Repairs to Mooring Base Facility Jetty

C.F.B. Esquimalt (Colwood) B.C.

("D - Jetty")

ENGINEERING REPORT

to

DEFENCE CONSTRUCTION (1951) LIMITED

September 17, 1970.

A.E. Insley, P. Eng.

Thurber Consultants Ltd.

FOUNDATIONS
LANDSLIDES
GEOLOGY
AIRPHOTO INTERPRETATION
EARTHWORK DESIGN
INSTRUMENTATION
LABORATORY TESTING
FIELD INSPECTION

THURBER CONSULTANTS LTD., Geotechnical Engineers

September 18, 1970.

File: 16-5-7

Defence Construction (1951) Limited,
4050 West 4th Avenue,
Vancouver 9, B.C.

Attention: Mr. J. C. Gillies

Reference: Repairs to Mooring Base Facility Jetty
C.F.B. Esquimalt (Colwood), B.C.

Dear Sir:

Enclosed with this letter are six copies of our report on our investigation of the above jetty. The purpose of our investigation was to examine the apparent unstable conditions of the slope under the jetty, to determine the reasons for these conditions, and to recommend appropriate action. Details are all described in the report.

We have recommended that no further work be carried out except for the immediate remedial work now in progress, and a regular annual inspection. However, for certain conditions described in the report further work of a major nature may be required.

I propose to call and review the report with you after you have had an opportunity to examine it.

Yours very truly,

Thurber Consultants Ltd.

A. E. Insley, P. Eng.

AEI:em
Encls :

H. W. Nasmith, P.Eng., B.A.Sc., (Geology) M.A.
A. E. Insley, P.Eng., B.A.Sc., D.I.C., M.Sc. (Eng.)
D. A. MacLean, P.Eng., B.Sc. (Civil), M.A.S.C.E.
G. F. Buck, P.Eng., B.Sc. (Civil), Ph.D.
G. C. Morgan, P.Eng., B.Sc., D.I.C.

628 Dupplin Road
VICTORIA
British Columbia
Phone: 388-5184

EDMONTON, Alberta

CONTENTS

1.0 INTRODUCTION

This report describes the study of the foundation conditions under a portion of the existing Mooring Base Facility Jetty (Colwood) at C.F.B., Esquimalt, B.C. and carried out by Thurber Consultants Ltd. in August and September, 1970.

The purpose of the study is to investigate the unstable conditions which are described herein, to determine the reasons for the existence of these conditions, and to recommend means for stabilizing the area as required.

2.0 AUTHORITY

Authority to proceed with the investigation was given in a letter dated August 13, 1970, from Mr. J. C. Gillies.

3.0 DESCRIPTION OF SITE

The Jetty is located on the western side of Esquimalt Harbour, near the entrance to the Straits of Juan de Fuca. The structure has a high load capacity timber and reinforced concrete deck, supported by driven precast concrete piles. It is "L" shaped with one side facing east and one facing north, known as Unit 1 and Unit 2 respectively. The deck is supported on pile bents which are at right angles to the shore. These are at 12 ft. centres under Unit 2, and at 6 ft. centres under Unit 1.

A service tunnel and coping wall runs along the shore side of the deck. These structures are supported on crushed rock. The coping wall is also supported by creosoted Douglas Fir piles which are spaced at 6 ft. centres under Unit 2 and at 3 ft. centres under Unit 1.

The shore slopes under the jetty are protected by a layer of large size riprap which was designed with a slope of 1:1 from the base of the coping wall. The position of Unit 2

is such that it is generally well protected from extreme wave forces, but Unit 1 and the east end of Unit 2 is exposed south east to a significant fetch across open sea. This direction is the source of strong winter winds, and attack from waves of moderate size. The severe storms from the southeast which occurred in this area on December, 1967, and December, 1969, would have a significant adverse effect on the slopes beneath the jetty.

Under the part of the service tunnel behind Unit 2, the rockfill support has dropped down a significant amount. At the most critical point there is a cavity about $1\frac{1}{2}$ ft. deep extending back about 7 ft. from seaward side under the service tunnel. At the west side or shore side of the service tunnel in this area, two sink holes or cavities have appeared in the pavement above. The larger cavity is about $6\frac{1}{2}$ ft. deep and the smaller one is $1\frac{1}{2}$ ft. deep. Mr. Armstrong has observed from quantity measurements for immediate remedial work, taken both before and after the December, 1969, storm, that the rock riprap surface dropped from $1\frac{1}{2}$ to 2 feet after this storm.

The photographs in Appendix A of this report show the condition of the foundation under the jetty at several locations. In some of the areas shown, considerable movement of the rock slope has occurred, while in others the slope is satisfactory. Photographs of the sink holes in the pavement are also included.

4.0 INVESTIGATION

A visual examination was made on June 1, 1970, by Mr. H. Armstrong of C.E.O. Esquimalt, Department of National Defence, and Mr. A. E. Insley of Thurber Consultants Ltd. The purpose of this visit was to make a visual assessment of the extent and possible causes of the present conditions and to consider the effect of several possible remedial measures. A survey below the water surface by soundings, and below the pavement surface

by drilling, was recommended to confirm the extent of the unstable conditions.

On August 17 and 18, 1970, following authorization of the work, cross sections under the northeast corner of the jetty were made using underwater soundings and conventional survey measurements. These were made midway between successive bents of piles and perpendicular to the shore. Representative cross sections of the most critical area between bents 4 and 5 and of a satisfactory area are shown in drawings 16-5-7-1 in Appendix B of this report.

The extent of the area where the service tunnel has lost rockfill support was also determined in the survey. Drawing 16-5-7-1 in Appendix B is a plan of the site showing the limits of this area and the extent of the cavities behind the service tunnel.

A drilling program was carried out on August 31, 1970, to determine if any cavities existed under the pavement on the shoreward side of the service tunnel, but which had not yet been detected. This operation was performed by using a hand held pneumatic drill to bore through the pavement to a depth of about 10 ft. By this method, the driller could detect a cavity and determine approximately the density of the underlying material. A series of 20 such holes were drilled, at 10 ft. centres, on the south side of the service tunnel beside Unit 2. The locations of these holes are shown in Drawing 16-5-7-1.

5.0 ENGINEERING EVALUATION

The results of the survey indicate that the riprap has dropped down along a significant length of the coping wall under Unit 2 and has assumed a flatter slope. The amount that it has been lowered varies considerably. It is greatest under the east end of Unit 2 and is least at the west end of Unit 2 and under Unit 1. At the most critical location the riprap has dropped about $4\frac{1}{2}$ ft. from the original elevation leaving the cavity under the

service tunnel. The crushed rock behind the riprap facing is exposed and is now subject to erosion at high tides.

The average gradient of the riprap rock slope where it has moved is about 2-1/4 horizontal to 1 vertical. For the remainder of its length under Unit 2 the rock slope is nearly 2:1. At no location examined is it now at the design grade of 1:1, and although this initial slope may have been constructed at a slope flatter than 1:1, the evidence shows the slope has moved down since construction to the present position.

The riprap is composed of angular rocks of 50 to 4000 lb. weight. The size of the adjacent crushed rock is given in the gradation curve in Appendix B. This material passes a 3 inch screen.

The riprap under Unit 1 is generally at a gradient of $1\frac{1}{2}$:1 and therefore steeper than under Unit 2. This is evidently due to the fact that the smaller space between piles is preventing rock movement down the slope.

No further cavities under the pavement were revealed by the drilling program, but it was determined that the rockfill behind the service trench is in a loose state.

Therefore, the slopes now existing have developed because of wave action which tends to develop the shore until an equilibrium grade is reached. This final slope depends on the size and grading of the shore material and the wave energy. For the southwest corner of the jetty, we consider a grade of 2-1/4:1 to be the equilibrium slope based on recognized engineering experience and practice. Therefore at this location, the riprap slope has nearly reached a stable condition.

Consideration has also been given to possible settlement of the fill material, or possible slope instability. However, a close examination revealed no supporting evidence

for this type of behaviour.

The immediate remedial measures now in hand by the Dept. of National Defence will involve filling the cavities under the service tunnel with concrete, and finishing with a vertical concrete face from the coping wall down to the top of the riprap. The sink holes will also be filled and paved. This will provide the required support to the service tunnel. The question arises whether further slope movements are likely which could dislodge this new concrete or leave it suspended.

It is considered that the most vulnerable portions of the riprap slope are now nearly to the gradient recognized as a stable slope. During the relatively short period of about 15 years this structure has been in service, it has experienced the highest winds, and highest wind-tide combination, over a record period in this area of about 70 years.

Further stabilizing measures at the dock which would provide for storms larger than yet experienced would consist of one of the following alternatives.

1. Construction of a $2\frac{1}{2}$:1 slope from coping wall to toe of slope. This would require additional rock riprap up to 3 feet thick or more. The deck of the jetty would have to be partly removed for rock placement.

2. Grouting the coarse riprap to cement it against individual rock action. Large quantities of grout would be required.

3. Decrease wave action by provision of a breakwater offshore (this would be useful to ships using the jetty as well as the jetty structure).

Approximate costs for present comparitive purposes only are as follows:

	Alternate	Approximate Cost
1.	More riprap	$200,000.
2.	Grout	$150,000.
3.	Breakwater	$200,000.

Therefore any one of these measures would be a major project and would only be justified to provide protection against a storm worse than yet experienced, since the slope

has already been adjusted by nature to the severe recent storms.

Therefore it is recommended that no further work be undertaken other than the immediate remedial measures now underway. An annual inspection should be made for several years to confirm that no new weak areas develop in the remedial work now underway.

6.0 CONCLUSIONS

1. The problem encountered at the southwest corner of the jetty where the rockfill support for the service trench has been withdrawn, is mainly due to wave action which caused rock movement down the slope.
2. It is considered that the original design slope for the riprap was too steep.
3. The existing slope varies from about 2:1 to 2-1/4:1.
4. The stable slope for the existing conditions based on recognized engineering practice is about 2-1/4:1.
5. The closely spaced piles under Unit 1 prevents erosion of the riprap slope by holding back the rocks.
6. The rock slope under the northwest end of Unit 2 shows no signs of instability.
7. The immediate remedial measures now planned will be adequate since the slope will be stable under a repetition of the recent severe storms. Although more serious storms are possible they are unlikely. Further protection and a larger safety factor requires expensive works. The required safety factor depends on operational requirements.
8. This examination suggests also that if movement occurs from the unlikely event of a more severe storm than yet experienced, the best course of action is to repair it at that time, and experience has shown that the jetty would not be damaged if the repair is prompt.

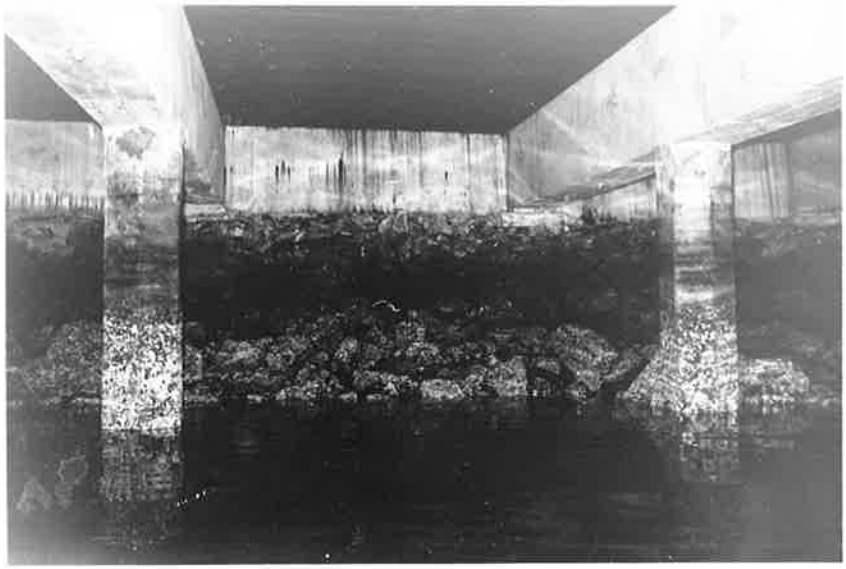
APPENDIX A

Photographs

View facing service tunnel and bents 4/5 showing how riprap has fallen about $4\frac{1}{2}$ ft. exposing piles and service tunnel.
TC 70/1/3

Similar view facing bents 5/6
TC 70/1/4

View facing service tunnel at bents 8/9 showing exposed piles and tunnel and crack in tunnel at construction joint.
TC 70/1/6



View facing service tunnel at bents 24/25 showing satisfactory conditions and no exposure of service tunnel and timber piles
TC 70/1/12

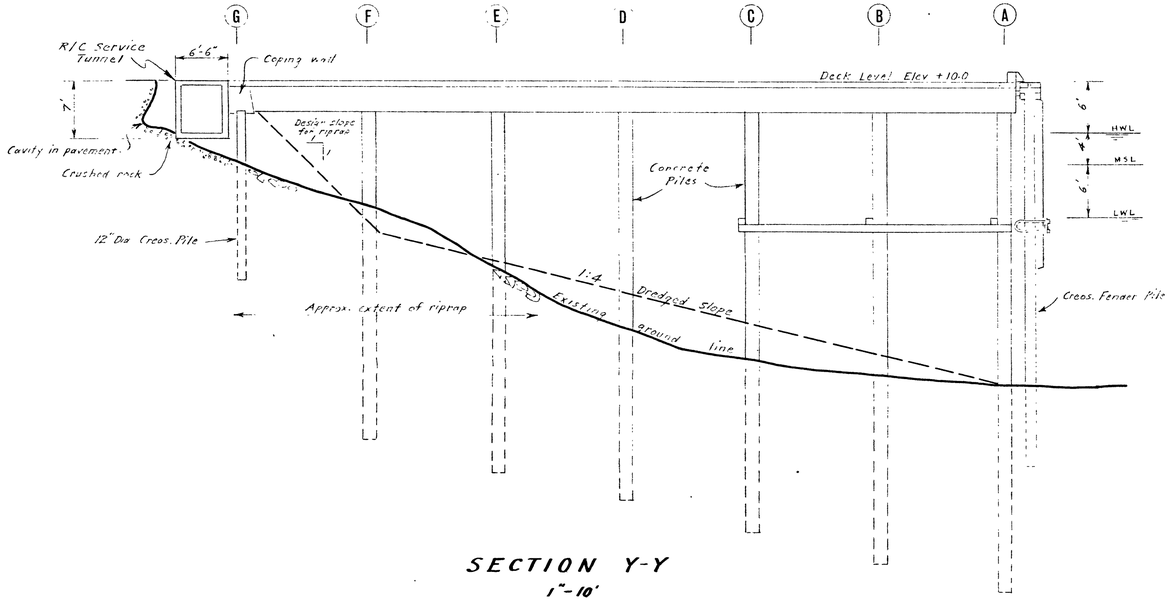
TC 70/1/3

TC 70/1/14

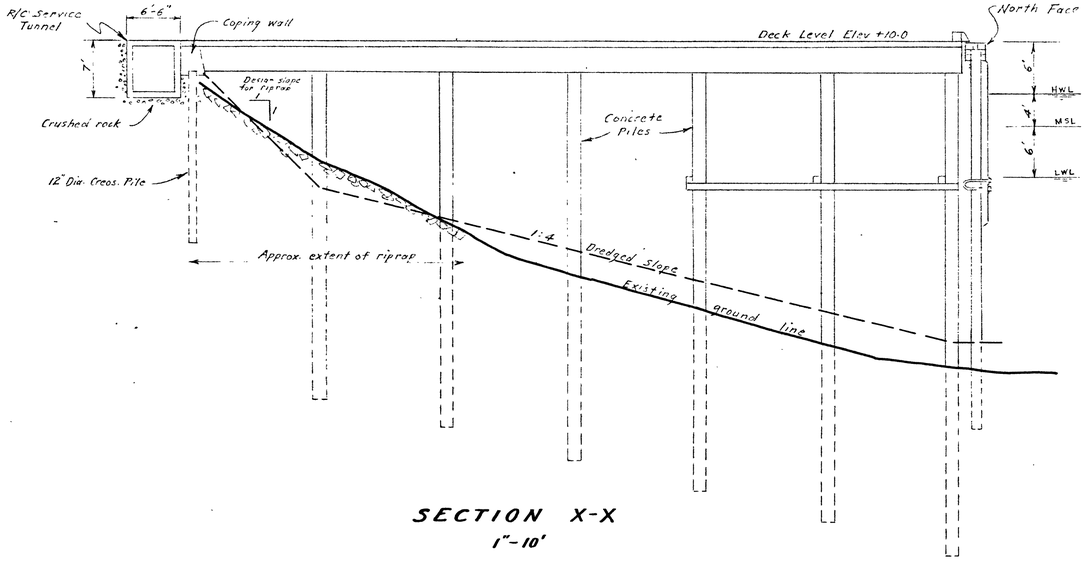
Views showing 2 sink holes in pavement on shore side of service tunnel beside unit 2.

APPENDIX B

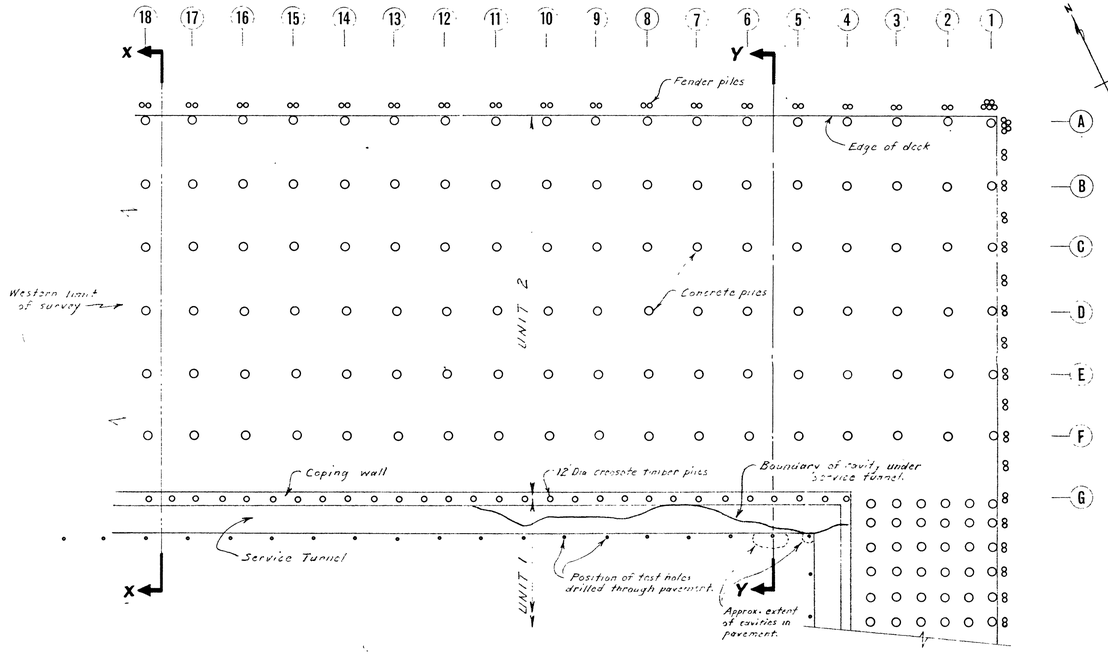
Drawings



SECTION Y-Y
1"-10'



SECTION X-X
1"-10'



PLAN
1"-20'

Notes.

Datum is based on Mean Sea Level.
Plan and Sections taken from Public Works of Canada Drwg No A.337 dated Oct. 19, 1954.

DEFENCE CONSTRUCTION (1951) LTD.

SITE INVESTIGATION AT
MOORING BASE FACILITY JETTY
C.F.B. ESQUIMALT (COLWOOD) B.C.

DRAWN	
TRACED	D.W.S.
DATE	Sept 21/70
APPROVED	T.C.
SCALE	Shown
DRAWING No.	76-5-7-1

THURBER CONSULTANTS LTD., Geotechnical Engineers