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northwest hydraulic consultants

MEMORANDUM

TO:	Andrew Gower, P.Eng. Wedler Engineering	**DATE:**	January 5, 2015
FROM:	Dave Mclean (NHC)	**NO. PAGES:**	4
CC:		**PROJECT NO.:**	3000276
		REF. NO.:	
RE:	**Lazo Road Erosion Bank Protection** **NHC Comments on Preliminary Design**		

1 INTRODUCTION

Wedler Engineering (Wedler) is designing shoreline protection for the City of Comox along a 700 m section of Lazo Road. Wedler retained NHC to conduct a coastal engineering study to assess the wave climate at the site and to provide comments on the draft design concept. NHC submitted a draft report on the wave climate on November 17, 2014. On December 5, the project's consulting team[1] met with the client to discuss a preliminary design concept. After this meeting, Wedler provided updated plans of the design concept on drawings V15-0196/A (Sheets 00 to 14) on December 29, 2014. The plans include a note indicating the drawings are for preliminary planning purposes only. The comments below are also intended for the preliminary planning stage of the design and do not constitute a check of the final design and technical specifications for the works.

2 REVIEW OF DECEMBER 10, 2014 STORM CONDITIONS

Southern Vancouver Island experienced a severe storm on December 9 to 10th, 2014 due to the passage of a low pressure system that generated strong southeast winds over the Strait of Georgia. The storm coincided with a period of "king tides". Previous experience has shown that the combination of strong southeast winds and high tides produces the most severe conditions for shoreline erosion on the east coast of Vancouver Island. Wave conditions at Lazo Road on December 10th were reviewed using video photos provided by Tim O'Brien. Preliminary wind and wave data were compiled from the Fisheries & Oceans buoy at Sentry Shoal. The observed ocean level during the storm was compiled from the Point Atkinson tide gauge. The maximum reported significant wave height was 2.7 m, with the highest waves occurring between 7AM and 10AM on December 10th. The return period of the waves was estimated to be 2 years. The highest ocean level reached 2.4 m (geodetic datum) at 9AM and remained above 2.0 m between 7AM and 11AM. In comparison, the design still water ocean level was 2.8 m (not accounting

[1] Andrew Gower of Wedler, Darryl Furey of Levelton Consultants, Warren Fleenor of Current Environmental and Dave McLean of NHC

for sea level rise). The joint probability of the storm event (combination of peak waves coinciding with high tide) is expected to be in the range of 0.25 to 0.2 (4 to 5 years return period).

The height of the wave runup (2% exceedence) was estimated to be 4.0 m for the December storm, causing significant overtopping along the northern section of the road between Station 1+580 to 1+700. The video imagery from the December storm showed spray and overtopping did occur along this section, which confirms the analysis. During more severe storms, general overtopping will occur in this section. For the 200 year storm condition, the maximum wave runup (2% exceedence) was estimated to range from 0.9 m (using the van Rijn equation) to 1.8 m (using methods in EM 1110-2-1100[2]). The corresponding maximum runup elevation ranges from 4.7 m to 5.6 m (Geodetic datum) for the future sea level rise scenario that was adopted in our wave climate study. Therefore, providing drainage control for the swash and spray will be a significant issue along this section of the road.

3 GENERAL COMMENTS ON DRAFT DESIGN

3.1 EXTENT OF RIPRAP PROTECTION AND LWD PROTECTION

In NHC's November 17 report we indicated either riprap or anchored LWD could be used for protecting higher elevation sections of the shoreline (between elevation 3.2 m to 4.4 m Canadian Geodetic Datum). In sections between 1+190 to 1+380 and between 1+570 to 1+700 there is not sufficient room to install both the LWD and the riprap revetment. In these sections, the LWD can be eliminated and the riprap revetment should be extended higher to prevent erosion from wave uprush. Proposed crest levels for the riprap revetment are shown in Table 1; these are between 0.4 m to 1.0 m higher than the levels shown on the draft plans.

The Large Woody Debris (LWD) erosion protection on Sheets 6 through 13 (cross sections) is shown aligned perpendicular to the shoreline (logs aligned up and down the slope of the bank). LWD revetments are generally aligned parallel to the shoreline, with one or two rows of logs placed to form the protection. The logs are ballasted with large riprap by cables. Sheets 2 through 5 indicate that Sheet 14 shows typical bank reinforcement structures for the LWD. However, the configuration of the proposed LWD revetments is not shown.

Based on our site inspections and analysis, the section between 1+390 to 1+490 could be protected using an anchored LWD revetment rather than a conventional riprap revetment. A bio-engineered approach is suitable for this section of the shoreline since the existing dune provides a relatively wide, higher elevation backshore between the road and the beach. NHC and Current Environmental have used this approach successfully in similar settings and have found it to be an effective, lower-cost solution that is compatible with Green Shores principles and has some advantage in terms of minimizing environmental impacts and environmental mitigation/compensation requirements. The design involve placing large LWD in rows parallel to the shoreline, with the logs anchored and ballasted with large rock that is buried below the general beach level.

[2] EM 1110-2-1100 2008. US Army Corps of Engineers, Update to the Shore Protection Manual.

3.2 Riprap Revetment

Sheet 14 provides typical bank reinforcement structures and indicates that the slope of the protection is to be "established in conjunction with DFO". The stable slope of the revetment is governed by hydraulic stability criteria and should be established using standard methods such as the Hudson equation (US Army Corps of Engineers), not biological criteria. Based on US Army Corps of Engineering coastal engineering design guidelines the revetment should not be steeper than 1V:2H. We suggest indicating the gradation of the rock using standard sizing specifications published by the BC Ministry of Transportation[3] or BC Ministry of Environment[4] guidelines for riprap design.

The design sections (Sheet 6 to Sheet 13) show the bottom elevation of the revetment at approximately elevation 0.5 m. Sheet 14 shows typical sections of the revetment with the bottom of the protection at elevation 1.0 m. Given that the general beach level near the protection is at approximately elevation 2.0 m, either value for the base of the protection is expected to be well below the anticipated beach scour level. However, a consistent elevation should be adopted.

Recommended minimum top elevations for the riprap are summarized in Table 1. The elevations vary along the project, depending on the height of the bank and whether or not additional LWD protection will be provided above the riprap.

3.3 Trail and Fill Sections

The plans show some portions of the proposed trail are located within the zone of wave runup. For example, the top of the trail is shown at approximately 4.1 m between Station 1+340 and 1+380 and approximately at elevation 3.2 m between Station 1+500 and 1+540. If possible, we suggest keeping the trail above elevation 4.4 m to reduce the chance of debris, spray and wave uprush damaging it and to reduce maintenance.

The plans also show granular fill with native seeding will be placed to establish a base for the trail between Station 1+200 to 1+380 and between Station 1+500 and 1+540. The fill is seaward of the riprap revetment and extends down to approximately elevation 2.5 m, which will expose it to relatively high energy wave runup. The material in the fill is not specified on the plans. The fill would have to be similar in size as the riprap to prevent erosion. Granular fill material, combined with vegetation planting, could be used for the portion of the slope above elevation 4.4 m.

[3] BC Ministry of Transportation and Infrastructure. 2012 Standard Specifications for Highways Construction, Volume 1, Section 205.
[4] BC Ministry of Environment, Lands and Parks, 2000. Riprap Design and Construction Guide.

Table 1: Proposed locations for using LWD revetment protection and conventional riprap revetment

Station	Top of Riprap Revetment (m CGD)	LWD Revetment
1+020 to 1+190	3.8	2 rows of LWD with riprap ballast
1+190 to 1+390	4.4	None
1+390 to 1+490	None	2 rows of LWD with riprap ballast
1+490 to 1+570	4.4	Intermittent, 2 rows of LWD with riprap ballast
1+570 to 1+700	4.0 (top of bank)	None

Suggested conceptual layouts of the riprap protection are shown on the attached sketches. These layouts are based solely on coastal engineering considerations at the site. An example of an LWD revetment has also been attached to illustrate the concept of using anchored LWD as an erosion control measure. The concepts may need to be revised to account for other planning and environmental factors at the project side and have been provided to help further refine the planning process. The concepts are not intended as final designs of the erosion protection.

*   *   *   *   *

If you have any questions, please do not hesitate to contact me at 250.754.6425.

Sincerely,

northwest hydraulic consultants ltd.

original signed by

Dave McLean, Ph.D., P.Eng.
Principal

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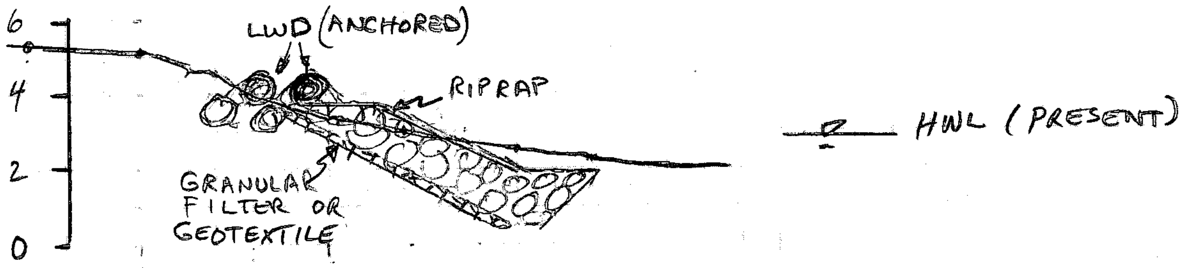
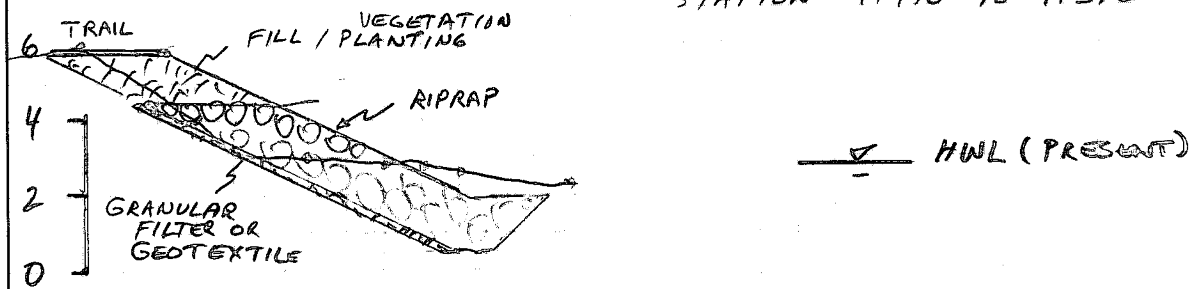
northwest hydraulic consultants	DATE JAN 5, 2015	BY DGM
CLIENT WEDLER ENGINEERING		JOB No.
SUBJECT CONCEPTUAL DESIGN OF EROSION PROTECTION - LA20 ROAD		

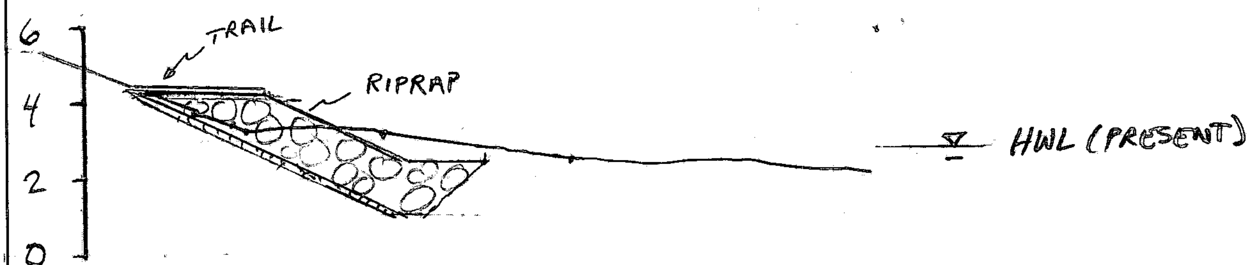
STATION 1+020 TO 1+190



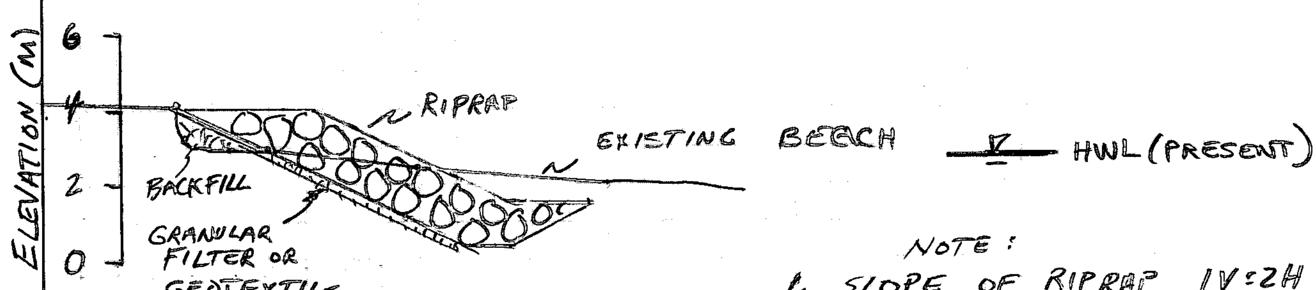
STATION 1+190 TO 1+390



STATION 1+490 TO 1+570



STATION 1+570 TO 1+700



NOTE:

1. SLOPE OF RIPRAP 1V:2H OR FLATTER
2. THICKNESS OF RIPAP REVETMENT 1.5 M (PENDICULAR TO SLOPE)
3. RIPRAP SIZE CLASS BC MOT CLASS 500 KG OR LARGER