HYDRAULIC MODEL STUDY
STEVESTON BENDWAY WEIRS

PRELIMINARY

WESTERN
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1.0 SUMMARY

Public Works Canada (PWC) on behalf of the Canadian Coast Guard (CCG), with the support of several engineering studies, developed a new training scheme for Steveston Bend to convert from a major to a minor shipping bend. The scheme called for a gravel berm supporting a multi-sectioned vertical steel or concrete wall, designed to reduce the annual maintenance dredging requirements by over 90 percent. Alternative schemes are presently being considered as a result of unanticipated shortages in the designated gravel reserves which led to cost overruns on the berm construction.

One of the alternatives considered is a series of bendway weirs. Bendway weirs are low elevation, totally submerged, structures placed at an angle of approximately 30 degrees to the flow, which reduce the characteristically large scour depths at the outside of bends and widen the channel near the inside of the bend, requiring less maintenance dredging. The bendway weir concept has not been tried in a tidal river like the Lower Fraser River. A hydraulic model study was considered necessary to demonstrate the feasibility and to develop comparative data for evaluation.

A test program was required to optimize the set of bendway weirs, to ensure adequate training of the river channel for navigation purposes at least cost, to determine the size of material required for stable weir construction and to assess the effect of the existing berm on the bendway weir scheme.

A hydraulic model study was commissioned to assist PWC with the design. The study was undertaken in an existing 1:150 horizontal scale tidal mobile bed hydraulic model with a vertical scale of 1:75. The model was used to assess the stability of the existing berm which is constructed of quarry tailings and Fraser River sand, to determine the feasibility of the bendway weir concept, to assist in the development of a bendway weir scheme as well as in the optimization of that scheme.

The results of the model study confirmed the stability of the berm presently in place, as well as the technical feasibility of the bendway weir concept. Although this concept was proven to be feasible even with a reduction in the number of weirs from 10 to 5, the study was by no means exhaustive, as a further reduction in the number of weirs remains a possibility. Hence further potential construction savings may be possible. The mobile bed tests indicated potential scour along the Albion Dyke similar in magnitude to the scour obtained with the multi-sectioned wall. Intertidal scour of approximately 1 meter should be expected to the end of the South Jetty.

Surface velocity contours obtained from the model indicated that the transverse velocity gradients in Steveston Bend will be reduced when bendway weirs are installed, rendering the velocities more evenly distributed laterally. In addition, the "dead" area behind the planned vertical training wall will be carrying flow, producing a more uniform flow and reducing the average flow velocities in the bend during the ebb tide. Flow velocities adjacent to the South Jetty will be measurably increased during the ebb tide.

The difference in the rates of energy dissipation of the Steveston Bend flows with and without bendway weirs has not been evaluated. It is presently not apparent therefore whether the flow distribution at Albion Dyke will be affected by the presence of a scheme of 5 bendway weirs.

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CONCLUSIONS AND RECOMMENDATIONS

2.1 Conclusions

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2.1.1 Berm Alone Tests

1. Berm tests confirmed that the quarry tailings, which were considered to have a median size of 100 mm with a range of 50 mm to approximately 270 mm, produced a stable berm for Hope discharges of up to 12100 cubic meters per second and a maximum tidal range of 4.7 m. This is approximately equivalent to a 15 year return period.

2. The tests further indicated that some settling must be expected to occur when the berm tailings are directly placed on an undisturbed sand bed and the interface between berm tailings and bed is above an elevation of -20 m LLW.

2.1.2 Berm with Bendway Weirs Tests

1. Mobile bed tests confirmed the feasibility of producing a self scouring channel using ten (10) bendway weirs in combination with the existing berm.

2. A limited number of optimization tests were undertaken. These tests demonstrated that the self-scouring characteristics of the navigation channel could be maintained when the number of bendway weirs was reduced from 10 to 5. The length of the second, third and fourth weirs required lengthening however.

3. On the basis of the limited tests undertaken the elevation of the berm has little influence on the effectiveness of the bendway weir scheme.

4. Intertidal scour of approximately 1 meter is expected along the South Jetty following construction of the recommended bendway weir scheme, which will cause further reductions in the average velocities through the bend.

5. Transverse velocities through 60 percent of the length of the bend will have a relatively uniform distribution while flow velocities adjacent to the South Jetty will be measurably increased during the ebb tide.

2.2 Recommendations

1. If no further testing is anticipated it is recommended that the scheme shown in Figure 9, consisting of the existing berm plus five (5) bendway weirs, be implemented for construction. The test results indicate that an average of less than 10,000 m^3 of maintenance dredging annually will be required upon implementation.

2. The recommended method of bendway weir construction is to excavate to an elevation of -20 m LLW before placement of the weir material.

3. It is recommended that the material used for berm construction has a median size of 100 mm and a minimum range of from 50 mm to 270 mm.

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3.0 INTRODUCTION

3.1 Rationale for Study

Public Works Canada (PWC) on behalf of the Canadian Coast Guard (CCG) commenced construction of a new multi-sectioned vertical training wall having an increased radius from 1600 to 3150 meters to improve navigation and drastically reduce maintenance dredging in Steveston Bend of the Fraser River by over 90 percent, see Figure 1. The wall sections, which had been designed and optimized using a hydraulic model, were to be driven into a submerged riprap armoured berm consisting mainly of St. Mungo Bend gravel.

PWC started to investigate alternative construction methods when the gravel reserves for berm construction were found to be much smaller than anticipated, necessitating the use of alternative sources of materials and construction methods and causing berm construction costs to escalate. A new structural design solution recently developed by the US Army Corps of Engineers using a hydraulic model and verified in several Mississippi River bends was suggested as an alternative to the vertical training wall. The Mississippi River solution consists of submerged angled weirs, called bendway weirs, placed throughout the river bend which reduce the characteristically deep and narrow trench, widen the navigation channel and re-direct the otherwise detrimental high flow velocities away from the outside of the bend. Bendway weirs are low elevation structures, totally submerged and relatively inexpensive to construct.

Western Canada Hydraulic Laboratories Ltd (WCHL), having completed the previous Steveston Bend model studies and still having the Steveston Bend model partially intact, was requested to undertake a test program to assess the effectiveness of bendway weirs in Steveston Bend. As the Butler type building housing the model was scheduled for demolition within 2 months, an accelerated test program was agreed upon.

3.2 Study Objectives

As a result of a review of the entire Steveston Bend training wall project, PWC is pursuing modified project objectives. These include an assessment of the stability and the effect on river training of the newly constructed berm, as well as an assessment of bendway weirs in combination with the existing berm.

The specific objectives of the model studies were :

3.2.1 Tests with Existing Berm Alone

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a) Test the hydraulic stability of the existing quarry tailings on the berm face as well as the sand backfill and test modifications if required.

b) Test the potential scour at the toe of the berm along the smooth face of the North Jetty downstream of structure #6, and the extent of potential erosion of the berm sand backfill if left unprotected.

c) Test the maximum flow velocities adjacent to and over the berm and between the berm and the North Jetty.

d) Test the expected "equilibrium" river channel dimensions following two different Hope freshets.

e) Determine the required amount of maintenance dredging within the navigation channel dimensions under the same freshet conditions as in d)

f) Determine the expected channel depth when capital dredging to -9.3 m is carried out to the southern boundary of the navigation channel and determine the annual amount of maintenance dredging required to maintain a depth of -8.1 m in the new channel.

g) Determine approximate velocity distribution patterns developed as a result of the presence of the berm and the effects on the fish migration window.

3.2.2 Tests with Berm and Bendway Weirs

The objectives of these tests was to develop an optimum set of bendway weirs which together with the existing berm will provide an adequate navigation channel with minimum maintenance dredging and minimum berm and weir maintenance. The specific objectives included:

a) Scoping tests to examine the effect of weir orientation, length, crest elevation and spacing with the objective of formulating an optimum weir layout.

b) Test the optimum layout for maximum velocities in the navigation channel, across the berm and between berm and North Jetty.

c) Test the potential scour at the toe of the berm along the smooth face of the North Jetty downstream of structure #6, and the extent of potential erosion of the berm sand backfill if left unprotected.

d) Assess local and uneven erosion and deposition patterns in the navigation channel with possible effects on downstream and upstream channel sectors.

e) Test the expected "equilibrium" river channel dimensions following two different Hope freshets.

f) Determine the required amount of maintenance dredging within the navigation channel dimensions under the same freshet conditions as in d)

g) Determine the expected channel depth when capital dredging to -9.3 m is carried out to the southern boundary of the navigation channel and determine the annual amount of maintenance dredging required to maintain a depth of -8.1 m in the new channel.

h) Determine approximate velocity distribution patterns developed as a result of the presence of the berm and the effects on the fish migration window.

i) Test different quarry run sizes to ensure adequate stability.

4.0 THE MODEL

4.1 Model Scales

The Steveston Bend model with short upstream and downstream tangent reaches was refurbished to the conditions existing during the 1990-1991 Steveston Bend Optimization test program with length and depth scales of 150 and 75 respectively, see Figure 2.

There are three general requirements which should be simultaneously considered in a moveable bed model design:

i) adherence to Froudian scaling if possible;
ii) geometric similitude of boundary roughness;
iii) dynamic scaling of sediment transport.

Each of these requirements is considered separately.

4.1.1 Froude Scaling

The transient nature of the flow in the Steveston Bend area as well as the non-uniformity of the main channel make it desirable to have equality in the ratio of gravitational to inertial forces in model and prototype. This implies Froude scaling, i.e. that the vertical scale ratio becomes a determinant for the velocity ratio, or

$$n_v = (n_h)^{1/2}$$

where n_v = velocity scale ratio

n_h = vertical or depth scale ratio

Deviations from Froude scaling will result in scale effects in the ratio of total energy head to pressure head. For example, for a 30 percent flow augmentation in the model the increased velocity head creates only a 1.5 to 2.0 percent increase in the total energy head in the model depending on the initial value of the Froude number.

4.1.2 Roughness Scaling

In wide rivers, where the width is an order of magnitude greater than the depth, geometric similitude of boundary roughness is an excellent substitute for Reynolds similitude provided turbulent flow is maintained in the model. The total roughness is often scaled by using either the Manning or the Chezy equation. Using the Chezy equation:

$$V = C\sqrt{(hS)} \tag{1}$$

where V = average cross-sectional velocity
C = Chezy transmission coefficient
h = average depth of flow
S = friction slope

4.1.2.1 Fixed Bed

The roughness of a fixed bed can be adjusted by using gravel, rods or strips or any other element offering resistance to the flow. The total roughness of bed and resistance elements must produce the correct friction slope. This results in the scale ratio of the Chezy factors:

$$n_C = (n_L/n_h)^{½} = (r)^{½} \tag{2}$$

where n_L = horizontal or length scale ratio
r = distortion ratio

4.1.2.2 Moveable Bed

The scale relationship expressed in equation (2) must also hold for a moveable bed. In addition to this, the ratio of grain resistance to total resistance at the bed should be the same in model and prototype to produce relatively equal drag forces of the flow on the particles. Expressing grain roughness in terms of the Strickler Formula:

$$C_g = K\,(h/d_p)^{1/6}$$

where C_g = Chezy coefficient for grains only

K = constant

d_p = average particle size

a relationship can be obtained for the scaling of the average size of sediment particles:

$$n_{d_p} = n_h^{4}/n_L^{3} = n_h/r^3 \qquad (3)$$

Both Chezy and Strickler formulas hold for turbulent flow only. Turbulent flow in the model must therefore be strictly adhered to, which is accomplished by ensuring that the shear velocity Reynolds number in the model exceeds the value of 100:

$$\frac{(u^*h)_m}{v} > 100 \qquad (4)$$

where u^* = $\surd(ghS)$ = shear velocity

v = kinematic viscosity

m = indicates model

4.1.3 Scaling of Sediment Motion

For the dynamic scaling of sediment transport one commonly relies on empirical relationships such as White's or Shield's tractive force analysis. These relationships are approximate and exact dynamic similitude can therefore not be expected. Moreover the relationship was derived for nearly uniformly sized particles so that additional uncertainties are introduced when an average size is used for both river and model sediment particles. Using Shield's functional relationships for the shear parameter

$$\frac{hS}{\Delta d_p}$$

and for the flow parameter:

$$\frac{u^* d_p}{v}$$

scale effects will be minimized if both parameters are scaled correctly. This results in the relationships:

$$n_\phi = n_h / (r.n_\Delta) \tag{5}$$

$$n_\phi = (r / n_h)^{½} \tag{6}$$

where $\Delta = \dfrac{R_s - R_w}{R_w}$

R_s = density of particles

R_w = density of fluid

It must be remembered that the energy expended to move grains is less than the total energy expended. The value of the shear parameter should therefore be reduced to account for this fact. This is sometimes done by the use of a ripple factor. In the previous section on roughness scaling this factor was taken into account by ensuring that the ratio of grain to total resistance is scaled correctly. Consequently the scale relationship obtained in equation (5) remains unaffected.

Shield's shear parameter can be used to derive a velocity scale relationship for sediment movement by substitution from the Chezy equation for the product hS:

$$n_v = n_C (n_{\Delta} \cdot n_{dp})^{½} \tag{7}$$

This is often called the "ideal" velocity scale relationship for sediment movement because it is this relationship that governs similarity with respect to bed topography.

4.1.4 Resultant Model Dimensions

The simultaneous solution of equations 1,2,3,5 and 6 with the constraint of equation 4 is not unique. This stems from the fact that there are only four independent variables, n_h, r, n_{d_p} and n_v. Constraints of space and required accuracy of vertical measurements in the model produced the scale relationships shown below.

Length	n_L	= 150	
Depth	n_h	= 75	
Distortion ratio	r	= 2	
Velocity	n_v	$= \surd n_h$	= 8.66
Area	n_a	$= n_L n_h$	= 11,250
Discharge	n_Q	$= n_v n_a$	= 97,428
Pressure	n_p	$= n_h$	= 75
Mass	n_m	$= n_L{}^2 n_h$	= 1,687,500
Volume	n_{VOL}	$= n_L{}^2 n_h$	= 1,687,500
Force	n_F	$= n_L{}^2 n_h$	= 1,687,500
Time	n_t	$= n_L / \surd n_h$	= 17.32
Frequency	n_f	$= 1/n_t$	= 0.0577

4.1.5 Application to Gilsonite as Bed Material

The above relationships will be valid as long as the ideal velocity scale for sediment transport equals the Froude scale. Since the gilsonite used as bed material was coarser than that required according to the ideal velocity scale, see Figure 3, the model flows required augmentation. The amount of augmentation required is discussed in the following paragraphs.

The roughness condition requires:

$$n_c = (r)^{0.5} = (150/75)^{0.5} = 1.414$$

However the fact that the model bed material due to its lightness is relatively significantly coarser than that of the prototype, the actual Chezy C ratio, n_c, is larger than 1.414.

Using the relationship of VanRyn and assuming a flat bed:

$$C = 18 \log (12h/3D_{90}) \tag{8}$$

the predicted Chezy C values are approximately 90 and 44 for prototype and model respectively, producing a value of $n_c = 2.05$.

Substitution of this value in the ideal sediment transport relationship of equation (7) results in:

$$n_u = 7.42$$

Adherence to this velocity scale would require a flow velocity augmentation of approximately 8.66/7.42 or about 17%.

The transport parameter increases with shear stress and approaches a maximum value of 25 when the bed becomes relatively flat at high river stage. The maximum value of d_b therefore does not occur during high freshet discharges. Rather the maximum value of d_b is reached when the product of the last two terms in equation (10) reaches a maximum. This maximum is reached when T assumes a value of 5, for which the maximum height of the bed forms is predicted to be d_b = 1.18 m, while the minimum is predicted to be d_b = 0.065 m when T = 24. The actual d_b value during the freshet will probably be much closer to the latter value. Assuming a mean value of 0.62 m for d_b, the equivalent k_s value becomes 0.09, while the Chezy value computes to C = 59.4.

The gilsonite bed features in the Steveston Bend model are typically 1.5 to 2.5 cm high and vary in length from 30 to 100 cm. The equivalent bed roughness computes to a value of k_s = 0.033 m and a Chezy value of C = 33.5.

The Chezy C ratio representative of the prototype sand bed and the model gilsonite bed computes therefore to:

$$n_C = 59.4/33.5 = 1.77$$

If this ratio is used in equation (7) the ideal transport velocity scale works out to:

$$n_u = 6.57$$

Adherence to this velocity scale would require a flow velocity augmentation of approximately 30%.

In view of the widely varying tidal discharges throughout the tidal cycle with equally varying bed roughnesses, a correct roughness scale cannot be selected and hence the ideal transport scale varies as well with the tidal cycle. It is for this reason that the accuracy in predicting the bed evolution is difficult to quantify. In some instances a combination of Froude scaled and augmented discharges are employed, while in others an augmented discharge scale is strictly adhered to.

Improved accuracy can only be obtained by means of model calibration during which the model simulates a specific prototype event which has been adequately documented.

4.2 Model Re-construction

The model was re-conditioned in WCHL's No. 1 building, Figure 2. The surface topography of Roberts Bank, Sturgeon Bank and the channel was constructed using sand-cement near the end of the bend where the model had been altered and the mobile bed material consisting of gilsonite with a size range of .1 to 4.0 mm, Figure 3, was placed to a depth of approximately -26 m LLW throughout the model.

4.3 Model Operation

The Froudian time scale of the model was:

$$n_t = \frac{n_L}{n_v} = \frac{150}{(75)^{1/2}} = 17.32$$

such that the prototype tidal period of approximately 24.8 hours was reduced in the model to approximately 86 minutes. Tidal flow reversals through the Steveston Bend were programmed on the micro-processor which controlled the movement of the valve actuators with the pump operating continuously. The downstream end of the model coincided with the return channel which was used as a gilsonite trap.

4.4 Instrumentation

Water surface elevations were measured using capacitive wire probes and wave followers calibrated with point gauges having a precision of ± .025 mm in the model. The overall accuracy in the prototype was ± 1 cm.

Cross sectional velocities were obtained using an Ott propeller meter with an accuracy of ± 3%, while surface velocities and flow patterns were obtained from photographs of confetti traces. Scour and deposition processes were measured using a rod profiler having a varying rod interval distance. An interval of 9 m prototype was used in the area of the berm to a distance of 253 m from the North Jetty. From here to a distance of 558 m a less dense interval of 19 m. was used while at distances greater than 558 m from the North Jetty an interval of 45 m was used. These intervals were chosen such that in the areas of complex bathymetry, ie. berm and submerged weirs a high degree of resolution could be achieved while in the areas of the south bank where the bed was much more uniform in nature a less dense array was required.

A portion of the model and the model control panel with data acquisition system are shown in Figure 4.

5.0 TEST PROGRAM

The main objectives of the test program were to:

1) test the stability and the effects on bed evolution of the existing berm consisting of quarry tailings materials having a D50 of 100 mm.

2) investigate the feasibility of and optimize, if possible, the deployment of bendway weirs to maintain a self-scouring navigation channel.

5.1 Tests with the Berm Alone

5.1.1 Without Maintenance Dredging (Tests 1 & 2)

The specific objectives of these two tests were to determine:

a) the hydraulic stability of the quarry tailings on the berm face as well as the stability of the sand backfill.

b) the extent and potential depth of scour at the toe of the berm, along the south face of the existing North Jetty and downstream of berm structure #6.

c) the effects the berm has on river velocities in the vicinity of the berm and the North Jetty.

d) The effects of the berm on long term river navigation channel depths in the absence of any maintenance dredging.

Test 1 Conditions:

- The river bed was moulded to the contours of the October 1992 river soundings.
- Hard points near the upstream end of the berm were installed.
- The existing berm was constructed to the laboratory floor (approximate elevation of -26 m LLW) using SG=1.34 coal, sized to represent St Mungo Bend gravel.
- No small groins were installed downstream of berm structure #6.
- A Hope discharge of 5700 cubic meters per second (cms) was simulated.
- The simulated tide was a Spring Tide from 0.2 m SH to 4.7 m SH.
- The model bed was profiled after 48 hours (prototype) of the testrun, followed by a further 48 hours.

Test 2 Conditions:

- This test was commenced using the evolved bed following test 1.
- A Hope discharge of 12100 cms was simulated.
- The simulated tide was a Spring Tide from 2.0 m SH to 4.8 m SH.
- The remainder of the test conditions were as in test 1.
- The model bed was profiled after a testrun of 48 hours and followed by a further test run of 48 hours.

5.1.2 With Maintenance Dredging to -8.1 m LLW (Tests 3 & 4)

The objective of these tests was to assess the effects of the existing berm in the presence of a pre-dredged navigation channel on maintenance dredging assuming the berm undergoes no further modifications.

Test 3 Conditions:

- The river bed was moulded to the contours of the October 1992 river soundings.
- Hard points near the upstream end of the berm were installed.
- The existing berm was constructed to the laboratory floor (approximate elevation of -26 m LLW) using SG=1.34 coal, sized to represent St Mungo Bend gravel.
- No small groins were installed downstream of berm structure #6.

- A 243 m wide navigation channel was dredged to a depth of 8.1 m LLW and located 30 m from the 3150 m radius bend.
- A Hope discharge of 5700 cubic meters per second (cms) was simulated.
- The simulated tide was a Spring Tide from 0.2 m SH to 4.7 m SH.
- The model bed was profiled at the end of the testrun (96 hours).

Test 4 Conditions:

- This test was commenced using the evolved bed following test 3.
- A Hope discharge of 12100 cms was simulated.
- The simulated tide was a Spring Tide from 2.0 m SH to 4.8 m SH.
- The remainder of the test conditions were as in test 3.
- The model bed was profiled after a testrun of 96 hours.

5.1.3 With Maintenance Dredging to -9.3 m LLW (Tests 5 & 6)

The specific objectives of these tests were identical to those of tests 3 and 4, but with an increased pre-dredged channel.

Test 5 Conditions:

- The test conditions were identical to those in test 3, except for the depth of the dredged river bed which was -9.3 m LLW and the Hope discharge with the tidal range which were identical to the conditions in test 4.

  Note: The results of this test were suspect as a portion of the berm near the downstream end was washed out during filling.

Test 6 Conditions:

- The test conditions were identical to those in test 4, except for the depth of the dredged river bed.

- This test was a repeat of test 5 with a repaired section of the berm near the downstream end. The repaired section of the berm was placed on top of the gilsonite bed in order to assess the local berm stability of the tailings berm when placed on top of the gilsonite bed rather than on the laboratory floor.

5.2 Tests with Bendway Weirs between Berm and North Jetty (Tests 7 & 8)

The specific objectives of the tests with the existing berm and the five bendway weirs between the berm and the North Jetty as shown in Figure 5 were to determine:

a) the hydraulic stability of the quarry tailings on the berm face as well as the stability of the sand backfill.

b) the extent and potential depth of scour at the toe of the berm, along the south face of the existing North Jetty and downstream of berm structure #6

c) The equilibrium river bed conditions in the navigation channel and south of the channel.

Test 7 Conditions:

- Five bendway weirs having side slopes of 3H:1V and a crest elevation of -11 m were constructed of coal representing quarry tailings and were placed on the laboratory floor in the locations shown in Figure 5.
- The remainder of the test conditions were as in test 1.
- The model bed was profiled at the end of the testrun (96 hours).

Test 8 Conditions:

- Test 8 was commenced using the evolved bed following test 7.
- The remainder of the test conditions were as in test 2.
- The simulated tide was a Spring Tide from 2.0 m SH to 4.8 m SH.
- The model bed was profiled at the end of the testrun (96 hours).

5.3 Berm with Bendway Weirs

5.3.1 Tests with Berm and Ten Bendway Weirs (Tests 9-12)

The objective of these tests was to determine the bed evolution resulting from the presence of ten (10) regularly spaced submerged weirs oriented at 30 degrees upstream from the normal as shown in figures 6 and 7, installed with a top elevation of - 11 m LLW throughout Steveston Bend, with each weir running from the North Jetty to near the edge of the navigation channel.

The specific goals of the tests were to determine:

a) the hydraulic stability of the quarry tailings on the weirs, on the weir noses, on the berm as well as the stability of the sand back fill.

b) the bed evolution throughout the bend, but specifically near the south end of the navigation channel

c) the effects of the terminal (south) ends of the weirs on eddy formation and on navigation for small vessels.

Test 9 Conditions:

- Ten bendway weirs having side slopes of 2H:1V and a crest elevation of -11 m were constructed of concrete sections with a 50 m long section of coal at the south end, representing quarry tailings. They were placed to a depth of -20 m LLW in the locations shown in Figure 6.
- The small groins downstream of berm #6 were installed using coal bed material simulating quarry tailings.
- The remainder of the test conditions were as in test 1.
- The model bed was profiled at the end of the testrun (96 hours).

Test 10 Conditions:

- Test conditions were as in test 9 with the exception that the concrete weir sections were nearly all replaced with coal representing quarry tailings.
- The lengths of the two downstream weirs, ie. weirs H and I were reduced from 263 m and 291 m to 230 m and 188 m respectively to reduce their scour effect and to alter the direction of the downstream flow to a flow more nearly parallel to the North Jetty, see Figure 7.

Test 11 Conditions:

- The berm and weir configurations were as in test 10.
- The flow and tidal conditions were as in test 2.

Test 12 Conditions:

- The flows were augmented by 30% to approach the ideal sediment transport velocity scale for bed evolution simulation.
- The coal representing the bendway weirs was replaced by rock to ensure the stability of the weirs.
- The remainder of the test conditions were as in test 11.

5.3.2 Tests with Berm and Five Bendway Weirs (Tests 13-18)

The objectives of these tests were identical to the objectives of the tests of section 5.3.1, except that the number of bendway weirs was reduced to five (5).

Test 13 Conditions:

- The bendway weir configuration was as shown in Figure 8.
- With the exception of the number of bendway weirs, the test conditions were as in test 12.

Test 14 Conditions:

- The terminal ends of the bendway weirs, consisting of rock, were replaced by 50 meter long sections of coal representing the quarry tailings.
- The berm elevation was lowered by 2 m to -13 m LLW throughout.
- The flow conditions were as in test 2.

Test 15 Conditions:

- With the exception of the lowered berm to elevation -13 m, the test conditions were as in test 12.

Test 16 Conditions:

- The test conditions were as in test 15 with the exception of the length of the weirs, which were changed as follows, see Figure 9:

Weir	Previous Length	New Length
C	279	324
E	303	393
G	255	347

Test 17 Conditions:

- Test 17 was commenced using the evolved bed following Test 16
- The rest of the conditions were as in Test 16.

Test 18 Conditions:

- Berm was raised back up to -11 m from -13 m LLW
- The bendway weirs were all raised by 1 m to an elevation of -10 m LLW
- The rest of the test conditions were the same as Test 16

6.0 TEST RESULTS

6.1 General

A total of 18 testruns were completed; 6 tests with the berm alone; 2 tests with 5 short bendway weirs between the berm and the North Jetty; 4 tests with 10 bendway weirs, one of which was with a 30% augmented discharge; finally 6 tests with 5 bendway weirs, five of which were with a 30% augmented discharge.

Although the purpose of the berm alone tests was twofold: to estimate the long term self scouring capability of the channel bend with the presently existing berm and to determine the longterm stability of the berm itself, the extent of the test program did not permit a thorough exploration of the former other than to indicate that the existing bend will have insufficient self scouring capability with an unknown annual maintenance dredging volume. The tests demonstrated satisfactory stability of the berm which was constructed of quarry tailings and river sand.

Pre-dredging did not result in any reduction in the required volume of maintenance dredging within the constraints of accuracy of test results.

The tests with the five short bendway weirs between the berm and the North Jetty demonstrated that the weirs limit the characteristic deep bend scour near the outside of the bend and widen the navigation channel through the bend. The depths in the widened channel remained insufficient however.

The present test program was undertaken primarily to evaluate the effectiveness of bendway weirs in developing a self scouring channel which will require little or no maintenance dredging. An initial configuration consisting of 10 bendway weirs was developed with the assistance of R. Davinroy of the U.S. Army Corps of Engineers. With minor modifications this configuration proved effective in providing a maintenance free navigation channel.

The test program has demonstrated that significant potential savings can be realized by reducing the number of weirs while maintaining a viable self scouring channel.

6.2 Tests with Berm Alone

6.2.1 Without Maintenance Dredging (Tests 1 and 2)

Bed profiles as well as surface velocity profiles were obtained for most tests. The results of Test 1 are shown in Appendix A, while the results of Test 2 are shown as Figures 10 to 16.

Scour Along the North Jetty

Scour was observed along the face of the North Jetty during Test 1 and also during Test 2. Approximately 3 m of scour occurred near the upstream end of the berm. The scour was observed to increase in the downstream direction with approximately 5 m of scour occurring near the end of the bend. The scoured trench along the North Jetty was widest at transects J8 & J9. The deepest scour occurred downstream of berm structure #6 where the river bed was scoured to the concrete floor at -26 m LLW.

Scour Along the Berm

The model construction material representing the berm material in these two tests simulated St Mungo Bend gravel with a D50 of approximately 20 mm, rather than the quarry tailings having a D50 of approximately 100 mm due to a misunderstanding at the start of the test program. The berm which consisted of this undersized material appeared to be stable except for two locations:

i) The berm elevation was reduced from -10 m LLW to -12 m LLW at the upstream portion of berm structure #2.

ii) The berm elevation was reduced by 1 m near berm structure #6 near a substantial scour hole.

Slight scour was also evident along the south face of the berm coincident with a parallel ridge of deposited material having top elevations above the floor of the navigation channel.

The berm stability was brought into question for two reasons:

1. The quarry tailings making up the existing berm are significantly coarser than the modeled material.

2. The berm in the model is placed on a concrete floor i.e. on bedrock; the berm in the field is not. How can the potential for berm undermining be assessed?

The berm material size was increased to represent the quarry tailings in the next test. To address the undermining question, two test sections were created which consisted of a length of berm having material of 1.34 SG coal placed directed on the gilsonite bed material at elevation -20 m LLW. During the tests one of the berm lengths failed while the other, near transect J10, appeared to be stable. Additional undermining tests were undertaken later in the test program.

PRELIMINARY

Scour of the Navigation Channel

The test results indicated that approximately 50,000 cubic meters of material would have to be dredged following Test 2 to obtain a depth of 8.1 m below LLW for the full 240 m width of the navigation channel, with all of this dredging being required along its south boundary.

Scour Along Roberts Bank

The area south of the navigation channel and the South jetty is relatively inactive. The area is estimated to scour up to approximately 1 m. Bed equilibrium conditions were generally reached after 48 hours i.e. the change in bed elevations was negligible between the 48 hour and 96 hour test durations.

6.2.2 With Pre-Dredging to -8.1 m LLW (Tests 3 and 4)

The bed profiles obtained with Tests 3 & 4 were generally very similar to the ones obtained with Tests 1 & 2 but with larger deposited ridges parallel to the berm. Approximately 98,500 cubic meters of material would have to be dredged following Test 4 to obtain a depth of 8.1 m below LLW for the full 240 m width of the navigation channel. Hence even though the navigation channel was pre-dredged to an elevation of 8.1 m below LLW, the maintenance dredging load was predicted to be approximately 50,000 cubic meters greater than without pre-dredging.

6.2.3 With Pre-Dredging to -9.3 m LLW (Tests 5 and 6)

The bed profiles obtained with Tests 5 & 6 were again very similar to those obtained with Tests 1 & 2, but with an even greater amount of 152,000 cubic meters of material to be dredged following Test 6 to obtain a depth of 8.1 m below LLW for the full width of the navigation channel. Hence the maintenance dredging load was again predicted to be considerably greater than without pre-dredging.

These discrepancies are believed due to the bed forms e.g. dunes and ripples which can cause periodic bed variations of an estimated 1 m to 1.5 m from the mean bed level. Assuming an affected channel width of 50 m, a total bend length of 1800 m and a bed elevation variation of +/- 1.25 m, the difference in total volume throughout the bend could be as much as 50 * 1800 * 2.5 = 225,000 cubic meters. Variations of a significant fraction of this volume must therefore be expected.

6.3 Tests with Bendway Weirs between Berm and North Jetty (Tests 7 & 8)

The stream lines of the flow above the bendway weirs was not as smooth as the stream lines south of the berm and appeared disorganized, an indication of high turbulence levels. Yet there was limited scour in the area between the berm and the North Jetty, indicating a stable local bed elevation.

The effect of the bendway weirs on the bed evolution manifested itself in a widening of the river channel, but the increase in width occurred south of the navigation channel, leaving depths in the navigation channel unaltered or even shallower. The amount of material that would have to be dredged to obtain a navigation channel depth of 8.1 m below LLW after completion of the tests was predicted to be 7,500 cubic meters.

6.4 Berm with Bendway Weirs

6.4.1 Tests with Berm and Ten Bendway Weirs (Tests 9 to 12)

6.4.1.1 Test 9, Figure 6

The results of Test 9 indicated that near the middle of the bend, ie. from transects J8 to J10 the bendway weir lengths were insufficient to produce sufficient self scour near the south edge of the navigation channel.

A severe scour hole developed downstream of bendway weir H which caused undermining of bendway weir I at its south end. Several of the concrete bendway weir sections were resting on the concrete floor (elevation -26 m LLW) and the bed material had disappeared.

Several other bendway weirs downstream of transect J10 were at the point of incipient failure as the channel bed had been scoured to elevations below -20 m LLW at several locations. Some of these failures were observed to be due to the presence of flow disturbances caused by the surface discontinuities of the concrete and coal bendway weir sections. Most of these discontinuities were eliminated in the next test.

Downstream of the bend (transect J12) a channel was scoured in a direction away from the North Jetty, suggesting the possibility of elimination or shortening one or two of the downstream bendway weirs.

The amount of material scoured from the navigation channel due to the ten bendway weirs was disappointingly small. Some 20,000 cubic meters of material remained to be dredged from the navigation channel.

6.4.1.2 Tests 10 and 11, Figure 7

All concrete bendway weir sections near the ends of the weirs were replaced with coal weirs, representing quarry tailings, while bendway weirs H and I were shortened by 33 m and 103 m respectively.

The results of tests 10 and 11 indicated improved structural stability of the bendway weirs and improved stability near the end of the bend due to the shortening of bendway weirs H and I. The improved structural stability was primarily due to elimination of many of the concrete bendway weir sections. Some erosion of the quarry tailings was still evident at the locations were coal and concrete bendway weir sections met. These tests also indicated improved self scouring characteristics of the channel with only 2000 cubic meters of material to be removed from the navigation channel. The area between the berm and the North Jetty was generally one of slight deposition. Areas of scour were generally immediately south of the berm followed by an area of slight deposition near the south edge of the navigation channel and in places scour of up to several meters further south of the channel. Scour of up to 1 m. was observed near the South Jetty.

6.4.1.3 Test 12 with Augmented Flow

The bendway weir configuration of Tests 10 and 11 was subjected to augmented flows of 30 percent which more nearly simulated the sediment transport process in Steveston Bend. (see section 4.1.5) This configuration produced a completely self scouring navigation channel throughout the length of the bend. Even at this augmented flow the berm remained stable. Some of the most upstream weirs exhibited slight erosion and some undermining at the weir noses. It is likely that the weir ends (noses) will suffer some degradation in the field as a result of the development of considerable scour holes immediately downstream of each of the bendway weirs. As the flow patterns develops in the downstream direction there appears to be a lessening of the frontal attack on the bendway weir noses such that stability becomes less problematic downstream.

The shortened bendway weirs H and I produced a channel downstream of the bend which was more nearly parallel to the North Jetty.

This test demonstrated the feasibility of using bendway weirs in shaping a self scouring navigation channel in the Steveston Bend, even in the presence of the existing berm.

To make the bendway weir system more economically feasible, additional tests were undertaken with the number of weirs reduced from 10 to 5.

6.4.2 Tests with Berm and Five Bendway Weirs (Tests 13 to 18)

6.4.2.1 Test 13, Figure 8

A reduced number of bendway weirs from 10 to 5 caused a slight deposition in the navigation channel requiring an estimated 1100 cubic meters of dredging between transects J9 and J10. A 4 m to 5 m deep scour hole was observed along the face of the North Jetty at transects J9a and J10. As the distances between the bendway weirs were approximately doubled, there was increased opportunity for the flow to produce scour holes downstream of each weir, resulting in relatively large scour holes. The upstream faces of the weirs exhibited some deposition. The nose of bendway weir I exhibited some damage due to downstream scour at the weir end.

6.4.2.2 Test 14, Figure 8

The berm elevation was lowered to -13 m LLW and the last 50 m to 100 m of each bendway weir was constructed using coal down to elevation -20 m, simulating the quarry tailings. The flow was not augmented as the stability of the weirs was being investigated.

The test demonstrated that the bendway weirs will be stable when constructed directly on a sand bed at elevation -20 m LLW. No erosion or undermining was evident at any of the bendway weirs including the weir noses.

As the D50 of the bed material was too coarse, the flow was unable to self scour the navigation channel. At the end of the test a total of 67,000 cubic meters of material would have had to be removed to dredge the 248 m wide channel down to -8.1 m LLW.

6.4.2.3 Test 15, Figure 8

Test 14 was repeated with rock rather than coal bendway weirs to prevent erosion and with an augmented flow of 30%, resulting in the removal of 62,000 cubic meters of sedimentary material, leaving 5000 cubic meters in the navigation channel at transects J9 and J10. As the weirs are relatively far apart (in excess of 300 m), they do not create a continuously scoured channel along the noses of the weirs as observed with the 10 weirs, but develop independent scour holes which do not extend to the next weir. Hence deposition resulted near the ends of weirs E and G. Correction of this deficiency required lengthening of weirs C, E and G.

6.4.2.4 Tests 16 & 17, Figure 9

With the bendway weirs extended as shown in Figure 9, Test 15 was repeated, resulting in a navigation channel requiring negligible dredging. The weir noses showed some sign of undermining but were all intact.

Test 16 was repeated to permit further examination of the weir nose erosion process and the long term development of the navigation channel. The results of Test 17 were almost identical to those of Test 16. The navigation channel was scoured another 500 cubic meters, but the final bed conditions looked identical.

6.4.2.5 Test 18, Figure 9

This test was undertaken with the 5 bendway weirs raised by 1 meter to an elevation of -10 m LLW with the berm at elevation -11 m LLW. Raising the weir by 1 meter did not have the expected effect of reducing the dredging load in the navigation channel. Although most of the scour depths below the navigation channel had increased, the depths at the south edge of the navigation channel at transects J9 and J9a were shallower resulting in an overall increased dredging load of 5800 m^3.

6.5 Velocity Contours (Isovels)

6.5.1 Introduction

Isovels were plotted representing the surface velocities for three tests: #2, #11 and #17.

The configuration of test #2 consisted of the berm alone at crest elevation -11 m LLW. The instantaneous discharge was 11192 cms, while the water surface elevation was 2.0 m LLW.

The configuration of test #11 consisted of the berm and 10 bendway weirs, all at crest elevation -11 m LLW. The instantaneous discharge was 11123 cms, while the water surface elevation was 2.05 m LLW.

The configuration of test #17 consisted of the berm at crest elevation of -13 m LLW and five (5) bendway weirs at crest elevation -11 m LLW. The instantaneous discharge was 11188 cms, augmented by 30%, ie 15700 cms, while the water surface elevation was 2.07 m LLW.

To compare isovels, the actual velocities of test 17 were reduced by 30%.

6.5.2 Comments on Isovels of Test #2 (Figure 31)

The isovels follow Steveston Bend in a well defined pattern, see in particular the isovels of 2, 1.75 and 1.5 m/s.

The isovels of 2.25 and 2.5 m/s give an indication that the flow impinges on the North Jetty somewhere between transects J8 and J8a.

The higher velocities, ie those greater than 2 m/s, are concentrated near the North Jetty, and largely follow the northern edge of the navigation channel, with typical velocities ranging from 2 to 2.5 m/s. The velocities decrease rapidly towards the southern edge of the navigation channel, with typical velocities ranging from 1 to 1.5 m/s.

The flow approaches the bend with a definite transverse velocity gradient from north to south. The gradient is actually more pronounced at the beginning of the bend (0.0055 m/s/m) than at the end of the bend (0.0019 m/s/m)

6.5.3 Comments on Isovels for Test #11 (Figure 32)

The velocities along the northern edge of the navigation channel are approximately 1.5 to 1.75 m/s, while at the southern edge they are 1 to 1.5 m/s. Velocities equal to or greater than 2 m/s are observed near the middle of the bend. High local velocities are observed behind and near the noses of the weirs, particularly near weirs J5a and J7. This pattern was observed in most of the tests at low water levels and high discharges.

6.5.4 Comparison of Test #2 and Test #11 Isovels

6.6 Observed Surface Flow Patterns

When the submerged weirs are present the flow through the bend can be classified into four zones, as follows (regardless of the Hope discharge):

Zone 1:

Defined as an undisturbed zone upstream of the first weir.
In this zone the streamlines develop "naturally" with minimum interference from the first weir.

Zone 2:

Defined as an undisturbed zone south of the southern limit of the navigation channel, all along the bend. In this zone the streamlines are relatively unaffected by the weirs.

Zone 3:

Defined as the zone between the southern limit of the navigation channel and the North Jetty, all along the bend.
Two subzones can be identified:

a) near and along the nose of the weirs, and

b) the zone between the North Jetty and subzone a).

Subzone a:

- A swirling pattern is observed at the nose of the weirs, all along the bend.
  The swirls transform into eddies when the discharge is high and the water surface elevation is low. This pattern is most pronounced when the Hope discharge is high. The swirls disappear within 200 m downstream of the last weir. The size of the largest eddies ranged from a length of 18 m to 30 m. They had an angular velocity of 2 rpm to 2.4 rpm.

Subzone b:

- A swirling pattern is also observed behind and at the middle of the weirs, together with a re-orientation of the velocity vectors causing a complex streamline pattern. At high discharges and low water surface elevations the swirls become eddies. The eddies in this zone are in general larger but have smaller angular velocity.
  The size of the largest eddies ranged from a length of 21 m to 47 m. They had an angular velocity of 1 rpm to 0.6 rpm.

<u>Zone 4</u>:

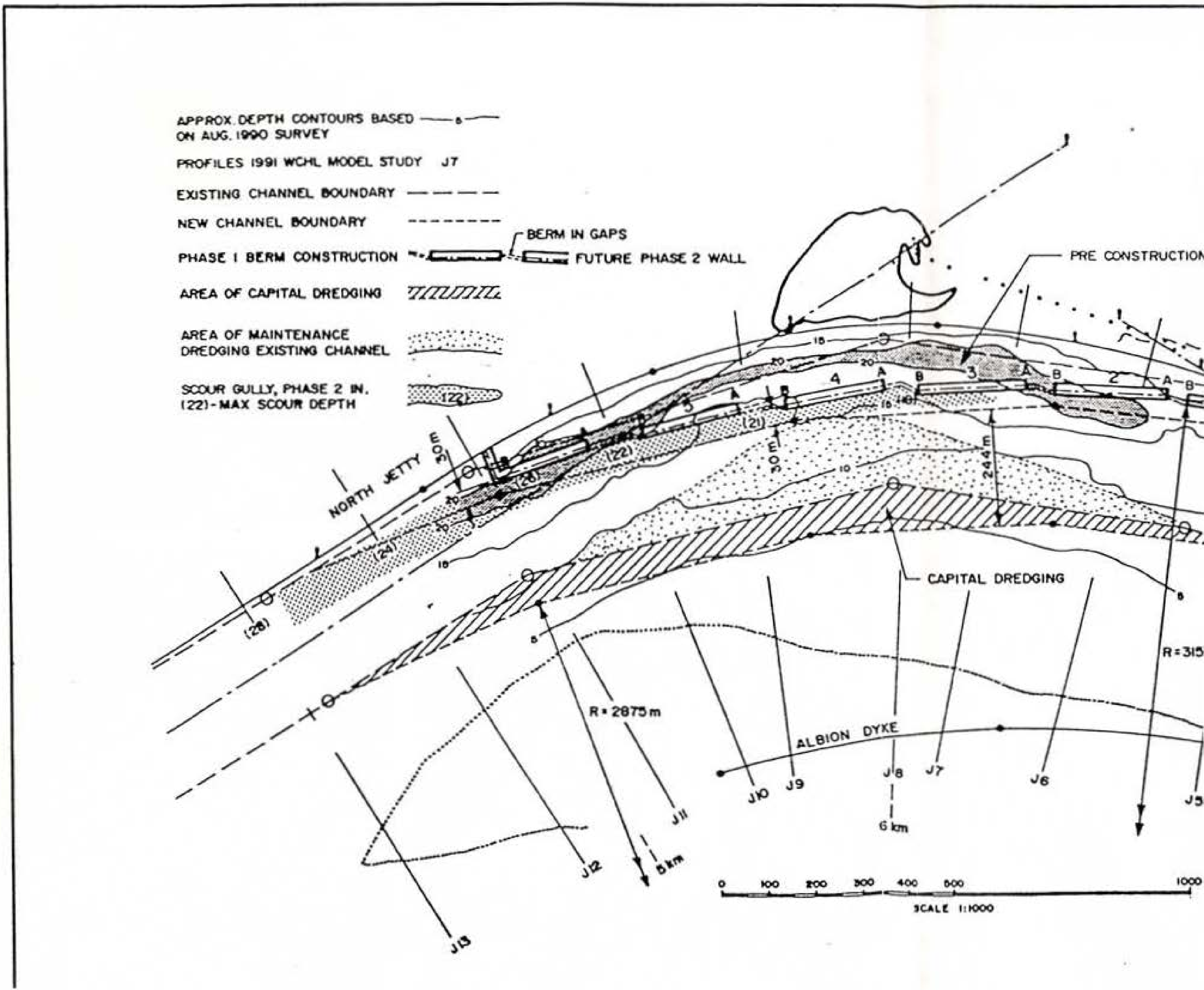
- In this zone which is the zone downstream of the last weir the swirling activity gradually dies down.

Prepared by ______________________

W. Schriek

Managing Director

PRELIMINARY



APPROX. DEPTH CONTOURS BASED ON AUG. 1990 SURVEY
PROFILES 1991 WCHL MODEL STUDY J7
EXISTING CHANNEL BOUNDARY
NEW CHANNEL BOUNDARY
BERM IN GAPS
PHASE I BERM CONSTRUCTION
FUTURE PHASE 2 WALL
AREA OF CAPITAL DREDGING
AREA OF MAINTENANCE DREDGING EXISTING CHANNEL
SCOUR GULLY, PHASE 2 IN. (22)-MAX SCOUR DEPTH
PRE CONSTRUCTION
NORTH JETTY
30 m
244 m
CAPITAL DREDGING
R=2875m
R=315
ALBION DYKE
J5
J6
J7
J8
J9
J10
J11
J12
J13
5 km
6 km
SCALE 1:1000
0 100 200 300 400 500 1000

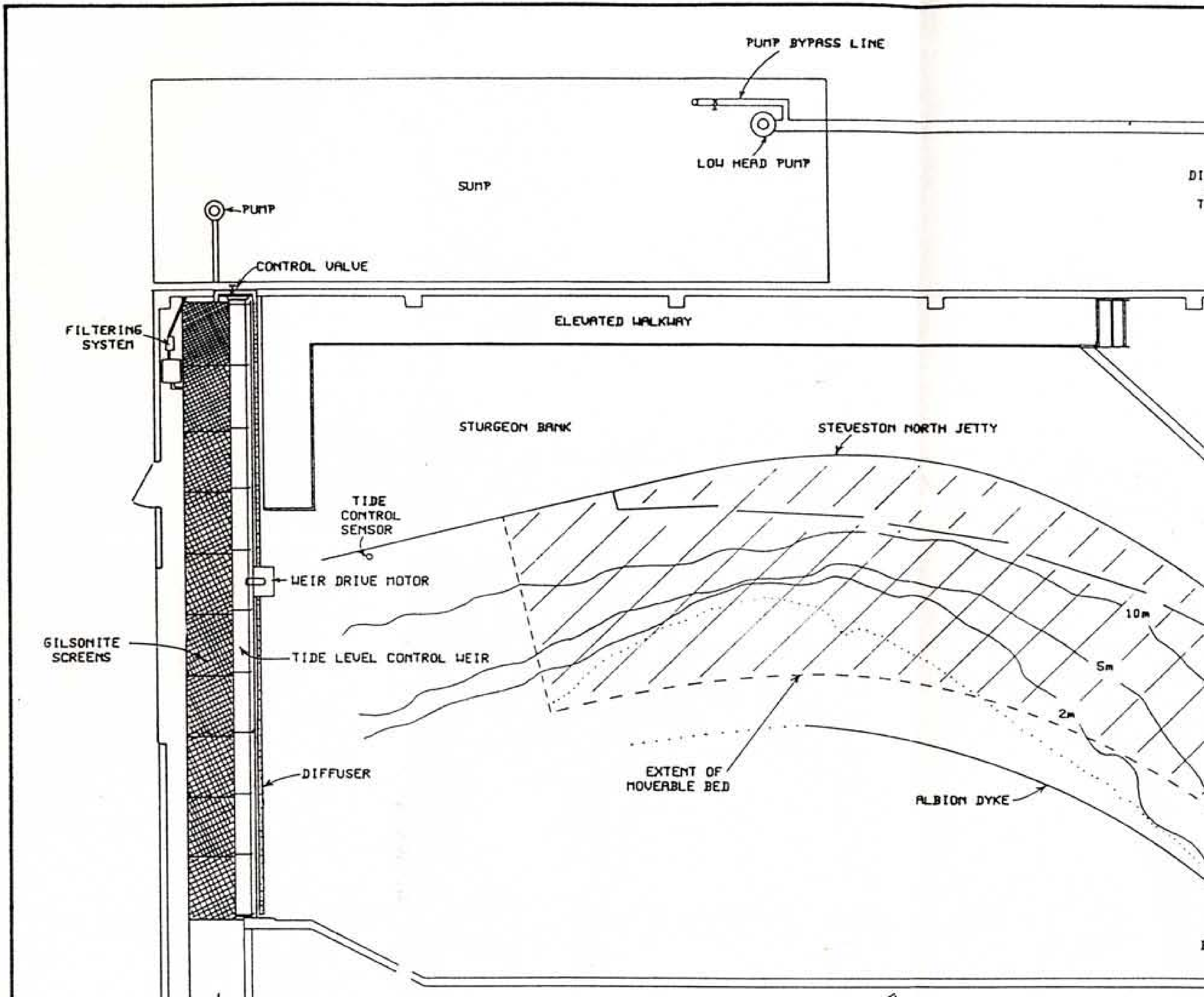
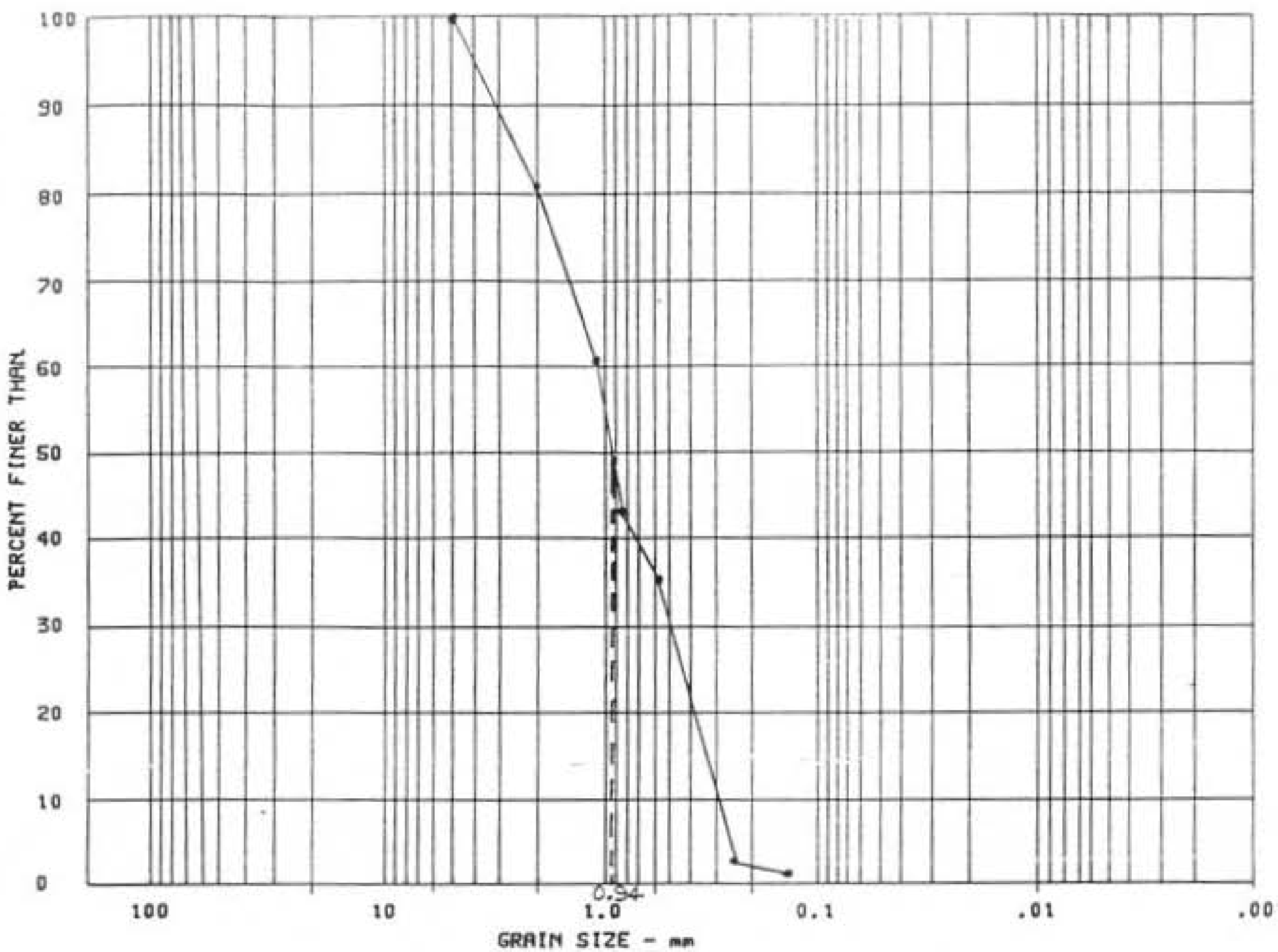

PUMP BYPASS LINE
LOW HEAD PUMP
SUMP
PUMP
CONTROL VALVE
FILTERING SYSTEM
ELEVATED WALKWAY
STURGEON BANK
STEVESTON NORTH JETTY
TIDE CONTROL SENSOR
WEIR DRIVE MOTOR
GILSONITE SCREENS
TIDE LEVEL CONTROL WEIR
10m
5m
2m
DIFFUSER
EXTENT OF MOVEABLE BED
ALBION DYKE

TABLE 1
TEST PROGRAM

Test No.	Hope Disch. m^3/s	Total Time hrs	Berm Elev. m LLW	Bendway Weirs No.	Weir Elev. m LLW	Weir Side Slopes	Weir Location	Berm/Bendway Weir Construction	Surface Vel.	Bed Contours
1	5700	96	-11	N/A				Coal berm to lab floor representing	Yes	No
2	12100	96	-11	N/A				St. Mungo Bend gravel.	Yes	No
3	5700	96	-11	N/A					Yes	No
4	12100	96	-11	N/A				Coal berm to lab floor representing	Yes	No
5	12100	96	-11	N/A				quarry tailings.	Yes	No
6	12100	96	-11	N/A					Yes	No
7	5700	96	-11	5	-11	3H-1V	Figure 5	Coal berm to lab floor representing	Yes	No
8	12100	96	-11	5	-11	3H-1V	Figure 5	quarry tailings. Concrete bendway weirs.	Yes	No
9	5700	96	-11	10	-11	2H-1V	Figure 6	Coal berm to lab floor. Concrete and coal bendway	Yes	No
10	5700	96	-11	10	-11	2H-1V	Figure 7	weir sections placed to a depth of -20 m LLW.	Yes	No
11	12100	96	-11	10	-11	2H-1V	Figure 7	Coal berm and bendway weirs.	Yes	No
12	15700	96	-11	10	-11	2H-1V	Figure 7	Coal berm and rock weirs.	No	No
13	15700	96	-11	5	-11	2H-1V	Figure 8	Coal berm and rock weirs.	No	No
14	12100	96	-13	5	-11	2H-1V	Figure 8	Coal berm and bendway weirs.	Yes	Yes
15	15700	96	-13	5	-11	2H-1V	Figure 8	Coal berm and rock weirs.	Yes	Yes
16	15700	96	-13	5	-11	2H-1V	Figure 9	Coal berm and rock weirs.	No	No
17	15700	96	-13	5	-11	2H-1V	Figure 9	Coal berm and rock weirs.	Yes	Yes
18	15700	96	-11	5	-10	2H-1V	Figure 9	Coal berm and rock weirs.	Yes	Yes

FIGURE 3



COBBLE	COURSE	MEDIUM	FINE	COURSE	MEDIUM	FINE	SILT SIZE	CLAY
SIZE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

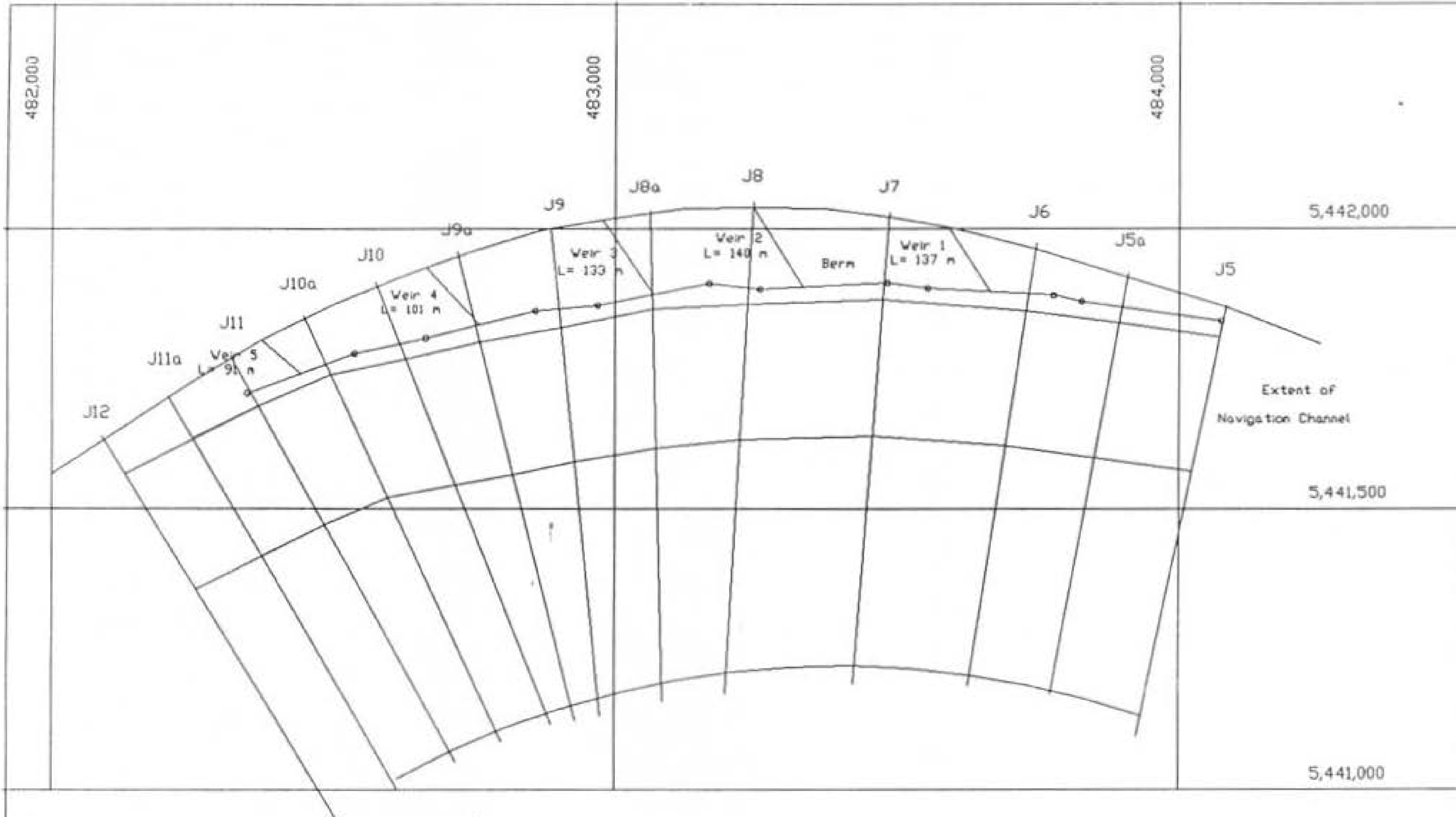
WESTERN CANADA HYDRAULIC LABORATORIES LTD.

1:150 SCALE MODEL

DATA AQUISITION AND MODEL CONTROL INSTRUMENTATION

STEVESTON BENDWAY WEIRS

TEST FACILITIES



Scale 1:10,000

Note: Bendway Weirs El.: -11 m LLW

Berm El.: -11 m LLW

STEVESTON BENDWAY WEIRS

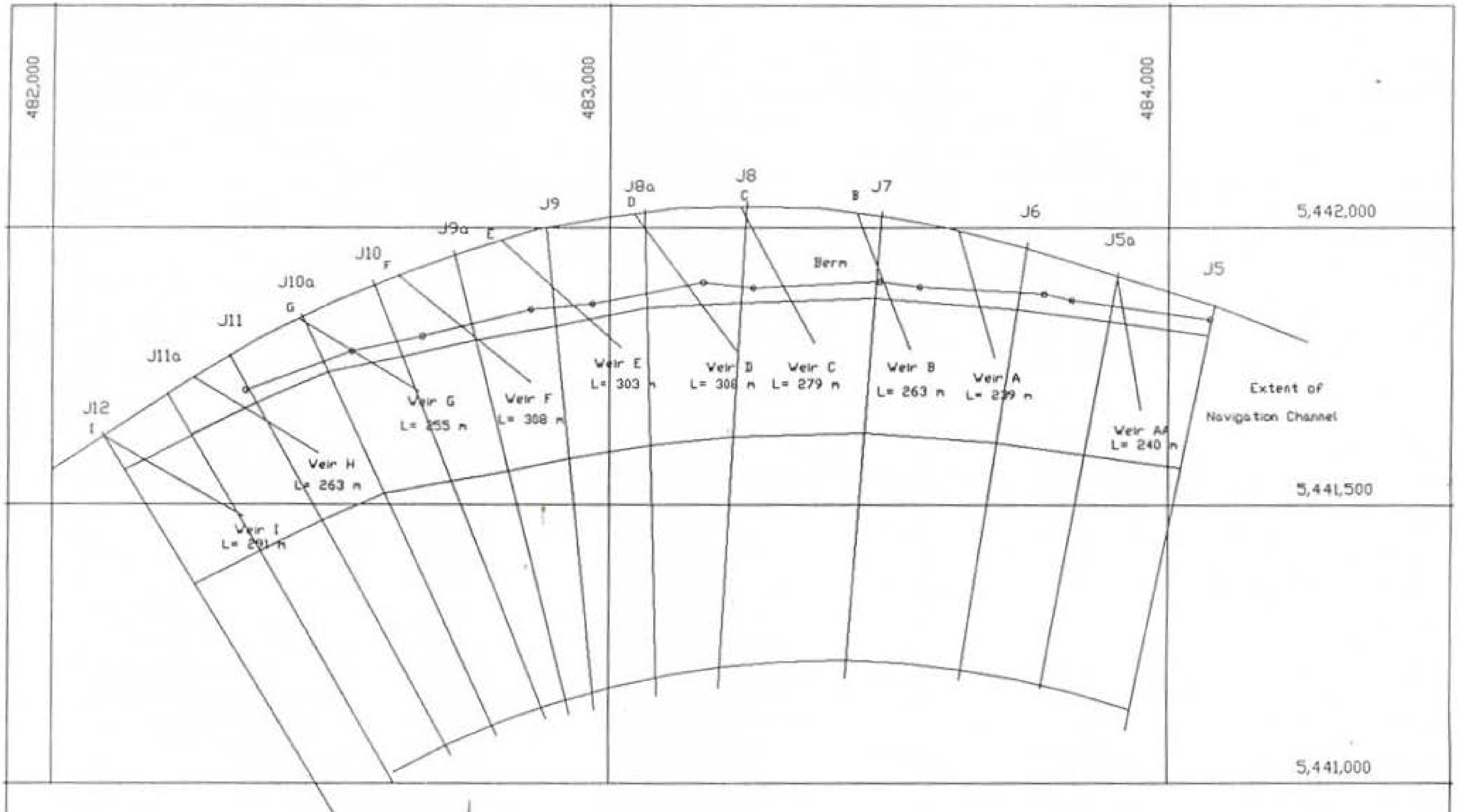
TEST #7 & #8

CONFIGURATION

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

FIGURE 5

FIGURE 6



Scale 1:10,000

Note: Bendway Weirs El.: -10 m LLW

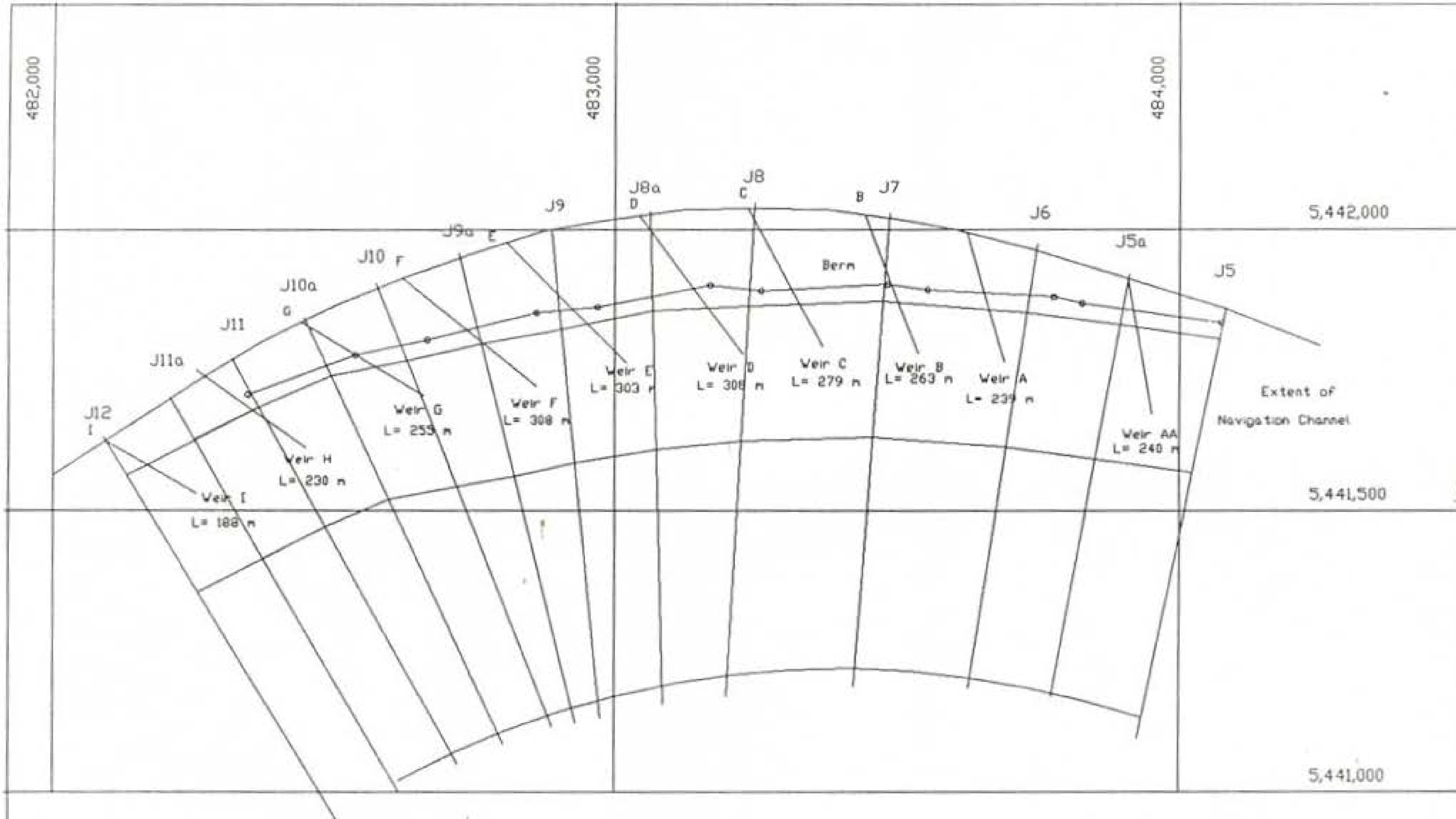
Berm El.: -11 m LLW

STEVESTON BENDWAY WEIRS

Test #9

CONFIGURATION

WESTERN CANADA HYDRAULIC LABORATORIES LTD.



Scale 1:10,000

Note: Bendway Weirs El.: -10 m LLW

Berm El.: -11 m LLW

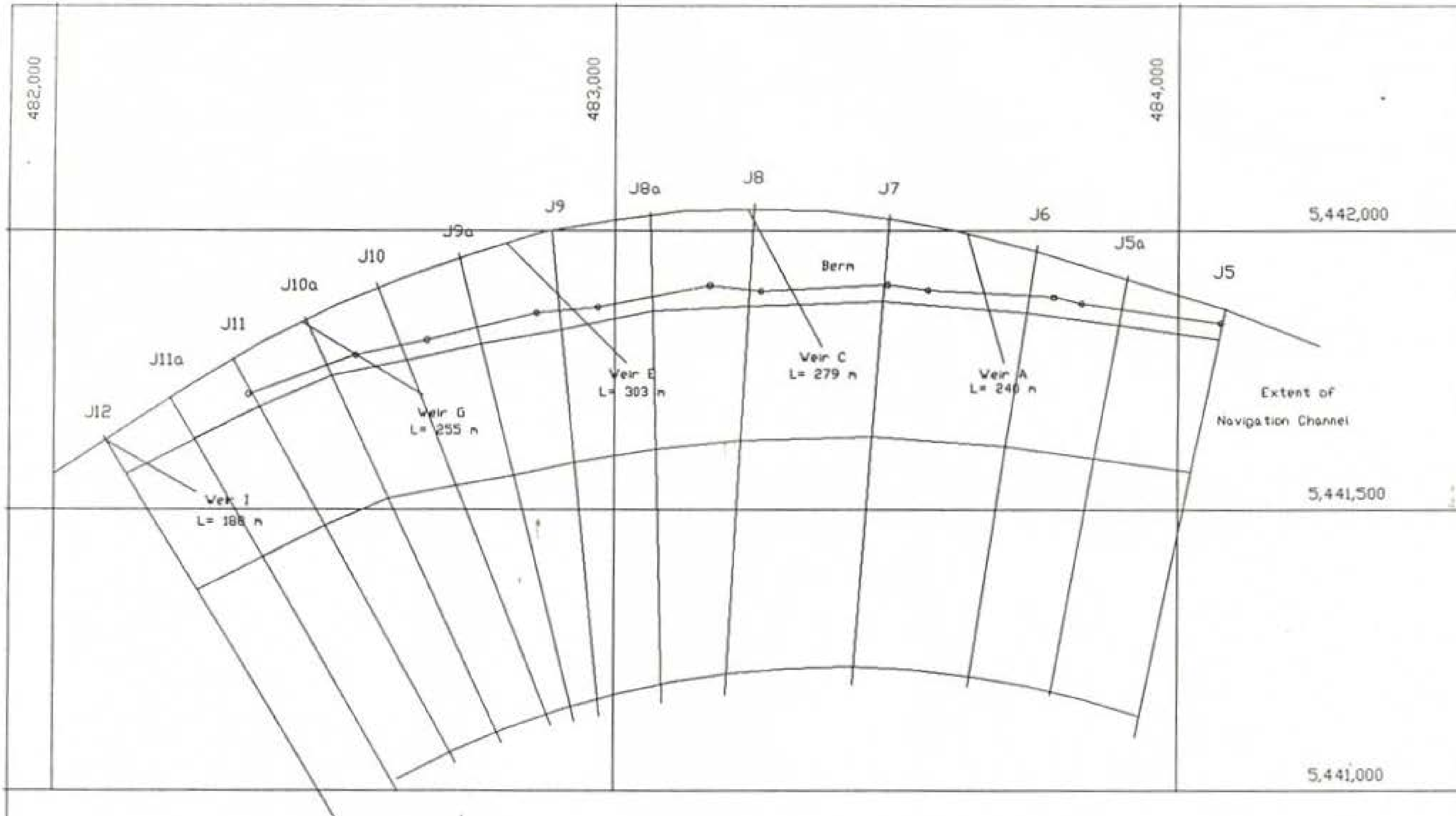
STEVESTON BENDWAY WEIRS

Test #10, #11, #12

CONFIGURATION

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

FIGURE 7



Scale 1:10,000

Note: Bendway Weirs El.: Test #13, -10 m LLW ; Test #14 & #15, -11 m LLW
Berm El.: Test #13, -11 m LLW ; Test #14 & #15, -13 m LLW

STEVESTON BENDWAY WEIRS

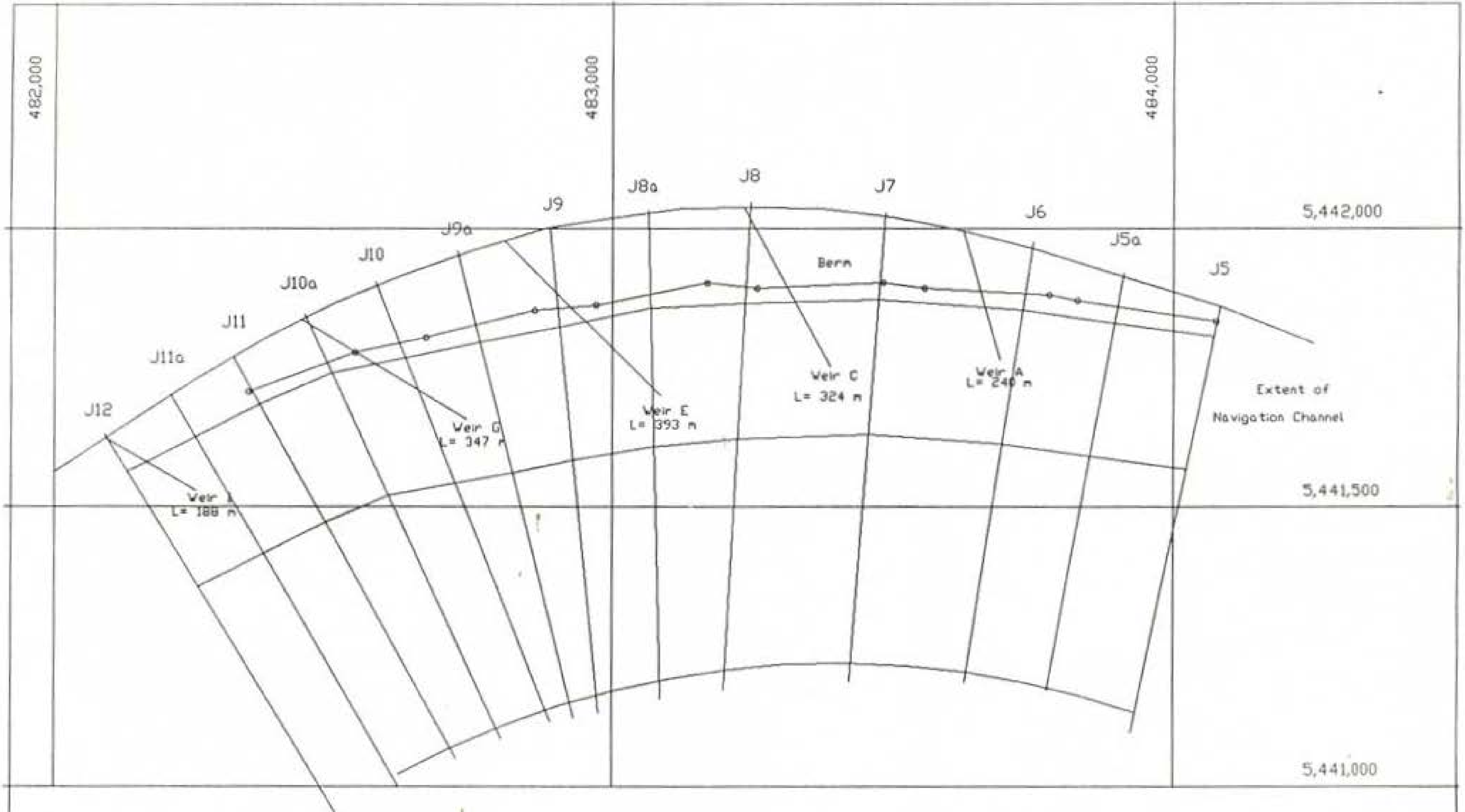
TEST #13, #14, #15

CONFIGURATION

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

FIGURE 8

FIGURE 9



Scale 1:10,000

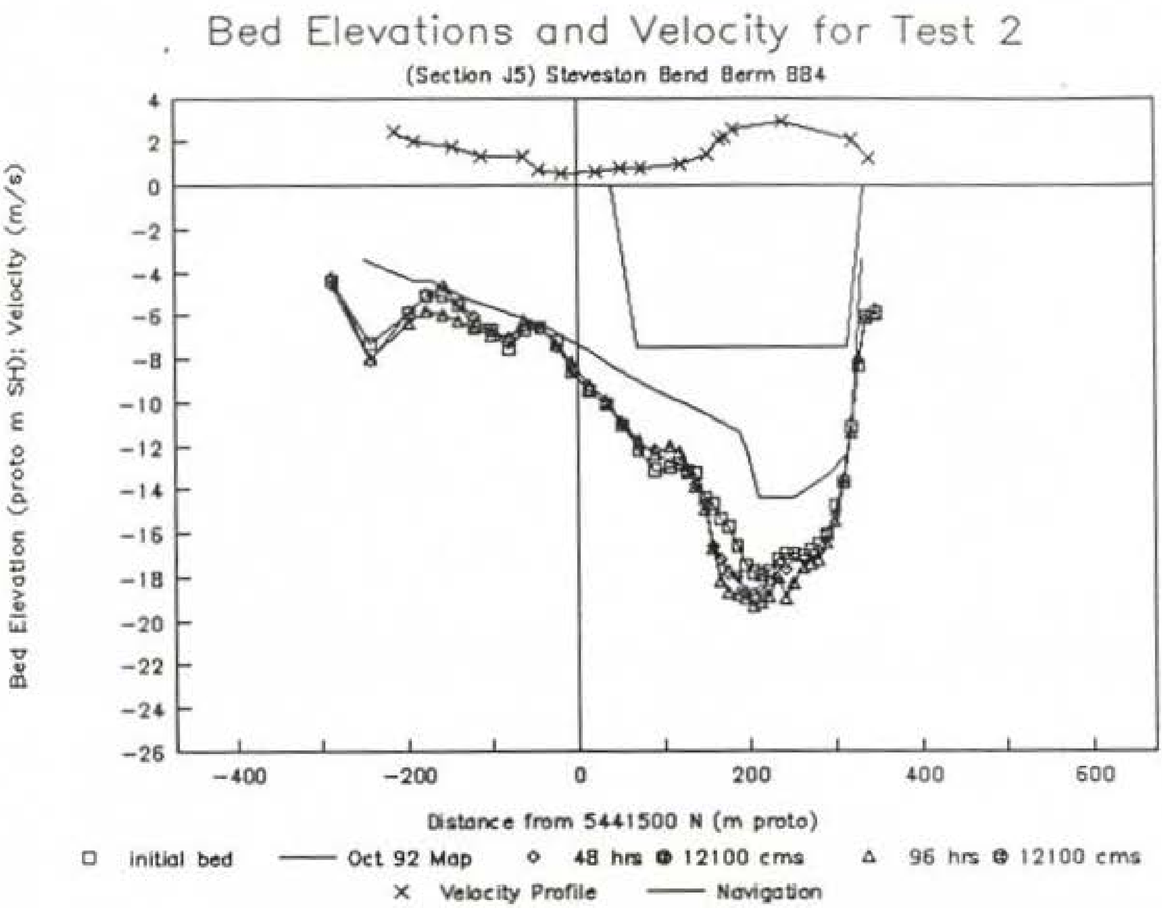
Note: Bendway Weirs El.: Test #16 & #17, -11 m LLW ; Test #18, -10 m LLW
Berm El.: Test #16 & #17, -13 m LLW ; Test #18, -11 m LLW

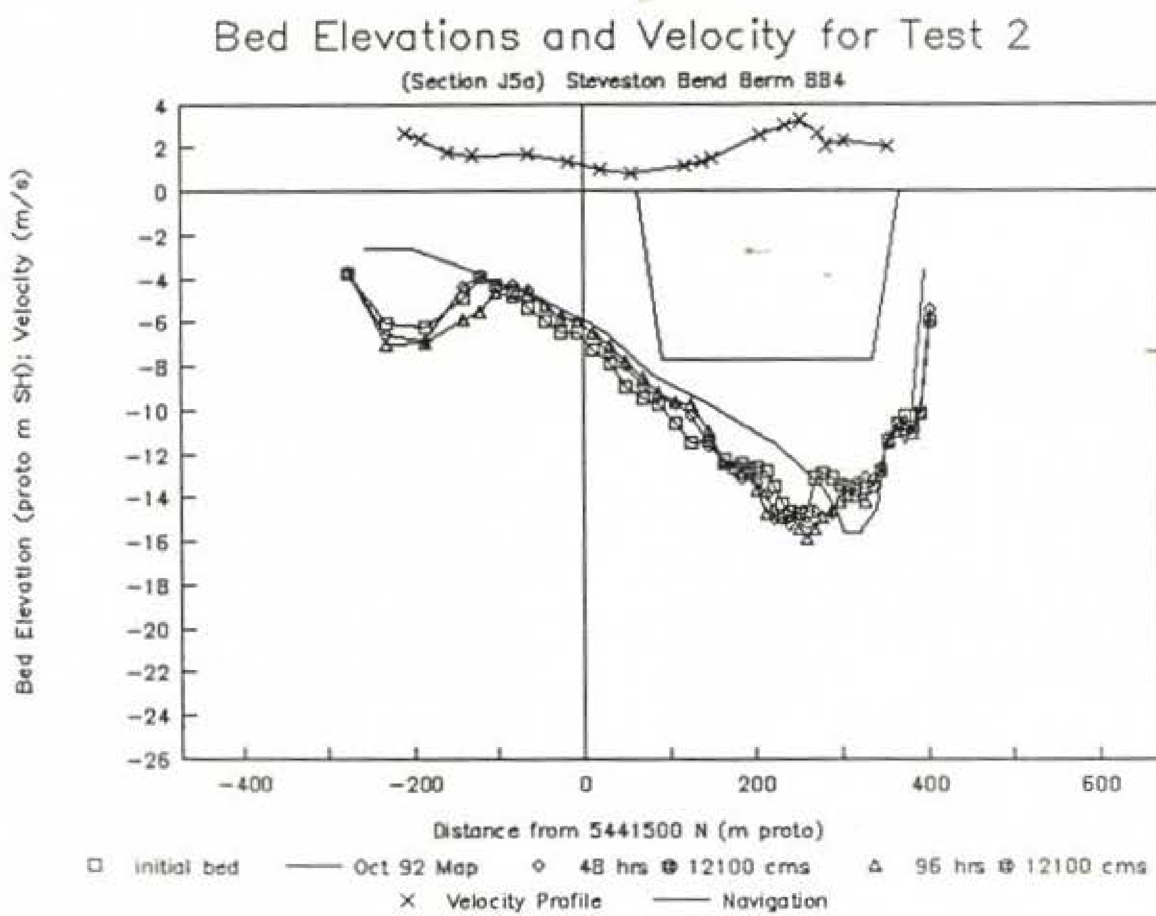
STEVESTON BENDWAY WEIRS

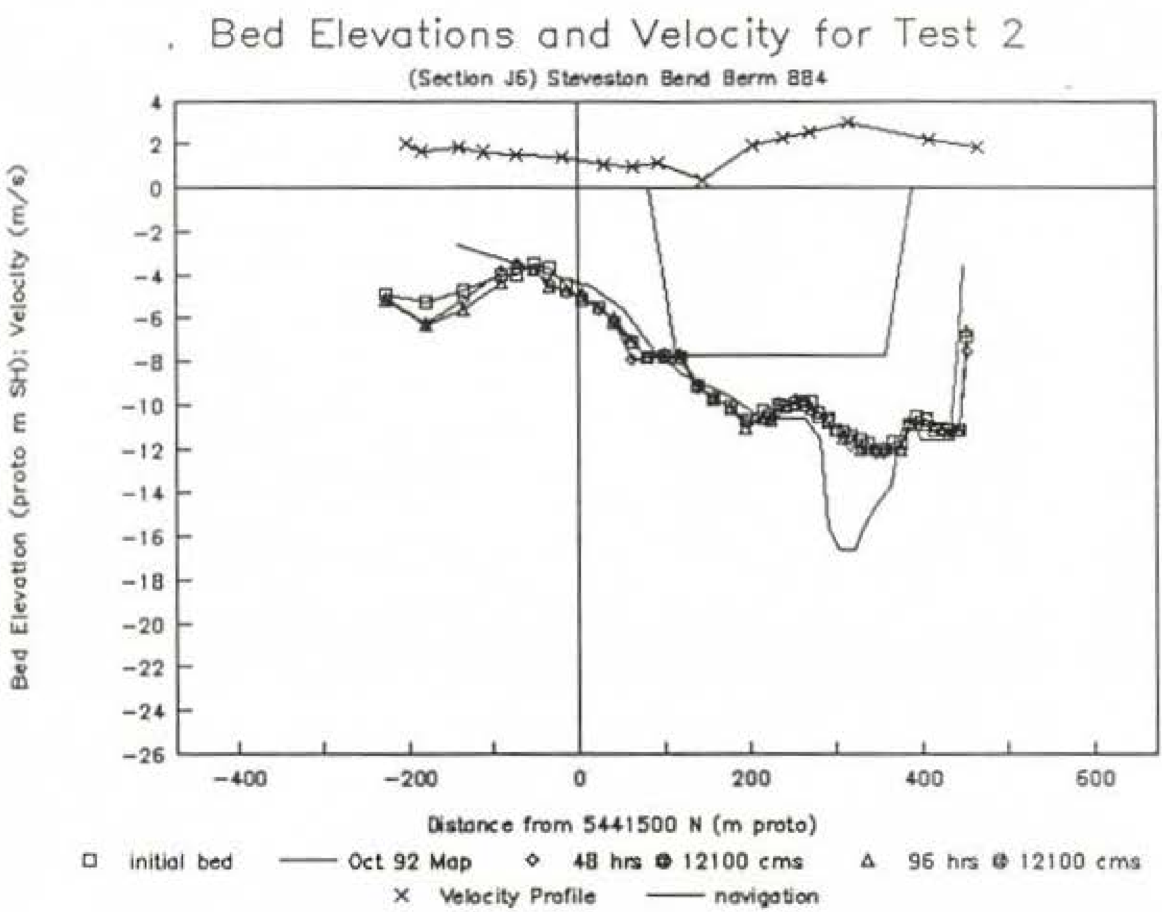
TEST #16, #17, #18

CONFIGURATION

WESTERN CANADA HYDRAULIC LABORATORIES LTD.


Bed Elevations and Velocity for Test 2
(Section J5) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
initial bed
Oct 92 Map
48 hrs @ 12100 cms
96 hrs @ 12100 cms
Velocity Profile
Navigation


Bed Elevations and Velocity for Test 2
(Section J5a) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
initial bed
Oct 92 Map
48 hrs @ 12100 cms
96 hrs @ 12100 cms
Velocity Profile
Navigation


Bed Elevations and Velocity for Test 2
(Section J6) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
initial bed
Oct 92 Map
48 hrs @ 12100 cms
96 hrs @ 12100 cms
Velocity Profile
navigation

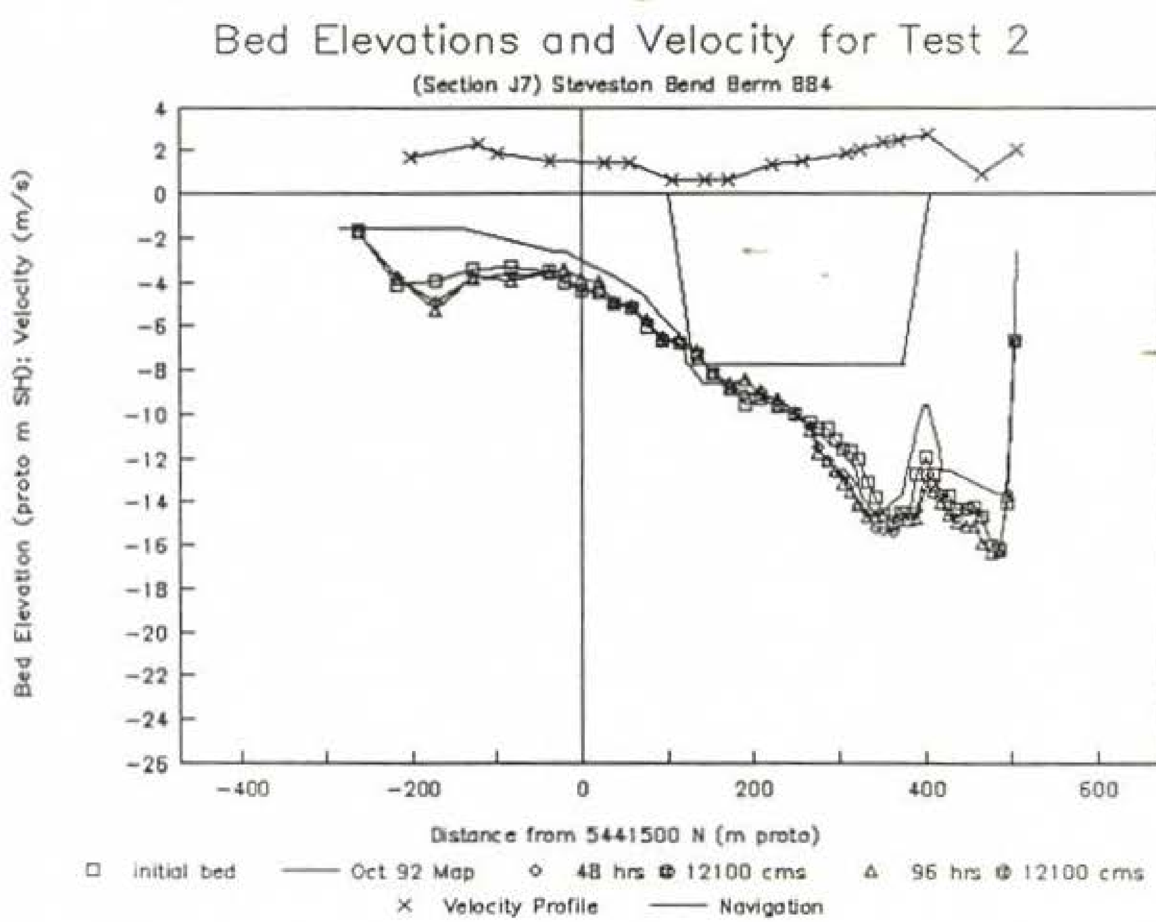
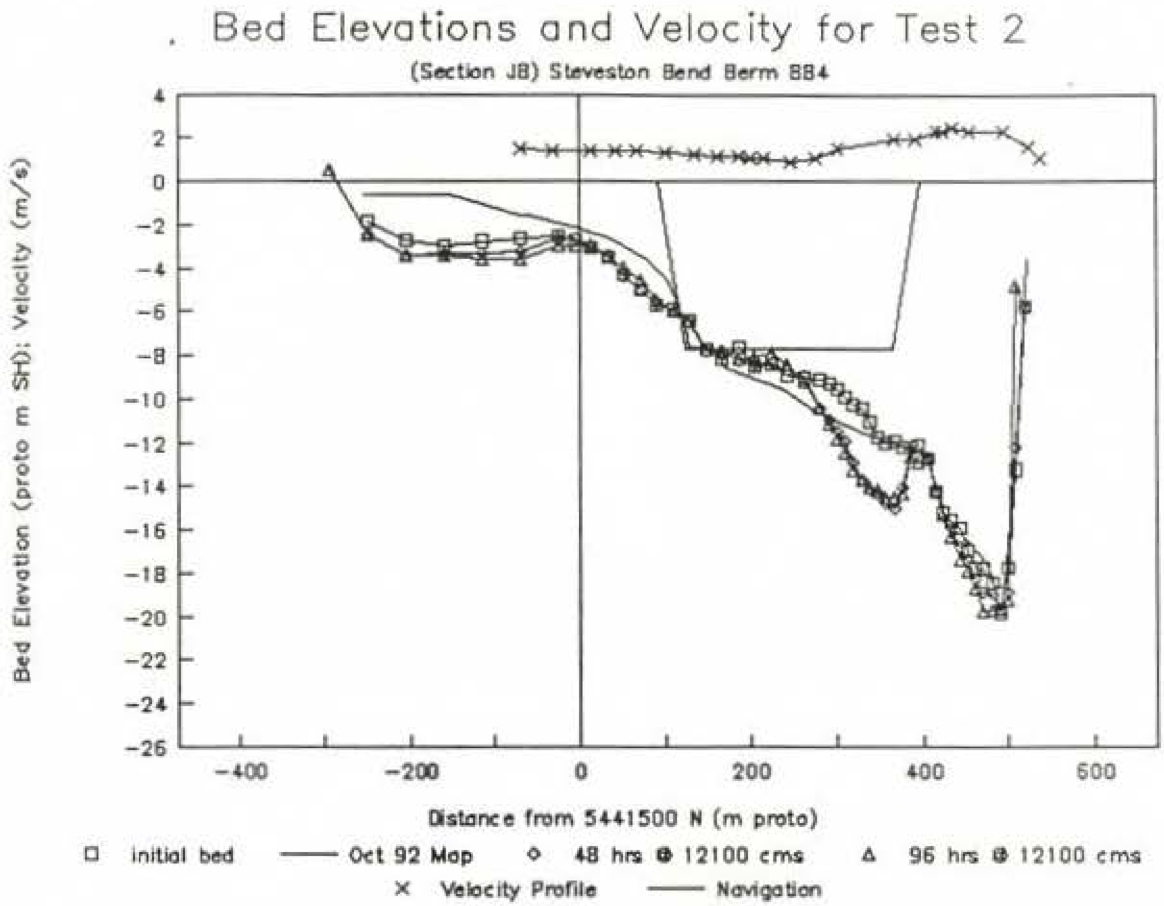

Bed Elevations and Velocity for Test 2
(Section J7) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
initial bed
Oct 92 Map
48 hrs @ 12100 cms
96 hrs @ 12100 cms
Velocity Profile
Navigation

FIGURE 12


Bed Elevations and Velocity for Test 2
(Section JB) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
Distance from 5441500 N (m proto)
initial bed
Oct 92 Map
48 hrs @ 12100 cms
96 hrs @ 12100 cms
Velocity Profile
Navigation

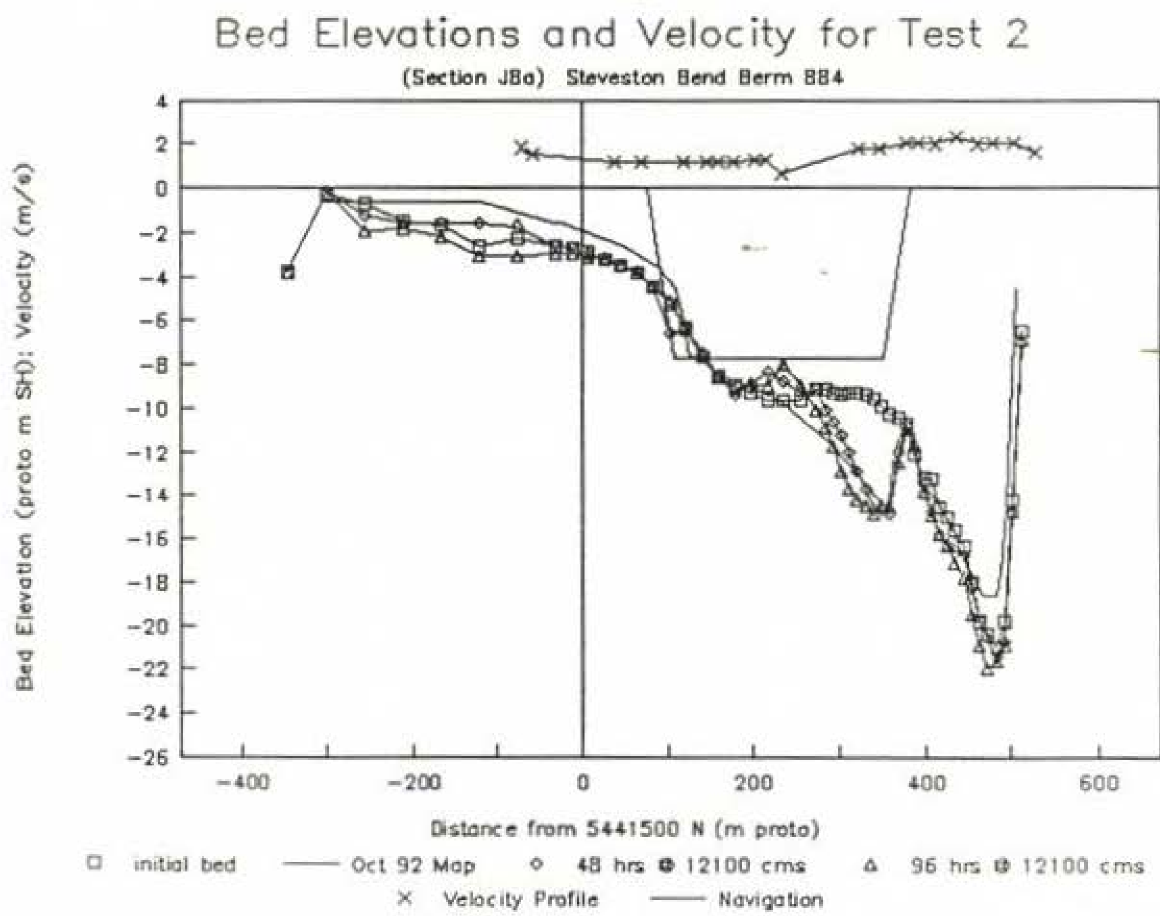
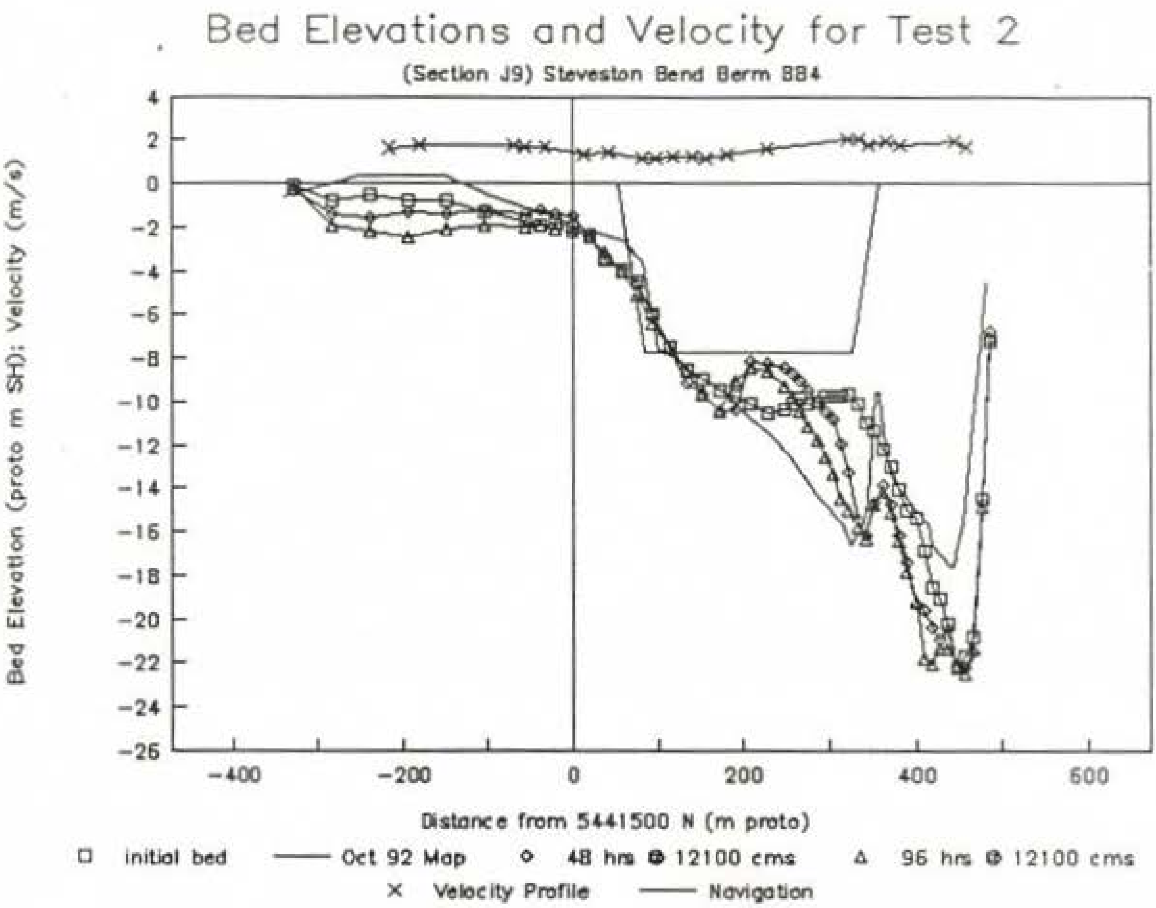
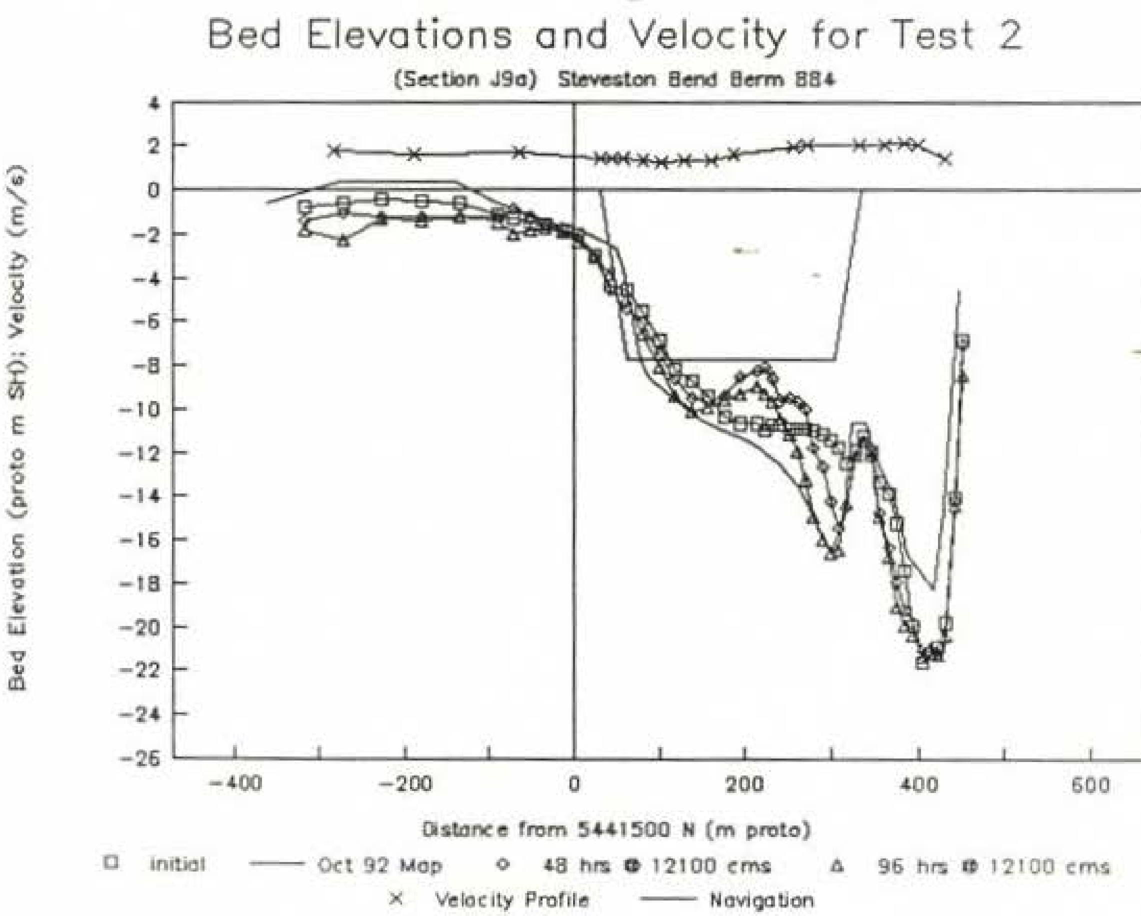

Bed Elevations and Velocity for Test 2
(Section JBa) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
Distance from 5441500 N (m proto)
initial bed
Oct 92 Map
48 hrs @ 12100 cms
96 hrs @ 12100 cms
Velocity Profile
Navigation

FIGURE 13



Bed Elevations and Velocity for Test 2
(Section J9) Steveston Bend Berm BB4
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
initial bed
Oct 92 Map
48 hrs @ 12100 cms
96 hrs @ 12100 cms
Velocity Profile
Navigation



Bed Elevations and Velocity for Test 2
(Section J9a) Steveston Bend Berm BB4
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
initial
Oct 92 Map
48 hrs @ 12100 cms
96 hrs @ 12100 cms
Velocity Profile
Navigation

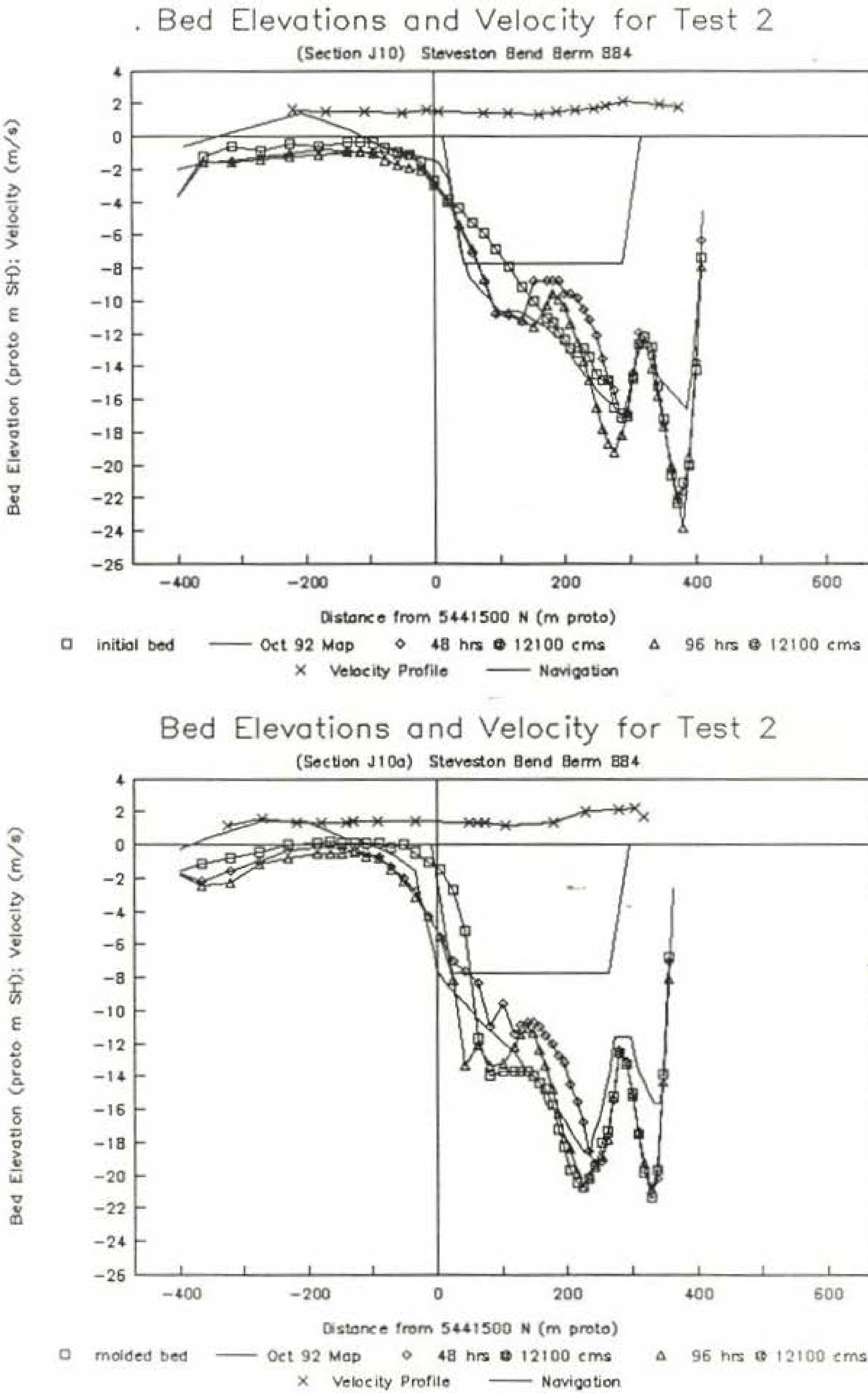
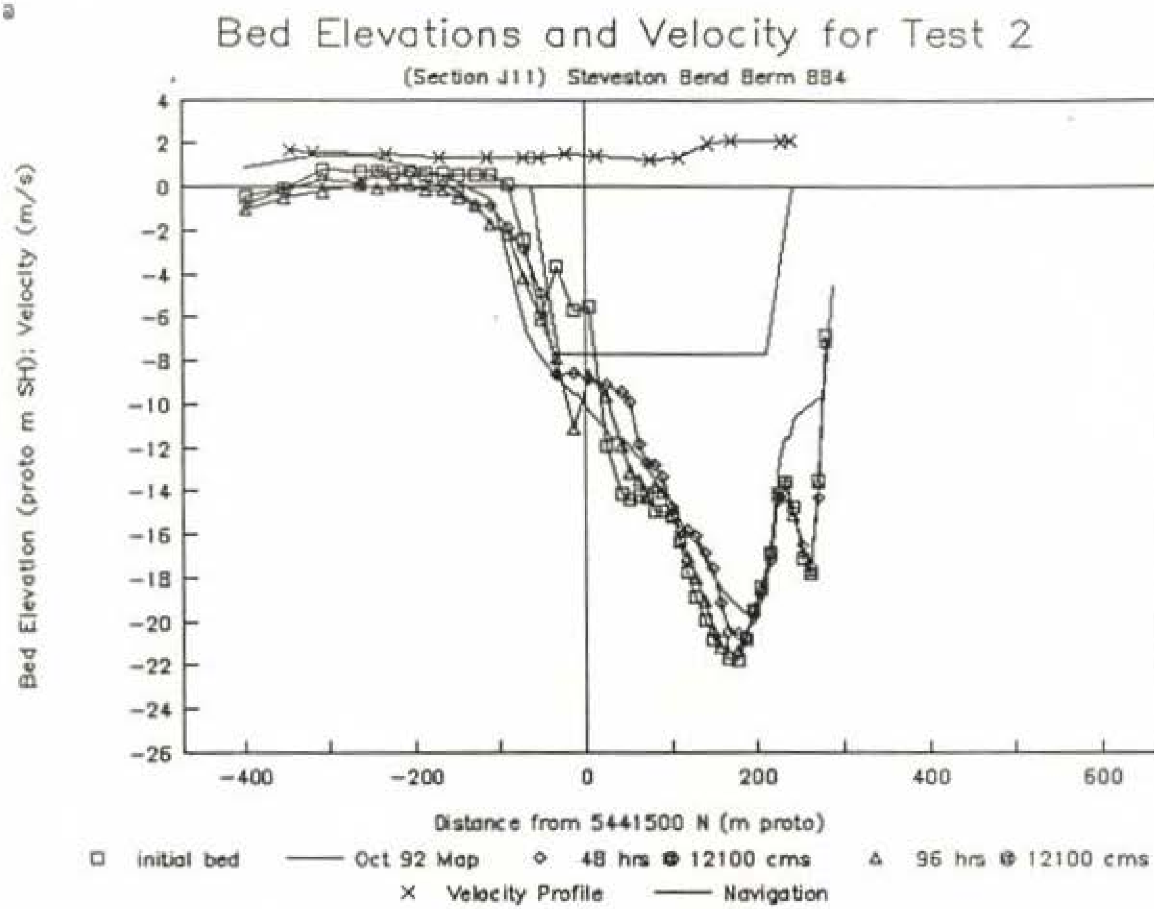
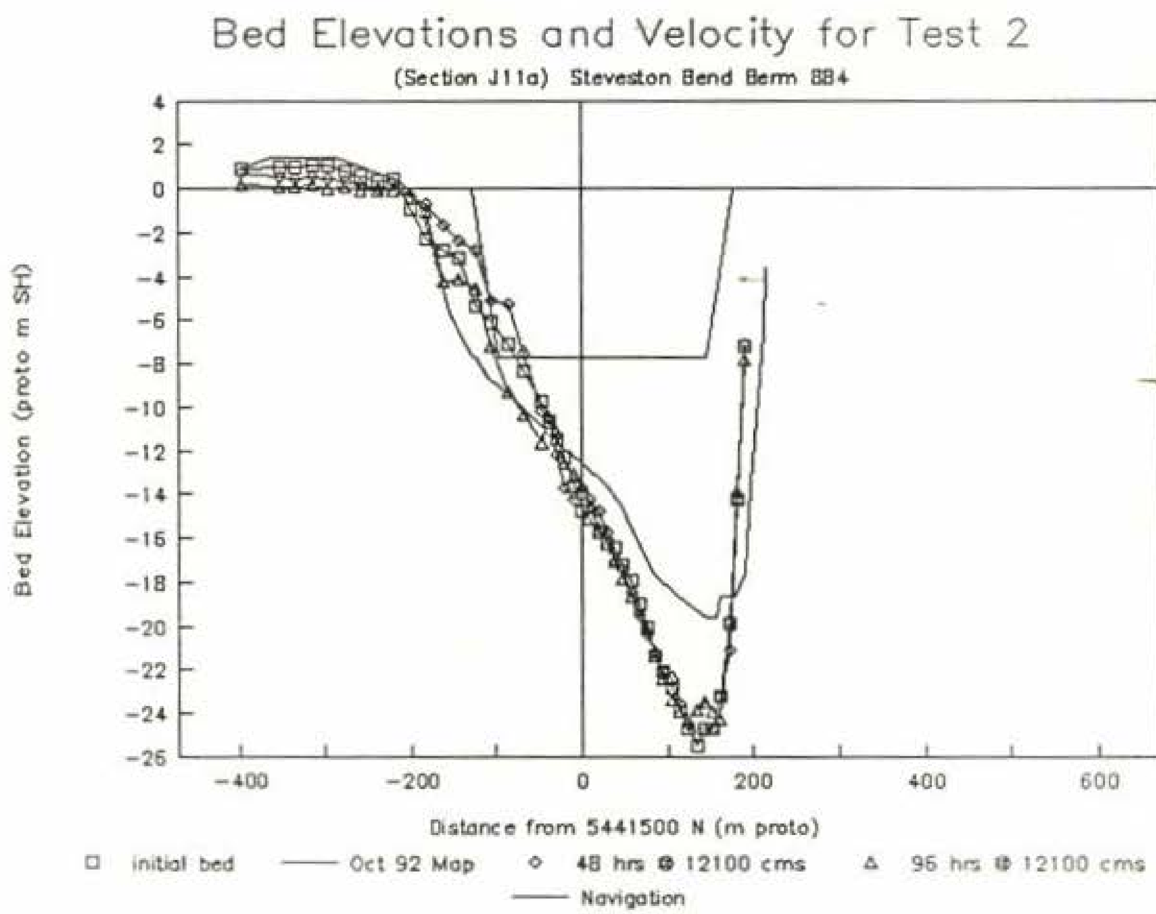

. Bed Elevations and Velocity for Test 2
(Section J10) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
Distance from 5441500 N (m proto)
initial bed
Oct 92 Map
48 hrs @ 12100 cms
96 hrs @ 12100 cms
Velocity Profile
Navigation
Bed Elevations and Velocity for Test 2
(Section J10a) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
Distance from 5441500 N (m proto)
molded bed
Oct 92 Map
48 hrs @ 12100 cms
96 hrs @ 12100 cms
Velocity Profile
Navigation

FIGURE 15


Bed Elevations and Velocity for Test 2
(Section J11) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
Distance from 5441500 N (m proto)
initial bed
Oct 92 Map
48 hrs @ 12100 cms
96 hrs @ 12100 cms
Velocity Profile
Navigation


Bed Elevations and Velocity for Test 2
(Section J11a) Steveston Bend Berm 884
Bed Elevation (proto m SH)
Distance from 5441500 N (m proto)
initial bed
Oct 92 Map
48 hrs @ 12100 cms
96 hrs @ 12100 cms
Navigation

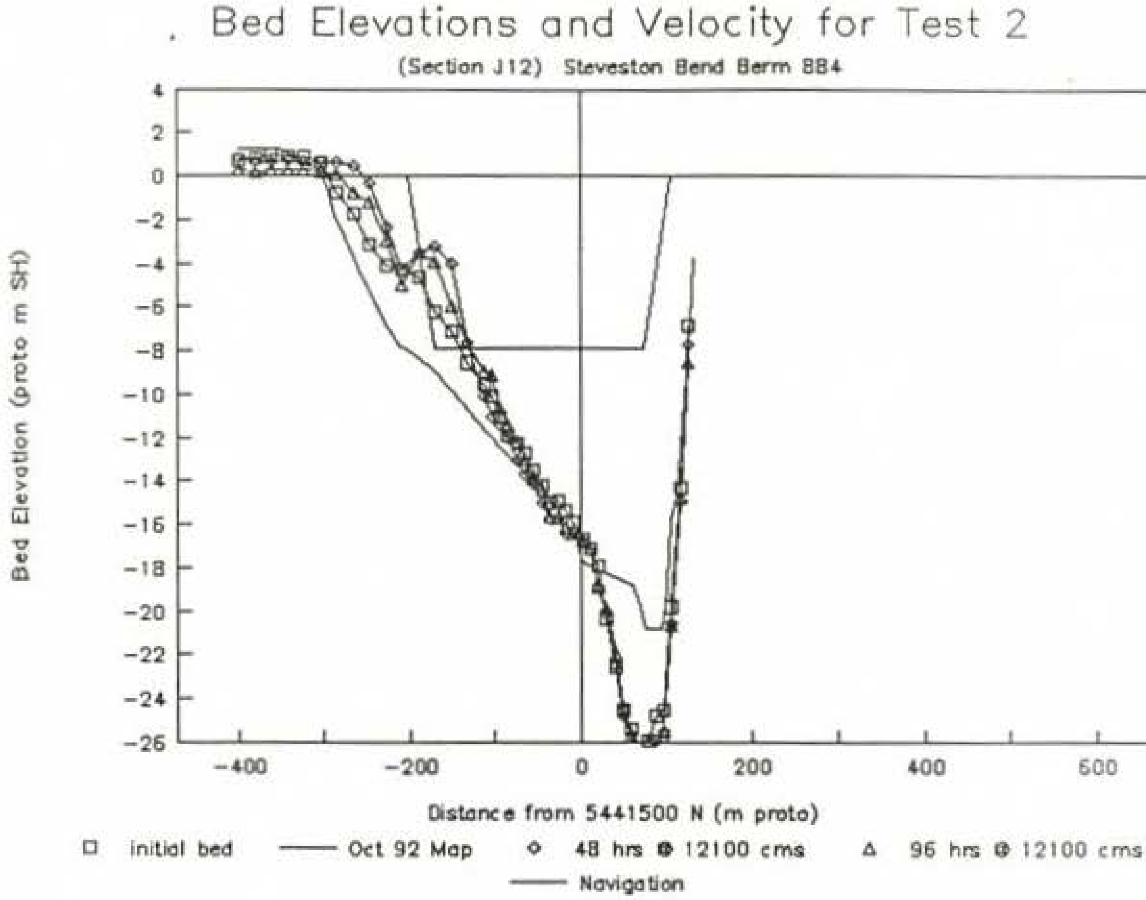
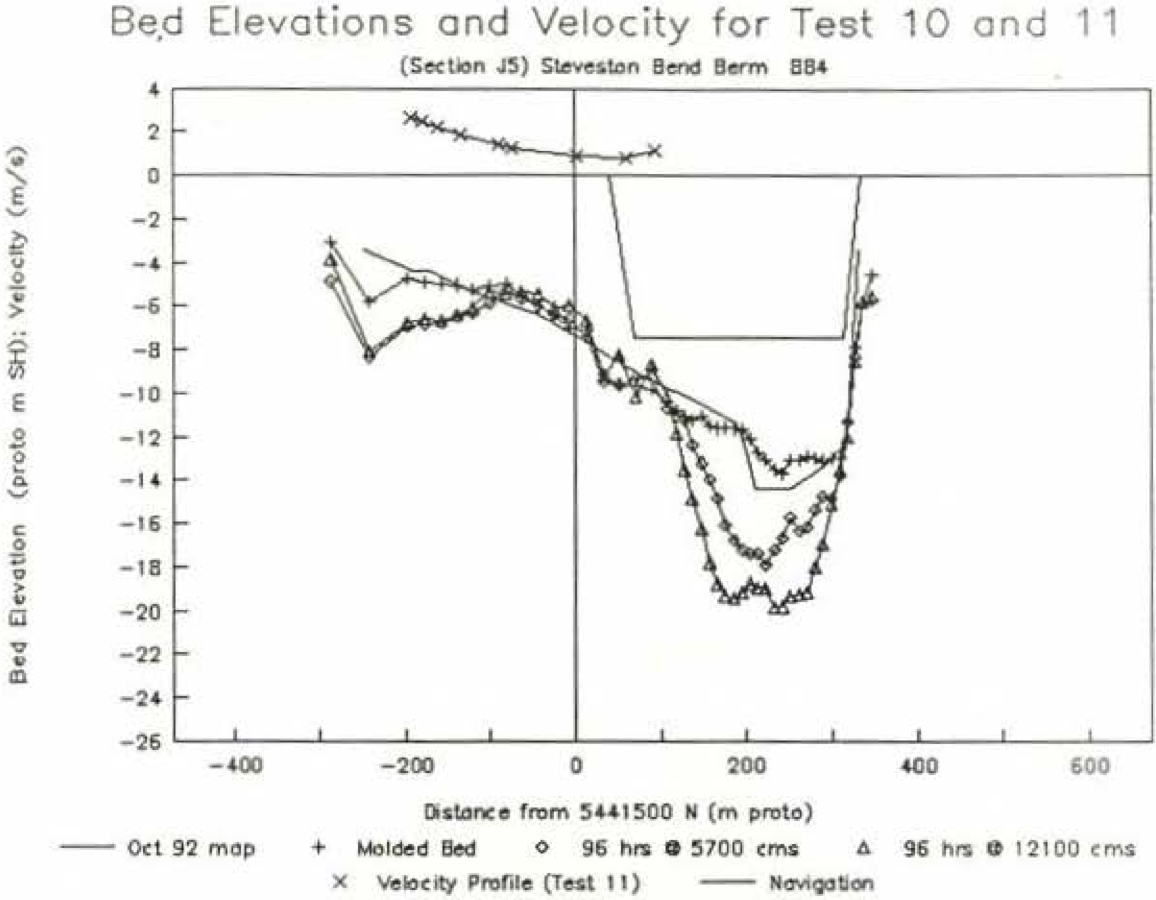

Bed Elevations and Velocity for Test 2
(Section J12) Steveston Bend Berm 884
Bed Elevation (proto m SH)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
initial bed
Oct 92 Map
48 hrs @ 12100 cms
96 hrs @ 12100 cms
Navigation

FIGURE 17


Bed Elevations and Velocity for Test 10 and 11
(Section J5) Steveston Bend Berm BB4
Bed Elevation (proto m SH); Velocity (m/s)
Distance from 5441500 N (m proto)
Oct 92 map
Molded Bed
96 hrs @ 5700 cms
96 hrs @ 12100 cms
Velocity Profile (Test 11)
Navigation

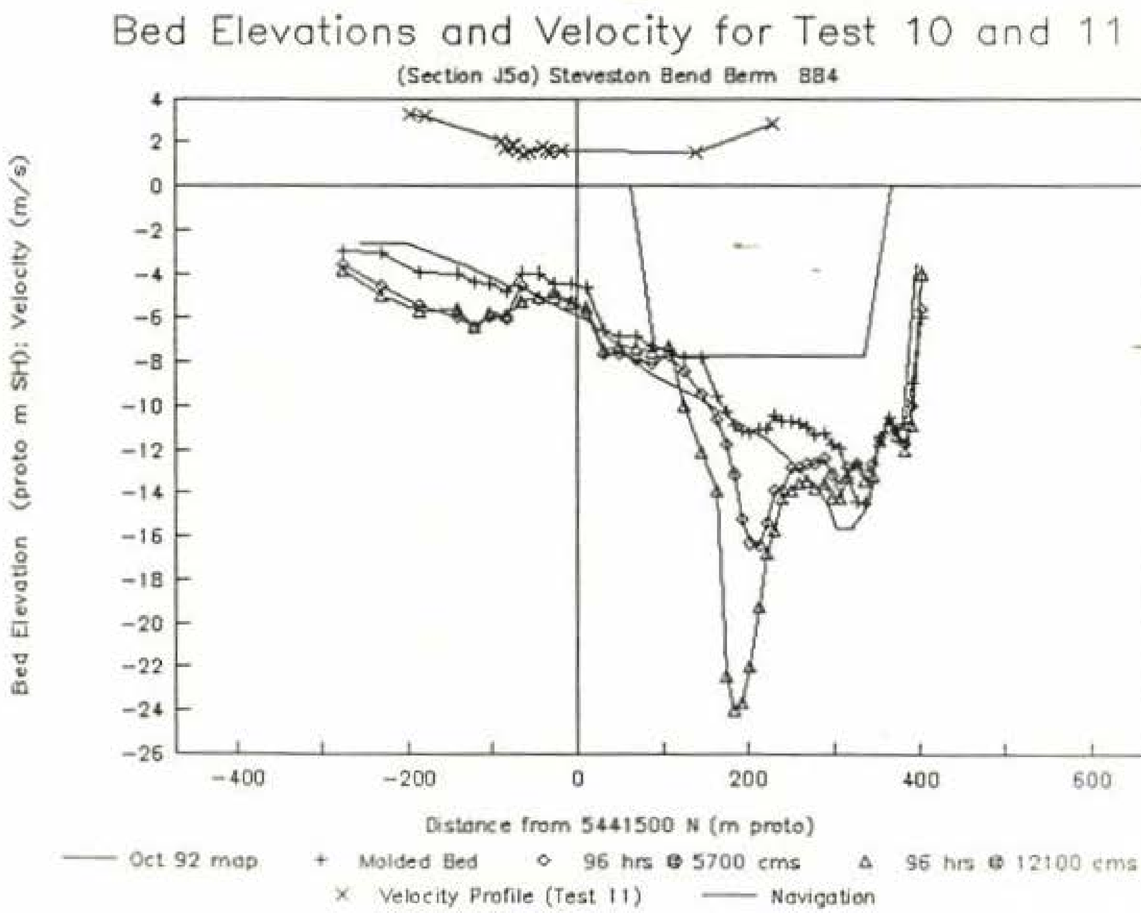
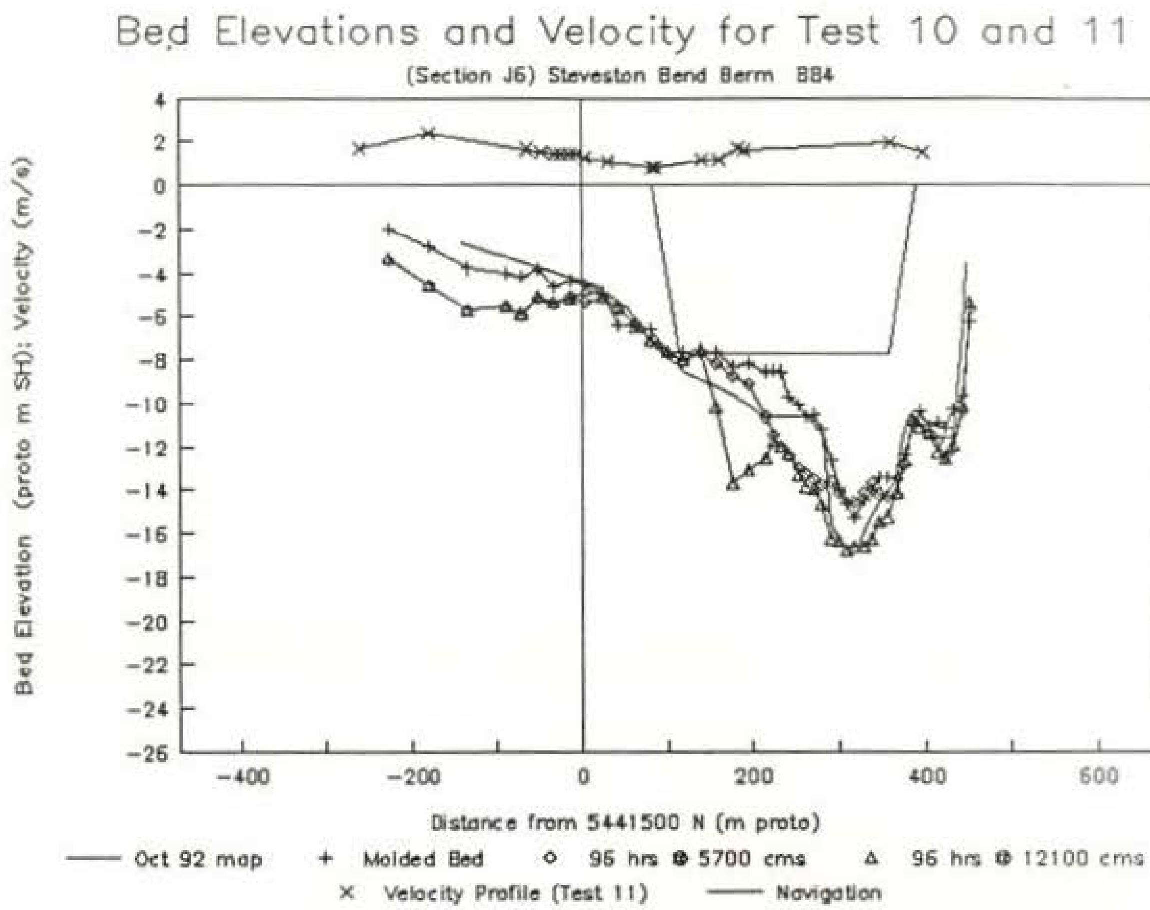
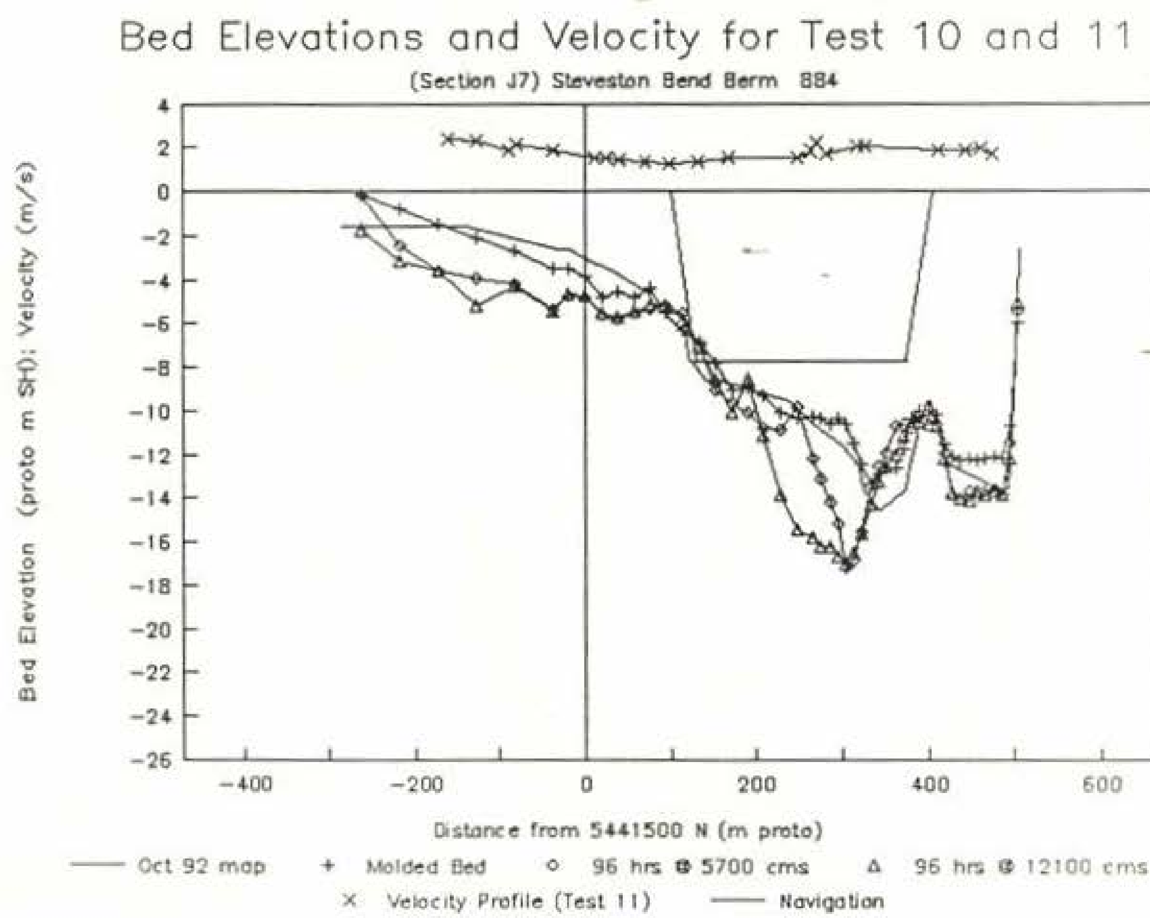

Bed Elevations and Velocity for Test 10 and 11
(Section J5a) Steveston Bend Berm BB4
Bed Elevation (proto m SH); Velocity (m/s)
Distance from 5441500 N (m proto)
Oct 92 map
Molded Bed
96 hrs @ 5700 cms
96 hrs @ 12100 cms
Velocity Profile (Test 11)
Navigation

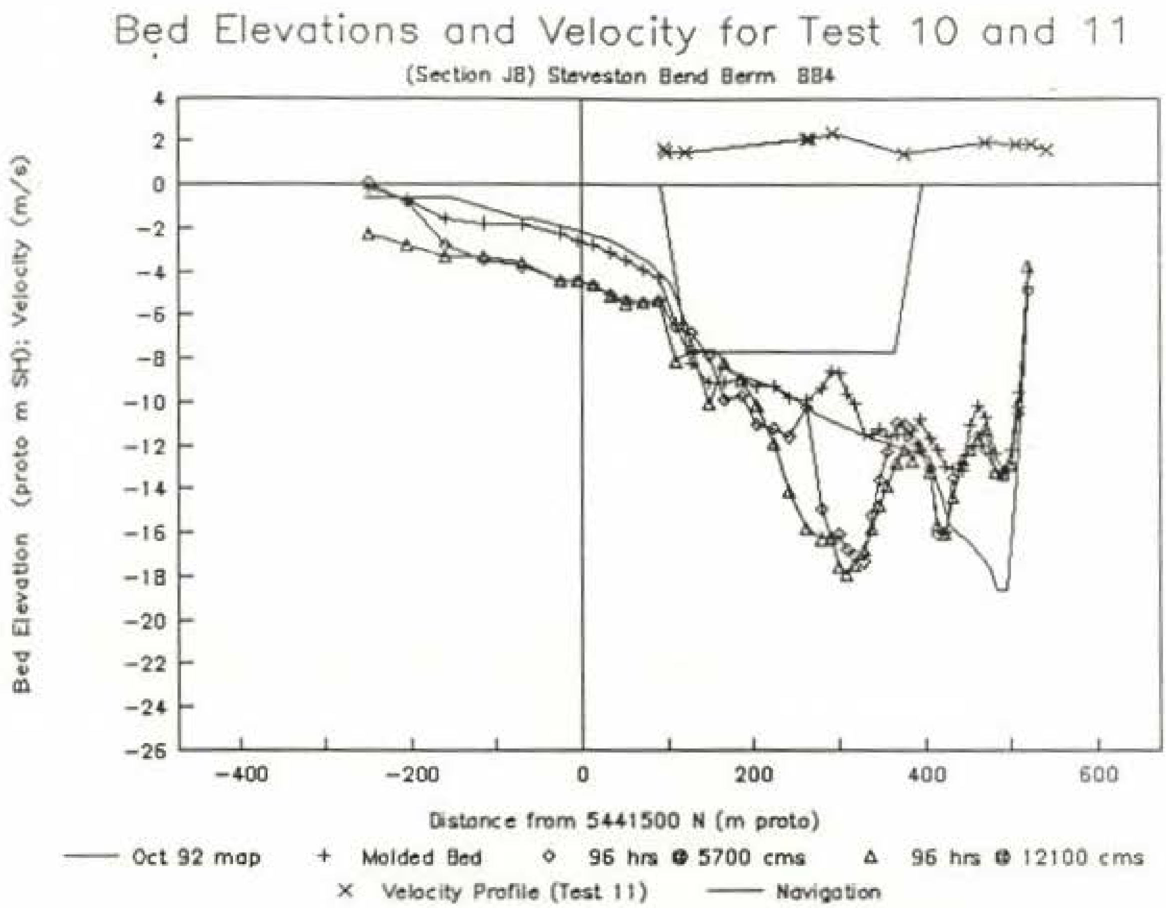
FIGURE 18



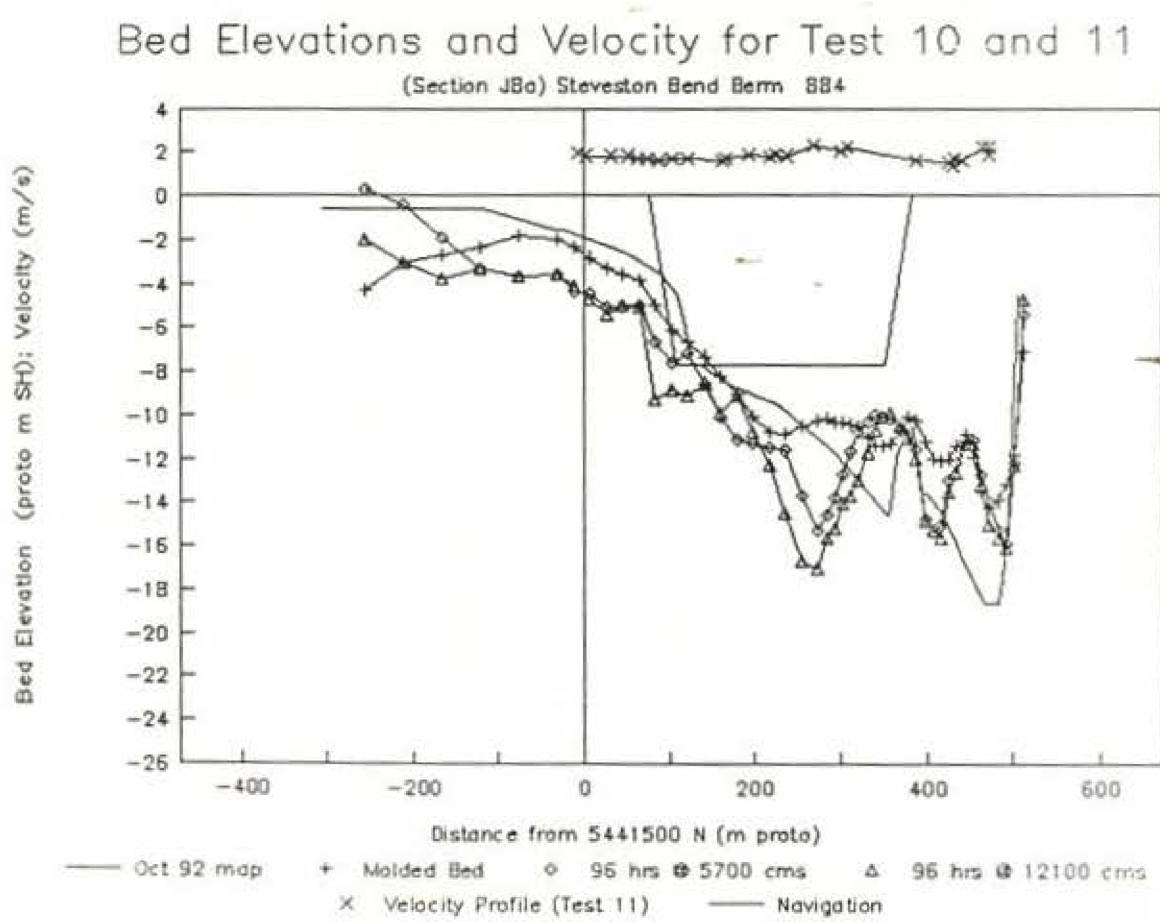
Bed Elevations and Velocity for Test 10 and 11
(Section J6) Steveston Bend Berm BB4
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
Oct 92 map
+ Molded Bed
◇ 96 hrs @ 5700 cms
Δ 96 hrs @ 12100 cms
× Velocity Profile (Test 11)
Navigation



Bed Elevations and Velocity for Test 10 and 11
(Section J7) Steveston Bend Berm BB4
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
Oct 92 map
+ Molded Bed
◇ 96 hrs @ 5700 cms
Δ 96 hrs @ 12100 cms
× Velocity Profile (Test 11)
Navigation



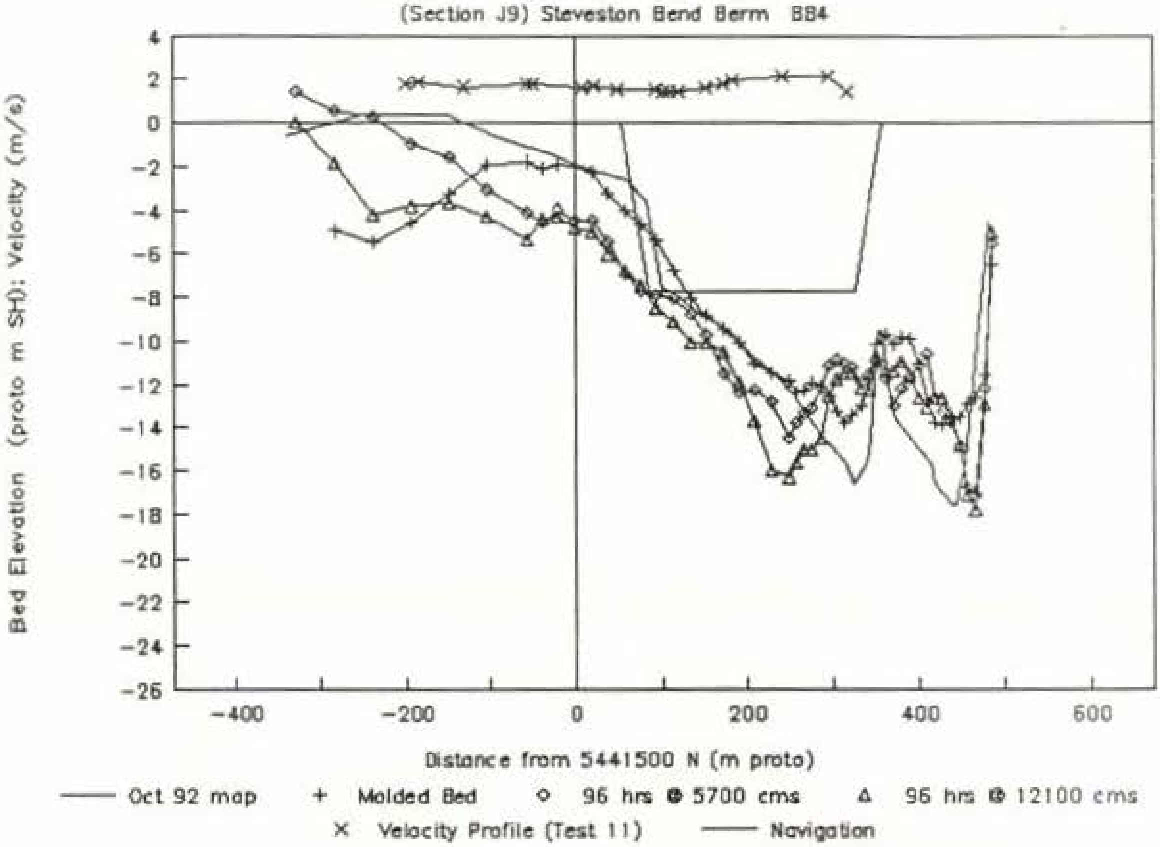
Bed Elevations and Velocity for Test 10 and 11
(Section JB) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
Oct 92 map
+ Molded Bed
◇ 96 hrs @ 5700 cms
Δ 96 hrs @ 12100 cms
× Velocity Profile (Test 11)
Navigation



Bed Elevations and Velocity for Test 10 and 11
(Section JBa) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
Oct 92 map
+ Molded Bed
◇ 96 hrs @ 5700 cms
Δ 96 hrs @ 12100 cms
× Velocity Profile (Test 11)
Navigation

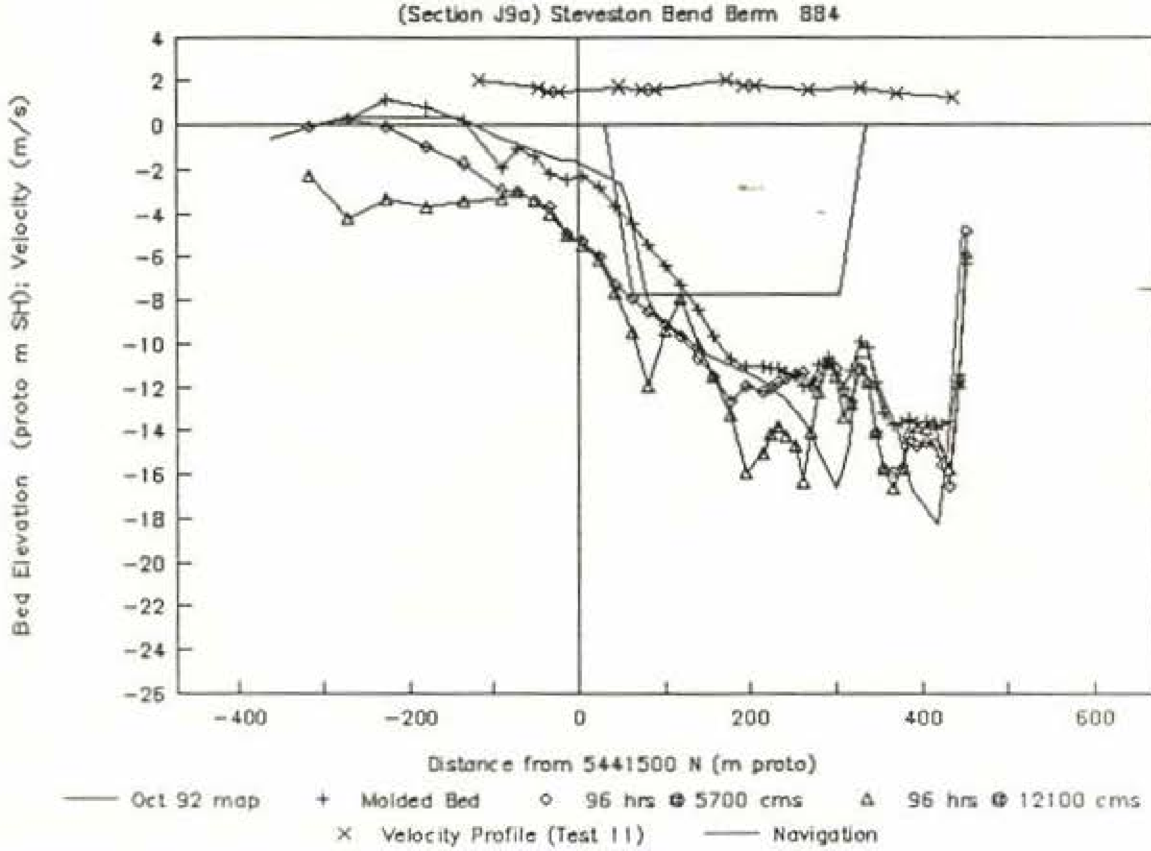
Bed Elevations and Velocity for Test 10 and 11

(Section J9) Steveston Bend Berm BB4



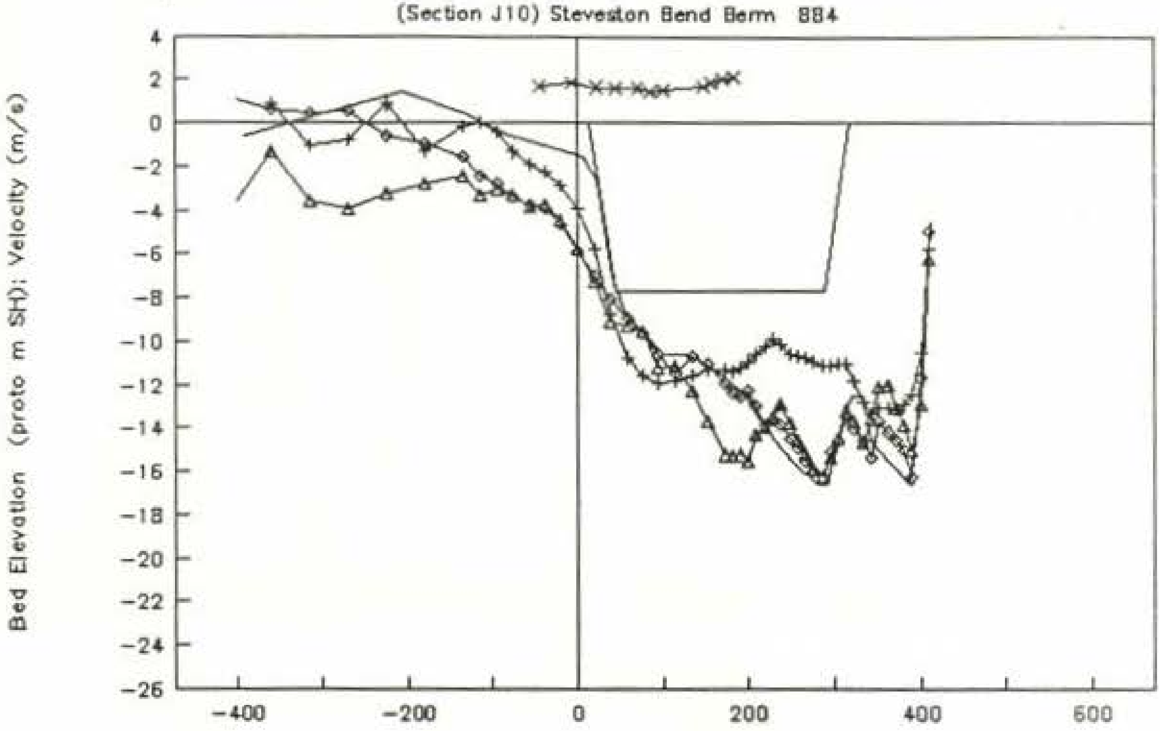
Bed Elevations and Velocity for Test 10 and 11

(Section J9a) Steveston Bend Berm BB4



Bed Elevations and Velocity for Test 10 and 11

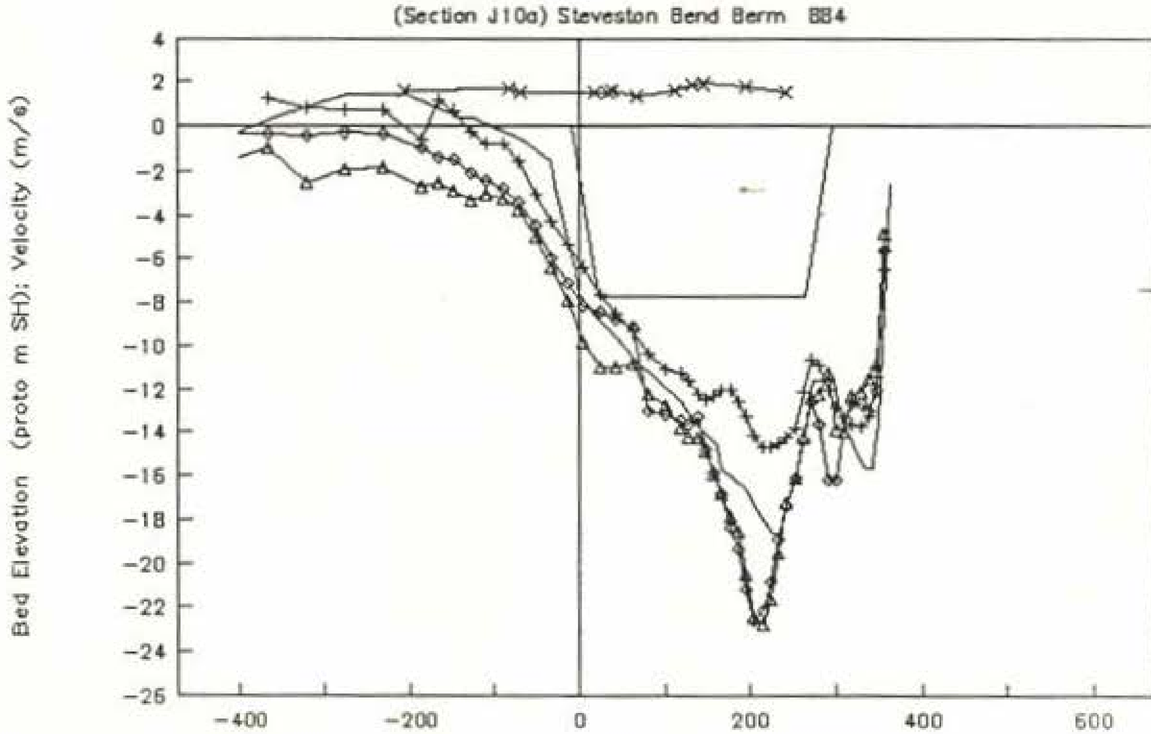
(Section J10) Steveston Bend Berm 884



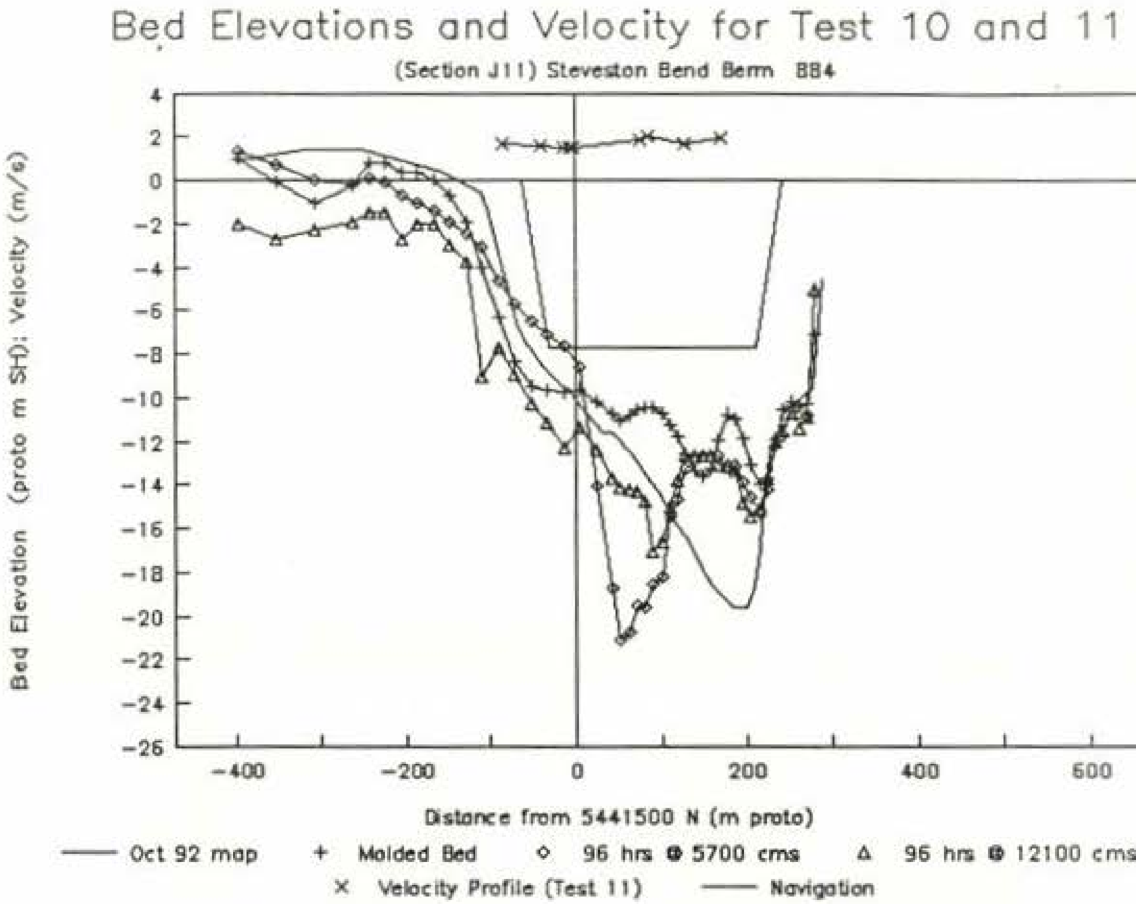
— Oct 92 map + Molded Bed ◇ 96 hrs @ 5700 cms △ 96 hrs @ 12100 cms
× Velocity Profile (Test 11) — Navigation

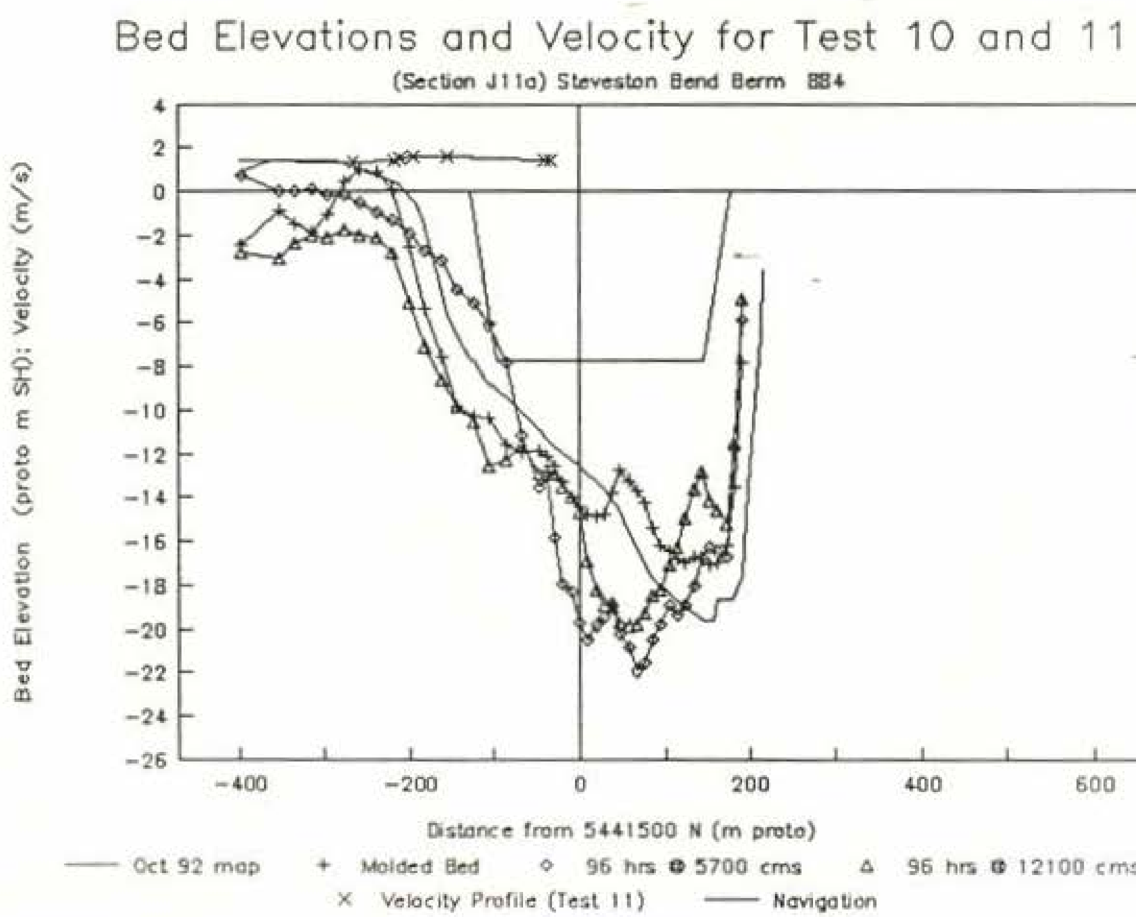
Bed Elevations and Velocity for Test 10 and 11

(Section J10a) Steveston Bend Berm 884



— Oct 92 map + Molded Bed ◇ 96 hrs @ 5700 cms △ 96 hrs @ 12100 cms
× Velocity Profile (Test 11) — Navigation


Bed Elevations and Velocity for Test 10 and 11
(Section J11) Steveston Bend Berm BB4
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
Oct 92 map
+ Molded Bed
◇ 96 hrs @ 5700 cms
Δ 96 hrs @ 12100 cms
× Velocity Profile (Test 11)
Navigation


Bed Elevations and Velocity for Test 10 and 11
(Section J11a) Steveston Bend Berm BB4
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
Oct 92 map
+ Molded Bed
◇ 96 hrs @ 5700 cms
Δ 96 hrs @ 12100 cms
× Velocity Profile (Test 11)
Navigation

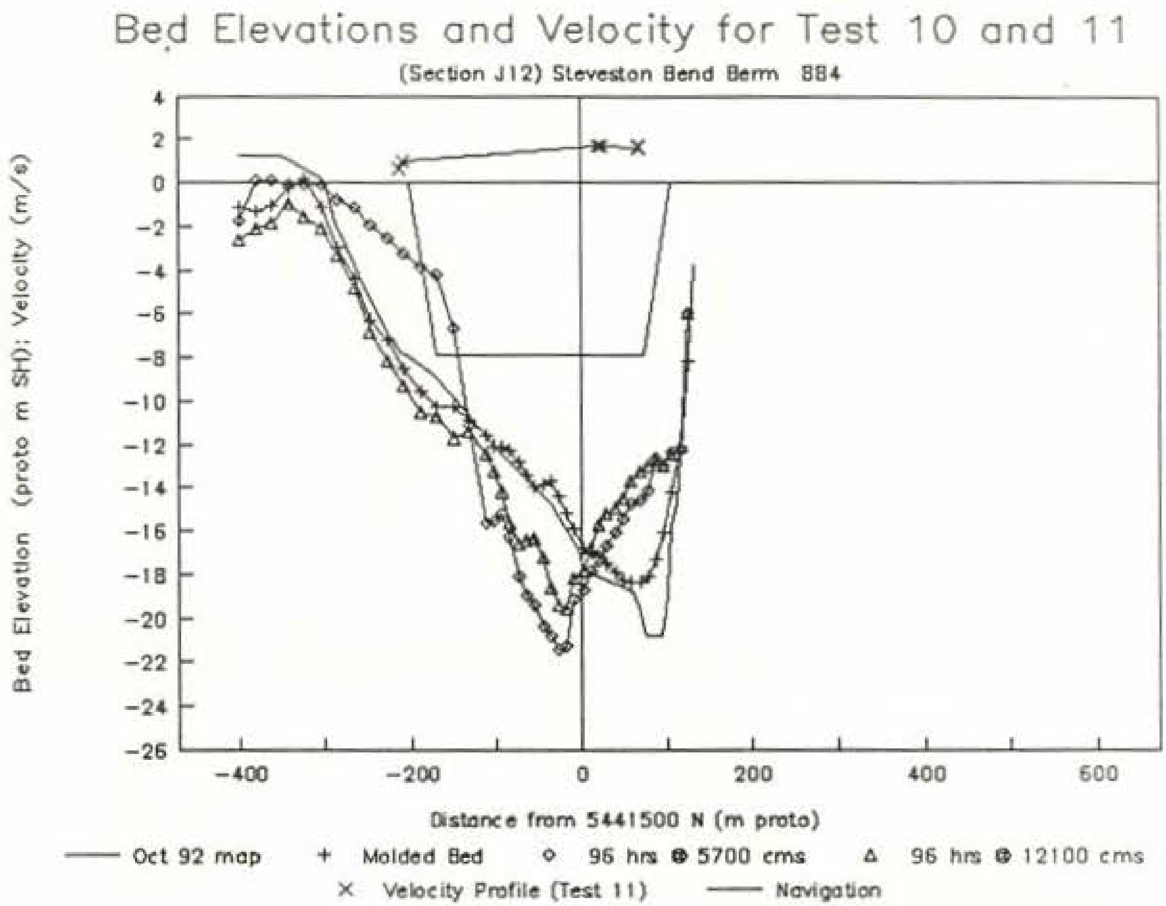
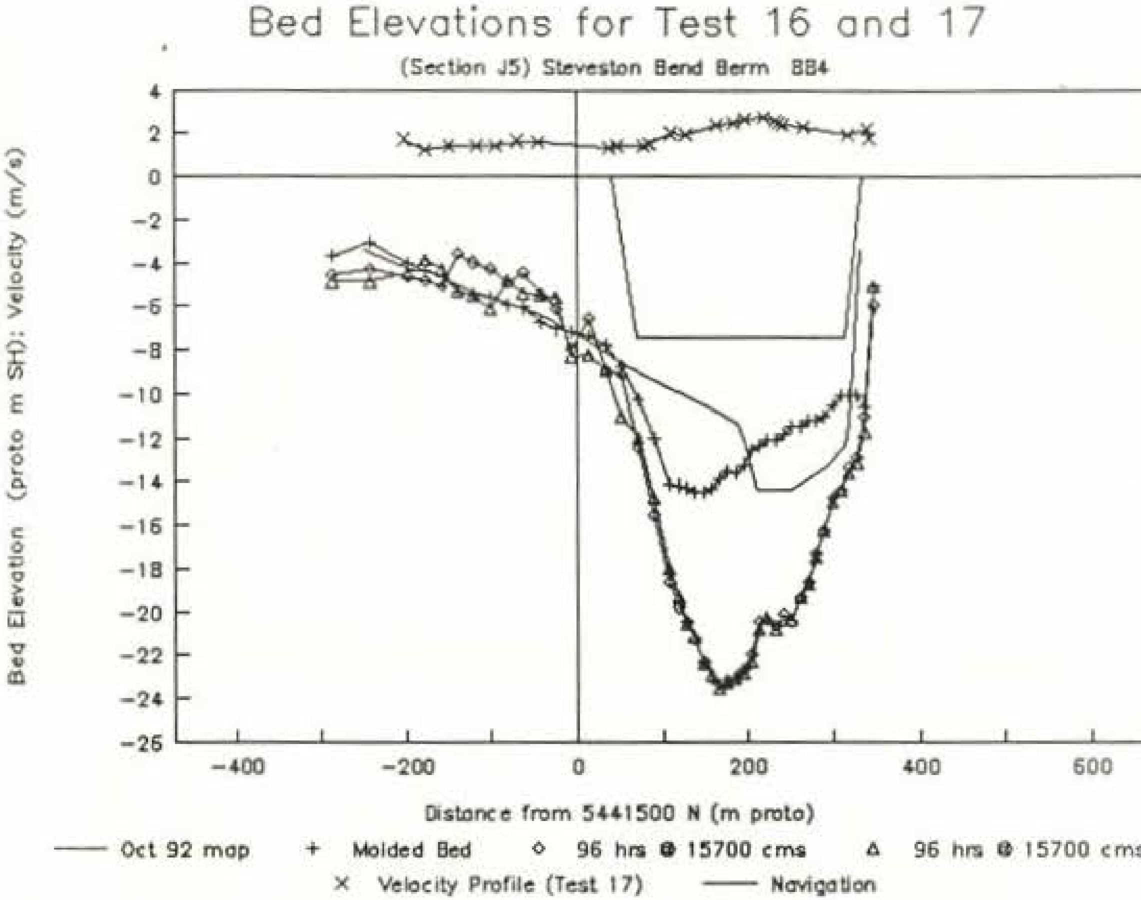

Bed Elevations and Velocity for Test 10 and 11
(Section J12) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
Oct 92 map
+ Molded Bed
◇ 96 hrs @ 5700 cms
Δ 96 hrs @ 12100 cms
× Velocity Profile (Test 11)
Navigation

FIGURE 24


Bed Elevations for Test 16 and 17
(Section J5) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
Oct 92 map
+ Molded Bed
◇ 96 hrs @ 15700 cms
Δ 96 hrs @ 15700 cms
× Velocity Profile (Test 17)
Navigation

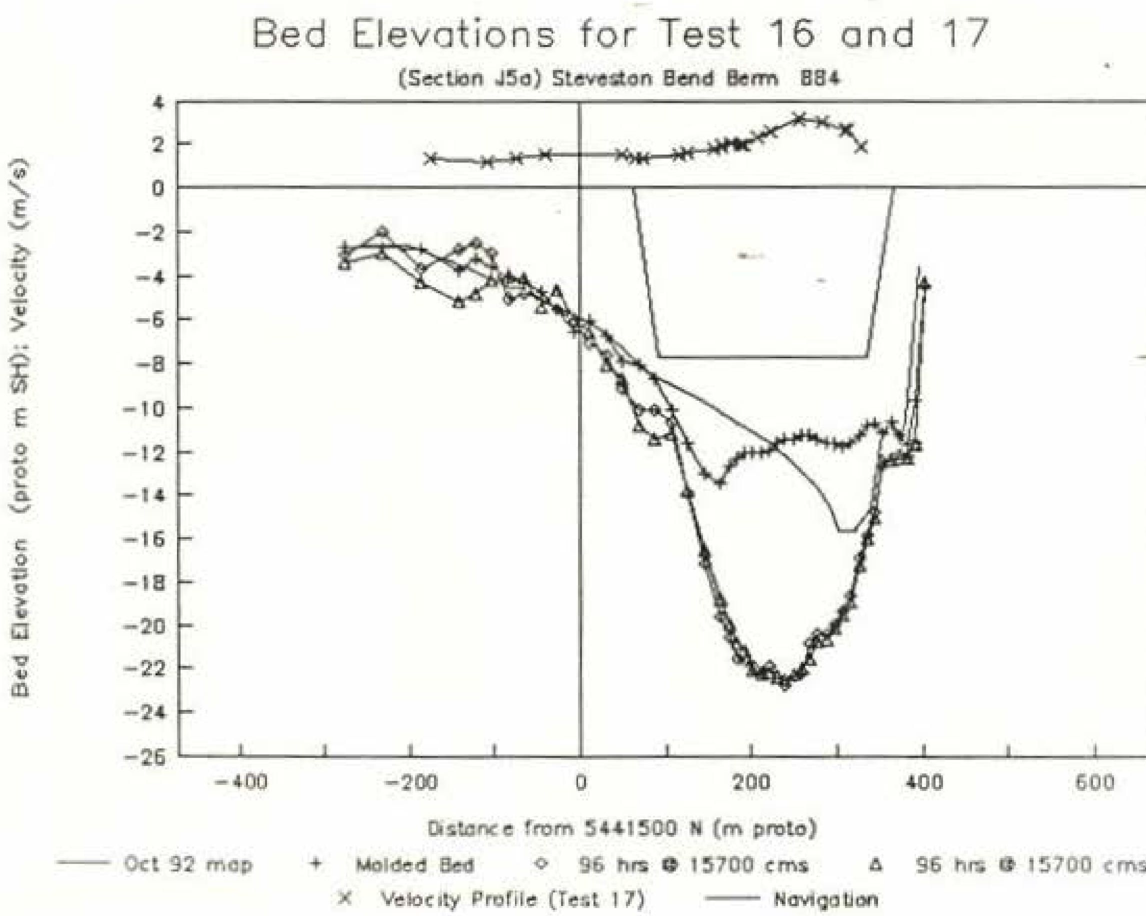
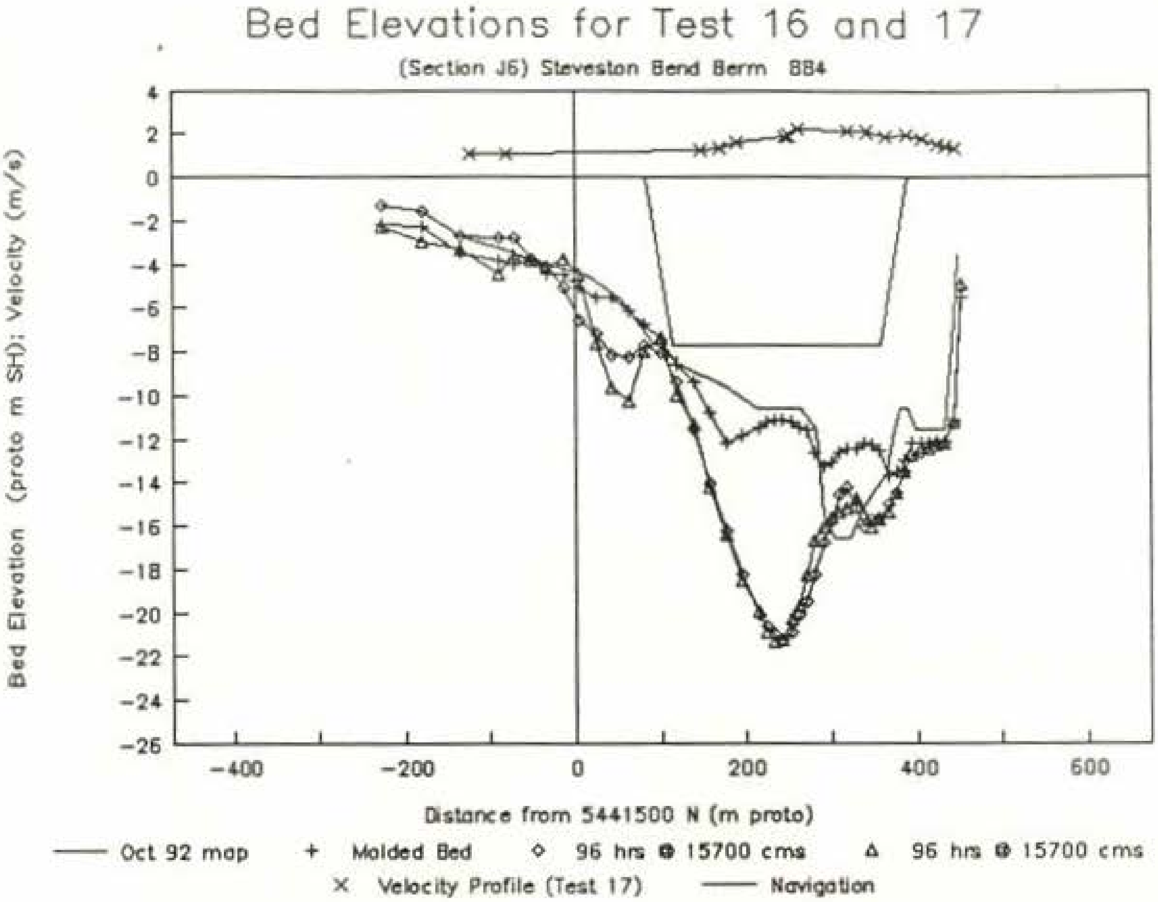
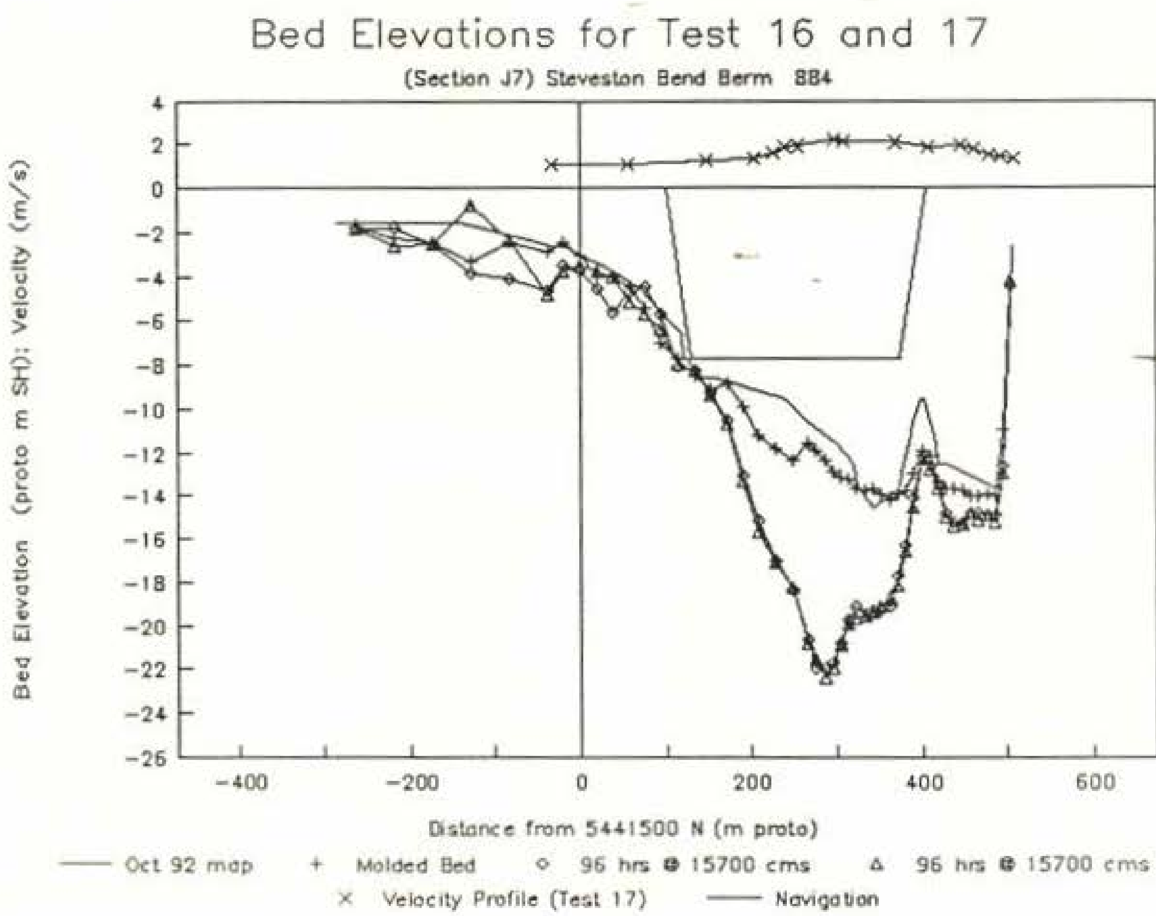

Bed Elevations for Test 16 and 17
(Section J5a) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
Oct 92 map
+ Molded Bed
◇ 96 hrs @ 15700 cms
Δ 96 hrs @ 15700 cms
× Velocity Profile (Test 17)
Navigation

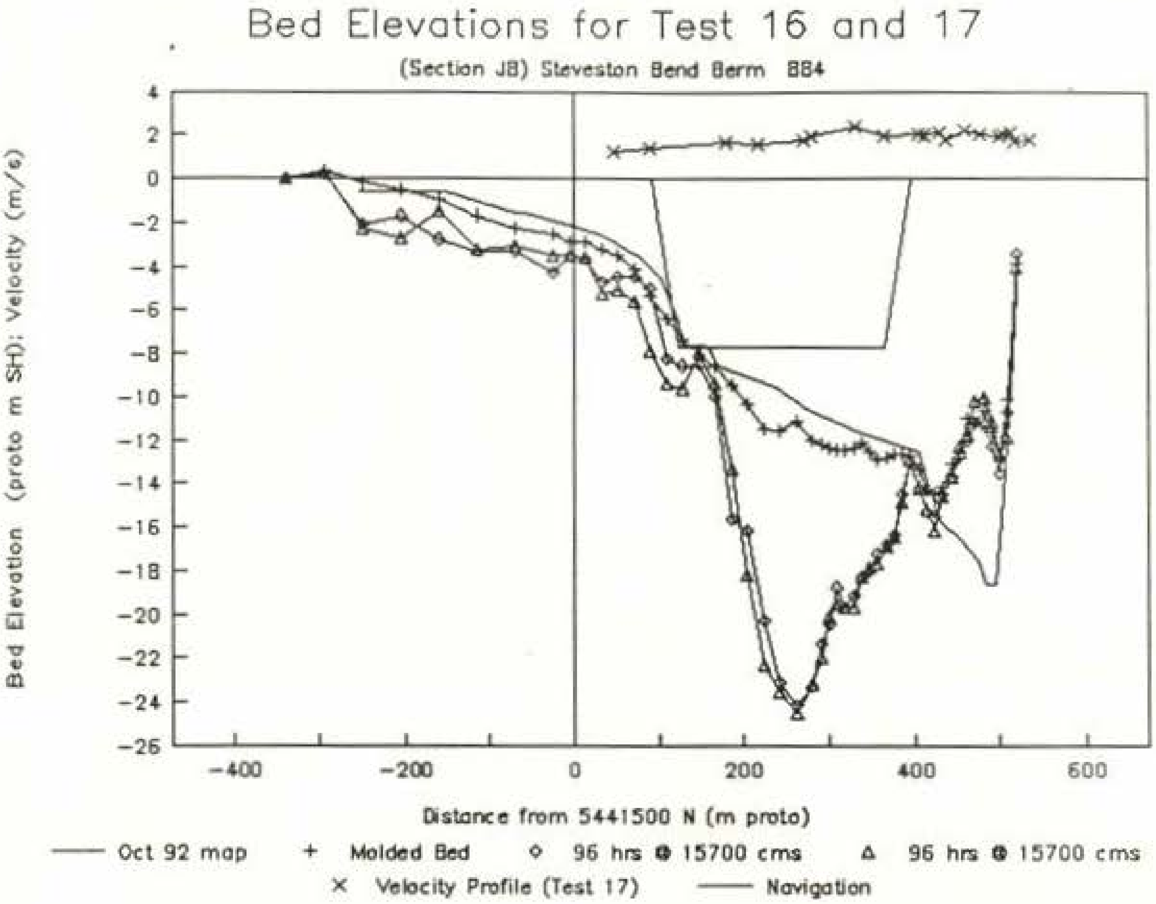
FIGURE 25

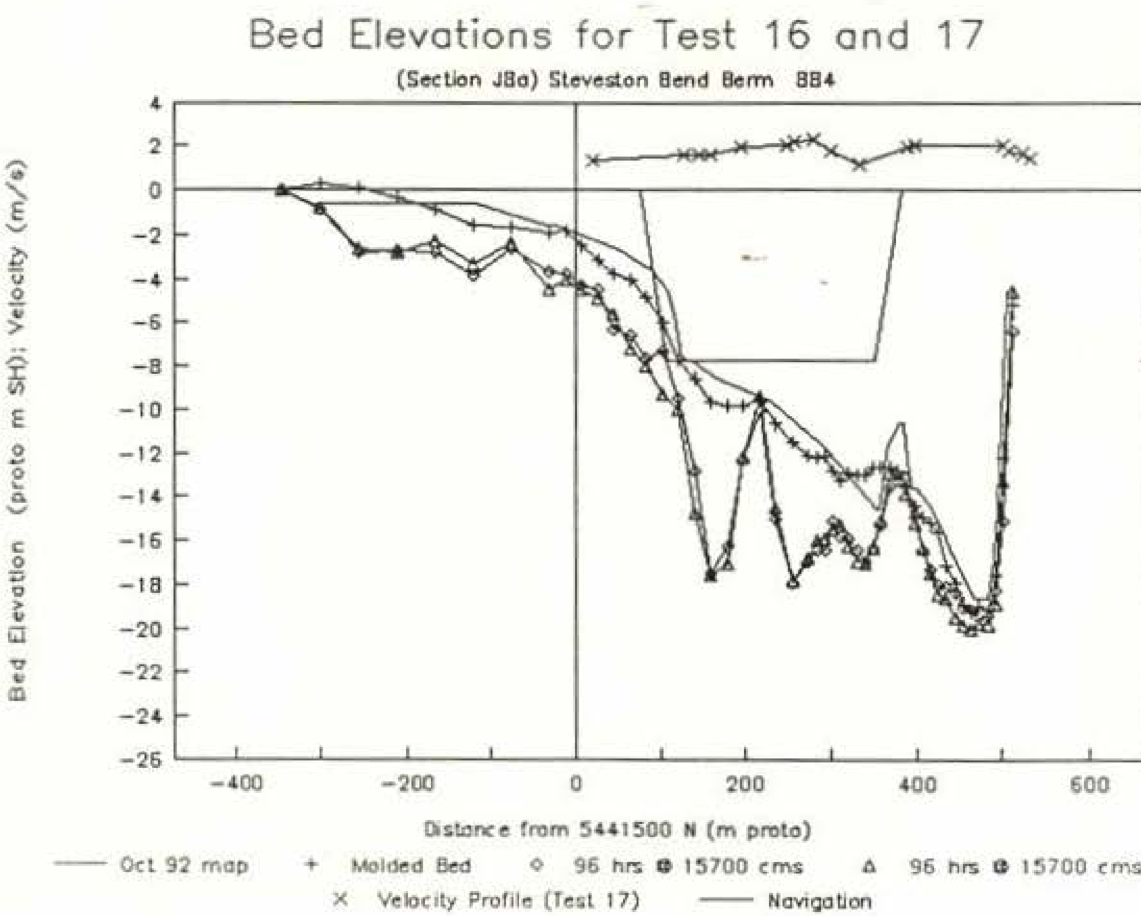


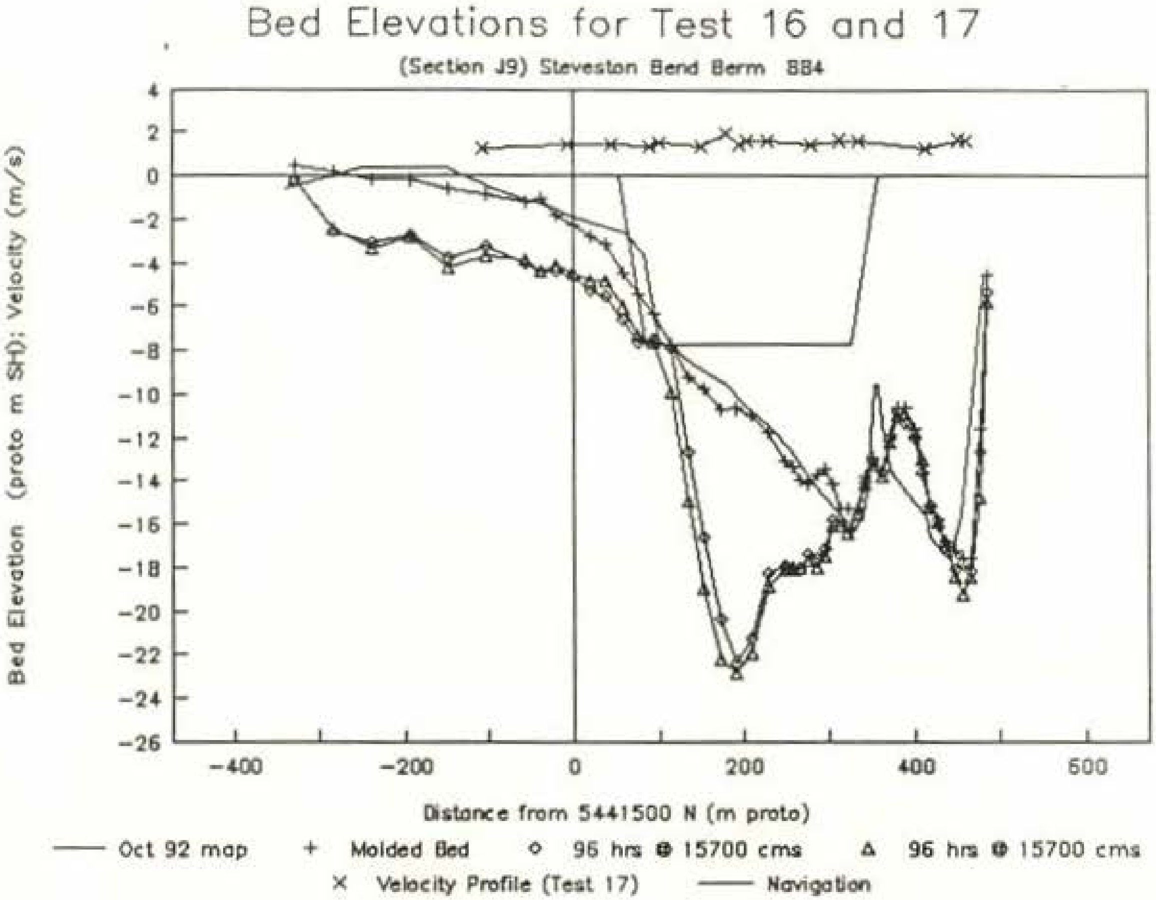
Bed Elevations for Test 16 and 17
(Section J6) Steveston Bend Berm 8B4
Bed Elevation (proto m SH); Velocity (m/s)
Distance from 5441500 N (m proto)
Oct 92 map
Molded Bed
96 hrs @ 15700 cms
96 hrs @ 15700 cms
Velocity Profile (Test 17)
Navigation

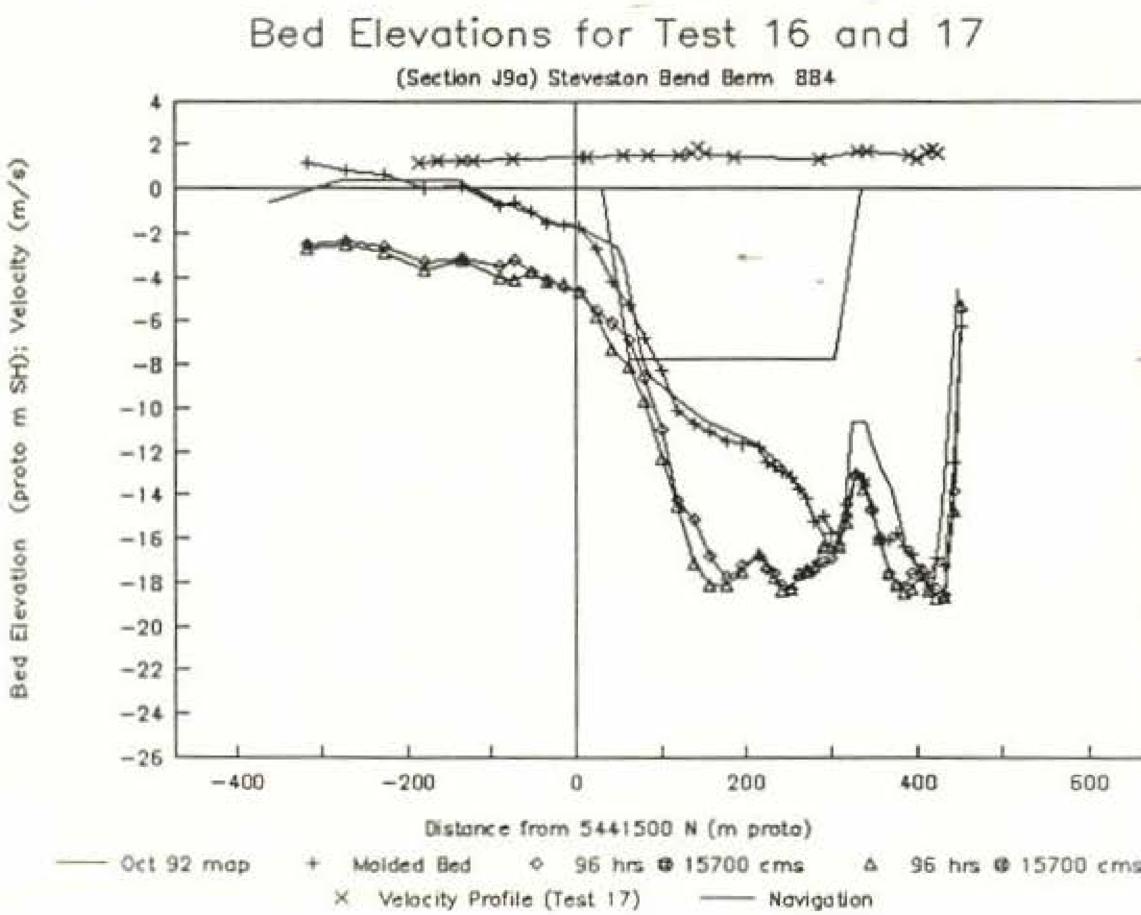


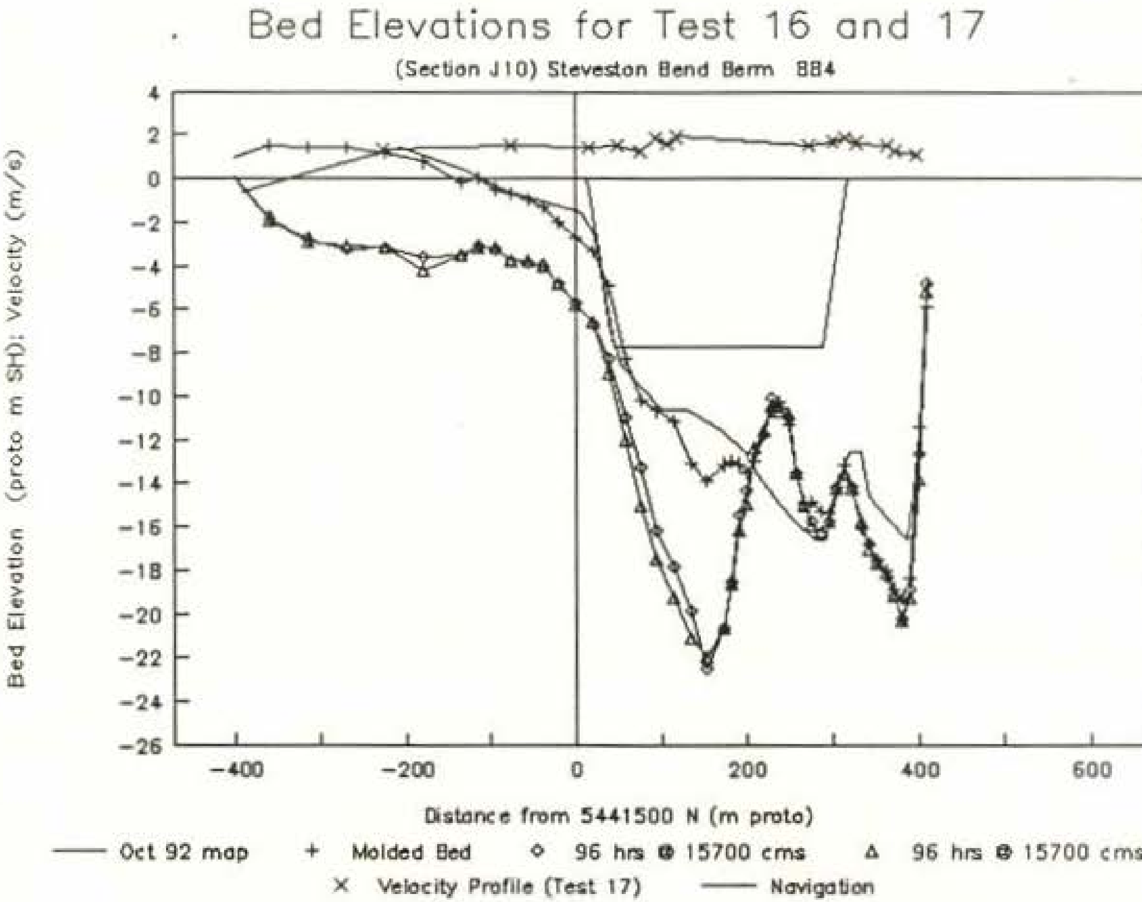
Bed Elevations for Test 16 and 17
(Section J7) Steveston Bend Berm 8B4
Bed Elevation (proto m SH); Velocity (m/s)
Distance from 5441500 N (m proto)
Oct 92 map
Molded Bed
96 hrs @ 15700 cms
96 hrs @ 15700 cms
Velocity Profile (Test 17)
Navigation


Bed Elevations for Test 16 and 17
(Section J8) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
Distance from 5441500 N (m proto)
Oct 92 map
Molded Bed
96 hrs @ 15700 cms
96 hrs @ 15700 cms
Velocity Profile (Test 17)
Navigation


Bed Elevations for Test 16 and 17
(Section J8a) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
Distance from 5441500 N (m proto)
Oct 92 map
Molded Bed
96 hrs @ 15700 cms
96 hrs @ 15700 cms
Velocity Profile (Test 17)
Navigation


Bed Elevations for Test 16 and 17
(Section J9) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
Oct 92 map
+ Molded Bed
◇ 96 hrs @ 15700 cms
Δ 96 hrs @ 15700 cms
× Velocity Profile (Test 17)
Navigation


Bed Elevations for Test 16 and 17
(Section J9a) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
Oct 92 map
+ Molded Bed
◇ 96 hrs @ 15700 cms
Δ 96 hrs @ 15700 cms
× Velocity Profile (Test 17)
Navigation


Bed Elevations for Test 16 and 17
(Section J10) Steveston Bend Berm BB4
Bed Elevation (proto m SH); Velocity (m/s)
Distance from 5441500 N (m proto)
Oct 92 map
Molded Bed
96 hrs @ 15700 cms
96 hrs @ 15700 cms
Velocity Profile (Test 17)
Navigation

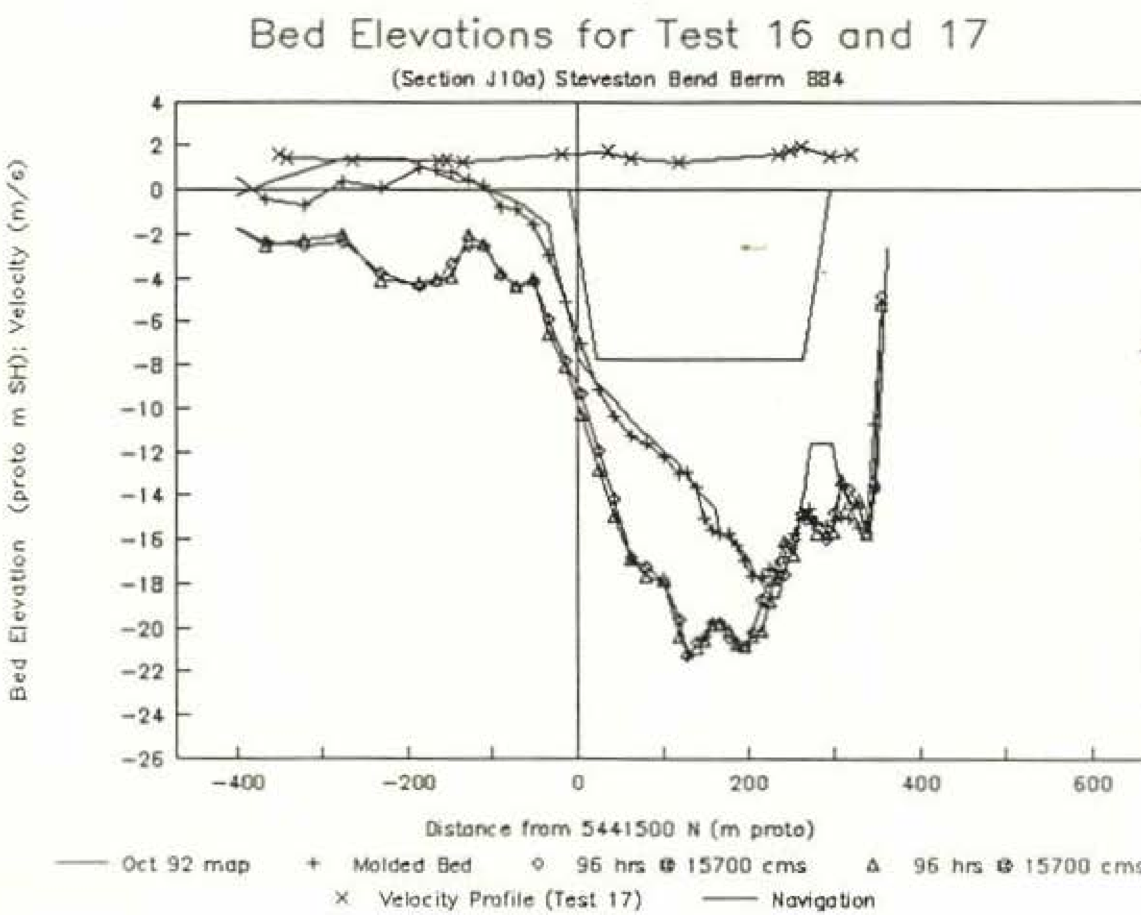
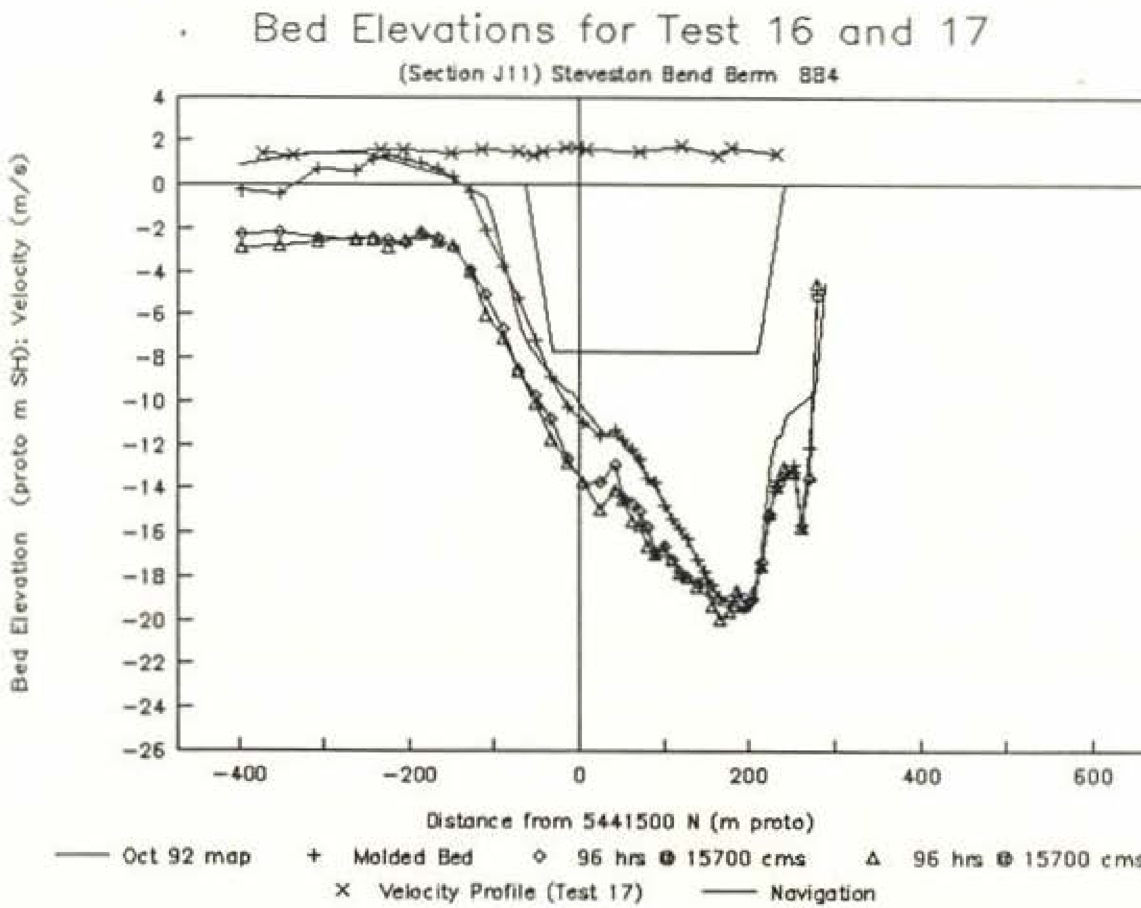
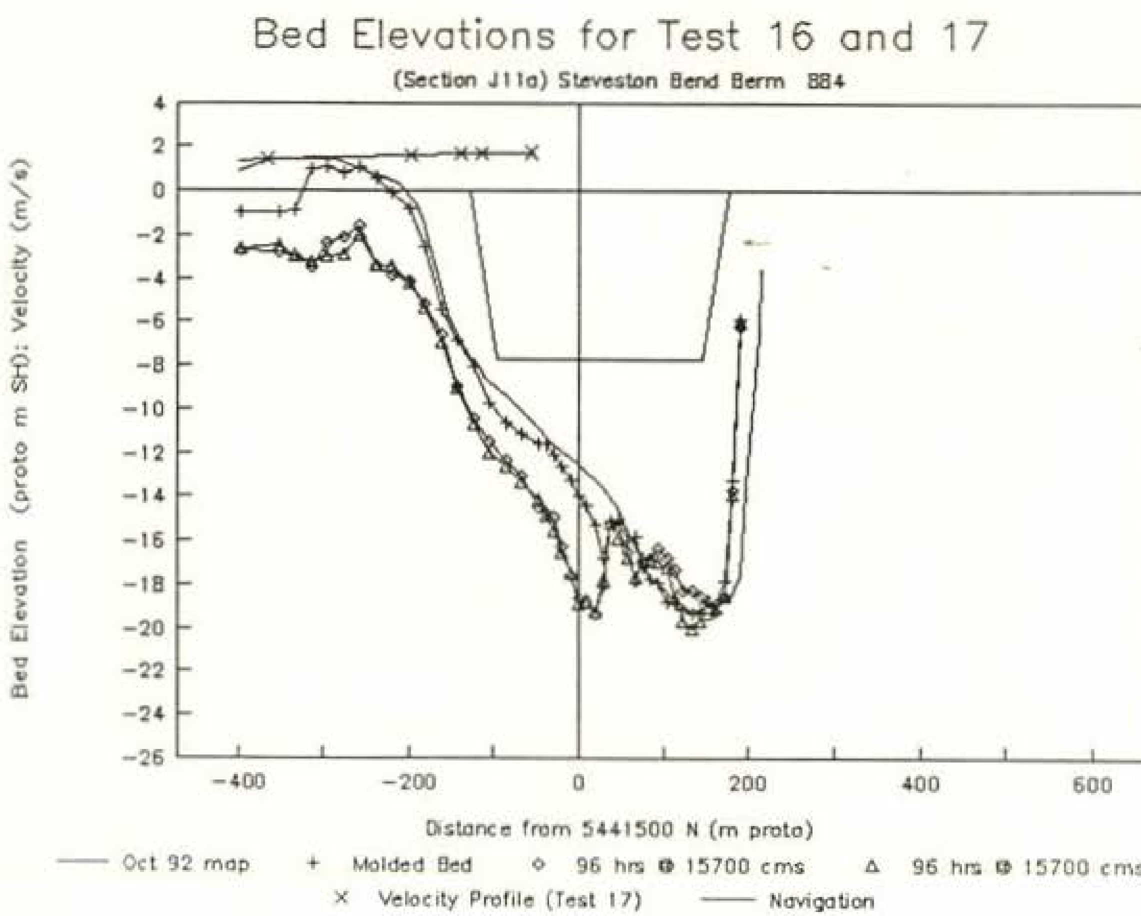

Bed Elevations for Test 16 and 17
(Section J10a) Steveston Bend Berm BB4
Bed Elevation (proto m SH); Velocity (m/s)
Distance from 5441500 N (m proto)
Oct 92 map
Molded Bed
96 hrs @ 15700 cms
96 hrs @ 15700 cms
Velocity Profile (Test 17)
Navigation

FIGURE 29



Bed Elevations for Test 16 and 17
(Section J11) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
Oct 92 map
+ Molded Bed
◇ 96 hrs @ 15700 cms
Δ 96 hrs @ 15700 cms
× Velocity Profile (Test 17)
Navigation



Bed Elevations for Test 16 and 17
(Section J11a) Steveston Bend Berm 884
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
Oct 92 map
+ Molded Bed
◇ 96 hrs @ 15700 cms
Δ 96 hrs @ 15700 cms
× Velocity Profile (Test 17)
Navigation

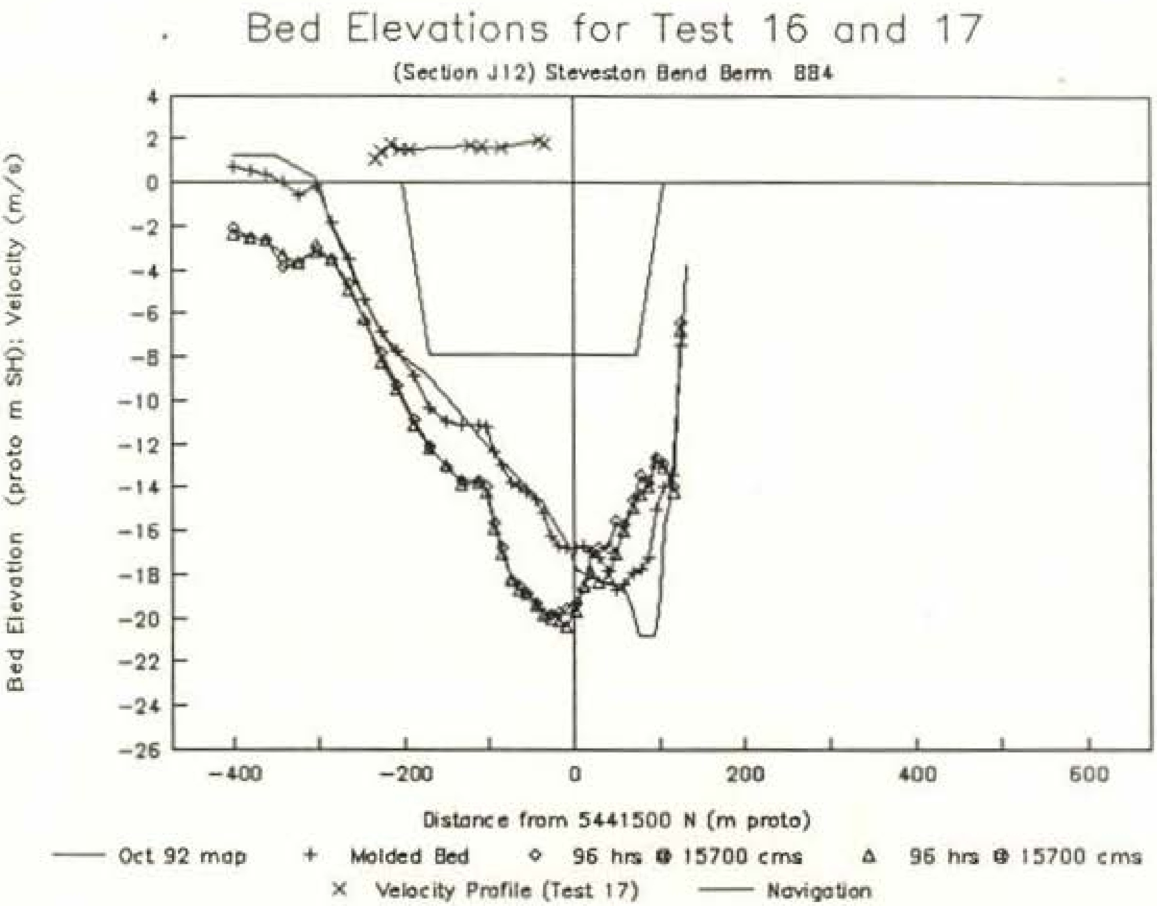
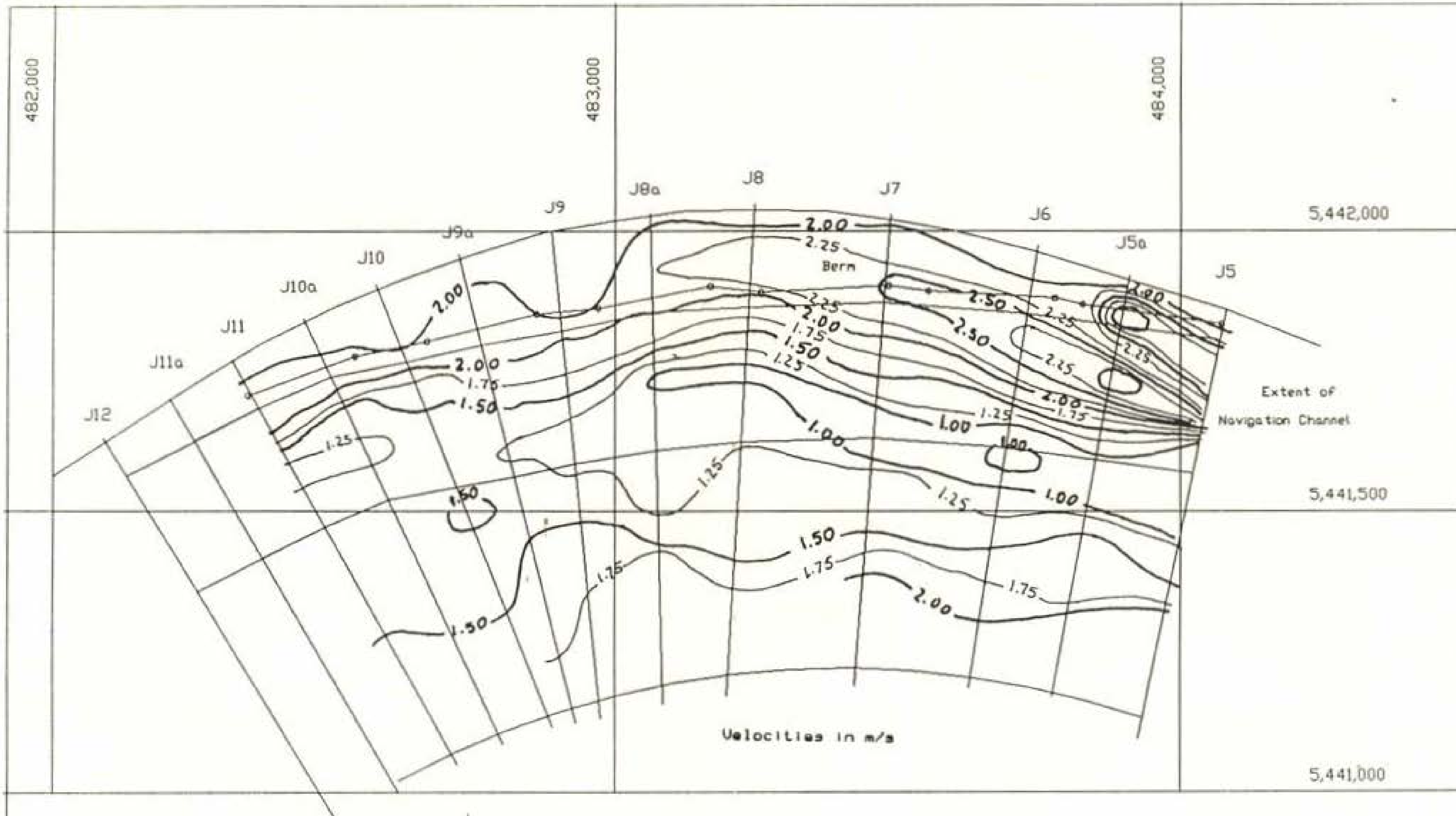

Bed Elevations for Test 16 and 17
(Section J12) Steveston Bend Berm BB4
Bed Elevation (proto m SH); Velocity (m/s)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-400
-200
0
200
400
600
Distance from 5441500 N (m proto)
Oct 92 map
Molded Bed
96 hrs @ 15700 cms
96 hrs @ 15700 cms
Velocity Profile (Test 17)
Navigation

FIGURE 31



Scale 1:10,000

Note: Berm El. -11 m LLW

Instantaneous Discharge: 11192 cms
Water Surface Elevation: 2.00 m LLW

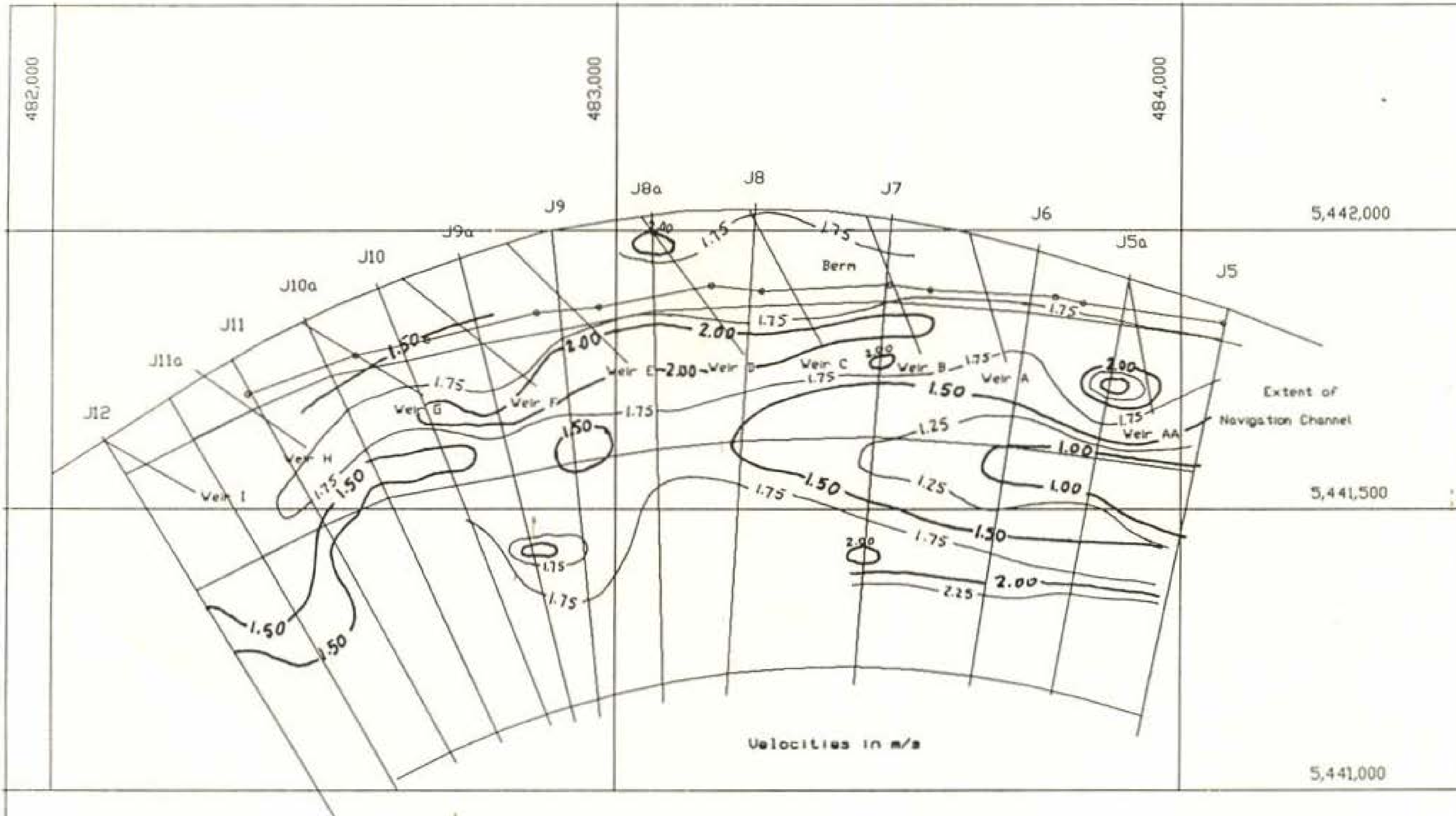
STEVESTON BENDWAY WEIRS

Test #2

VELOCITY CONTOURS

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

FIGURE 32



Scale 1:10,000

Note: Bendway Weirs El.: -10 m LLW

Berm El.: -11 m LLW

Instantaneous Discharge: 11123 cms

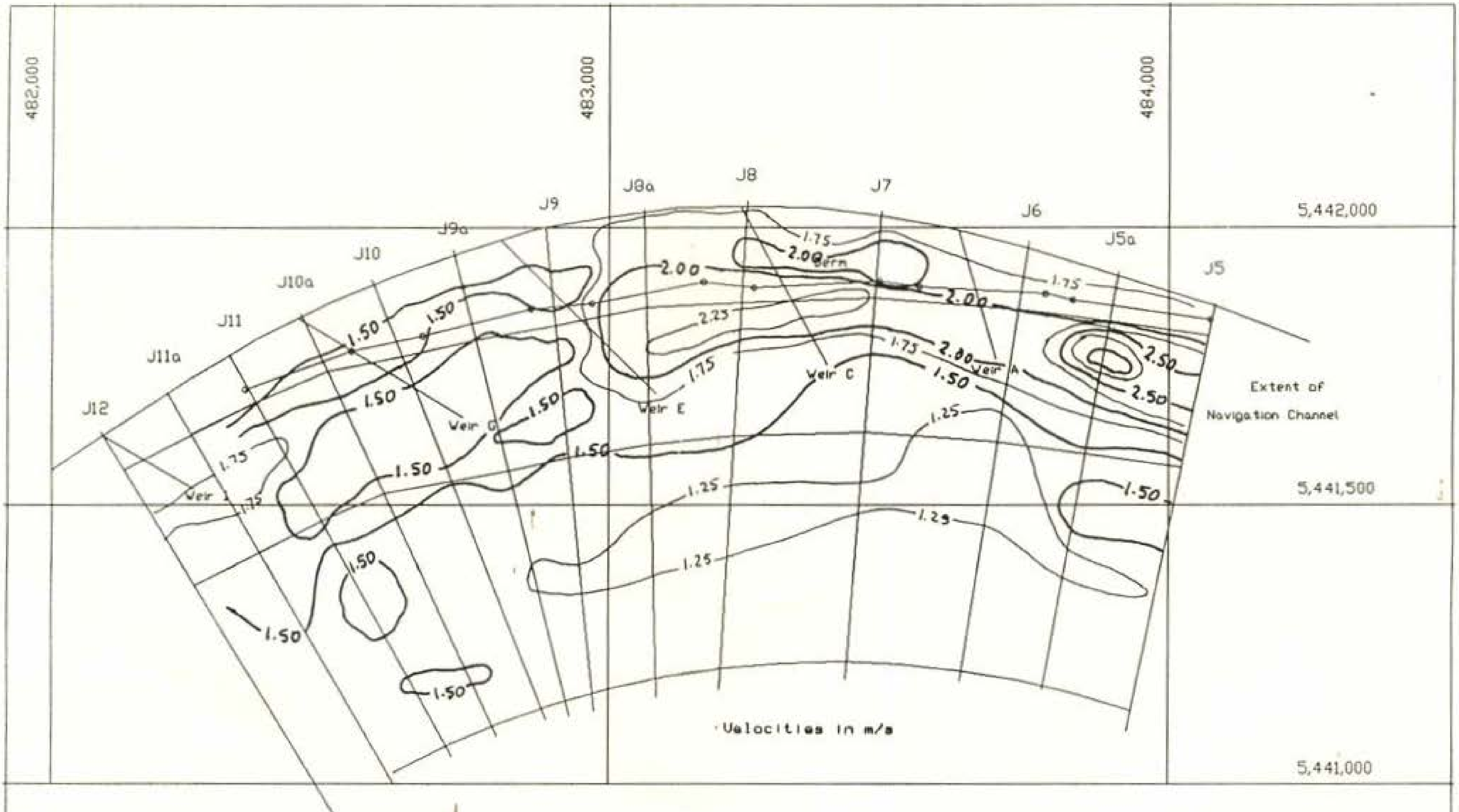
Water Surface Elevation: 2.05 m LLW

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

STEVESTON BENDWAY WEIRS

Test #11

VELOCITY CONTOURS



Scale 1:10,000

Note: Bendway Weirs El.: Test #16 & #17, -11 m LLW ; Test #18, -10 m LLW
Berm El.: Test #16 & #17, -13 m LLW ; Test #18, -11 m LLW

Instantaneous Discharge: 11188 cms
Water Surface Elevation: 2.07 m LLW

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

STEVESTON BENDWAY WEIRS

Test #17
VELOCITY CONTOURS

FIGURE 33