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CCIW BREAKWATER MODEL TESTS

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MANAGEMENT PERSPECTIVE

Cost-effective designs for coastal and hydraulic structures can often benefit from a physical scale model study. The rehabilitation of the CCIW breakwater is a case in point. Instead of evaluating conventional structural repair options only, several options involving the use of a Floating Tire Breakwater (FTB) were investigated in a physical hydraulic model in the NWRI Hydraulics Laboratory. At an out-of-pocket cost of about $15,000 to conduct the model study, a potential savings of $800,000 may be realized if the FTB option is selected. Greater awareness of such modelling techniques should be encouraged.

Dr. John Lawrence
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SOMMAIRE

On a souvent avantage à recourir à une étude sur maquette pour une conception rentable d'ouvrages hydrauliques et côtiers. C'est le cas de la restauration du brise-lames du Centre canadien des eaux intérieures (CCEI). Au lieu d'évaluer seulement les options habituelles de réparation de l'ouvrage, on a étudié plusieurs options sur maquette au laboratoire d'hydraulique de l'Institut nationalde recherche sur les eaux (INRF), dont l'utilisation d'un brise-lames fait de pneus flottants. Moyennant un déboursé d'environ 15 000 $ pour réaliser l'étude sur maquette, on pourrait épargner 800 000 $ si l'option du brise-lames en pneus flottants était retenue. On devrait encourager une plus grande diffusion de ces techniques de modélisation.

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ABSTRACT

Physical hydraulic model tests were conducted to measure wave transmission and force characteristics of a partial vertical thin-walled breakwater. Tests were run for the breakwater in its as-designed condition as well as in its present deteriorated state in which many of the wave-reflecting panels on its windward face have fallen off. Several options involving the use of a Floating Tire Breakwater (FTB) moored in front of the structure were also tested. A conceptual design was made for an FTB to restore an acceptable wave climate in the lee of the structure.

RÉSUMÉ

On a procédé à des tests sur maquette hydraulique pour évaluer la transmission des vagues et les caractéristiques de force d'un brise-lames partiel vertical à parois minces. On a réalisé des tests sur la maquette de brise-lames dans les conditions de conception et dans l'état actuel détérioré. (Un bon nombre des panneaux de réflexion des vagues de la façade au vent sont tombés). On a également testé plusieurs options prévoyant l'utilisation d'un brise-lames fait de pneus flottants, amarré sur le devant de la structure. Selon une étude, on pourrait utiliser un brise-lames de pneus flottants pour rétablir un régime de vagues acceptable sous le vent de l'ouvrage.

INTRODUCTION

Berthing for marine research vessels at the Canada Centre for Inland Waters (CCIW) in Burlington, Ontario is protected from waves and ice by a 518 m long breakwater (Figure 1). This 22 year old structure has deteriorated and is no longer providing a satisfactory wave climate at the CCIW wharf. Public Works Canada (PWC) was asked to investigate means of rehabilitating the breakwater and to prepare appropriate plans and specifications.

As described in a report by Allen (1971), the breakwater consists of a series of I-beam piles with a 12:1 slope at 3 m centres on the windward side, concrete-filled pipe piles at a slope of 12:5 on the leeward side, a 1.9 m wide poured-in-place concrete cap encasing the tops of the piles, and precast concrete panels which extend down from the concrete cap but do not reach the bottom (Figure 2). This type of structure is known as a partial vertical thin-walled breakwater.

The concrete panels are suspended from the cap beam by two steel hanger rods and are attached to the I-beams by clamp plates and bolts with lock washers. Only months after construction began in 1968, cracks in the concrete cap were observed, as well as loosening of the nuts securing the panel connections. At that time an engineering investigation concluded that the design of the panel connection was inadequate due to larger than anticipated deflections of the structure. The breakwater has performed adequately over its 22 year life, however, panels have been falling off and are doing so at an increasing rate. Consequently, the wave climate in the lee of the structure is no longer satisfactory.

The National Water Research Institute (NWRI), located at CCIW, includes the Hydraulics Laboratory, administered by the Research and Applications Branch (RAB). Building and Properties Services of the Staff Services Division, NWRI,

requested RAB to undertake a physical hydraulic model study related to the rehabilitation of the breakwater. The purpose of the hydraulic model study is to allow PWC to evaluate the most feasible option for the repair of the existing breakwater. Based on an underwater assessment, two major repair options were identified:

1) Stiffening the existing pile system using the existing structural breakwater configuration and the same design principles. A preliminary cost estimate to rehabilitate the breakwater in this manner, including buttressing the structure to provide additional rigidity and replacing the missing panels, was $2 million.

2) Using an energy-absorbing system, such as a floating breakwater, to decrease the wave forces acting on the existing breakwater.

Both options eliminate the primary cause of the damage which has been attributed to excessive deflection of the breakwater under severe wave conditions.

UNDERWATER SURVEYS

Each panel attached to the breakwater piles is 3.0 m wide, 6.71 m long and 0.25 m thick. A few observations from shore have been documented and they reveal the following number of missing panels at the surface:

Dec 27/68	1	panel (partially completed)
Nov 7/88	10	panels (3 were not installed by design)
Nov 17/88	11	"
April/89	11	"
March/90	15	"
May 10/90	16	"

Eight of the missing panels occur in a 39 m long damaged section (13 panels) that has moved laterally about 0.6 m to windward. Unfortunately, this section is directly opposite the normal mooring site of the Limnos, CCIW's largest research vessel. This section was the site of a detailed underwater inspection in April 1989 using video obtained with NWRI's remotely operated camera MURV. Subsequently, in March 1990, a similar inspection of the whole breakwater was completed. These inspections revealed that, in general, if the panel is missing at the surface, the underwater part is also missing. However, in several cases, the panel has slid down to the bottom and is still attached, acting as a submerged panel. In addition, many of the remaining panel connections are in poor condition. The worst section has three adjacent panels missing and one of the next two panels missing, giving four of five consecutive panels missing.

FLOATING BREAKWATERS

Floating breakwater technology has evolved considerably since 1968 when the CCIW breakwater was designed. Some examples of field experience since 1964 with floating breakwaters (floating tire, concrete caisson, A-frame) in North America are summarized by Nece et al. (1988). Since the maximum fetch within Hamilton Harbour to the CCIW breakwater is only 7100 m, floating breakwaters can be a feasible component of the rehabilitation.

Floating breakwaters reflect and/or dissipate some of the incident wave energy and transmit the rest. Their performance is characterized by a transmission coefficient C_t which equals the transmitted wave height H_t divided by the incident wave height H_i. As a partial vertical breakwater, the CCIW breakwater also transmits part of the incident wave energy, by it passing under the panels which don't extend to the bottom.

Floating Tire Breakwaters (FTBs) have proven to be environmentally-friendly and very cost-effective when constructed using state-of-the-art guidelines (Bishop et al. 1983). The 35,000 tire FTB at LaSalle Park Marina in Hamilton Harbour has functioned very well and has met expected performance criteria since being installed in April 1981 (Bishop 1985). The first major maintenance was performed in drydock in November 1990, consisting of the replacement of some conveyor belting, bolts and washers used for connections. In addition, the FTB has been relocated from November to April each year to another site in the Harbour to protect it from ice floes. A 7,200 tire FTB at Morch Marine in the Moira River estuary at Belleville, Ontario was installed in 1985. It too has performed well and, aside from its first winter in 1986, has been left year-round at its normal mooring site. Ice from the river and the Bay of Quinte has not caused any problems for the FTB.

There are several different designs for FTBs but by far the most common is the so called "Goodyear" design. It consists of modules, each containing 18 tires, interconnected to form a flexible mat as shown in Figure 3. Its performance is a function of the ratio of wavelength L to the breakwater beam dimension B; for L/B less than 0.8, C_t is less than 0.5 (Bishop 1985). Other designs that incorporate pipes and denser arrays of tires (Harms et al. 1981, Pierce 1984) may provide the same attenuation with smaller beams, although at a higher overall cost. Both the LaSalle Marina and Morch Marine breakwaters are of the Goodyear type.

WAVES

A rudimentary wave measurement program was undertaken by Mackenzie (1969) at CCIW between June 21 and October 25, 1969. Water level pressure gauges were used to measure the water level fluctuations but the results were not corrected by the pressure response factor K_p. Accordingly, the recorded wave heights should be divided by K_p which varies from approximately 0.60 to 1.00 for the

depths of submergence encountered. Tabulated wave heights are the largest in each hour. For the gauge located at the midpoint of the CCIW wharf, the largest recorded wave height was 0.58 m (during a northerly wind).

The CCIW breakwater is designed for westerly waves. Model tests by Brebner (1968) determined that the as-designed breakwater would have a transmission coefficient of 0.15 for a 4 s incident wave. In its as-designed condition, the CCIW breakwater was considered to provide a satisfactory wave climate for the berthing of the research vessels along the CCIW wharf. However, as some of the wave-reflecting panels fell off, the transmitted wave energy during westerly storms increased. On 19 March 1986, during a 35 knot westerly wind, the CSS Bayfield had a hole punched in her side while rolling at the wharf. The damage was probably caused by a piece of driftwood being lodged between the wharf and the side of the ship.

Southerly winds can also cause wave agitation problems along the CCIW wharf and the small boat basin. These problems were addressed in an earlier model study after which it was concluded that an additional breakwater need not be constructed (Dick 1970).

During easterly storms, long period (7 s) waves from Lake Ontario propagate through the Burlington Canal. These waves diffract and enter the area on the lee side of the CCIW breakwater, causing large vertical movements of ships moored along the CCIW wharf. In order to determine if roughening part of the wharf with rubble would effectively dissipate wave energy from easterly storms, calculations were performed following the methods in Bishop (1987).

The combination of long wave period, deep water and wide gap between the wharf and the CCIW breakwater make rubble-lining ineffective. The most likely way to improve the wave climate along the wharf for easterly, as well as southerly, storms would be to build a bottom-resting breakwater at the south end of the CCIW wharf extending out to the CCIW breakwater. This was rejected by Dick (1970) for reasons of cost and increased ice formation north of such a breakwater.

For westerly storms, waves at the CCIW breakwater have been hindcast. The longest straight-line fetch is 7100 m to the west-southwest (Figure 4). The open water area narrows considerably at a line joining Willow Point and the Centennial Dock. Accordingly, an effective fetch of 5300 m to this line has been used.

Two wind speeds were selected for hindcasting waves. These wind speeds, U, are representative of overwater conditions at a height of 10 m above the water:

1. U = 25 m/s = 90 km/hr = 56 mph = 49 knots
2. U = 15 m/s = 54 km/hr = 34 mph = 29 knots

The maximum hourly southwest wind speed recorded at nearby Hamilton Airport between 1969 and 1979 was 89 km/hr. Although a comprehensive statistical frequency analysis of the wind data was not done, the 25 m/s speed from the WSW can be considered to be approximately a one in 10 year storm (see also Donnelly 1969). Wind speeds of 15 m/s or more from the WSW usually occur during several storms each year.

Six sets of wave prediction equations have been used to estimate characteristic wave height H_{mo} and peak period T_p: those of SMB, Donelan and JONSWAP as compared by Bishop (1983), the shallow water SMB equations (U.S. Army, CERC 1977) for a constant water depth of 15 m, and deep and shallow water equations in ACES version 1.04 (U.S. Army 1990):

	U = 25 m/s		U = 15 m/s	
	H_{mo} (m)	T_p (s)	H_{mo} (m)	T_p (s)
SMB (deep water)	1.44	4.39	0.79	3.36
SMB (d = 15 m)	1.35	4.11	0.77	3.21
Donelan	1.25	3.81	0.66	2.89
JONSWAP	0.93	3.19	0.56	2.69
ACES (deep water)	1.26	3.52	0.64	2.81
ACES (d = 15 m)	1.21	3.37	0.63	2.69

Based on comparisons of measured and predicted wave data (Bishop 1983, Bishop et al. 1989), and on the author's experience, it was decided to average the predictions of the SMB shallow water and the Donelan equations to arrive at the design waves. This results in 1.3 m/4.0 s and 0.72 m/3.1 s.

WAVE HEIGHT CRITERION

Discussions were held with Mr. Bob Marshall, Regional Marine Superintendent, Dept. of Fisheries and Oceans and with Capt. M.C. Birchall of the CSS Bayfield. Capt. Birchall said that he could tolerate wave heights of 0.6 m but not 0.9 m in the lee of the CCIW breakwater. He said that this wave height could be considered the significant (or characteristic) wave height.

Waves transmitted past the CCIW breakwater are reflected by the vertical wall at the CCIW wharf. Assuming a reflection coefficient of unity, the transmitted wave height H_t will be doubled by reflection. If it is further assumed that the acceptable characteristic wave height in the lee of the CCIW breakwater can be up to 0.75 m during a 1 in 10 year storm, the transmitted characteristic wave height must be less than or equal to 0.375 m.

HYDRAULIC MODEL

It was decided to construct a two-dimensional model of the CCIW breakwater. Model car tires with an outside diameter of approximately 8.5 cm were available in the Hydraulics Laboratory. This is representative of 1:8 scale car tires so a model length scale of 1:8 was used.

The hydrographic field sheet shows water depths of 7.0 to 8.8 m below datum along the length of the breakwater. Lake Ontario mean monthly water levels vary about datum from -0.3 m to +1.6 m, with an average of about +0.5 m. A design water depth of 8.0 m was chosen, giving a model depth of 1.0 m. Depths along the southwest fetch vary from 8 m to 23 m. For a 4 s wave period, the ratio of depth to wavelength is 0.32, which indicates that the effects of refraction and shoaling can be considered negligible. Therefore, the harbour bottom contours were not modelled, instead the model was built with a flat bottom.

In order to minimize wave reflection problems in the laboratory, it was decided to construct the model in a wave basin (Figure 5). The breakwater model was installed in a 2 m wide channel that had been built as an extension to the basin. Waves from the 5 m long wave generator were dissipated by a crushed stone beach at 6:1 on either side of the test channel. Reflected waves from the breakwater model were attenuated by diffraction when they propagated out of the channel, by floating horsehair mats attached to the perimeter of the basin, and by making the concrete block wave guidewalls semi-permeable. This was accomplished by turning alternate blocks in the top two courses on their side so that the open faces allowed some wave energy to be transmitted through the walls. Wave energy transmitted past the breakwater model was dissipated by a crushed stone beach at 4:1 at the end of the channel.

The portable Kelk wave generator has optimal performance characteristics when operating in a water depth of 0.6 m. Therefore, it was placed on specially constructed concrete pads 0.4 m high.

Fetch-limited irregular waves with a DHH spectrum (Donelan et al. 1985) were generated using GEDAP wave generation and analysis software obtained from the National Research Council Hydraulics Laboratory in 1984. The design wave sequences were 200 s long with random phasing. Data was sampled for 4096 scans at 0.045 s.

Waves were measured using capacitance probes with electronics designed by M. Pedrosa, Research Support Division, NWRI. Calibration was done by raising and lowering the probes using spacers in still water at a constant depth. Over the two month testing period, repeated calibrations showed a maximum difference between calibrations of 1.1%. Forces were measured using load cells manufactured by Interface, Inc. model SM-250 (1112 N range). The manufacturer's calibrations were used after partial verification tests using a pulley system with loads up to 133 N.

In order to avoid having to submerge any of the load cells, the model was not cantilevered from the bottom as in prototype, but rather was suspended from a support frame (Figure 6). The model breakwater and support frame were designed by N. Madsen of NWRI's Research Support Division. Engineering drawings are available. To summarize, the model is essentially a rigid structure supported by two linear bearings. The bearings restrain the structure but permit it to transmit horizontal forces to a load cell on either side of the support frame.

The sum of the forces on these two horizontal load cells gives the total horizontal force exerted on the 2 m wide model. In addition, the frame is pivoted about the centreline of the supporting tube. The model was balanced in its operating position and then was restrained by a vertical load cell. Moment loads due to wave action are transmitted through the frame to the vertical load cell. The moment arm from the centreline of the tube to the vertical restraining load cell is 1.35 m. The maximum vertical force F_v times the moment arm 1.35 m equals the horizontal component of the wave load on the breakwater face times its moment arm. The depth in model units from the tube centreline at which the horizontal force acts is $1.35(F_v/F_h)$, making the simplifying assumption that the maximum horizontal and vertical forces occur at the same time. This depth needs to be multipled by 8 to get the corresponding prototype value.

The CCIW breakwater model was tested with several different combinations of missing panels. Each prototype panel was modelled by three model panels, each of which was only one third the scaled length of the prototype panel. In this

way, a top panel could be removed while still leaving the lower two panels attached to represent a prototype panel that had slipped to the bottom. As shown in Figure 6, the panels were numbered sequentially from left to right, top to bottom.

During the course of running the model tests, the following interested parties observed the model:

Mr. R. Marshall, Reg'l Marine Superintendent, DFO
Capt. M.C. Birchall, CSS Bayfield, DFO
Capt. W. Corkum, Ass't Marine Superintendent, DFO
Mr. J. Hall, Director, Small Craft Harbours, DFO
Mr. D. Blanchard, Small Craft Harbours, DFO
Mr. E. Leesti, Ontario Region, PWC
Mr. P. Does, Ontario Region, PWC
Mr. J. Kahale, A & E Services, PWC
Mr. R. Laporte, A & E Services, PWC
Mr. N. Dezeeuw, Head, Building & Property Services, NWRI
Mr. J. Smith, A/Head, Staff Support Division, NWRI
Dr. R. Daley, Executive Director, NWRI

RESULTS

Tests have been run using three different design waves (DW): DW1 represents a very severe condition for which some preliminary tests were run, DW2 represents the in 1 in 10 year storm, and DW3 represents a fairly frequent storm (as discussed under the section on Waves). The GEDAP software was used to separate incident and reflected wave spectra. Average results in prototype units for incident wave height H_i, peak energy period T_p and transmitted wave period T_{pt} are given below:

Design Wave	H_i (m)	T_p (s)	T_{pt} (s)
DW1	1.01	4.77	4.77
DW2	0.98	3.98	4.17
DW3	0.73	3.38	3.26-4.17

The response of the wave generator limited the wave height that could be attained for the two largest design storms. However, results from DW2 can be used to assess performance during the 1 in 10 year design storm. The transmission coefficients C_t given in Table 1 for DW2 tests can be applied to the hindcast wave height of 1.3 m to determine the corresponding transmitted wave height. The force results in Table 1 are in prototype units. The depth (d) is the vertical distance in prototype units from the top of the breakwater coping to the point where the resultant horizontal force acts.

First of all, it is interesting to note the performance of the breakwater in its as-designed condition. From test J06T02, C_t = 0.23 for DW2. This is slightly greater than the value of 0.15 reported by Brebner (1968). His tests were conducted at a scale of 1:24 using monochromatic waves. Using the present value of 0.23 gives H_t = 1.3(0.23) = 0.30 m. After reflection, the wave height in the lee of the breakwater would be 0.60 m. This agrees closely with Capt. Birchall's estimate of acceptable wave conditions. For design storm 2, H_t = 0.17 m, so that after reflection the wave height would be 0.33 m.

Two mooring configurations for the FTB were tested. Conventionally, FTBs are moored with anchors on the windward and leeward sides, using mooring lines with a scope of at least four. In the model, this was represented by tying the FTB with nylon ropes to bolt anchors in the wave guide walls. However, since the CCIW breakwater is basically still structurally sound, an alternate mooring arrangement was tested in which the FTBs leeward side was tied up to the CCIW

breakwater and no windward anchors were used. This has the potential for significant cost savings by omitting many anchors and mooring lines as well as their placement using a barge. In this configuration, it was observed that the FTB tended to bunch up or compress slightly.

As expected, the forces exerted on the breakwater are less when the FTB is moored away from the CCIW breakwater compared to the case when the FTB is moored to the CCIW breakwater. However, in both cases there is a significant reduction in the forces compared to the no FTB case. Presumably, use of an FTB with no other maintenance to the CCIW breakwater would prolong the life of the remaining panel connections. For DW2, use of a 14x7 module FTB moored away reduced the horizontal force exerted on the as-designed CCIW breakwater from 361.2 kN/m to 198.9 kN/m. Since wave forces are roughly proportional to the square of the wave height, the percentage reduction would be even greater if the full design wave height representative of 1.3 m could have been realized in the model.

Two different sizes of FTB were tested: 12x7 modules and 14x7 modules. These represent prototype beam dimensions of approximately 22.6 m and 26.3 m respectively. All available tires were used in the latter model, so larger beams could not be tested.

Eventually, if no maintenance is done, all panels can be expected to fall off. When waves propagate through a structure with gaps in it, each gap constitutes the origin of a wave diffraction pattern. However, no attempt was made to make detailed measurements of diffracted wave patterns. Instead the wave probe on the lee side of the breakwater remained in one position, at the centreline, 0.77 m from the model's front face. Therefore, under these conditions, the measured "transmitted" wave height is really a combination transmitted-diffracted wave height, and it is strictly valid only at that point. It is safe to say that, due to diffraction effects, this measured wave height is larger than that which would reach the CCIW wharf before reflection. Diffraction and reflection from the CCIW wharf will combine to produce a complicated short-crested (three-dimensional) wave pattern behind the breakwater. For the

cases with diffraction, the model measurements can be used to give an indication of relative wave transmission. For example, for DW2 with a 14x7 module FTB moored away, C_t is 0.35 with panels 2-4 out, and can be compared to tests without diffraction such as 0.16 with all panels on, 0.45 with panels 1-5 out, 0.49 with panels 1-10 out, and finally to 0.53 with all panels out.

Table 1. Summary of CCIW Breakwater Model Test Results

FILENAME	WAVES	C_t	F_h (kN/m)	F_v (kN/m)	d (m)	TEST CONDITIONS
M02T02	DW1	0.27	314.6	247.6	5.84	CCIW b/w as designed = BWAD
J06T02	DW2	0.23	361.2	247.6	4.72	BWAD
J06T03	DW3	0.23	191.0	180.7	7.52	BWAD
M04T04	DW1	0.25	270.1	189.4	4.88	BWAD with 12x7 module FTB moored to
M04T02	DW1	0.23	216.3	149.5	4.80	BWAD with 12x7 module FTB moored away
J06T04	DW2	0.17	253.2	164.9	4.32	BWAD with 14x7 module FTB moored to
J06T06	DW2	0.16	198.9	145.2	5.20	BWAD with 14x7 module FTB moored away
J06T05	DW3	0.13	111.1	83.7	5.44	BWAD with 14x7 module FTB moored to
J06T07	DW3	0.12	89.3	64.3	5.04	BWAD with 14x7 module FTB moored away
M04T05	DW1	0.45	172.8	131.1	5.52	12x7 FTB moored to, with panels 2 & 4 out
M14T02	DW2	0.37	198.9	116.0	3.60	12x7 FTB moored to, with panels 2 & 4 out
M14T03	DW2	0.38	154.4	105.2	4.64	12x7 FTB moored away, with panels 2 & 4 out
M22T02	DW3	0.25	72.7	52.0	5.04	12x7 FTB moored away, with panels 2 & 4 out
J08T03	DW2	0.38	161.8	103.4	4.24	14x7 FTB moored to, with panels 2-4 out
J12T03	DW2	0.35	127.2	90.6	4.96	14x7 FTB moored away, with panels 2-4 out
J08T02	DW3	0.22	53.8	52.2	4.40	14x7 FTB moored to, with panels 2-4 out
J12T02	DW3	0.25	61.4	42.8	4.80	14x7 FTB moored away, with panels 2-4 out
M22T03	DW3	0.26	63.0	45.3	5.04	12x7 FTB moored away, with panels 2-4 out
J05T03	DW2	0.50	109.8	70.9	4.24	14x7 FTB moored to, with panels 1-5 out
J05T04	DW2	0.45	85.2	58.6	4.72	14x7 FTB moored away, with panels 1-5 out
J05T02	DW3	0.38	46.8	32.5	4.80	14x7 FTB moored to, with panels 1-5 out
J12T05	DW2	0.42	89.1	64.8	5.12	14x7 FTB moored away, panels 2-4 & 7-9 out
J12T04	DW3	0.25	41.0	29.2	4.96	14x7 FTB moored away, panels 2-4 & 7-9 out
M22T04	DW3	0.29	43.8	31.7	5.12	12x7 FTB moored away, panels 2-4 & 7-9 out
J05T06	DW2	0.48	75.5	39.7	2.96	14x7 FTB moored to, panels 1-5 & 7-9 out
J05T05	DW2	0.50	42.8	31.2	5.20	14x7 FTB moored away, panels 1-5 &7-9 out
M28T03	DW2	0.58	81.9	41.0	2.72	12x7 FTB moored to, panels 1-5 & 7-9 out
M28T02	DW2	0.52	46.6	34.3	5.28	12x7 FTB moored away, panels 1-5 & 7-9 out
M28T05	DW3	0.44	37.4	20.7	3.28	12x7 FTB moored to, panels 1-5 & 7-9 out
M24T03	DW3	0.36	19.7	16.9	6.56	12x7 FTB moored away, panels 1-5 & 7-9 out
J05T07	DW2	0.53	51.2	21.5	1.84	14x7 FTB moored to, panels 1-10 out
J05T08	DW2	0.49	19.2	17.4	7.12	14x7 FTB moored away, panels 1-10 out
J12T06	DW3	0.36	8.7	10.2	10.0	14x7 FTB moored away, panels 1-10 out
J12T08	DW2	0.43	77.6	56.1	5.12	14x7 FTB away, panels 2-4,7-9 & 12-14 out
J12T07	DW3	0.26	35.8	25.3	4.96	14x7 FTB away, panels 2-4,7-9 & 12-14 out
J12T11	DW2	0.53	5.1	3.1	3.76	14x7 FTB moored away, panels 1-15 out
J12T10	DW3	0.37	2.3	1.5	7.20	14x7 FTB moored away, panels 1-15 out

DISCUSSION

For the smaller design storm (DW3), a 14x7 module FTB moored away would result in a combined characteristic wave height (including reflection) of no more than 0.55 m at the CCIW wharf, even if all the panels are off the CCIW breakwater.

For the larger design storm (DW2), a 14x7 module FTB moored away would result in combined characteristic wave heights (including reflection) at the CCIW wharf no larger than the following:

Panels out	H (m)
1-15	1.38
1-10	1.27
1-5	1.17
2-4,7-9&12-14	1.12

Clearly, these wave heights exceed the suggested acceptable limit of 0.75 m. None of the tests reproduce exactly the conditions that now exist at the damaged section of the CCIW breakwater. The latter is indicative of the worst section with three adjacent panels missing. However, the resulting wave height is quite conservative because it virtually ignores the height-reducing effects of diffraction. It is possible that the use of an FTB of this size might result in a satisfactory wave climate at the CCIW wharf if most of the panels remain attached. And it is likely that the use of an FTB moored away would prolong the life of the panel connections.

Use of an FTB moored to the windward side of the CCIW breakwater might even accelerate the rate at which the panels drop off due to it rubbing against the panels; as a worst case, all the panels might fall off. In such a case, what

size of FTB beam would be required to achieve H_t = 0.375 m? The results of the final two tests with all panels off can be used to check the wave transmission characteristics of the FTB relative to the design curve in Bishop (1985). For DW2, using a wavelength calculated by linear theory, and the measured beam dimension, one gets L/B = 0.92, and the design curve gives C_t = 0.6; this compares with C_t = 0.53 from test J12T11. Similarly, for DW3, L/B = 0.68, C_t design = 0.40 and J12T10 gives C_t = 0.37. Therefore, it appears that the design curve gives results that are slightly conservative for the conditions at the CCIW breakwater. In order to get C_t = 0.375/1.3 = 0.29, the design curve modified by this experience gives a required beam of 39.6 m or 21 modules.

CONCLUSIONS

A Goodyear-design FTB can provide the desired wave attenuation at the CCIW breakwater site. A beam dimension of about 40 m (21 modules) would be required in the worst case of all the existing wave-reflecting panels having fallen off. A smaller beam, perhaps 26 m (14 modules), might also be adequate if it is assumed that most of the panels remain on the CCIW breakwater.

To protect the whole 518 m length of the breakwater would require about 250 modules in length. At 20 tires per module (including two link tires), the total number of tires needed to construct the FTB would be 105,000 for the wider beam, or 70,000 for the narrower one. PWC has prepared a Class "D" estimate for the wider beam option at $1.2 million.

Since tires are now considered a liability for waste disposal and many sanitary landfills will not accept them, local tire disposal costs vary from $1 to $5 per tire. This presents a revenue-generating potential for any FTB project. One of the key factors in realizing this potential is to have a long lead time to procure the tires.

Due to the increasing rate of deterioration and the unacceptable performance of the breakwater, an emergency project should be considered. The damaged section of the CCIW breakwater, in which 8 of the breakwater's total 16 missing panels are located, should be protected by a minimum 14 module (beam) by 25 module (length) FTB moored to the CCIW breakwater as soon as possible. This 50 m long section would constitute one tenth of the whole project, would protect the degraded berthing area along the wharf, and would also provide valuable experience in the following areas:

1. Collecting tires for revenue
2. Ice-FTB interaction
3. Mooring the FTB directly to the CCIW breakwater

4. Does the FTB help to decrease the rate at which the panels fall off?
5. Beam dimension required.

If it is not possible to use the waterlot on the windward side of the CCIW breakwater, the FTB could be moored on the CCIW breakwater's leeward side. Of course, this would require the wider 21 module beam FTB, would not slow the rate of deterioration of the CCIW breakwater, and would reduce the open area for manoeuvering ships.

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Many people provided valuable input to this study. Special thanks to Dave Beesley for his conscientious work as head technologist. George Voros and Chris Lomas helped to construct the model. Niels Madsen designed the support frame and breakwater model. Terry Nudds helped with the instrumentation. Jay Doering helped with the GEDAP software.

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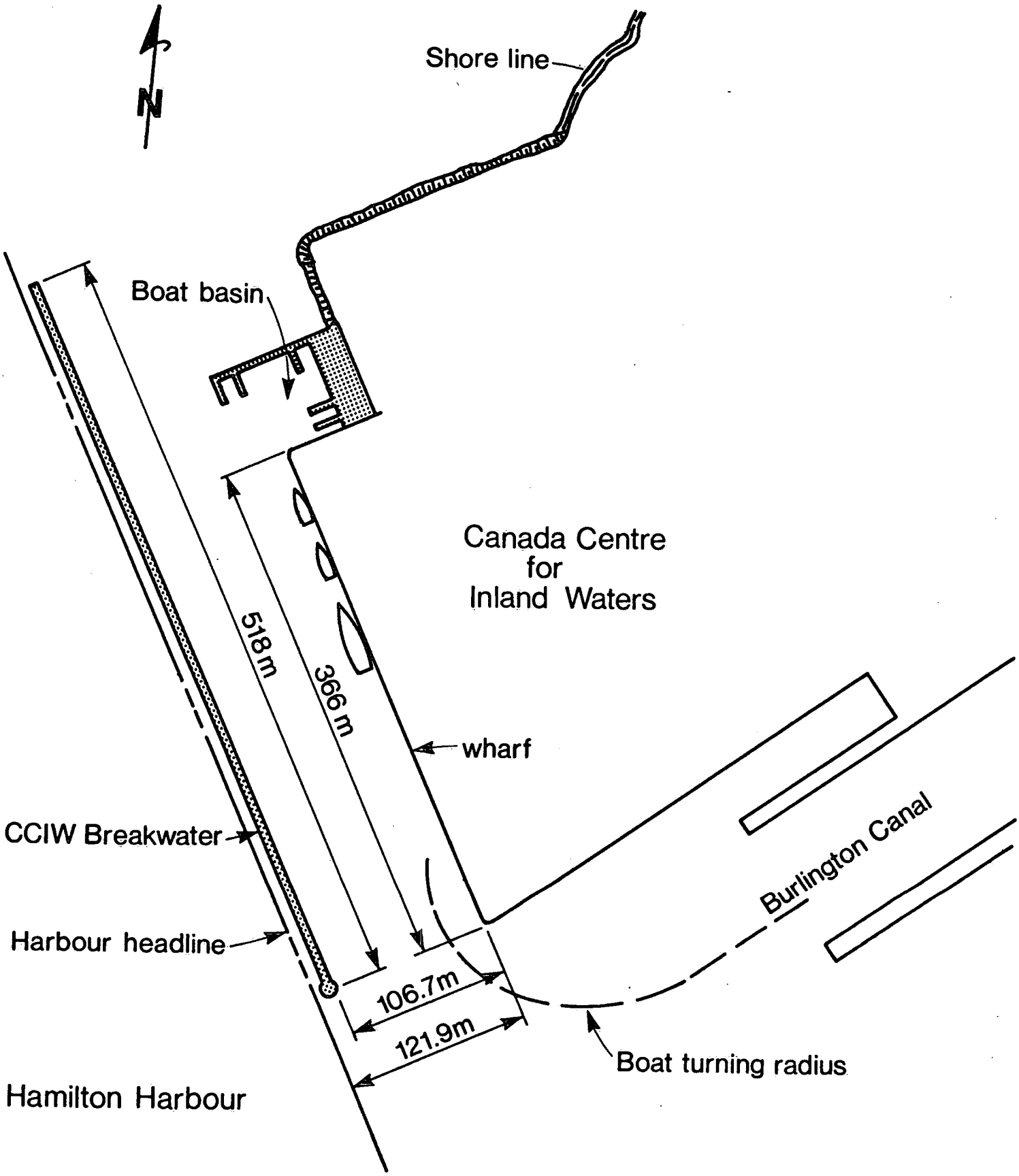
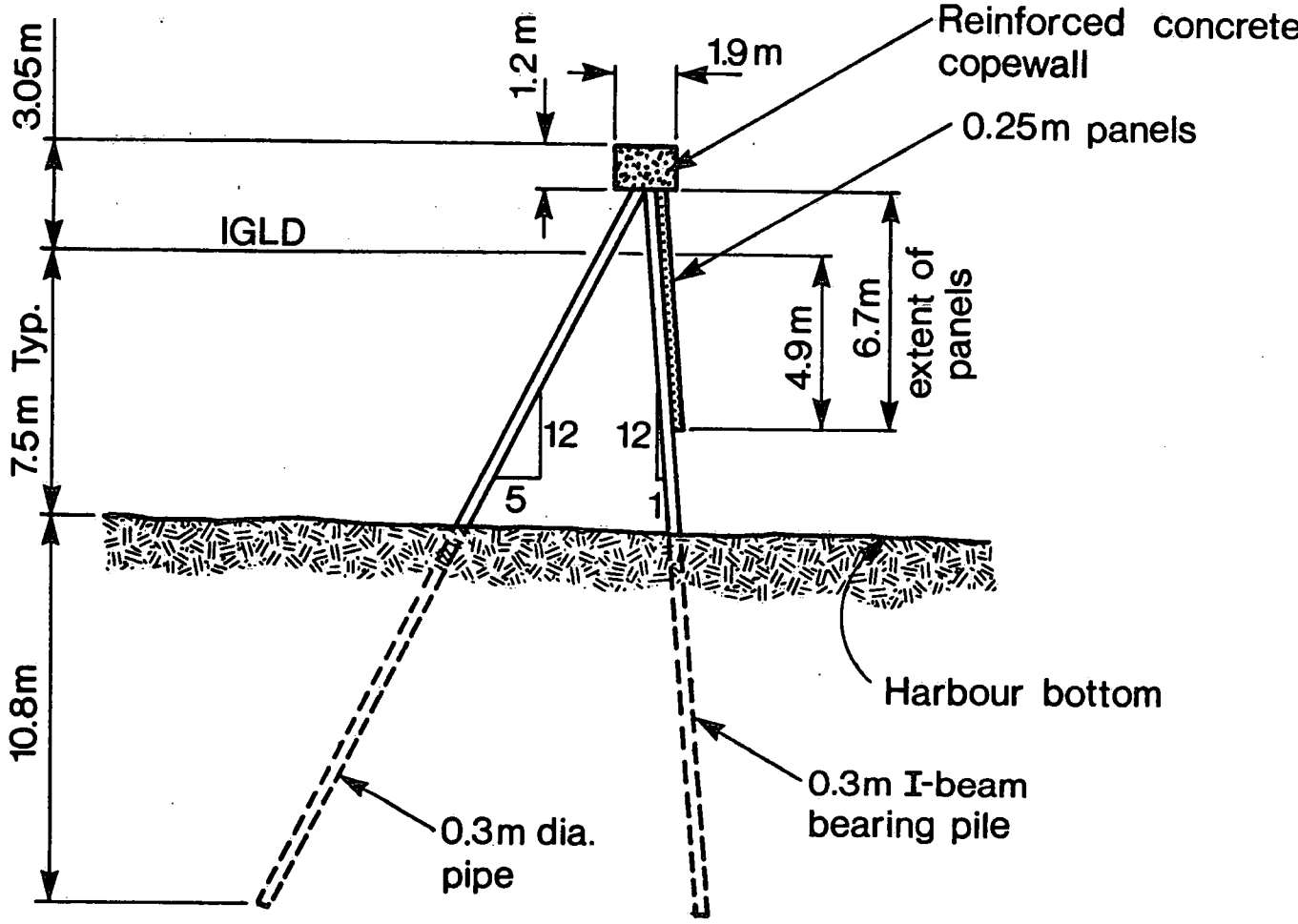
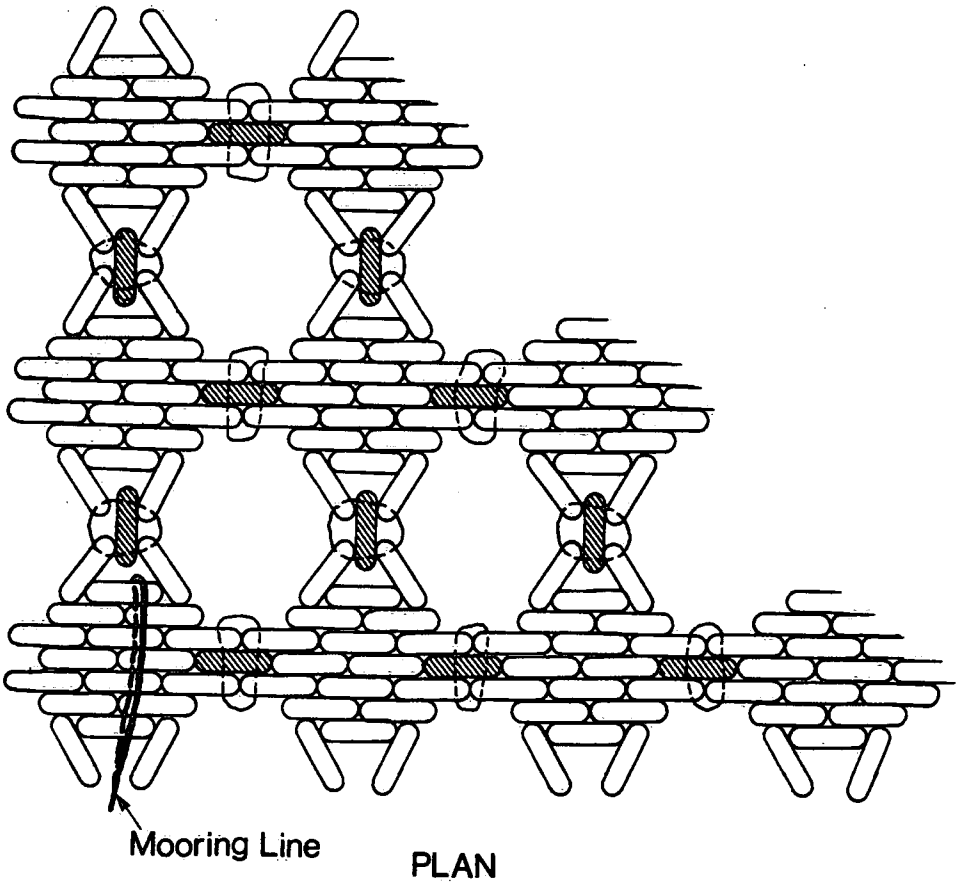


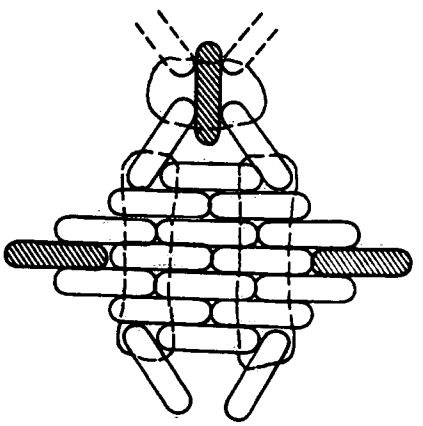
Figure 1 CCIW Marine facilities (after Hardtke 1968)



Note: pile bents at 3.05m cts

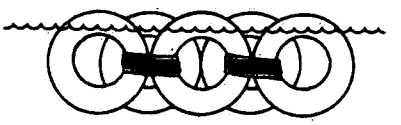
Figure 2 Section through CCIW Breakwater (after Hardtke 1968)



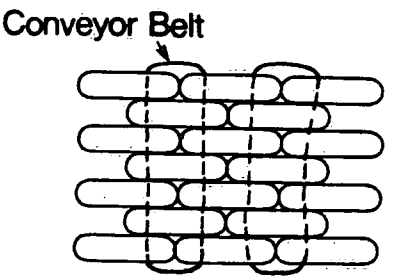


18 Tire Module Detail

Note: each tire equipped with some form of supplemental flotation, tires shown cross-hatched interconnect modules

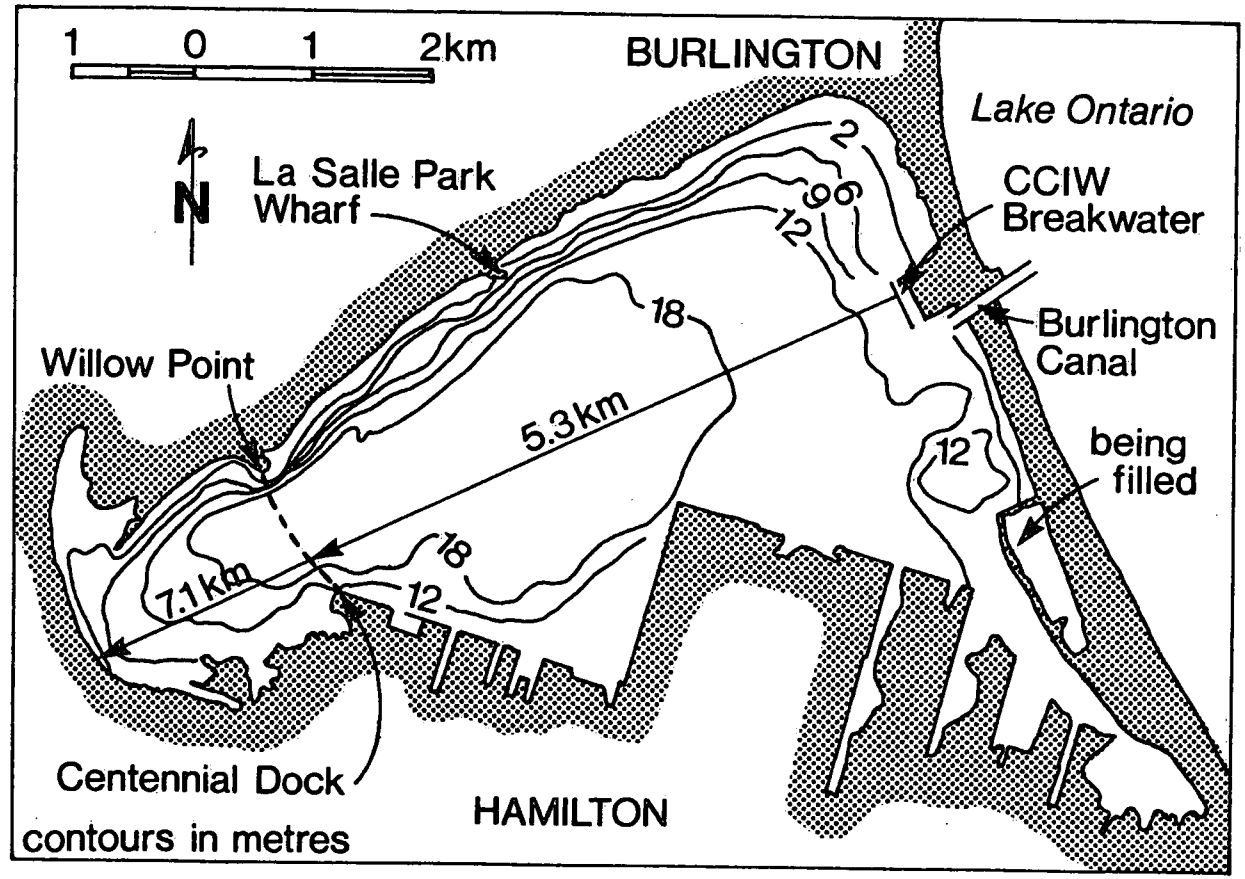


ELEVATION



PLAN

Figure 3. Detailed arrangement of tires in a Goodyear FTB (Bishop 1985)



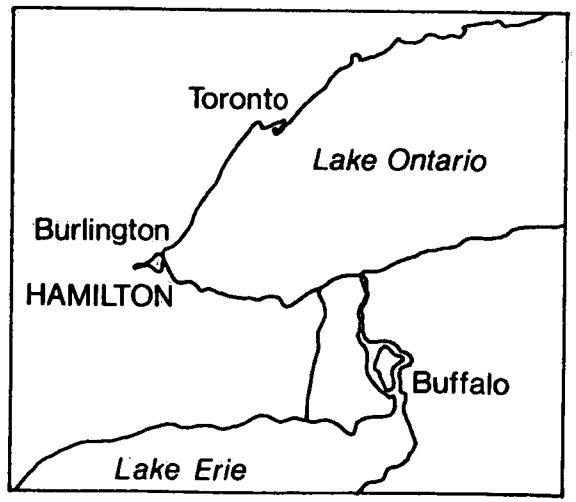


Figure 4 Hamilton Harbour

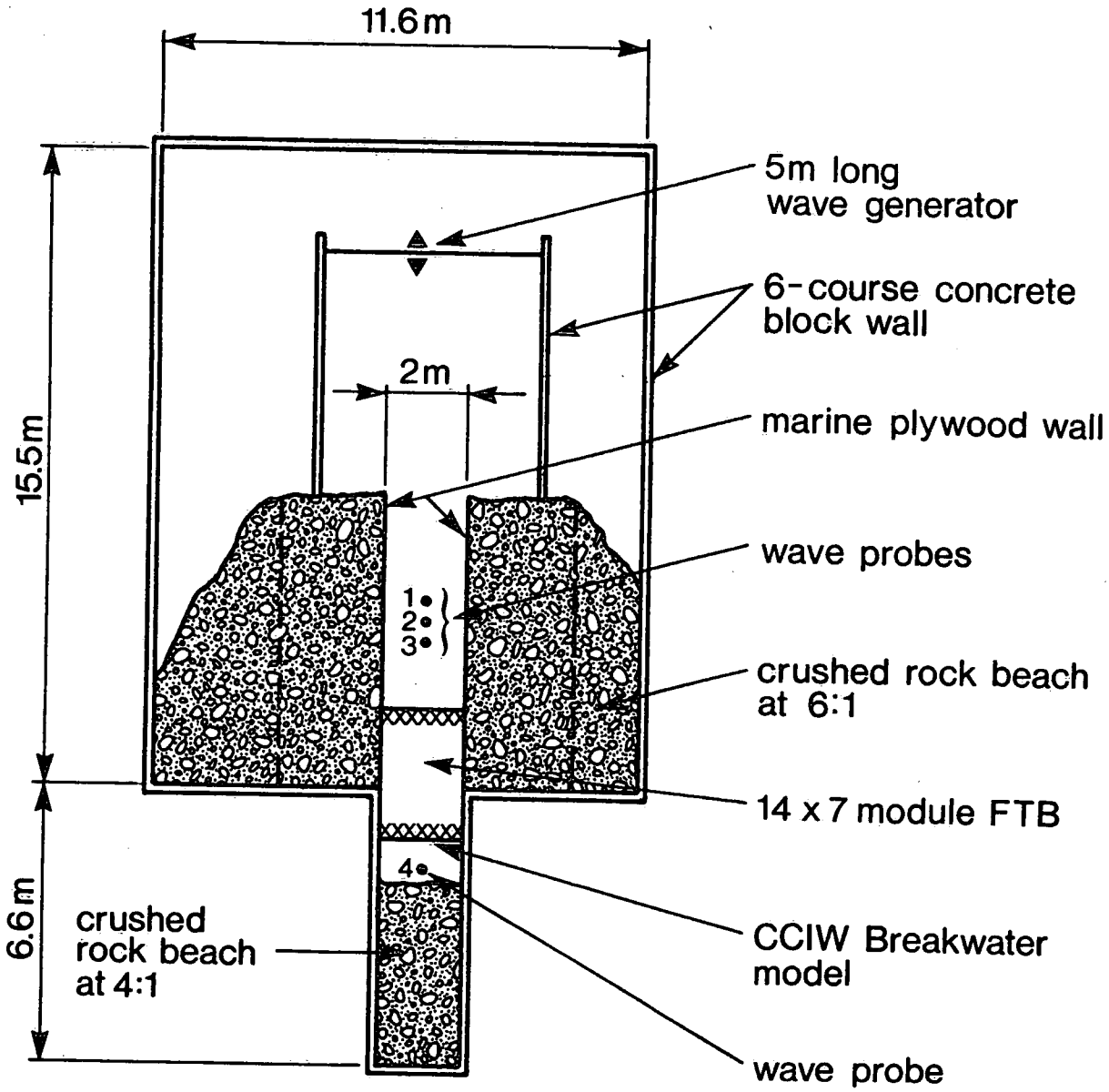


Figure 5 Layout of wave basin

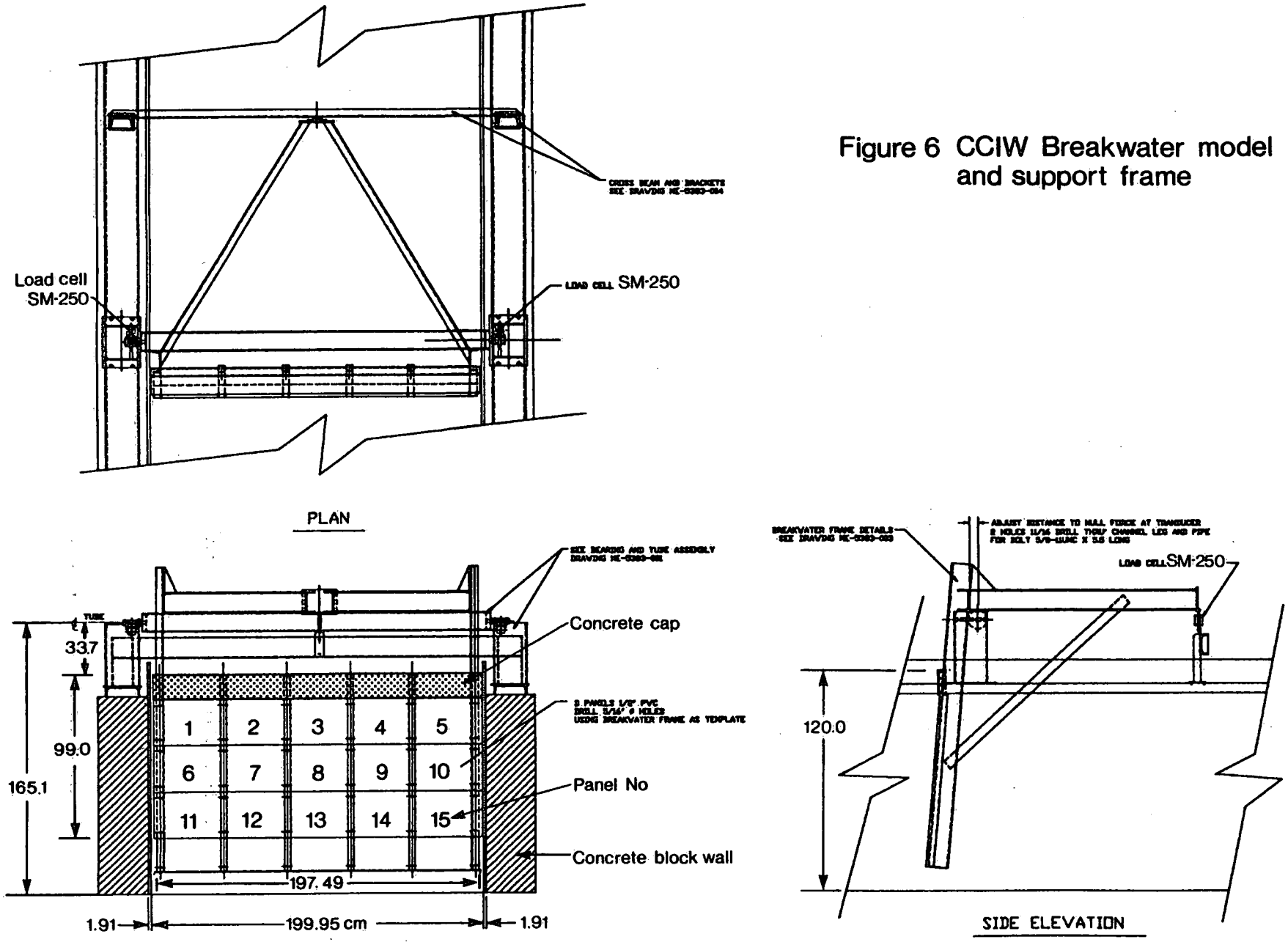


Figure 6 CCIW Breakwater model and support frame

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