Design and construction of a 3 km (2 mi) long shoreline protection system for the City of Luna Pier

N. K. BECKER AND H. R. PATTERSON
N.K. Becker & Associates Ltd., Consulting Engineers, 2468 Ouellete Avenue, Windsor, Ont., Canada N8X 1L4

AND

J. A. MCCORQUODALE
Department of Civil Engineering, University of Windsor, 401 Sunset Avenue, Windsor, Ont., Canada N9B 3P4

Received December 3, 1984
Revised manuscript accepted January 30, 1986

In 1982, N.K. Becker & Associates Ltd. designed a precast concrete flood protection and beach restoration system for the 3 km (2 mi) long Lake Erie shoreline of the City of Luna Pier, Michigan. The construction of these works, which included beach sills, shorewalls, and a marina basin, was completed in 1984.

The shorewall system is unique in that it was designed of interconnected, open-ended, earth-filled, thin-walled precast concrete cylinders, placed on-end and in-line to form a continuous wall. This seawall design was selected by the City of Luna Pier over alternative systems because it was estimated to cost 50% less than conventional steel sheet piling or cast-in-place concrete systems.

During the final design, hydraulic model tests were carried out at both the University of Michigan and the University of Windsor to study the uprush, scour, longshore transport, and wave pressure characteristics of the proposed system. The methods used to design this economical and functional flood protection system are presented along with an analysis of its performance to date.

Key words: beach, beach sill, flushing, hydraulic model testing, ice, longshore drift, overtopping, precast cellular concrete wall system, scour, shorewall, uprush, water levels, waves, wave deflector.

En 1982, N.K. Becker & Associates Ltd. ont conçu un système de protection contre les inondations, en béton précoulé, et un système de restauration de la plage pour les 3 km (2 mi) du côté du lac Érie adjacents à la cité de Luna Pier au Michigan. La construction de ces travaux comprenant un mur de soutènement pour le sable de la plage, des murs de protection et une marina, fut completée en 1984.

Le système de mur de protection est unique dans sa conception : il est composé de cylindres en béton précoulé aux parois minces, ouverts aux extrémités, remplis de terre et reliés bout à bout pour former un mur continu. Ce mur de protection fut choisi par la cité de Luna Pier parmi d'autres variantes parce que son coût fut évalué à 50% de celui des solutions conventionnelles telles que l'utilisation de palplanches métalliques ou de béton coulé sur place.

Durant la conception finale, des tests sur des modèles hydrauliques furent menés à l'Université du Michigan ainsi qu'à l'Université de Windsor. Ces tests étudièrent la montée soudaine, l'érosion, les alluvions et les caractéristiques de la pression des vagues pour le système de protection proposé. Les méthodes utilisées pour la conception de ce système, économique et fonctionnel, de protection contre les inondations sont présentées avec une analyse à date de ses performances.

Mots clés: plage, murs de soutènement pour le sable de plage, affleurement, expérimentation du modèle hydraulique, glace, mouvement côtier, système de mur cellulaire en béton précoulé, érosion, mur de protection, montée soudaine, niveaux de l'eau, vagues, brise-lame.

[Traduit par la revue]

Can. J. Civ. Eng. 13, 301–309 (1986)

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Introduction

In 1979, a field prototype of a shore protection system consisting of structurally interconnected, open-ended, earth-filled, precast concrete cylinders installed on-end was installed along Lake St. Clair (McCorquodale and Becker 1980). This installation has been monitored since then and has performed well under both record high lake levels and severe ice loading conditions.

The success of the prototype system has lead to the construction of several other shoreline protection systems utilizing this concept. Similar installations are now in service on the shores of Lake Erie, Lake St. Clair, the Detroit River, and their tributaries. These installations include beach sills, shorewalls, marina breakwalls, docks, and groynes.

In 1982, N.K. Becker & Associates Ltd. were retained to undertake the design of a complete flood protection and beach restoration system for the 3 km (2 mi) long Lake Erie shoreline of the City of Luna Pier in Michigan. This paper sets out the design criteria, the hydraulic model testing program, various construction details, and early performance results of the system.

Luna Pier — the project setting

The City of Luna Pier is spread thinly along 3 km (2 mi) of Lake Erie shoreline in Michigan, just north of Toledo, Ohio (see Fig. 1).

During the 1920's, a wooden pier that included a promenade with a terrazo dance floor was built 60 m (200 ft) into Lake Erie. A devastating ice storm destroyed much of the pier in 1939. Ice storms and wave action in subsequent years combined to obliterate the pier from the shoreline.

The city has experienced four major floods since 1950. These floods have threatened the safety of city residents, caused property damages in excess of $1.5 million (U.S.), and disrupted power, water, and sewerage services for lengthy periods.

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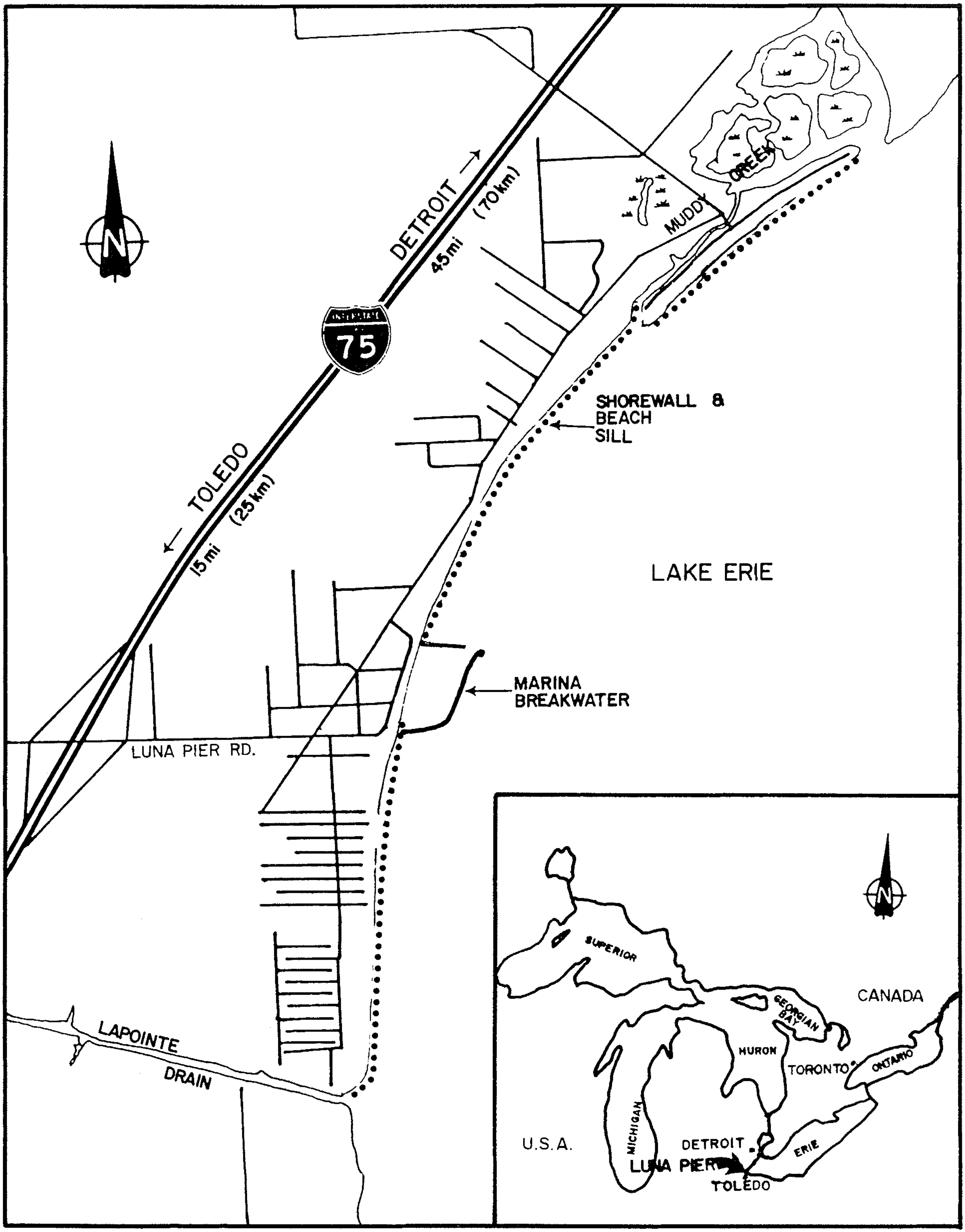

FIG. 1. Location of Luna Pier, Michigan, U.S.A.

Because of the massive flooding that occurred in the city in November 1972 and March 1973 as a result of record high lake levels, the U.S. Army Corps of Engineers under "Operation Foresite" constructed temporary rock-crib gabions and rubble-mound revetments along most of the city's shoreline. This emergency shoreline protection work was intended to provide the city with temporary flood protection. The rock-crib gabions had an expected service life of 5 years.

The rock-cribs stood 1.9 m (6 ft) above the level of the residential properties and were built within 1.5 m (5 ft) of many existing homes. People complained that the rock-cribs limited their recreational use of the waterfront and made it difficult to

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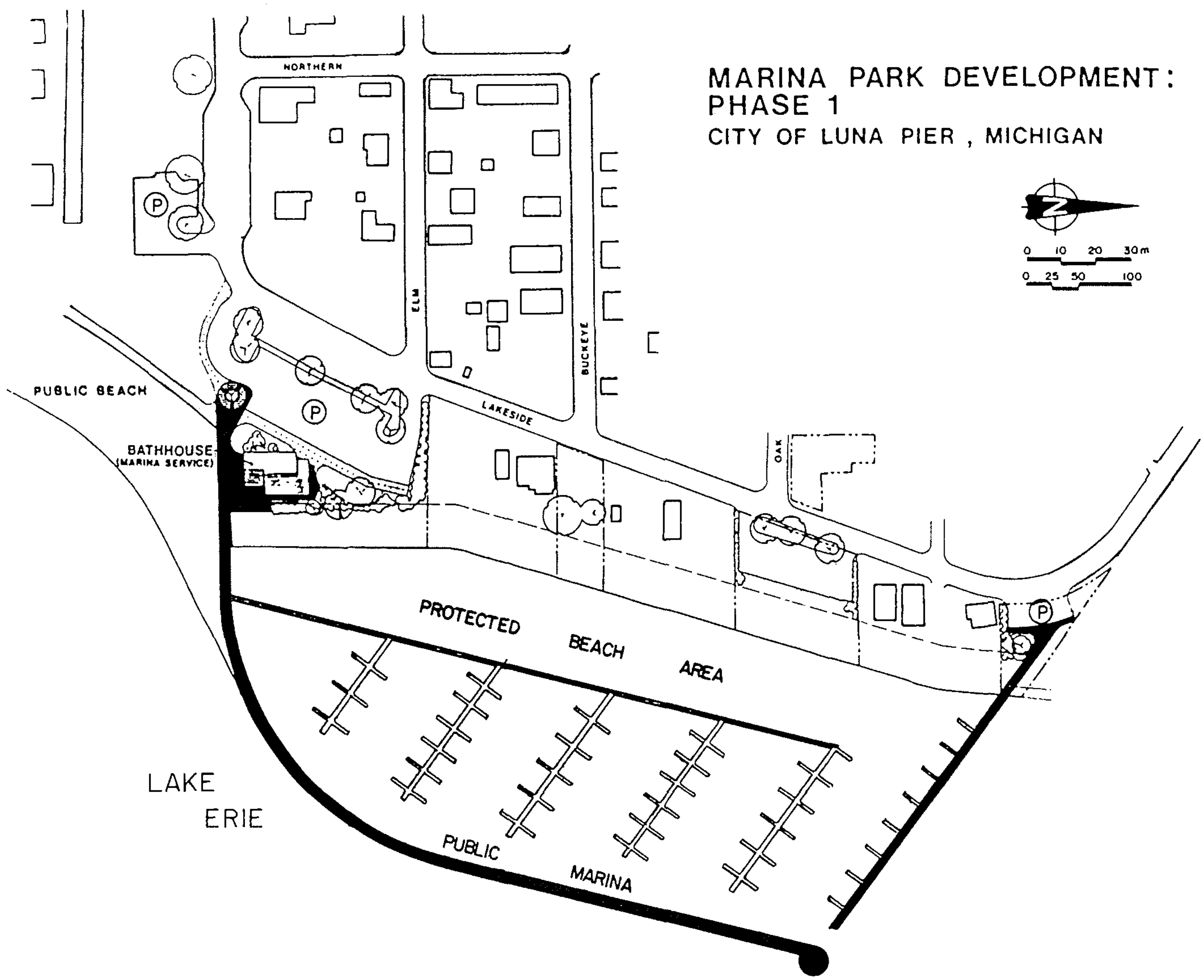

FIG. 2. Plan view of proposed marina and public beach area for City of Luna Pier.

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see the lake. Property values dropped and some residents moved out of the city.

Early in 1979, the city council began to search for a permanent solution to the city's long-term flooding problems. The rock-cribs had outlived their useful design life of 5 years and were deteriorating rapidly. They could no longer be relied upon to provide flood protection for the city during the persistently high water levels of Lake Erie.

Alternative designs for the city's proposed shoreline protection system were developed by the city's engineers during 1980 and 1981. The alternatives considered included a steel sheet piling wall, a reinforced concrete shorewall, and a clay-core, rubble-mound revetment. The construction costs (in U.S. dollars) for these alternatives were estimated at \$8.3 million for a steel wall, \$6.5 million for a reinforced concrete wall, and \$3.1 million for a clay-core rubble-mound revetment. None of these alternatives were acceptable. The steel and concrete alternatives exceeded the city's budget of \$3 million and many residents objected to the appearance of the rubble-mound revetment alternative.

In 1982, the city retained N.K. Becker & Associates Ltd. to design a permanent shoreline protection system for the entire city using the precast cellular concrete wall system this firm had researched and developed.

During the conceptual design phase, the project was expanded to include a marina and a protected public beach area. The marina was to be sheltered by a 300 m (900 ft) long curving pier that would project 100 m (300 ft) offshore into Lake Erie. Figure 2 shows a plan view of the proposed marina and public beach area.

Shorewall system design concept

The shorewall was designed of structurally connected, precast concrete pipe sections. A beach sill designed of similar precast units was also to be constructed parallel and in front of the shorewall to trap sand within a beach area. A cross-sectional view and a longitudinal profile of the beach sill/shorewall system are illustrated in Fig. 3 and 4 respectively.

Marina wall design concept

Since the marina breakwall would be located offshore, in deeper waters, the sliding forces and overturning moments due to wave and ice attack would be much greater than those acting on the shorewall. The marina wall was therefore designed of

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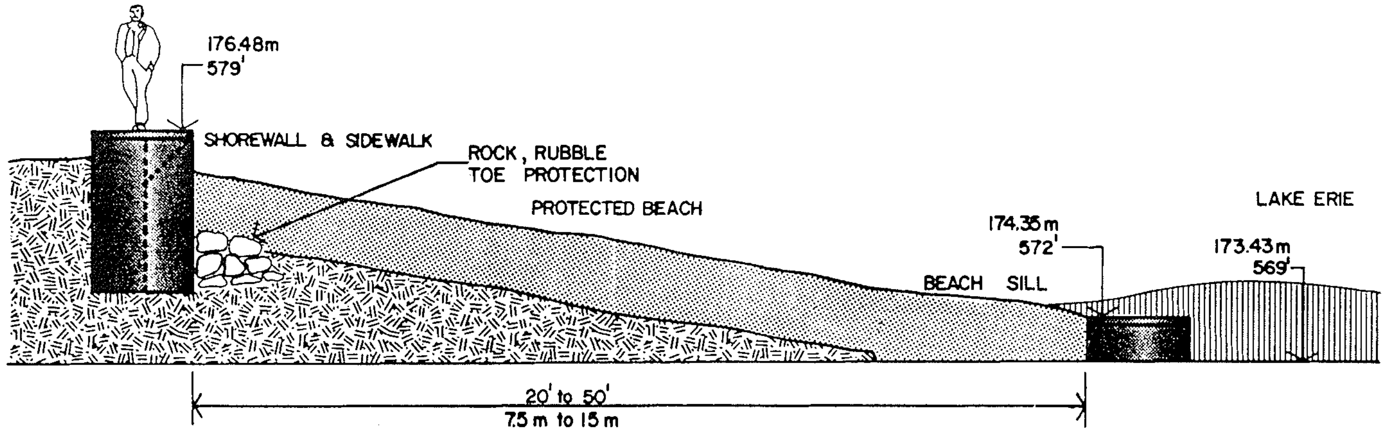
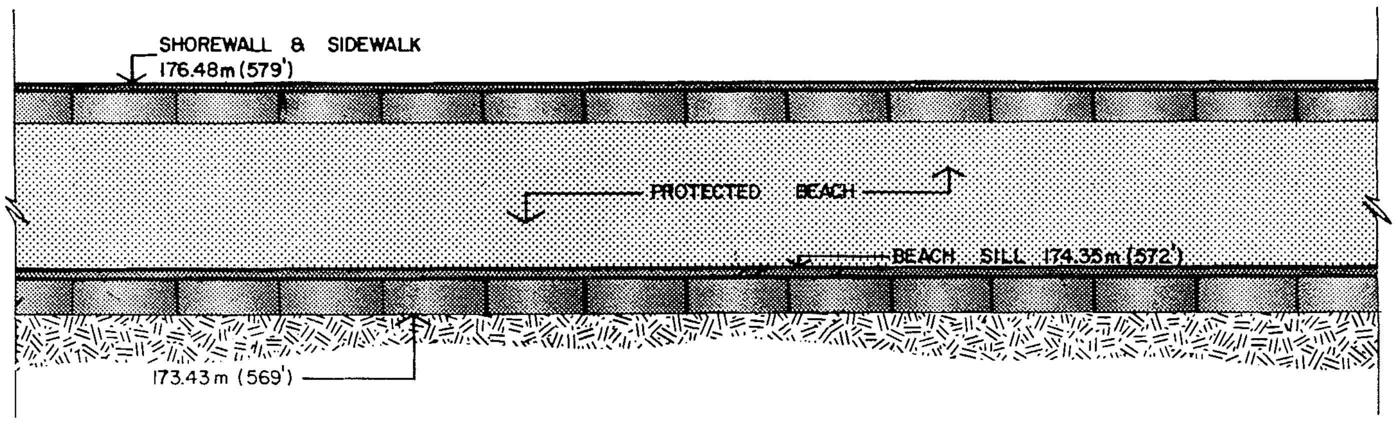


CROSS-SECTION

FIG. 3. Typical section of Luna Pier beach sill.



FRONT ELEVATION

FIG. 4. Longitudinal profile of cellular shorewall.

two parallel rows of in-line precast concrete cylinders. The units in the front row were designed to be structurally interconnected to adjacent units and tied back by a reinforced concrete rigid frame to the units in the back row. The marina wall concept is illustrated in Fig. 5. The breakwater was also designed to support emergency vehicles.

Design criteria

In addition to normal hydraulic and structural design criteria, the design of the shore protection system had to satisfy other criteria that were imposed by the city and various other government agencies. The following list sets out a number of the more important design criteria, many of which are site specific:

(a) the structure must withstand wave and ice forces and associated overturning moments based on the wind, wave, and water level conditions for a storm return period of 20 years;

(b) environmental impacts on fish should be minimal;

(c) soil bearing capacity must not be exceeded;

(d) the shorewall must be aesthetically pleasing, provide people with easy access to the beach, include safety features, such as anti-slip surfaces and no protruding/sharp edges, to promote recreational and leisure activities;

(e) the top of the shorewall must not exceed elevation 176.48 m (579.0 ft) International Great Lakes Datum (I.G.L.D.);

(f) subject to the height restriction, the system should include design features to minimize the frequency of overtopping of the dike wall by wave action;

(g) the shorewall must be able to withstand the abrasive forces of sediments in waves, resist chemical attack, withstand rigorous freeze–thaw cycles, and must have a design life of 20 years;

(h) the shorewall must be aligned so that some of the property lost to erosion would be reclaimed from the lake and to encourage a natural buildup of sand between the shorewall and beach sill;

(i) wave activity in the marina basin must be controlled;

(j) the marina basin must be self-flushing;

(k) the marina breakwall cap should be capable of supporting the weight of emergency vehicles;

(l) the marina entrance/exit must provide safe access for boats during storm conditions.

Because of the length of the project, the imposed design constraints, and the uniqueness of the design concept, the development of the final design involved considerable research and testing. Previous experience with the performance of prototype installations showed that hydraulic model testing would be required to fine-tune the design.

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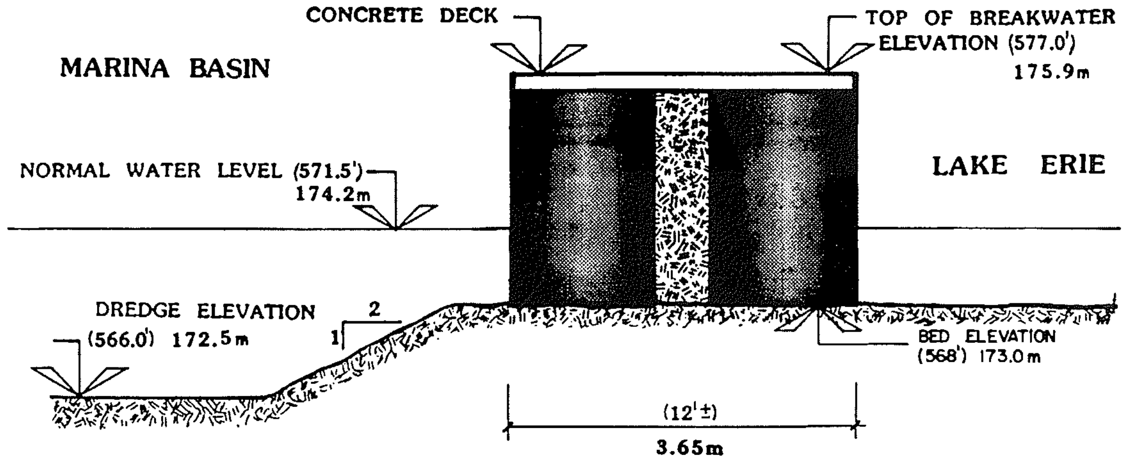

FIG. 5. Marina wall concept.

Oceanographic information

Based on the International Great Lakes Datum (I.G.L.D.), the water depth of this reach of Lake Erie shoreline levels off at 6.5 m (20 ft) approximately 600 m (2000 ft) offshore.

In the nearshore zone, there are normally three distinct sand bars. The size of these bars varies widely from storm to storm, season to season, and year to year. The nearshore lake bed slope is approximately 1:20.

Water levels

The U.S. Army Corps of Engineers continuously monitor the water levels in Lake Erie. Table 1 shows the return periods and water elevations based on the International Great Lakes Datum (I.G.L.D.) 1955. These elevations are instantaneous and include a storm setup of approximately 0.8 m (2.5 ft), but exclude the actual height of waves and wave setup. At the time of the design, the record high water level, set in 1973, of 175.52 m (575.85 ft) (I.G.L.D.) was equivalent to about a 1:50 year return period. After the completion of this project, a new record lake level of 175.75 m (576.6 ft) (I.G.L.D.) was recorded on 31 March 1985.

Wind information

The prevailing wind direction in this area is from the west. This wind direction is offshore and is therefore not a factor in the shorewall design. The wind directions producing the highest waves at Luna Pier occur from 12° south of east to approximately 25° east of north. Direct wave attack on the shorewall occurs at approximately due east. Maximum storm setup also occurs as a consequence of these wind directions owing to a fetch of over 320 km (200 m).

Generally, violent storms have short durations, usually in the order of 1–2 h. However, experience has shown that heavy winds with no precipitation do last longer.

Wave characteristics

The maximum wave height that can reach the shoreline depends upon the water depth, which in turn depends on the lake level. Based on the bed contours, fetch, and wind data, a refraction analysis confirmed that, for Luna Pier, a maximum offshore wave height of 2.4 m (8 ft) in a direction 12° south of east could develop (U.S. Army Corps of Engineers 1977). The combined effect of shoaling, refraction, and breaking would reduce this wave height in the nearshore zone. Table 1 shows the maximum unbroken wave height for the respective return period and water level elevation (Snell Environmental Group 1983).

TABLE 1. Water level information

Return period	Water level elevation (I.G.L.D.) with storm setup, m (ft)	Maximum unbroken wave height, m (ft)
Low level	174.0 (571.0)	0.8 (2.5)
1:2 year	174.7 (573.3)	1.4 (4.5)
1:10 year	175.2 (574.9)	1.8 (6)
1:20 year	175.4 (575.3)	2.0 (6.5)
1:100 year	175.7 (576.4)	2.4 (8)

Four types of waves could impact the proposed shorewall system. They are (1) unbroken waves, (2) breaking waves (plunging or spilling type), (3) travelling breakers, and (4) broken waves.

The pressures exerted on the proposed structures by these waves were determined both through the use of a pressure transducers installed in a hydraulic model and through calculations using established formulae (U.S. Army Corps of Engineers 1977).

Wave period characteristics for this reach of Lake Erie range from approximately 4.5 to 8 s. The short-period waves (4.5 s) usually occur at the beginning of a storm, whereas the longer-period waves tend to occur at the end of a storm. However, there is some randomness in the wave period in that short- and long-period waves can and do occur at any time during a storm.

Sediment climate

In general, the grain size for the sand found along the Luna Pier shoreline ranged from 0.08 to 1 mm (0.003 to 0.04 in.) with the d_{16} being 0.2 mm (0.008 in.) and the d_{84} being 0.4 mm (0.016 in.). However, there were a few isolated locations along the shoreline where a coarser beach material was found. The grain size of this material ranged from 3 to 13 mm (0.1 to 0.5 in.).

The net longshore drift of sediment has been observed to be from north to south. The source of this sediment is generally from the Detroit River, the mouth of which is located approximately 40 km (25 mi) due north of Luna Pier. Much of this material originates from Lake St. Clair and Lake Huron. However, under certain wind and wave conditions, the longshore drift is reversed, from south to north. Offshore sand bars are the source of a very small amount of beach material for the Luna Pier shoreline.

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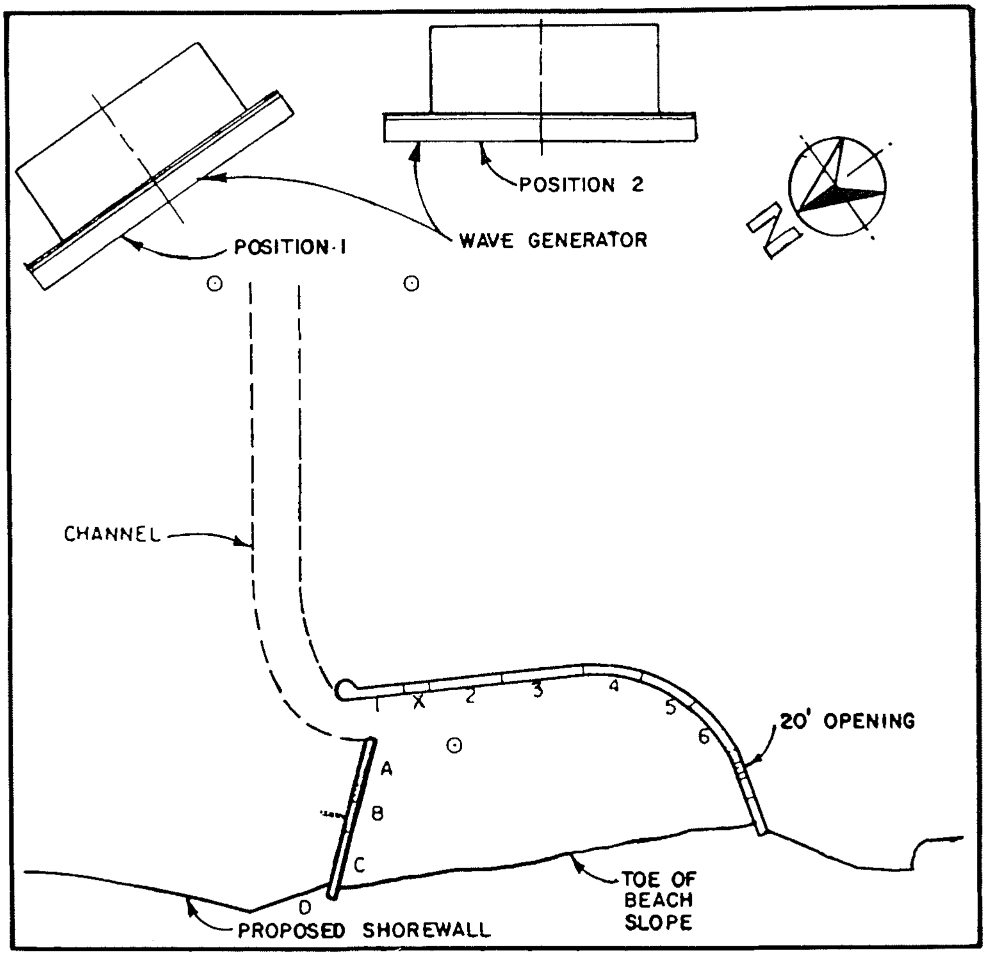

FIG. 6. Plan view of model in wave tank at University of Michigan.

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The net sediment transport rate was estimated to be between 45 and 180 t/day (50 and 200 tons/day) based on observations during a 4 month period in the spring of 1985.

Elevation of the top of the shorewall

At the request of lakefront property owners who wanted a reasonable view of the lake, City Council agreed to limit the top of the proposed shorewall to elevation 176.5 (579.0 ft) (I.G.L.D.), 0.6 m (2 ft) below the height of the rock cribs that had been installed along their shoreline during "Operation Foresite."

Hydraulic model testing

An extensive hydraulic model testing program was used to study the performance of the design concept. Two separate models, each with a different scale, were tested because of the size and complexity of the project. The smaller-scale model concentrated on the three-dimensional hydrodynamics of the marina and harbour while the larger-scale model was used to study the two-dimensional problems associated with the marina breakwater and the shoreline protection system.

Luna Pier harbour hydraulic model study

The Luna Pier harbour model testing was carried out at the G.G. Brown Lake Hydraulics Laboratory at the University of Michigan by the Snell Environmental Group of Lansing under the direction of N.K. Becker & Associates Ltd.

The harbour model was tested in a 12.8 m (42 ft) by 13.1 m (43 ft) indoor wave tank at the scale of 1:44. This model scale was selected to incorporate as large a section of the shoreline as possible to check the effects of the transitions to the shorewall, located to the north and south of the proposed marina. Wave reflection and diffraction patterns were studied to determine locations where potential problems could occur. Figure 6 shows a plan view of the model in the wave tank.

The wave machine was positioned in such a manner as to test the performance of the model under wave attack from due east and from 25° south of east. The maximum wave height studied in the model was 39 mm (1.53 in.), which corresponded to a prototype wave height of 1.7 m (5.6 in.), which corresponded to a prototype wave height of 1.7 m (5.6 ft). The wave period in the model ranged from 0.75 to 1.06 s, which corresponded to prototype wave periods between 5 and 7 s. These design parameters were selected for the purpose of studying the *worst* design conditions.

The marina harbour model was used to study alternative harbour entrance configurations, wave attenuation inside the harbour during periods of extreme high water, and the flushing action of the marina basin itself.

Three alternative harbour entrance designs were tested in the model. Structural considerations dictated that the elevation of the harbour breakwater be set lower than the shorewall in order to reduce overturning moments due to wave pressures and ice pressures. The model tests indicated that significant overtopping of the breakwater would occur at 1 in 10 year wave and lake level frequencies. This overtopping would raise the water level inside the marina basin, create some turbulence inside the

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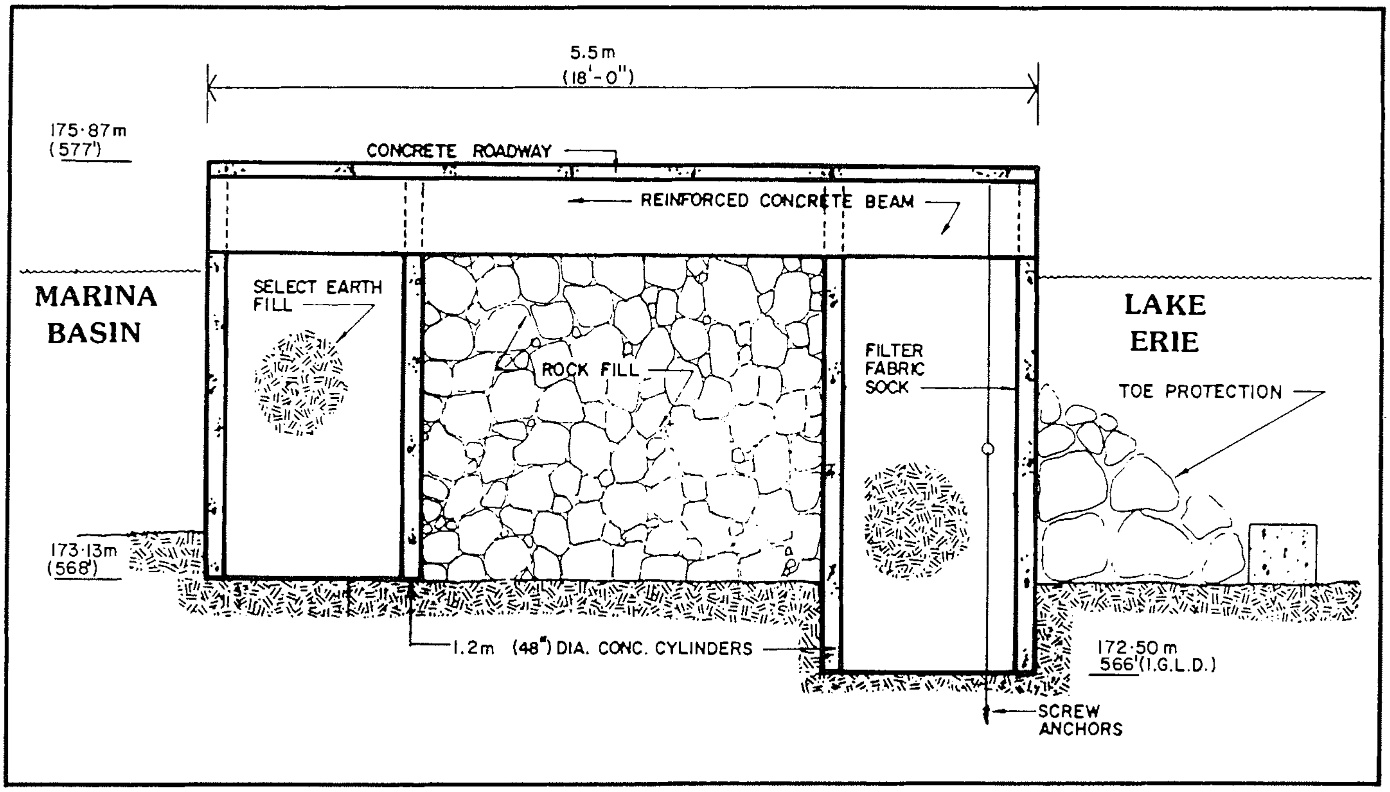

FIG. 7. Final design of marina breakwater.

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basin, and result in strong currents at the entrance.

Modifications were made to the structural design of the concrete cap on the breakwater during the testing to limit waves inside the harbour to approximately 0.3 m (1 ft) for a 1 in 10 year water level and a wave height of 1.8 m (6 ft).

Alternative harbour entrance designs were tested in the model for optimum flushing of the dredged marina opening. It was found that an 18 m (60 ft) harbour opening with the front breakwall projecting 15 m (50 ft) past the north wall provided the best design.

The flushing action of the marina basin was determined using coloured dye at a number of strategic locations. The dye test results suggested that there would be a potential for some sediment accumulation inside the marina due to overtopping waves, but that the water from the overtopping waves would keep the harbour entrance clear of sediments as the water flushed into the lake. The dye tests also confirmed that gaps in the marina wall near the shoreline would be necessary to prevent stagnation areas in the marina basin and to reduce the velocity of water through the marina entrance as a result of overtopping waves.

The harbour model testing also provided information concerning the propagation of oblique waves along the exterior of the marina breakwater. The model test showed that without the installation of strategically placed groynes (Fig. 6) to block these waves, localized overtopping and scour would occur both upstream and downstream of the marina, where the marina wall returns to the beach sill.

Large-scale wave flume testing

Comprehensive, two-dimensional model tests were also carried out using the 15 m (50 ft) long, 0.6 m (2 ft) wide, and 1 m (3.5 ft) deep wave flume at the Civil Engineering Laboratory of the University of Windsor. The bed of the wave flume was covered with sand and sloped to simulate the lake bottom. The waves were generated by an adjustable stroke and depth paddle operated by a 5 hp variable speed electric motor.

Model scale

The model was designed according to the Froude law with an undistorted scale of 1 : 12.9. This scale was dictated by the use of 115 mm (4.57 in.) outside diameter commercially available plastic pipe for the cylinders. The flume width modelled approximately 7.6 m (25 ft) or 5.5 precast unit widths of the prototype shorewall system. The half unit was desired since it demonstrated the activities in both the fluted (or connection) areas and at the front of the shorewall units. For testing purposes, the beach sill and shorewall were tested together and the marina wall was tested alone.

The water levels, wave heights, breaker type, and wave periods were varied for each test to simulate some 50 possible conditions.

Model test results and final design

The bottom elevations of the precast sections, the marina wall, beach sill, and shorewall were established based on the scour depths observed during the hydraulic model tests plus some freeboard. The stability of this shorewall system depends on the base since severe toe scour or poor soils will result in the upsetting and (or) settling of the sections of the wall. The upsetting of the wall would have catastrophic results, while settlement would be unsightly and could pose a threat to pedestrian traffic from projecting sections of the concrete cap.

Marina breakwall

The marina wall was tested to determine the type of toe protection and cap design required.

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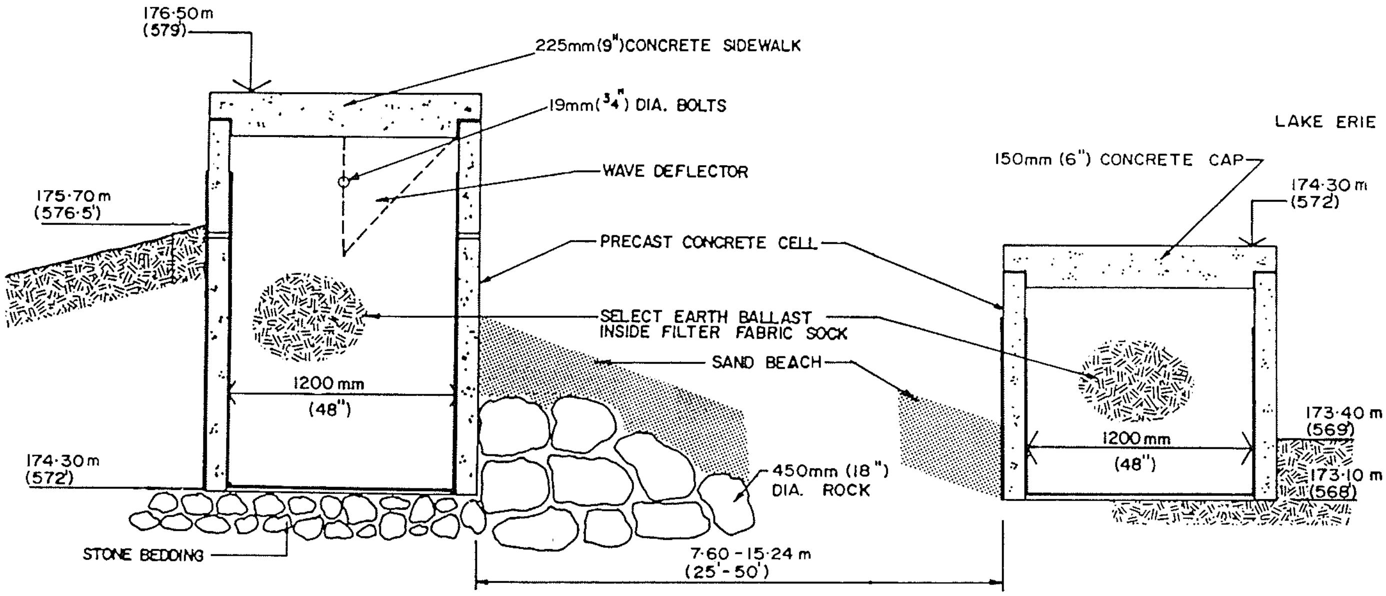

FIG. 8. Final design of shorewall/beach sill.

For the marina breakwall with a top elevation of 175.9 m (577.0 ft), the elevation of deepest scour was 172.5 m (566.0 ft), which occurred for storm levels with return periods in the range of 2–20 years. This scour resulted from 1.2–1.5 m (4–5 ft) waves breaking directly on the wall. The larger waves, 2.1–2.4 m (7–8 ft), did not cause as much scour since most of these waves overtopped the wall.

The lakeside face of the breakwall cap was designed to conform to the rounded shape of the precast cells in order to avoid the large uplift forces due to breaking waves and ice. In addition, the concrete cap was designed to accommodate emergency vehicles and pedestrians. A series of galvanized steel screw anchors were attached to the underside of the concrete cap and were screwed into the clay bed to help resist the forces of waves and ice.

Figure 7 illustrates the final design of the marina breakwall.

Beach sill

The original concept assumed a top elevation of 175.3 m (575.0 ft) (I.G.L.D.) for the beach sill. Through the extensive model testing, it was found that 174.4 m (572.0 ft) was the best elevation for several reasons:

(a) less toe scour occurred and no toe protection was required at the beach sill;

(b) more overtopping of the sill at normal lake levels increased the entrapment of sediment materials for the beach;

(c) at the 1 in 5 to 1 in 20 year water levels, waves began to break at the sill with most of the breaking force being dissipated on the beach area and very little on the shorewall itself. This reduced scour at the toe of the shorewall.

The tests indicated that at the 1 in 100 year water level, the beach sill units would be buried by material from the toe of the shorewall and offshore sediments.

Beach area

Model tests showed that a minimum separation of 6 m (20 ft) between the shorewall and the beach sill would be required to reduce the effect of waves breaking directly on the shorewall. This breaking action could greatly increase toe scour. The *maximum* separation is not critical. To help control longshore material transport along the beach area, it was found that the groynes should be installed from the shorewall to the beach sill at 60 m (200 ft) spacings.

Shorewall

Since the top elevation of the shorewall was set by City Council at 176.5 m (579.0 ft), it was only necessary to determine a cap design and toe protection requirements for the shorewall design itself.

No scour occurred at the toe up to the 1 in 10 year water level. In fact, there was a buildup of material at the toe due to the run-up of waves that overtopped the beach sill. It is important to note that after the occurrence of storms at water levels greater than the 1 in 20 year level during which severe toe scour does occur, the hydraulic model tests showed that sediment materials would eventually refill these scour areas.

The scour elevations for the 1 in 20 year water level ranged from 174.9 m (573.7 ft) for short-period (high-frequency) waves down to 173.9 m (570.9 ft) for long-period (low-frequency) waves. Also, at this water level, there was a small amount of wave overtopping and a large amount of spray (up to 10 m in the air), which would be carried over the wall by wind. Overtopping and splash could only be controlled by raising the elevation of the top of the shorewall from 176.5 to 178.3 m (579.0 to 585.0 ft).

It was found that an overhanging wave deflector on the lake side of the shorewall cells would greatly reduce overtopping and splash, but toe scour would increase approximately 150 mm (6 in.). This scour would result from breaking waves reflecting off the underside of the concrete cap. With the installation of wave deflectors in the fluted areas of the units, the maximum scour elevation was found to be elevation 174.3 m (572.0 ft).

At the 1 in 100 year level, the shorewall would experience some damages. The extent of the damages would depend on the length of the storm. The damages to the shorewall might include a lifting of the wave deflector/concrete cap and tipping of the wall as a result of severe toe scour.

The final design of the shorewall and beach sill that evolved through the hydraulic model testing is illustrated in Fig. 8.

Wave forces

During the course of the model tests, wave pressures were

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measured in order to obtain magnitude of the force at the marina wall face. The maximum wave force on the marina wall was approximately 15 kN/m (1000 lb/lineal ft) which is about 1/10 of the Minikin force calculated using the method described in the *Shore protection manual* (U.S. Army Corps of Engineers 1977).

The forces on the shorewall and beach sill would be much smaller than those acting on the marina wall because much of the wave energy would dissipate on the beach area rather than directly on the walls. In addition, larger waves impact on the marina wall since it is situated in deeper waters.

Filter sock

An important feature in this shorewall system is the installation of a filter fabric sock in the interior of the precast concrete units. This sock prevents the select backfill materials from being leached out of the inside of the cells when water levels fluctuate.

Construction methods

Construction of the Luna Pier project started in September 1983 and was completed in December 1984. The final cost of the project including the marina breakwall was $2 800 000.00 (U.S.)

All of the work was installed using conventional land-based equipment, which included hydraulic backhoes, rubber-tired loaders, bulldozers, and associated equipment. The contractor also custom-designed and fabricated several items of equipment to facilitate the installation of the precast concrete cells. This enabled the contractor to achieve daily installation rates of up to 150 lineal m (500 lineal ft) of shorewall and 245 lineal m (800 lineal ft) of beach sill.

Shorewall performance

The performance of the installed sections of shorewall under severe storm conditions in November 1983 and February 1984 has validated the design assumptions and the results of the hydraulic model testing.

During the course of the construction work, the shorewall was subjected to three storms with water levels equivalent to a 1 in 20 year return period and one storm with a water level equivalent to a 1 in 50 year return period. The completed structures sustained no damage in these storms. The shorewall was subjected to a minor amount of wave overtopping, mostly in the form of a fine spray.

In January and February 1985, the wall was subjected to a combined wave and ice attack with water levels near the 1 in 100 year level. Ice accumulated and subsequently overtopped the marina breakwall and portions of the shorewall. However, the structures were not damaged and the city was not flooded.

On 31 March 1985, the shoreline protection system was subjected to the highest storm lake level ever recorded on this section of Lake Erie. The return period exceeded 100 years. The shorewall was overtopped by waves to the extent predicted by the model tests and a limited amount of toe scour occurred. The shorewall experienced localized damage to the concrete cap on a portion of the shorewall near the south end of the project. However, the structures remained upright and functional throughout the storm, with most of the damage being cosmetic rather than structural.

A large section of the beach sill at the northern end of the project was completely covered by sand in 1984. In addition, a significant amount of sand was deposited by a longshore transport and waves along the beach. This confirms that, under normal conditions, the beach sill will perform as the model studies indicated.

Summary

The Luna Pier shoreline protection project has proven to be both economical and functional. Besides eliminating the unsightly rock-crib gabions, rubble-mound revetments, and failing shorewalls that posed a barrier to the recreational use of the city's Lake Erie shoreline, the shore protection works will greatly reduce the potential for flooding, has enhanced property values, and has helped to restore the city's beaches.

The model testing and prototype performance has confirmed that the beach sill and shorewall as designed will provide the city with greater flood protection than a conventional sheet piling wall or a rubble-mound revetment at a lower cost.

Acknowledgements

The hydraulic model studies were carried out at the University of Windsor and at the University of Michigan.

McCorquodale, J. A., and Becker, N. K. 1980. Precast cellular shorewall. Canadian Society for Civil Engineering Conference, Winnipeg, pp. S/12:1–S/12:8.

N.K. Becker & Associates Ltd. 1983. Hydraulic model testing—City of Luna Pier shore protection works. N.K. Becker & Associates Ltd., Windsor, Ont. 90 p.

Snell Environmental Group. 1983. Luna Pier Harbour—hydraulic model study. Snell Environmental Group Lansing, MI, 30 p.

U.S. Army Corps of Engineers. 1977. Shore protection manual. U.S. Government Printing Office, Washington, DC, Vols 1–3, 1105 p.

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