Name in News

A fast-track approach
to design and construction

Vancouver's Floating Concrete Heliport

By Thomas W. R. Taylor and H. Roger Woodhead

Describes the design and construction of one of the world's newest and most unique heliports, a floating facility constructed from styrofoam-filled cellular concrete. Some of the interesting aspects of construction are highlighted and the economic benefits of this use of reinforced concrete are discussed.

In June 1986, the Port of Vancouver, British Columbia, Canada, called tenders for a design-build contract to construct what is believed to be the world's first floating concrete heliport. The Vancouver Floating Heliport, a cellular structure as large as an American football field, was in revenue service before the end of the same year.

General requirements

In July 1986, Dillingham Construction Ltd., of North Vancouver, was awarded a contract to construct the facility. The consulting engineers, Taylor Peach and Associates Ltd., of Vancouver, who had developed the conceptual design for the bid, began immediate work on the detailed design, since a tight schedule to complete the project before the end of the year called for a fast-track approach.

Keywords: concrete construction; connections; floating bodies; heliports; pontoons; post-tensioning; structural design.

The contractor's plant, located on the north shore of Burrard Inlet, the main harbor of Vancouver, includes a drydock that was of a size and shape suitable to construct the heliport facility. The completed heliport was to be located on the south shore of the same body of water, as shown in Fig. 1.

The Port of Vancouver defined the plan dimensions and preferred freeboard in the tender documents. The length and width were dictated by the requirement that three landing pads had to be operated simultaneously. The pads were to be designed for the operation of helicopters of 5.4, 7.7, and 22.7 Mg (6, 8.5, and 25 tons) gross weights. The sum of the required pad sizes (each by regulatory code to be 1.5 times the size of the appropriate helicopter) and the clearances between each pad determined the length. The size of the largest pad determined the width. The resulting structure was approximately 86 m long by 33 m wide (282 x 108 ft).

In addition, the Port specified that the design was to be capable of expansion at the largest pad to permit even larger machines to use the facility in the future. For esthetic reasons, the owner required that the freeboard be as low as practical. A design freeboard of approximately 0.8 m (32 in.) was selected.

The conceptual design of the heliport was such that it could be constructed in individual pontoons in the drydock. The sections would then be floated out and joined together. The maximum pontoon size was tailored to the size of the drydock. The width was such that sections 16.5 m (54 ft), half the width of the completed float, could be constructed.

The length of the drydock was shorter than the heliport so that a transverse pontoon had to be built in addition to the two longitudinal pontoons. This pontoon also was designed to be 16.5 m (54 ft) wide. Pontoon sizes and assembled dimensions are shown in Fig. 2.

The freeboard specified by the owner and the overall aim of minimizing construction costs dictated that the completed float should be as shallow as possible. Preliminary analysis and design indicated that an average structural depth of 1.83 m (6 ft) would satisfy the freeboard specification. It was also sufficient to provide enough overall stiffness and section modulus to carry stresses from wave motion without excessive deflection or cracking. Further, it meant that the pontoon did not require prestressing, which reduced the overall cost.

To insure positive flotation, the pontoons were filled with blocks of styrofoam (Fig. 3) so that water could not accumulate in the cells. It

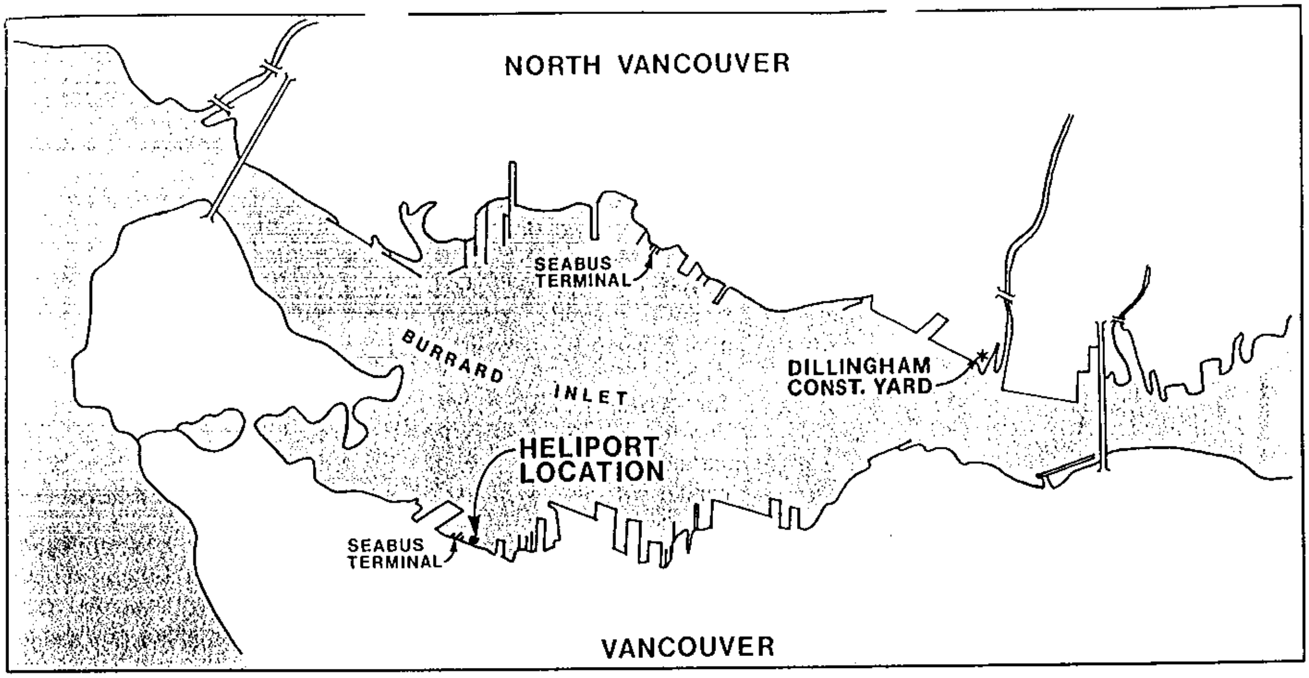


Fig. 1—Vancouver harbor and facility location.

also eliminated the need to form the interior walls and the soffit of the top slab. Epoxy-coated reinforcing was used to improve the durability of the structure as indicated in Fig. 4, which shows placement of the base slab of one of the pontoons.

Analysis and design

Once the general requirements had been set, detailed design proceeded. Although all environmental loading conditions were investigated during analysis, it was apparent that the most severe stresses in the assembled float resulted from wave action. Waves encountered at the heliport were generated primarily by wind and passing marine traffic.

Waves due to wind were typically of a very short length and low amplitude because of the protected location and the relatively short reach to the adjacent shore. The most severe wind-generated waves at the moored location resulted from winds from the north-west to the north-east. The strongest winds in Vancouver were typically from the opposite directions.

Waves generated by marine traffic, however, had a higher amplitude and much longer wave length. The effects of these waves on the structure were investigated in detail, since the facility was to be located adjacent to the south terminal of the Vancouver Sea Bus. The

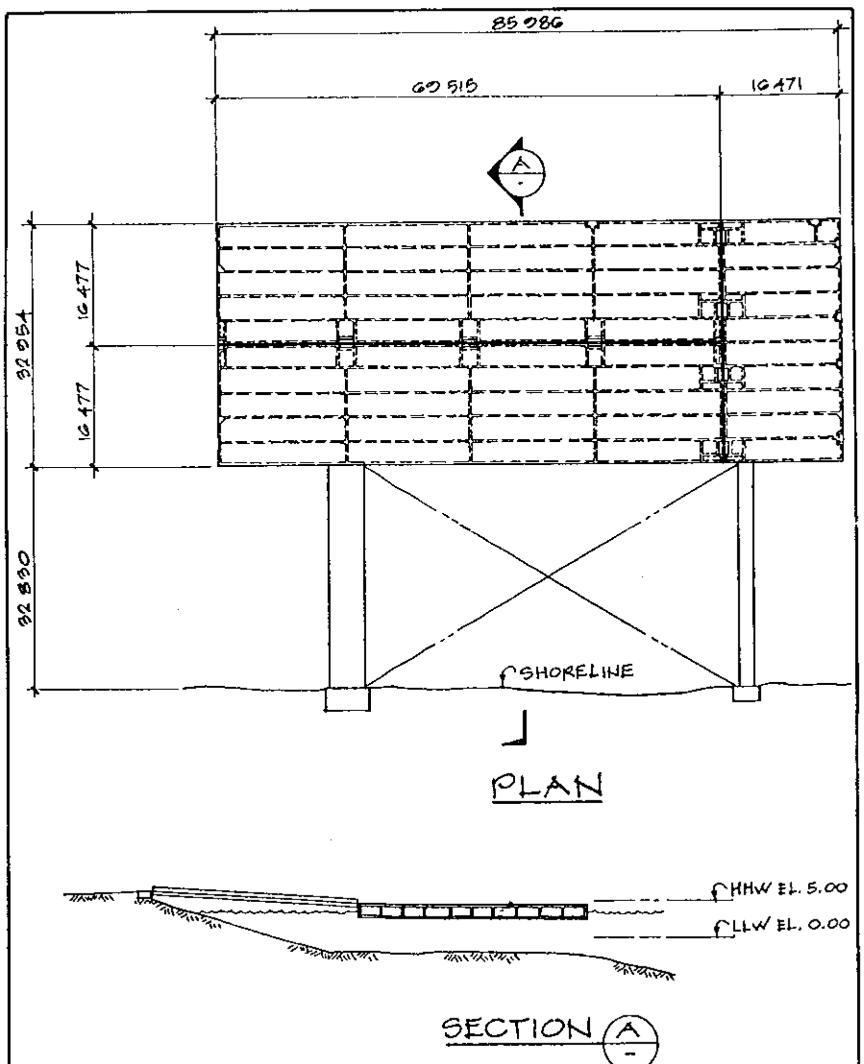


Fig. 2—Plan of the heliport.

bow wave generated by the sea bus had an amplitude of about 0.6 m (2 ft), a wave length of about 10.7 m (35 ft), and typically would strike the float at about a 45 deg angle.

The analysis of the response of the structure to loads induced by wave motions was very complex. Preliminary analysis was carried out using the cosine wave formula

$$M_w = \frac{BH\lambda^2}{2763.5}\left[\cos \pi \frac{L}{\lambda} + \frac{\pi L}{2\lambda}\left(\sin \pi \frac{L}{\lambda}\right) - 1\right]$$

where
L = float length in ft
B = float width
H = wave amplitude
λ = wave length
M_w = bending moment in ton-ft

Since the waves could strike the face of the float at any angle from zero to 90 deg, the wetted distance from wave crest to wave crest along the exposed face of the structure might be considerably greater than the actual wave length of the input wave. Hence, a very long, narrow float would be subject to bending moments that would result if the wave length was equal to the length of the structure.

During preliminary one-directional analysis, the effects of waves having wave lengths varying from 3 m (10 ft) to the length of the structure were investigated. It was quite apparent that the most severe loading would result if a wave having the same wave length as the structure length struck the float so that the angle of incidence to the long side was zero. Also quite obviously, such a situation would never occur. Therefore, a considerable degree of engineering judgment was required to determine what represented realistic wave loadings.

The structural design developed essentially from the analysis of a space structure supported on elastic foundations of varying stiffness representing the uplift from the water.

The numbers and placement of internal longitudinal walls was governed by the stresses due to landing loads on the deck and hydrostatic pressures on the bottom slab. A series of continuous longitudinal walls spaced transversely at about 3 m (10 ft) was determined to be the optimum. Thicknesses of 150 mm (6 in.) for both the top and bottom slabs were consistent with the stresses from one-way bending action. In determining hydrostatic pressures, it was assumed that the float was immersed to deck level.

Analysis for shears and moments in the top slab was carried out using influence surface techniques. This was considered to be the most rapid method in determining the governing positions of the various specified live loads.

The owner had called for the facility to be initially moored close to the downtown shoreline. Future development plans for the site, however, required that the heliport be capable of being moved to an alternative, currently undefined, site within the harbor.

Particular care must be exercised to insure that joining procedures are well thought out. Time spent in design is cheap in comparison to the cost of possible delays if such details cause construction difficulties.

The initial mooring was attained by securing the float to two onshore pile-supported piers using two gangways. In addition to providing access, the gangways acted as articulated stifflegs to keep the float positioned parallel to the shore. Movement of the float parallel to the shore was restrained by a pair of wire rope spring lines connected between the two gangways.

To provide future moorage capability, hawsepipes were cast into the float to accommodate a chain anchorage system. Particular care was paid to the design and detailing of the gangway connections, not only to permit adequate movement, but also to accommodate the high-cycle fatigue and impact loads that were generated by the structure's response to waves.

Joining the pontoons

As stated previously, the heliport was assembled from three pontoon sections. The two longitudinal pontoons were essentially mirror images of one another. The only differences were the inclusion of embedded hot-dipped galvanized fittings to connect the gangways to one pontoon and embedded sleeves for future expansion in the other. A transverse drainage slope of 75 mm (3 in.) was built into each pontoon.

The assembly of large floating concrete sections without the benefit of match casting can be a financial nightmare. As a result, particular attention was paid in the design to joining details that would permit simple and accurate assembly. A series of five "joining wells" were provided adjacent to the common edge of the longitudinal pontoons. The wells were designed so that all work associated with assembling the pontoons into a single float could be accomplished "in the dry" by the contractor.

The joining wells were designed with locally thickened walls to carry the wave-generated moments and shears from one pontoon to the other. To reduce the weight of the thickened walls, styrofoam slab inserts were cast within the concrete where stress levels permitted.

The initial mating of the pontoons was assisted by pintles and sleeves cast into both ends of each pontoon. It was determined that a 100 mm (4 in.) maximum diameter tapered solid pintle would be adequate to prevent relative movement between the pontoons and carry all shears in the protected lagoon during curing of the concrete closure placements. Fig. 5 shows details of the joining wells.

During the joining process, a compressible seal was placed along the contact surfaces of the joining wells. After the pontoons were brought together, a series of post-tensioning bars were inserted into the top row of post-tensioning sleeves and partially tightened (Fig. 6). Portable ballast blocks were then placed along the outer edges of the pontoons to close the seal at the

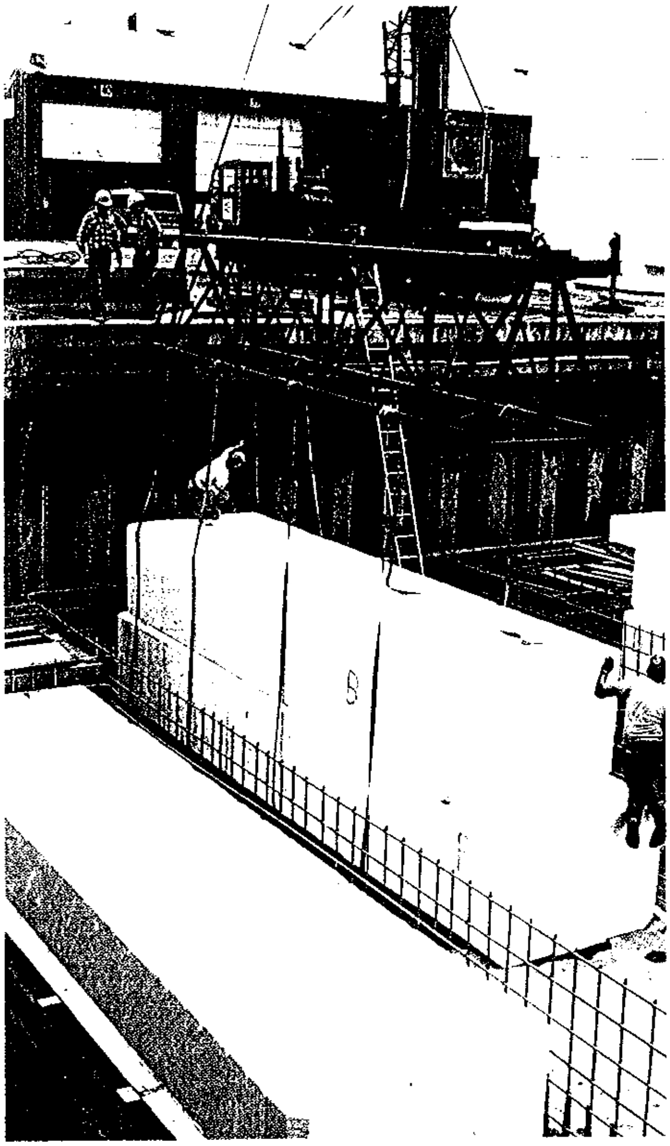
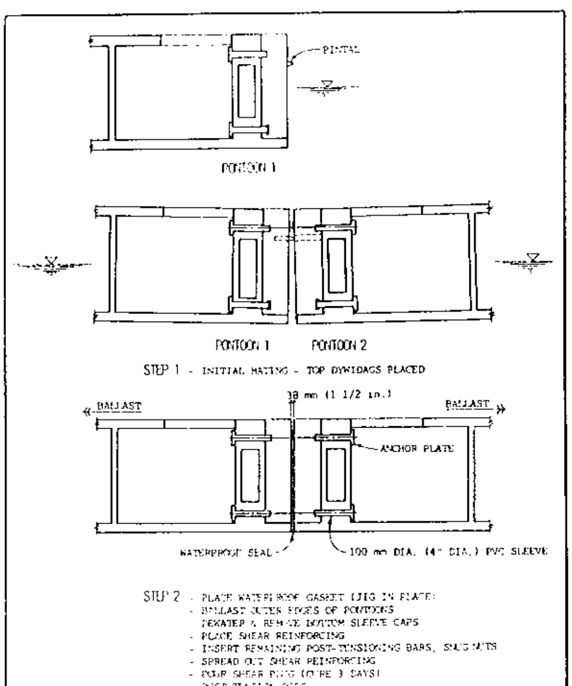
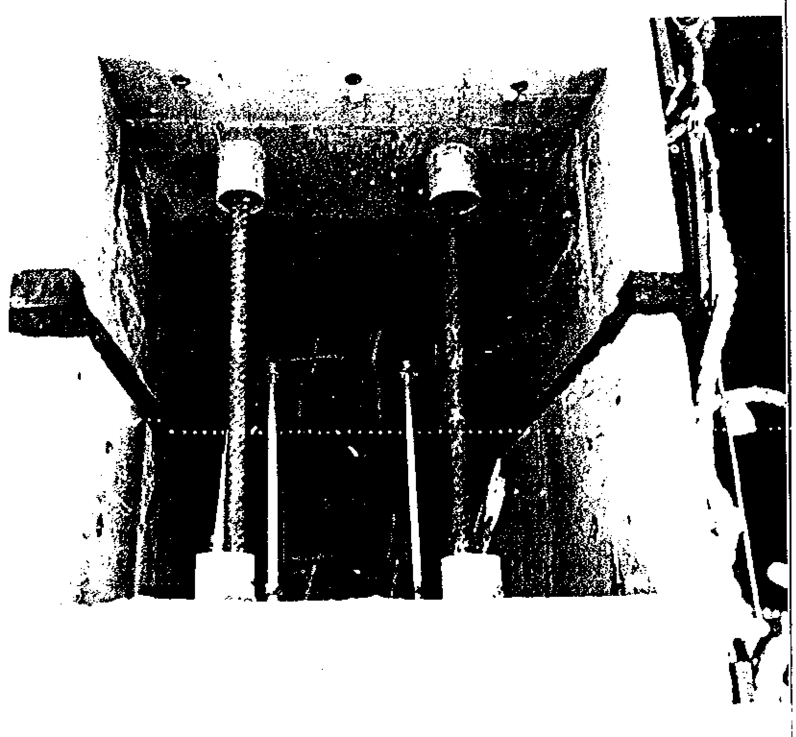


Fig. 3—(top, left) Styrofoam blocks being placed.
Fig. 4—(top, right) Base slab of pontoon being placed.
Fig. 5—(bottom, left) Pontoon joining wells.
Fig. 6—(bottom, right) Detail of joining well.





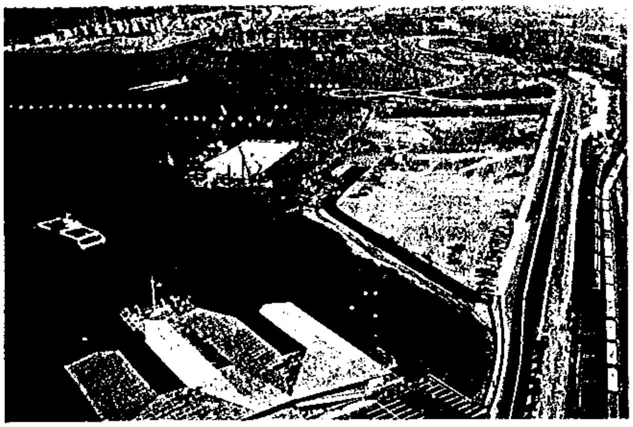


Fig. 7—Installation of heliport.

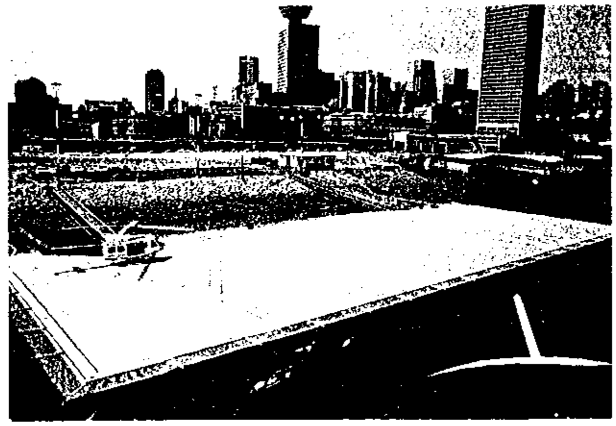


Fig. 8—Completed heliport from incoming helicopter.

bottom of the joining wells. The compartment between the pontoons at each joining well was thus made temporarily watertight.

After dewatering these compartments, the waterproof covers, which had been placed over the outer ends of the bottom row of post-tensioning ducts in the drydock, could be removed and the bottom row of post-tensioning bars inserted and partially tightened.

Before the closure concrete could be placed, shear reinforcing stirrups had to be placed in the joining compartment. Then, after the concrete was cast and cured, post-tensioning rods were stressed to complete the assembly of the first two pontoons. The flotation of the assembly was then checked against the design. Since the third (transverse) pontoon was being constructed in the drydock while the longitudinal pontoons were being connected, time was of the essence if depth modifications were to be made to the third pontoon to assure level flotation of the finally completed structure. The two joined longitudinal pontoons had essentially the same freeboards along transverse cross sections.

However, due to the added weight of the joining wells located along the face that was to be connected to the transverse pontoon, the assembly was deeper in the water at that end. An in-house computer program had previously been developed to predict the flotation characteristics of each pontoon individually and the assembly as a whole. A slight modification to this program permitted the flotation to be updated with actual measured values at each corner of each pontoon. Thus it was possible to accurately predict final flotation characteristics and make the anticipated depth changes to the third pontoon.

An interesting aspect of the flotation was that all pontoons floated deeper in the water than the actual material quantities and measured density of the water would indicate. Actual quantities of concrete placed agreed with design calculations, as did steel and styrofoam quantities. Based on observed conditions of freeboard, the unit weight of concrete including reinforcing was determined to be close to 2725 kg/m^3 (170 lb/ft^3) as opposed to the 2400 kg/m^3 (150 lb/ft^3) assumed in the design.

The top slabs of all pontoons were preducted and blocked out for the installation of landing lights and grounding terminals at each of the three pads. In addition to the landing lights that were pilot activated, the completed facility had fire-fighting and fueling facilities and a beacon approach system. The access gangways were fully articulated at each end to allow restricted movement due to tide and currents. Fig. 7 shows the heliport being installed on the waterfront.

At this writing, the facility has been so successfully received and used that the owner is already assessing the need for expansion. A helicopter's eye view of the completed heliport is shown in Fig. 8.

Specifications

Overall length	85.98 m (282 ft)
Overall width	32.94 m (108 ft)
Mean depth	1.8 m (6 ft)
Design freeboard	0.8 m (32 in.)
Top and bottom slabs	152 mm (6 in.)
Interior longitudinal walls	102 mm (4 in.)
Transverse walls	152 mm (6 in.)
Specified concrete strength	40 MPa (6000 psi)
Reinforcing steel	epoxy coated
Post-tensioning	Dywidag (pontoon connections)
Quantities:	
concrete	1130 m^3 (1477 ft^3)
reinforcing steel	268 Mg (295 ton)
styrofoam	4000 m^3 (5230 ft^3)

Conclusions

In designing such a facility, particular care must be exercised to insure that joining procedures and details for connecting pontoons are well thought out and as simple as possible. Time spent in design is cheap in comparison to the cost of possible delays if such details cause construction difficulties.

If the facility is moored to the shore by rigid stiffleg struts, the designer must thoroughly investigate the fatigue loads and impact loads that result. The use of reinforced concrete in connection with styrofoam blocks for positive flotation for the Vancouver Floating Heliport has proven to be an economical and durable solution for such an application. The overall completed facility cost of just over \$1.6 million (Canadian), or about \$570/m^2 (\$53/ft^2), (including shore facilities) compares very favorably with the undeveloped cost of the adjacent land. By specifying construction of a floating concrete facility, the Port of Vancouver realized these advantages:

- low initial cost;
- a short construction schedule to minimize interim costs;
- a facility that left existing valuable downtown land available for development;
- a portable facility that allows future movement if site priorities change;
- a low maintenance structure;
- high deck friction to reduce the risk of falls by users;
- good resistance to fuel spills and excellent resistance to heat damage in the event of fire;
- an esthetically attractive structure that fits in well with the shoreline facilities.

Owner:	Vancouver Port Corporation, Vancouver, B.C.
Contractor:	Dillingham Construction Ltd., North Vancouver, B.C.
Consulting engineers:	Taylor Peach & Associates Ltd., Vancouver, B.C.
Subconsultants:	ABAM Engineers Inc., Federal Way, Washington

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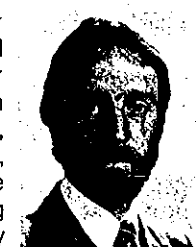

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