Swimming Pool Design Guidance

This resource contains copies of the following two articles, which appeared in the Winter 1996 and Winter 1997 issues of the *Journal of Coastal Research* (JCR):

Scour Impact of Coastal Swimming Pools on Beach Systems, by Soronnadi Nnaji, Nur Yazdani, and Michelle Rambo-Roddenberry (JCR, Winter 1996)

Conceptual Breakaway Swimming Pool Design for Coastal Areas, by Nur Yazdani, Soronnadi Nnaji, and Michelle Rambo-Roddenberry (JCR, Winter 1997)

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Scour Impact of Coastal Swimming Pools on Beach Systems

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ABSTRACT

NNAJI, S.; YAZDANI, N., RAMBO-RODENBERRY, M., 1996. Scour impact of coastal swimming pools on beach systems. *Journal of Coastal Research,* 12(1), 186–191. Fort Lauderdale (Florida), ISSN 0749-0208.

Swimming pools have become an integral part of habitable coastal construction. These pools frequently increase the turbulance of floodwater during a tropical storm or hurricane. This results in an increase in the scour potential under and around swimming pools. This paper demonstrates that a suitable scour model for seawalls from literature is applicable to coastal swimming pools, including over-topping and corner effects. The model predicts substantial scour around pools for typical storm waves and water levels. Optimum sizing and siting for coastal swimming pools are also discussed.

ADDITIONAL INDEX WORDS: *scour, swimming pools, dune systems, erosion, flooding.*

INTRODUCTION

The State of Florida has an extensive tidal shoreline. In recent years, this shoreline has been subjected to rapid development and construction due to a massive population influx. Swimming pools have become essential accessories attached to habitable coastal construction in terms of property value and the tourism industry in Florida. Virtually all of these pools are situated seaward of the habitable structure.

The Federal Emergency Management Agency (FEMA) oversees the construction of all structures (including pools) in the Coastal High Hazard Areas (V-zones) in order for these structures to be insured under the National Flood Insurance Program (NFIP). These requirements are contained in 44CFR Section 60.3 which states that all new construction and substantial improvements in Zones V1–V30, VE, and V shall have the area below the lowest floor level, either free of obstruction or constructed with non-supporting breakaway walls or similar structures.

If a swimming pool is placed below the level of a coastal building, but above natural grade, it may behave as an obstruction to the free flow of flood water. A large object, such as a swimming pool, placed above the natural grade may increase the turbulence of the floodwater, resulting in an increase in the scour potential under and around pools and around the pile supports.

The objective of this paper was to formulate a wave scour model around coastal swimming pools. Optimum siting, sizing, and design conditions for coastal pools need to be considered in order to minimize unwanted scour effect on beach/dune systems and adjacent structures. Siting aspects include the encroachment, orientation and elevation of pools, while sizing aspects include the shape and depth of pools.

The effect of a swimming pool type massive structure on coastal topography during a storm has been apparent over the years; however, documentation of this effect has started only recently. No basic research has been performed to attempt to understand this effect and to determine methods to minimize such costly damage.

LITERATURE REVIEW

Various studies have been performed on scour around coastal structures, such as piers, abutments, piles, pipelines, and seawalls during extreme flooding. Several reports on scour evaluation and methods for predicting scour around coastal structures were reviewed, including: EADIE and HERBICH (1987), FROEHLICH (unknown), HERBICH (1968, 1981), IBRAHIM and NALLURI (un-

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known), JAIN and FISCHER (1979), KADIB (1963), KAWATA and TSUCHIYA (1988), KHANBILVARDI *et al.* (1988), BREUSERS (1972), HANCU (1971), FOTHERBY (1992), FOWLER (1992, 1993), SHEPPARD and NIEDORODA (1990), RANCE (1980), and RICHARDSON (1993). However, no previous or continuing studies were found which address scour around coastal swimming pools. Additional studies have been performed in the Netherlands, U.K., Japan and Norway. Most previous studies dealt with scour around other types of coastal structures. The applicability of these models to coastal swimming pools is questionable, especially because of the dissimilarity of geometric parameters.

The authors were seeking a scour prediction model that included most of the pertinent variables associated with coastal swimming pools and yielded reasonable results. A model developed by HERBICH (1984) for ultimate scour depth at seawalls was found to be one such model. This model was developed using Prandtl's boundary layer theory along a flat plate, from the definition of stream function and from the continuity equation between a section before scouring and a section when the ultimate scour is reached. POWELL (1987) cites the limitations of the Herbich equation are as follows: (1) the equation was only validated by a few model tests, which were affected by scaling errors; (2) the equation predicts the scour averaged over a distance, rather than the depth of toe scour; and (3) the equation was derived for nonbreaking waves and flat sea beds.

DEVELOPMENT OF SCOUR MODEL

Pertinent variables for predicting scour around coastal swimming pools (based on literature review) may include the following: wave height, median sediment diameter, sediment density, fluid density, shape factor, velocity of flow, wave length (L), wave period (T), time (t), acceleration due to gravity (g), and structure height. A scour model for coastal swimming pools was developed in this paper based on the scour equation from HERBICH (1984). The Herbich equation for ultimate scour depth at seawalls is as follows:

$$S = (d - A/2)\left[(1 - C_r)u_\cdot \cdot \left\{3/4C_D\,\rho\frac{\cot\theta}{d_{50}(\gamma_s - \gamma)}\right\}^{1/2} - 1\right] \tag{1}$$

in which d = depth to still water level (SWL) at the wall, A = wave height at the wall = $H_I + H_R$ (incident wave height + reflected wave height), C_r = reflection coefficient = H_r/H_I, $u_\cdot$ = local horizontal velocity parallel to the bottom, C_D = coefficient of drag, ρ = density of water, θ = angle of repose of the sediment, d_{50} = mean diameter of the sediment, γ_s = specific weight of the sediment, and γ = specific weight of the water.

From DAS (1990), sand grain diameter ranges from 0.075 to 4.75 mm (0.003 to 0.19 inch) (Unified Soil Classification System), and the specific gravity of light colored sand may be assumed to be about 2.65. The authors assumed the median sand grain diameter to be 0.5 mm (0.02 inch). From HERBICH *et al.* (1984), the coefficient of drag depends on the Reynolds number and the shape factor of the sediment. The authors assumed a value of 0.7 for the coefficient of drag for coastal swimming pools in the turbulent zone. This value was assumed from HERBICH *et al.* (1984) for an average sediment shape factor of 0.7. The local horizontal velocity parallel to the bottom, $u_\cdot$, depends on the water depth to wave length ratio (d/L). This ratio determines whether the condition is shallow water ($d/L < 1/25$), transitional water ($1/25 < d/L < 1/2$), or deep water ($d/L < 1/2$). The following expression for $u_\cdot$ for shallow water conditions is applicable for coastal swimming pools:

$$u_\cdot = H/2(g/d)^{1/2}\cos\theta_p \tag{2}$$

in which H = wave height, d = water depth to SWL, and θ_p = phase angle. The phase angle θ_p can be expressed as $2\pi x/L - 2\pi t/T$, where x is the horizontal distance travelled by the wave. The wave length can be found by the following equation for shallow water:

$$L = T(gd)^{1/2} \tag{3}$$

The angle of repose of the sediment is found from the relationship of components in Figure 1 (HERBICH, 1984). In this figure, the drag force F_D and the weight of the particle W may be expressed as:

$$F_D = C_D\rho(u_\cdot^2/2)(\pi d_{50}^2/4) \tag{4}$$

$$W = \pi d_{50}^3(\gamma_s - \gamma)/6 \tag{5}$$

From Figure 1,

$$\theta = \tan^{-1}[0.75\ C_D\rho u_\cdot^2/d_{50}(\gamma_s - \gamma)] \tag{6}$$

From the Shore Protection Manual (U.S. Army

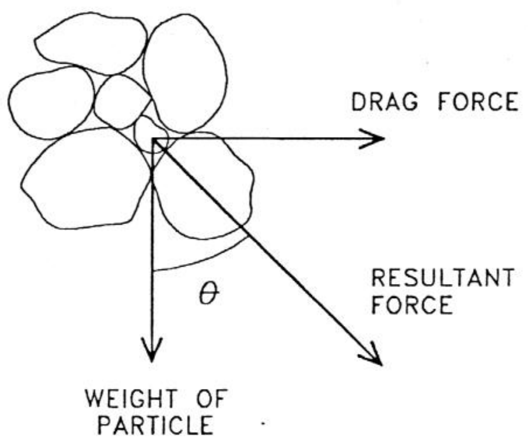

Figure 1. Initiation of sand particle movement.

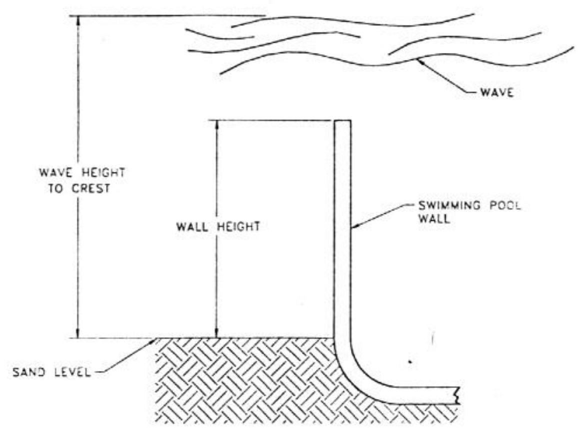

Figure 2. Overtopping of pool wall by wave.

1984), the reflection coefficient C_r, is the ratio of the reflected wave height to the incident wave height. For perfect reflection, where the reflected wave height equals the incident wave height, C_r is unity. This coefficient depends on the geometry and roughness of the reflecting wall and possibly on the wave steepness and the "wave height-to-water depth" ratio. C_r should be taken as 1.0 for walls; a value less than 0.9 should not be used for design purposes.

Scour around swimming pools may differ from scour around seawalls because of two reasons. A swimming pool has corners, while a seawall is assumed to have an infinite length. Also, in the Herbich equation (Equation 1), there is no consideration for overtopping by a wave. As shown in Figure 2, overtopping is very probable for coastal swimming pools in case of an extreme storm.

From physical modeling by RANCE (1980) for large objects, scour depth at the corners was as much as 18 times the scour depth in front of the wall. The authors believe that this ratio may be large because the sand that is removed from the front of the wall is replaced by the sand removed from the corners. The scouring effect at the corners may cause an increase in the scour depth at the corners and a decrease in the scour depth in front of the wall. Thus, the authors believe that the scour depth predicted by Equation 1 may be a good representation of the average scour depth along the wall for swimming pools.

As for the overtopping condition, for vertical walls, the least wave attack behind the wall and the largest scour depth in front of the wall were observed to occur when the top elevation of the wall was one wave height above the SWL (KADIB 1963). The largest wave attack on the area behind the wall and the smallest scour in front of the wall were observed when the top elevation of the wall was at a half wave height below the SWL. From these observations, it may be inferred that maximum scour occurs before overtopping in general. Thus, the authors assumed that the scour depth predicted by Equation 1, even if overtopping occurs, will provide an approximate conservative prediction for the average scour depth along the length of a coastal swimming pool wall.

Because the Herbich equation was derived for non-breaking waves, the predicted scour depth will probably be smaller than the actual scour depth under breaking wave conditions. This is due to the fact that breaking waves cause greater scour than non-breaking waves.

Predicted scour depths from Equation 1 for a depth to SWL of 1.8 m (6 ft) and various values of the coefficient of reflection is presented in Figure 3. Because C_r is to be taken as 1.0 for design purposes, a graph of scour depth versus wave height for several values of depth to SWL is presented in Figure 4 for C_r equal to 1.0. In these figures, the negative sign represents scour, or sand being removed from the front of the wall. The general trend is the decrease in scour depth with the increase in wave height for a given depth to SWL. Also, for a given wave height, the scour depth increases, with an increase in depth to SWL. As the reflection coefficient increases, the scour depth increases until H/d (wave height/water

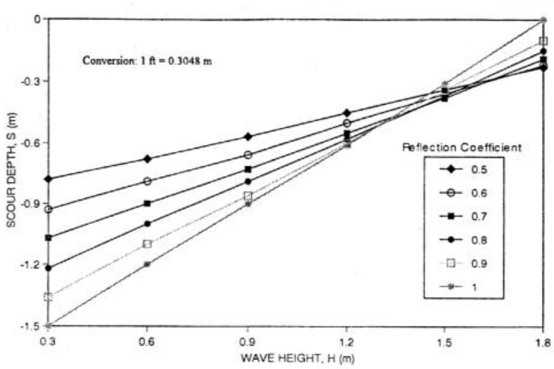

Figure 3. Scour depth vs. wave height, for depth to still water level of 1.8 m.

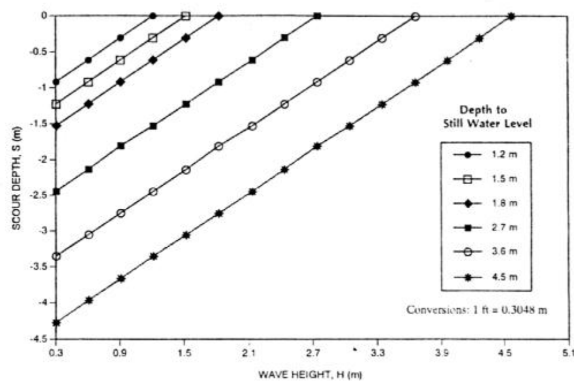

Figure 4. Scour depth vs. wave height, for reflection coefficient (C_r).

depth to SWL) is approximately 0.7, which is generally accepted as the initiation point for breaking waves (Figure 3). It is interesting to note that the curves for the reflection coefficient cross at the approximate value for H/d of 0.7, and the scour depth begins to decrease with an increase in reflection coefficient. It should be noted that beyond the approximate value of 0.7 for H/d, Equation 1 may no longer be valid, because it was developed for nonbreaking waves and not for breaking waves.

OPTIMUM SITING AND SIZING CONDITIONS

The variables in Equation 1 for scour prediction are: wave period, wave height, depth to SWL, and reflection coefficient. The first three variables are site specific to a particular pool; therefore, general conclusions cannot be made based on these variables. Thus, the authors were not able to recommend general optimum siting/sizing aspects for pools based on this equation. Other sources were utilized to make general conclusions for optimum siting/sizing aspects.

RANCE (1980) studied scour around large objects through physical modeling. His observations are reproduced in Table 1. The following conclusions may be made for coastal swimming pools based on Rance's observations: (1) a round swimming pool is expected to experience approximately half the scour around a square pool; and (2) rotating a square pool so that the wave angle of attack is 45 degrees causes approximately 40% more scour than a pool with a zero degree angle of attack.

The following conclusions were made by RICHARDSON (1993) for riverine piers: (1) an increase in pier width causes an increase in scour depth; (2) with a zero degree angle of attack, pier length does not significantly affect local scour depth; if the pier is skewed, doubling the pier length increases scour depth by 33%.

The following general conclusions about the effect of siting/sizing aspects on scour around coastal swimming pools are based on Richardson's observations: (1) a small angle of attack causes the least scour; and (2) it is best to place the side of the pool with the smaller dimension perpendicular to the flow; for example, placing the longer side perpendicular to the flow causes 2.5 times the scour as placing the shorter side perpendicular to the flow for a length to width ratio of 4; and (3) a smaller length to width ratio causes less scour.

CONCLUSIONS

Scour around and under coastal structures such as piers, abutments, piles, pipelines and seawalls has been extensively studied in the U.S.A. and in countries such as the Netherlands, the U.K., Ja-

Table 1. *Scour around objects with diameter larger than a tenth of a wavelength (RANCE, 1980).*

	Shape	Maximum Scour Depth	Horizontal Extent of Scour
Flow Direction →	○	0.064 Dp*	0.75 Dp
	◇	0.180 Dp	1.00 Dp
	□	0.128 Dp	0.75 Dp

*Dp = equivalent diameter of structure

pan and Norway. The applicability of these models to a coastal swimming pool type structure has not been investigated. The Herbich equation (1984) for ultimate scour depth at seawalls contains most of the pertinent variables for scour around coastal swimming pools. This equation does not include the effects of a wave overtopping the pool wall or the effects of the corners of the wall. The extent that these two parameters affect around coastal swimming pools is expected to be negligible. The Herbich equation is not intended for use with elevated pools. A round swimming pool is likely to experience approximately half the scour experienced by a square pool. The angle of attack of the wave directly affects scour around coastal pools. A zero degree angle of attack is likely to result in least scour. Placing the smaller dimension of a rectangular pool parallel to the shore (or perpendicular to the wave) is beneficial in controlling scour around coastal pools. The conclusions made in this study are strictly based on theoretical studies and scour models for non-swimming pool structures. Physical modeling of coastal swimming pools is needed to validate pool scour models reported in this paper.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

A = wave height at the wall;
C_D = coefficient of drag;
C_r = reflection coefficient;
d = depth to still water level;
d_{50} = mean diameter of the sediment;

F_D = drag force;
g = acceleration due to gravity;
H_I = incident wave height;
H_r = reflected wave height;
L = wave length;
S = ultimate scour depth;
T = wave period;
u. = local horizontal velocity parallel to the bottom;
x = horizontal distance traveled by the wave;
θ = specific weight of the water;
γ_s = specific weight of the sediment;
θ = angle of repose of the sediment;
θ_p = phase angle;
ρ = density of water

CONCEPTUAL BREAKAWAY SWIMMING POOL DESIGN FOR COASTAL AREAS

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ABSTRACT

Swimming pools have become an essential attachment to most habitable coastal construction such as hotels, condominiums and single family residences. A large swimming pool type structure may obstruct the free flow of flood water and increase the turbulence. This in turn may increase the scour potential and the wave/debris action on the building and foundation. A conceptual breakaway concrete swimming pool design is described herein. It is demonstrated that this pool will withstand everyday factored water/soil loading, but will collapse and breakaway under extreme wave action, thereby minimizing the detrimental effects of a solid pool.

ADDITIONAL INDEX WORDS: Key-words: Breakaway, swimming pool, scour, coastal construction.

INTRODUCTION

The State of Florida has an extensive tidal shoreline. In recent years, this shoreline has been subjected to rapid development and construction due to a massive population influx. Swimming pools have become essential accessories attached to habitable coastal construction in terms of property value and the tourism industry in Florida. Virtually all of these pools are situated seaward of the habitable structures.

The Federal Emergency Management Agency (FEMA) oversees the construction of all structures (including pools) in the Coastal High Hazard Areas (V-zones) in order for these structures to be insured under the National Flood Insurance Program (NFIP). These requirements are contained in 44CFR Section 60.3 which states that all new construction and substantial improvements in Zones V1-V30, VE, and V shall have the area below the lowest floor level either free of obstruction, or constructed with non-supporting breakaway walls or similar structures.

If a swimming pool is placed below the level of a coastal building, but above natural grade, it may behave as an obstruction to the free flow of flood water. A large object, such as a swimming pool, placed above the natural grade may increase the turbulence of the floodwater, resulting in an increase in the scour potential under and around pools, and around the pile supports. The extra turbulence created by the presence of the pool structure may also cause increased wave and debris action on the elevated portion of the building or other adjacent structures and foundations.

Coastal swimming pools should withstand everyday water and soil loads with an adequate factor of safety, but should collapse and break away in case of a 100-year flood event without acting as an obstruction to the flow of floodwater. If pools located below the base flood elevation in V-zones were designed to disintegrate and not cause water build-up or act as debris on upland sructures or their piles during a specified storm, the detrimental effect on the beach/dune system or adjacent structures would be drastically reduced. Swimming pools designed to be frangible will help preserve the integrity of the beach/dune system and other structures in extreme flooding conditions.

The effect of a swimming pool type massive structure on coastal topography during a storm has been apparent over the years; however, documentation of this effect has started only recently. No basic research has been performed on understanding this effect, or on ways to minimize such costly damage.

DATA ON EXISTING POOLS

The Florida Department of Environmental Protection (FDEP) is responsible for permitting of coastal construction in the coastal zone. Permitting files from FDEP were searched to investigate common scenarios for swimming pools on the Florida coast. Important variables that were recorded include: the shape, dimensions, orientation to the Coastal Construction Control Line (CCCL), location relative to CCCL, maximum depth, 100 year storm surge, distance above or below the sand level and material used. Pool data for 23 swimming pools located in coastal regions of Florida are presented in Table 1. Data was gathered from the FDEP permitting files for the last four years.

From Table 1, it is observed that only one of the pools is fiberglass; the remainder are concrete or gunite. The distribuation of the shapes of the pools is: 70% rectangular, 13% kidney, 4% oval, 9% odd and 4% round. The average largest dimension is 34.4 feet; the average smallest dimension is 17.4 feet.

BREAKAWAY POOL LAYOUT

To force breakaway mechanism in a coastal swimming pool under an extreme storm, joints at 2 ft. on center in the top 3 ft. of the pool walls will be assumed. The ACI Code minimum required flexural reinforcement will be used. Splices will be provided at 3 ft. below the top of the wall. This depth corresponds to the depth at shallow ends for most coastal swimming pools. To provide a failure mechanism at the bottom of the wall near the deep end, another splice will be provided above the floor/wall joint when the depth is 5 feet and more. The depth of 5 feet was chosen so that the bar that extends below the splice at 3 feet could be more than 2 feet long. The vertical joints will allow the walls to breakaway vertically. The splices will allow the walls to break horizontally.

BREAKAWAY POOL DESIGN

Swimming pools have been built from several materials, which include concrete, fiberglass, timber, masonry, and vinyl. The FDEP considers timber pools as frangible because they are vinyl-lined. The authors spoke with many pool builders about typical construction practices. Most of them liked the on-site ease and rapid construction of concrete or pressure sprayed (gunite) pools.

The authors suggest that fiberglass or timber be used for frangible pools because they breakaway easily and result in smaller and lighter debris. However, for pool owners who wish to build a concrete pool, the authors present a recommended breakaway design methodology. It is entirely possible to develop other equally effective breakaway designs for concrete pools.

EVERYDAY LOAD DESIGN

A swimming pool must be able to withstand everyday maximum loading. For pools situated above ground, these loads include the water load inside the pool when it is full, as shown in Fig. 1. The total load is:

$$W_A = 0.5\gamma_w H^2 \text{ per unit width of wall} \tag{1}$$

in which γ_w = unit weight of water, and H = height of pool.

The bending moment at the pool base is given by:

$$M_A = 0.083\gamma_w H^3 \text{ per unit width of wall} \tag{2}$$

For a below ground pool, the maximum everyday forces are caused by soil outside the pool when it is empty, as shown in Fig. 2. This force and the corresponding moment are expressed as the following for a 32° coefficient of internal friction for soil:

$$W_B = 0.235\gamma_s H^2 \text{ per unit width of wall} \tag{3}$$

$$M_B = 0.078\gamma_s H^3 \text{ per unit width of wall} \tag{4}$$

in which γ_s = unit weight of soil.

The ground water table was assumed to be low, which would cause negligible force on a below ground pool. For higher water levels the pool should remain filled with water to prevent it from floating up. A floating pool is likely to crack and will rarely settle back in the original position after flooding subsides.

The everyday maximum forces and moments expected on the pool wall are presented in Tables 2(a) and (b). The waterload on an above ground pool is slightly higer than the soil load on a below ground pool; the two forces just act in opposite directions. Therefore, only the design of an above ground pool with water load is presented herein.

Design shear forces and moments with ACI load factors on a 2 foot width of pool wall are shown in Table 2(c). Corresponding vertical steel design at the splice (3 feet from top) and at the bottom (6 feet from top) are also presented. Two #4 bars are needed at the splice to satisfy ACI code limitation for maximum spacing. Typical sections chosen for the breakaway concrete pool are shown in Fig. 1. Wall panel design layout showing joints and bar splices are shown in Fig. 2. Pool wall and floor reinforcement details are shown in Fig. 3.

WAVE LOADING

The forces from breaking waves may be found from the Minikin Method, which is "based on observations of full-scale breakwaters and the results of Bagnold's study," and is presented in the Shore Protection Manual (1984). Because this method can result in wave forces that may be 15 to 18 times those for nonbreaking waves, the Shore Protection Manual warns that this method be used with caution. The variables are: the depth to the still water level (SWL) at the pool wall, the slope of the shore in front of the pool, and the wave period. The forces and moments on a typical pool wall for a 6 second conservative wave period are presented in Table 3.

Non-breaking waves obviously cause smaller forces on a pool than breaking waves. The non-breaking wave forces can be estimated from the Miche-Rundgren Method contained in the Shore Protection Manual. These forces depend on the free wave height, the depth of water to the SWL, the wave period, the wave reflection coefficient and the height of the wall above ground. The calculated non-breaking wave forces for a 6 second wave, a reflection coefficient of unity and the wall height equal to the water depth are presented in Table 4. The last condition represents no overtopping of the wall by the wave.

VERIFICATION OF BREAKAWAY

A comparison of Tables 2 and 3 reveals some interesting conditions. Breaking waves during a storm are expected to generate shear forces and bending moments which in most cases will easily exceed those caused by the everyday forces. This observation is valid for most water depths of 4 ft. or more and wall heights of 5 ft. or more. Non-breaking waves generate forces and moments on the pool wall which may exceed the everyday forces and moments if the water depth is generally 6 ft. or more or the wave height is 2.5 ft. or more. These critical water/wave depths are situation specific, i.e., they may occur if the shore slope is high and the pool is close to the water line. The wave height also depends on the intensity of the storm.

It may be inferred that the breakaway pool design described herein is expected to perform well in many coastal situations under an intense storm. The strength of the designed pool under wave action is found to be less than the strength needed for everyday loading, for most conditions. Therefore, the pool is expected to withstand the daily normal loading, while it is expected to breakaway along lines of weaknesses under extreme wave action. It is understood that many simplifying assumptions were made and parametric values assumed in the design of the breakaway pool, changes in which will affect the design and the validity of the breakaway criteria. Only a conceptual breakaway pool design is detailed herein, which shows that it is possible to design a frangible pool for coastal areas.

IMPACT OF DEBRIS ON FOUNDATION

If a pool is designed to be frangible, it is likely to breakaway in several pieces during an extreme flooding. It is possible that the broken debris may be carried by wave action and impact on the adjacent house or foundation. The foundation should be designed with proper consideration for this impact force from a frangible pool.

There are many variables which are likely to influence the magnitude of the debris impact force, such as the size of the pieces that will break away, the velocity of the broken pieces, the wave height and wave depth, the amount of time the broken pieces remain in contact with the foundation, and the manner in which the pieces come in contact with the foundation. The position of the pieces in the wave is also a factor for transitional or deep water.

Simplifying assumptions were made in order to develop an expression for the debris impact force on adjoining foundations. It was assumed that the pool wall will break into 2 foot by 3 foot by 6 inch thick pieces (according to the breakaway design for concrete pools developed in this study) and will impact at a velocity equal to the velocity of the water (a conservative assumption).

From Impulse-Momentum relationships (Beer, 1988):

$$\int F\,dt = mv \tag{5}$$

in which F = impact force, dt = increment of time, m = mass of broken piece, and v = velocity of piece when it comes in contact with the foundation.

The velocity of the piece, assuming shallow water conditions, is as follows (Herbich, 1984):

$$v = H/2\,(g/d)^{1/2}\cos\theta \tag{6}$$

in which H = wave height, d = depth to SWL, θ = phase angle of wave, and g = acceleration due to gravity = 32.2 ft/sec^2.

For maximum velocity, assuming $\theta = 0$ degrees:

$$\int F\,dt = 13.98\,(0.5H)(g/d)^{1/2} \tag{7}$$

Values of the impulse force from Eq. 7 for various values of $\sqrt{H/d}$ are shown in Table 5. If a frangible coastal concrete pool is designed, the adjacent foundation should be designed to withstand debris impact forces similar to the presentation in this table.

CONCLUSIONS

The following conclusions may be made based on the findings of the study:

1. There have been no previous or continuing studies which address frangibility criteria for coastal swimming pools.
2. Most coastal swimming pools are rectangular; the average dimensions are about 17 feet by 34 feet. Almost all coastal pools are made of concrete or gunite. The average distance from the CCCL is 112.2 feet; and the average maximum depth is 5.75 feet. The average storm surge is 6.7

feet above the grade. These conclusions are based on a survey of 23 coastal pools from the Florida Department of Environmental Protection permit files.

3. Most coastal pool builders like the ease of working with gunite.
4. It is feasible to theoretically and practically design and construct a good and safe breakaway swimming pool made of concrete. A good breakaway concrete pool design includes vertical joints and splices in the reinforcing steel.
5. Scour that causes undermining of the pool wall may cause failure. For example, for the concrete swimming pool design, a 6 foot wall undermined approximately 3 feet will fail due to the weight of the water inside the pool.
6. The debris from a breakaway pool may impact the pool or house foundation due to wave and current action. The foundation must be designed to withstand the debris impact force from a frangible pool.
7. The authors recommend that for high hazard areas, in which frangibility is desired, fiberglass or plywood be used for the pools. If the pool must be concrete, a design such as the one presented in this report may be used as an option. If a concrete pool is to be situated above ground, the authors recommend that the pool be no more than 3 feet above ground.

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These figures were used in this paper:

dt = increment of time;
F = impact force;
g = acceleration due to gravity;
H = wave height;
M_A = bending moment at pool base (above ground pool);
M_B = bending moment at pool base (below ground pool);
m = mass of broken piece;
W_A = water load inside pool when full (above ground pool);
W_B = water load inside pool when full (below ground pool);
γ_s = unit weight of soil;
γ_w = unit weight of water;
θ = phase angle of wave;
ν = velocity of piece on impact.

Table 1. Florida Coastal Swimming Pool Characteristics

Pool #	Shape	Dimensions	Orientation to CCCL	Loc. Rel. to CCCL	Max. Depth	100 Year Storm Surge (NGVD)	Bottom Elev. (NVGD)	Material
1	Rect.	20'x40'	10'//	<47' seaward	6'	13.2'		shotcrete
2	Rect.	18'x26'	18'//	<158'	6'	12.8'	5'	shotcrete
3	Rect.	17'x42.5'	42.5'//	<162.5'	6'		10.5'	conc. shell on grade
4	Rect.	18'x38'	38'//	<9'	5'	12.3'	13'	conc. shell & stem wall
5	Rect.	20'x40'	40'//	<77'	8'		4.5'	reinf. gunite shell
6	Rect.	20'x60'	60'//	<64'	8'	12.6'	0.0'	6" conc. shell
7	Rect.	20'x32'	20'//	<7'	6'	11.3'	10.4'	5" reinf. conc. shell
8	Rect.	15'x40'	40'//	<125'	6'	11.4'	16'	conc. shell
9	Rect.	18'x38'	38'//	<183'	6'	12.2'	-1.0'	6" reinf. conc. shell
10	Kidney	14'x28'	28'//	<340'	6'	14.8'	0.6'	4-6" conc. shell
11	Rect.	14'x28'	14'//	<9'	5.5'	14.7'	1.2'	6" conc. shell
12	Rect.	10'x28'	28'//	<300'	4'	12.2'	5.8'	manufact. fiberglass
13	Rect.	17'x29'	29'//	<75'	6'	12.5'	4.7'	8" conc. shell
14	2 Rect.	12'-18'x24	24'//	<94'	5.5'	13.1'		5" conc. shell
15	Kidney	20'x31'	31'//	<132'			12'	conc. shell
16	Rect.	19'x41'	19'//	<138"	6'	13.1'	6.5'	6" conc. slab shell
17	Odd	35'x44'	35'//	<216'	8'	12.3'	2.0'	conc. shell
18	Rect.	18'x40'	8'//	<60'	4.5'		6.0'	conc. shell
19	Round	16'x20'	16'//	<20'	5'	14.7'	0.0'	6" gunite shell
20	Rect.	14'x27'	app. 45 deg.	<19'	5'	12.5'	4.3'	6" shotcrete
21	Odd	22'x33'	33'//	<295'	5.5'	12.8'	2.0'	conc. shell
22	Oval	16'x26'	26'//	<38'	2.5'		11'	reinf. gunite shell
23	Kidney	15'x36'	36'//	<12'	6'	11.5'	9.8'	4"-6" conc. shell

Table 2. Everyday Maximum Forces and Moments on Pool Wall

Wall Height (ft)	Above Ground Pool (lb/ft width)	Below Ground Pool (lb/ft width)
4	499	451
5	780	705
6	1123	1015
7	1529	1382

(a) Everyday Forces

Wall Height (ft)	Above Ground Pool (lb/ft width)	Below Ground Pool (lb/ft width)
4	666	602
5	1300	1175
6	2246	2030
7	3567	3224

(b) Everyday Moments at the Base

Depth from Pool Top (ft)	Ultimate Shear (lb)	Ultimate Moment (lb-ft)	Reinforcement Design
3	786	786	1 - #4 (2 - #4 provided)
6	3145	6290	3 - #4

(c) Ultimate Shears and Moments in Wall (on 2' width)

Table 3. Breaking Wave Forces & Moments on Pool Wall

Shore Slope	Depth to SWL (ft)						
	1	2	3	4	5	6	7
0.00	31	125	281	499	780	1123	1529
0.01	136	683	1787	3552	6083	9506 .	13881
0.02	140	709	1861	3712	6380	10038	14739
0.03	146	741	1950	3896	6710	10611	15648
0.04	153	779	2049	4097	7065	11221	16601
0.05	162	820	2157	4312	7443	11861	17607
0.07	181	912	2394	4780	8258	13236	19726
0.10	213	1067	2792	5561	9607	15478	23187

(a) Forces on Pool Wall (lb/ft)

Shore Slope	Depth to SWL (ft)						
	1	2	3	4	5	6	7
0.00	10	83	281	666	1300	2246	3567
0.01	102	1100	4459	12078	26257	49853	85766
0.02	105	1142	4656	12660	27633	52854	91462
0.03	110	1199	4898	13337	29176	56112	97537
0.04	117	1267	5173	14088	30852	59593	103942
0.05	124	1343	5475	14900	32647	63273	110713
0.07	142	1514	6146	16681	36547	71217	125072
0.10	172	1810	7291	19690	43075	84295	148702

(b) Moments on Pool Wall (lb-ft/ft)

Table 4. Non-breaking Wave Forces & Moments on Pool Wall

Free Wave Height (ft)	Depth of Water from SWL (ft)						
	1	2	3	4	5	6	7
0.5	48.4	110.9	178.6	250.0	300.9	>	>
1.0	>	191.7	306.0	428.7	553.1	677.1	795.9
1.5	>	>	426.6	598.1	766.7	953.6	1132.1
2.0	>	>	537.6	750.0	975.5	1206.7	1434.7
2.5	>	>	>	894.4	1160.9	1441.7	1728.7
3.0	>	>	>	>	1336.6	1661.2	1995.9
4.0	>	>	>	>	>	2076.8	2489.0

(a) Forces on Pool Wall

Free Wave Height(ft)	Depth of Water from SWL (ft)						
	1	2	3	4	5	6	7
0.5	19.0	97.9	243.3	461.6	631.7	>	>
1.0	>	152.1	387.5	760.3	1257.5	1866.8	2552.7
1.5	>	>	514.6	1007.5	1671.1	2615.8	3715.6
2.0	>	>	634.3	1222.7	2045.5	3179.2	4554.1
2.5	>	>	>	1428.5	2390.5	3665.2	5317.9
3.0	>	>	>	>	2692.6	4124.1	5970.7
4.0	>	>	>	>	>	5056.0	7179.4

(b) Moments on Pool Wall

> Beyond Range for Nonbreaking Waves

Table 5. Impulse on Foundation From Debris

$H/\sqrt{d}$*	Impulse ($\int Fdt$) (lb-sec)
0.5	20
1.0	40
1.5	60
2.0	80
2.5	100
3.0	120
3.5	140
4.0	160
4.5	180
5.0	200

* H and d are in feet