sandy gravel, from the trench up to elevation -3 ft MLLW.

Cap construction will require an estimated 2,100 cy of sandy gravel and approximately 290 cy of cobbles.

4 LONG-TERM MONITORING

Long-term cap integrity will be verified by measuring the cap surface thickness at various locations within the cap area. Measurements will be performed approximately 1, 2, and 5 years following completion of the cap constuction, and then evaluated based on the findings.

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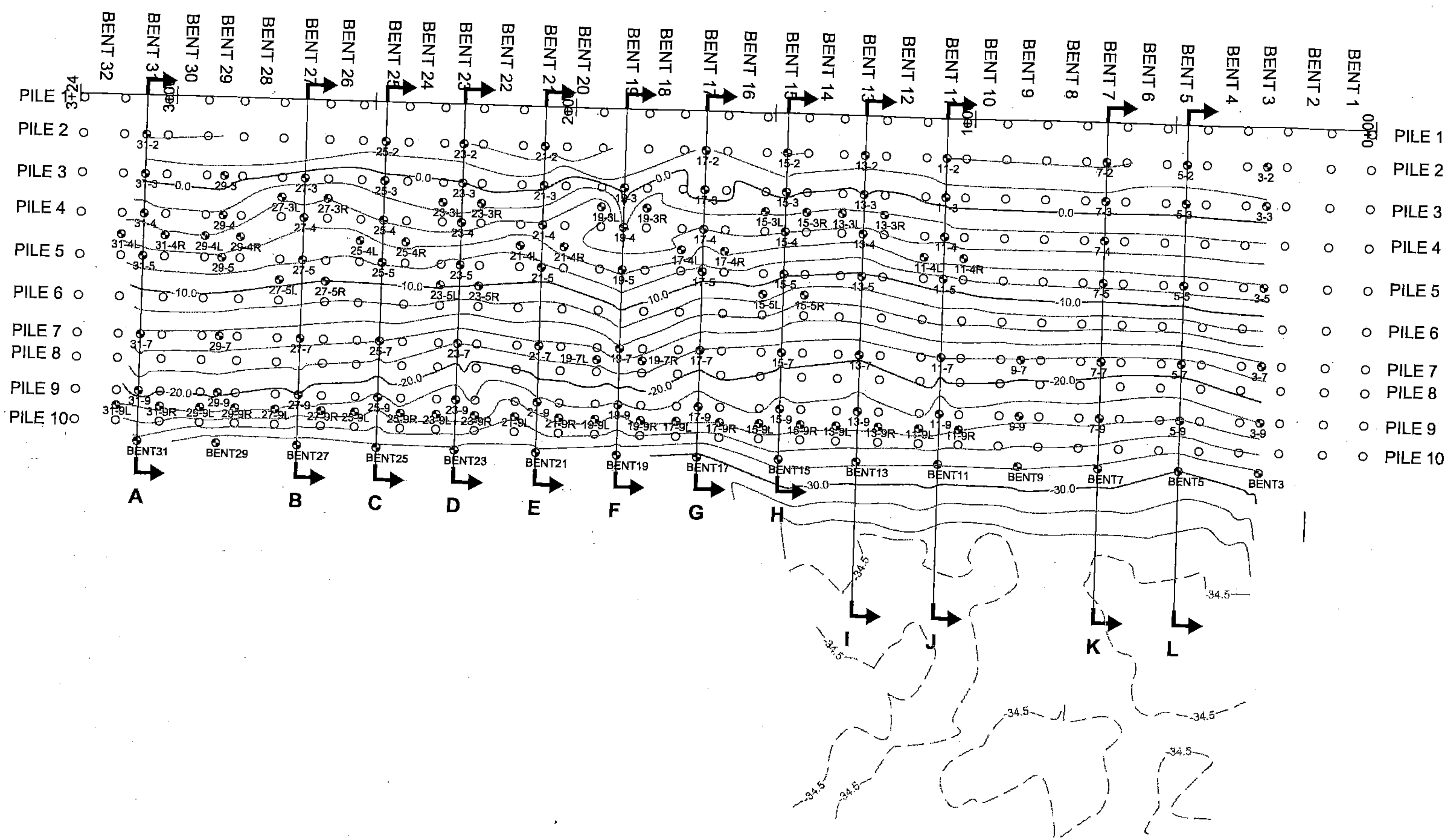

Note: Cross Sections shown in Appendix A.

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Figure 2
Under-Pier Bathymetry
Manke Lumber
Hylebos Waterway Wood Waste Removal

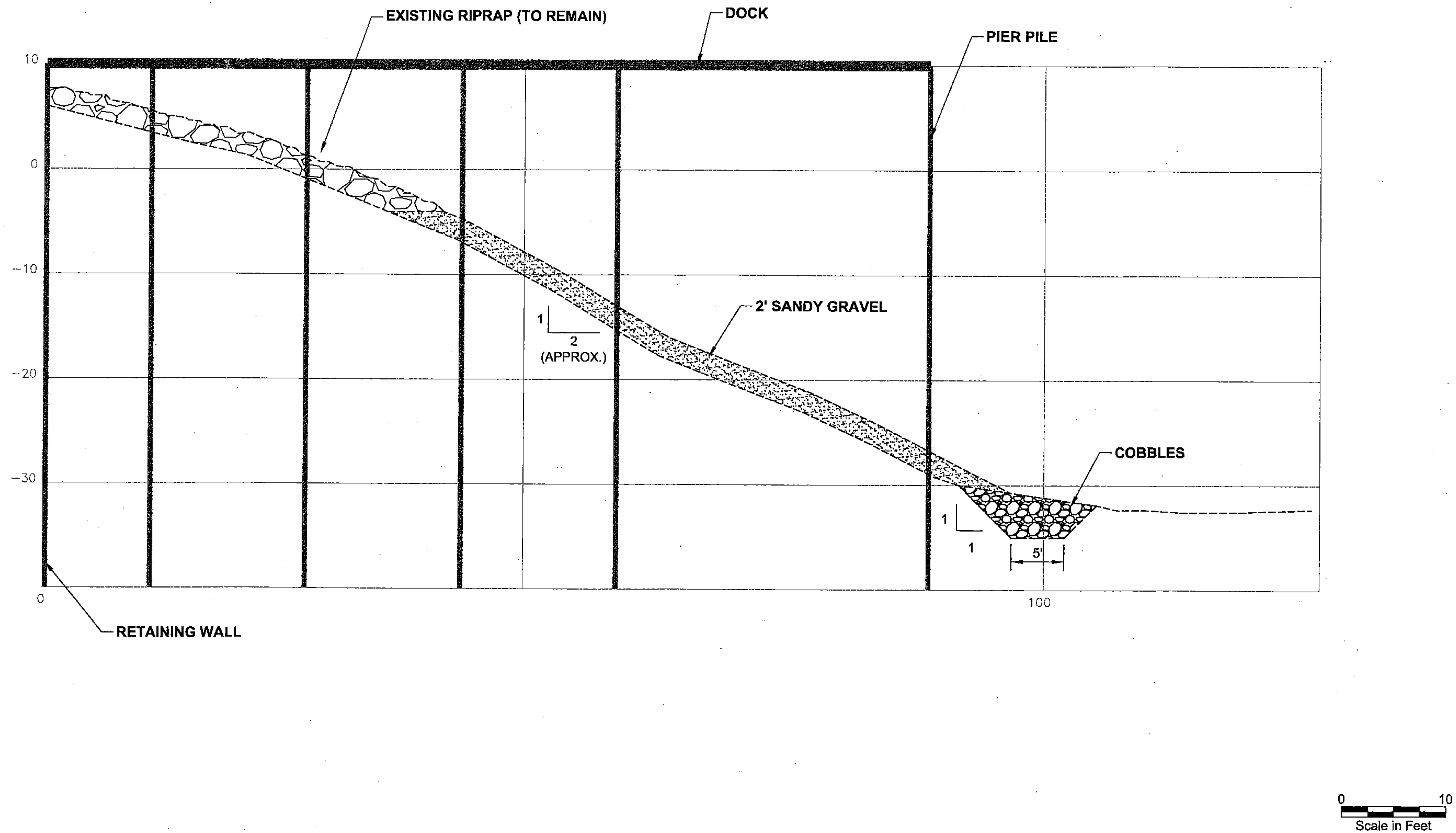

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Figure 3
Typical Cap Cross Section
Manke Lumber
Hylebos Waterway Wood Waste Removal

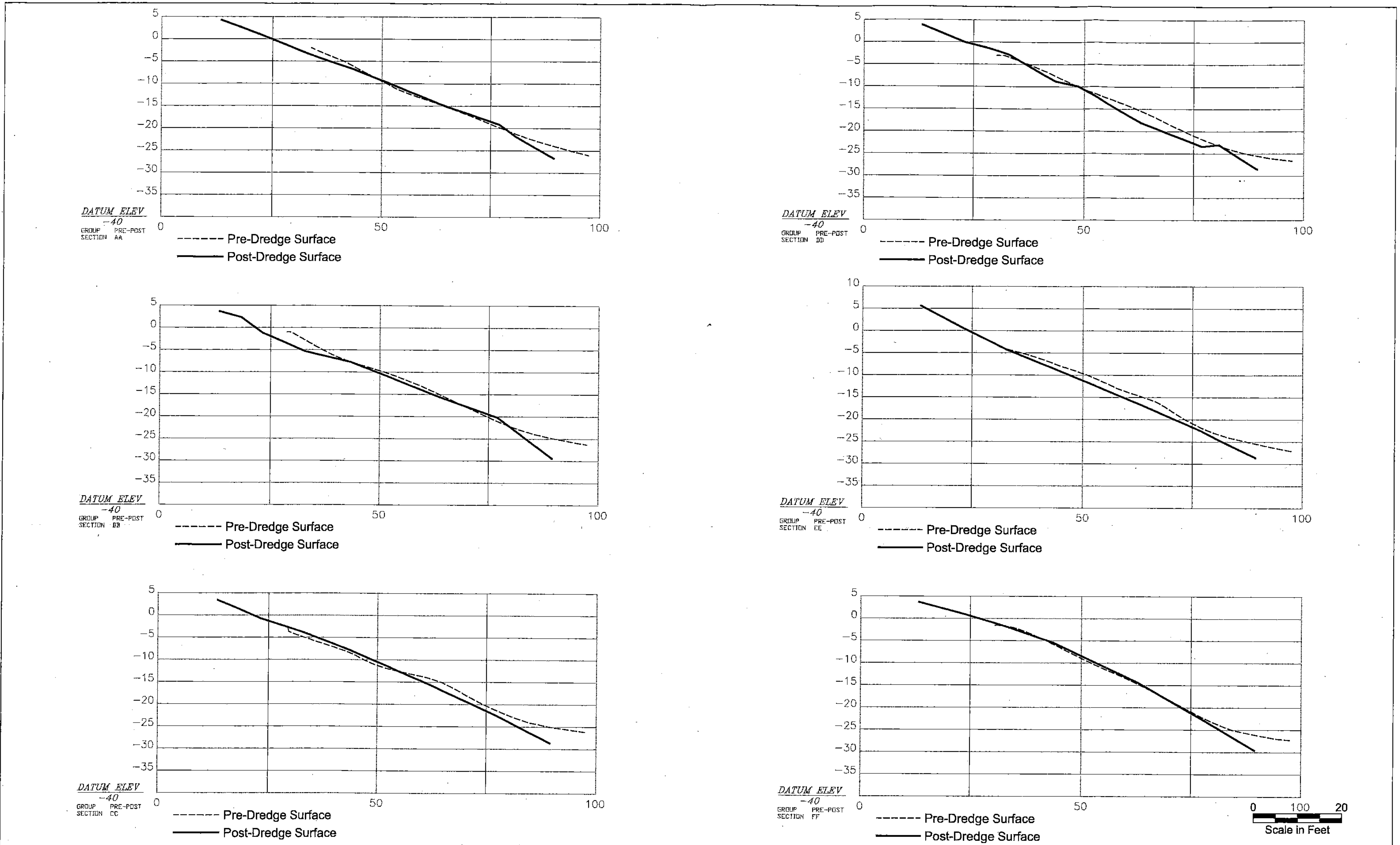

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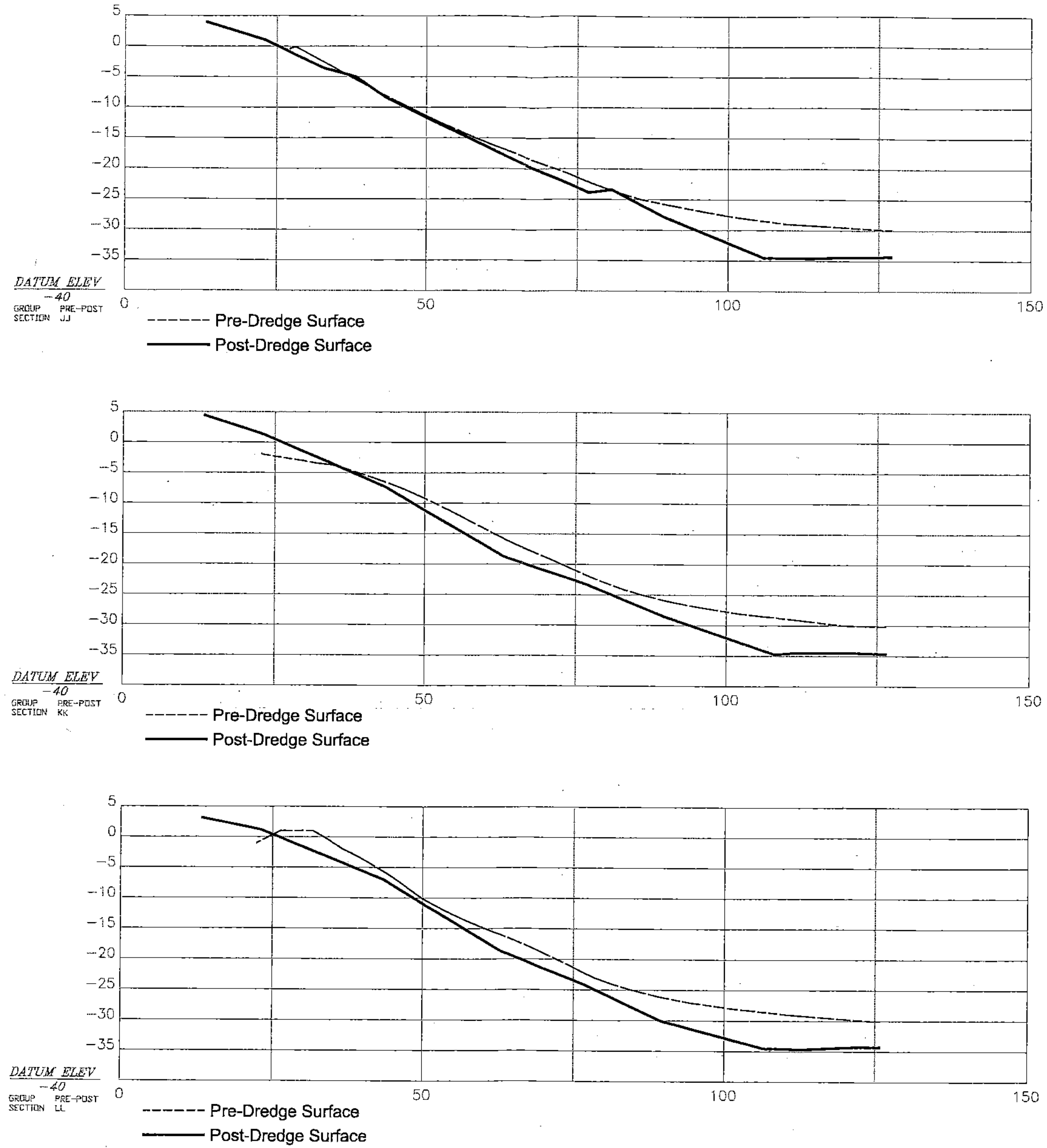
Figure A-1
Cross Sections A through F
Manke Lumber
Hylebos Waterway Wood Waste Removal



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Scale in Feet

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Figure A-3
Cross Sections J through L
Manke Lumber
Hylebos Waterway Wood Waste Removal