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Scour Impact of Coastal Swimming Pools on Beach Systems

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ABSTRACT

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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:

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

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_* = 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_*, 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_* for shallow water conditions is applicable for coastal swimming pools:

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

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:

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

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:

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

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

From Figure 1,

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

From the Shore Protection Manual (U.S. Army

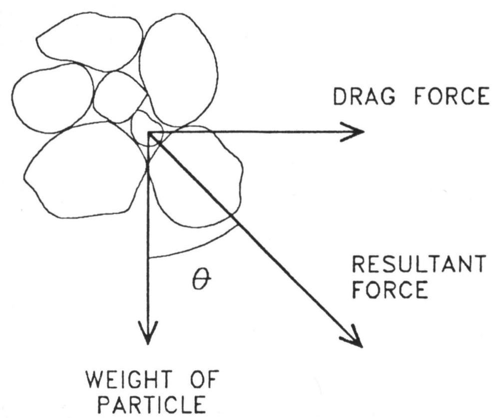

Figure 1. Initiation of sand particle movement.

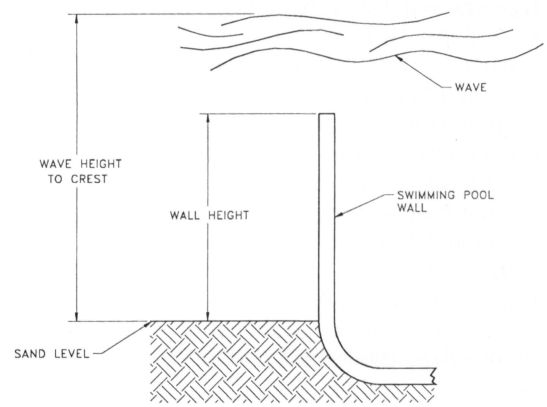

Figure 2. Overtopping of pool wall by wave.

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

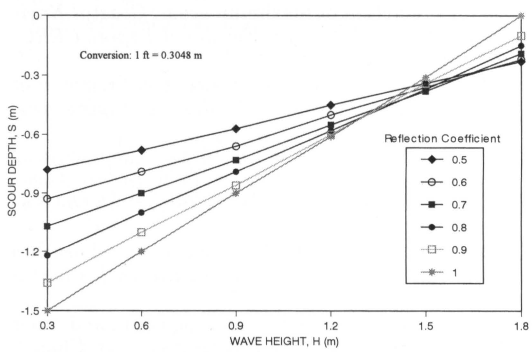

Figure 3. Scour depth vs. wave height, for depth to still water level of 1.8 m.

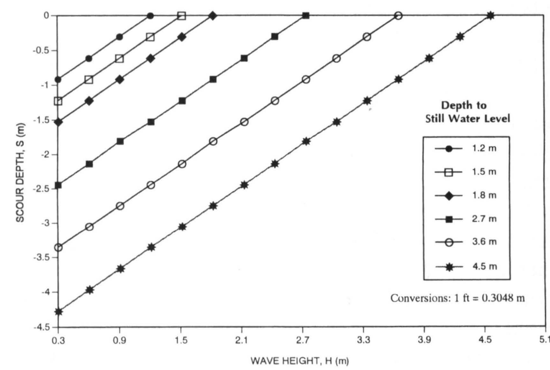

Figure 4. Scour depth vs. wave height, for reflection coefficient (C_r).

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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LITERATURE CITED

BREUSERS, H.N.C., 1972. Local Scour Near Offshore Structures. *Offshore Hydrodynamics Symposium, Proceedings, Oosterveld, M.W.C., Wageningen, Netherlands,* August 25-26.

DAS, B.M., 1990. *Principles of Geotechnical Engineering.* 2nd ed., Boston: PWS-Kent.

EADIE, R.W. and HERBICH, J.B., 1987. Scour About a Single, Cylindrical Pile Due to Combined Random Waves and a Current. *Proceedings, Twentieth Coastal Engineering Conference,* V.3.

FOTHERBY, L.M., 1992. *Footings, Mats, Grout Bags, and Tetrapods: Protection Methods Against Local Scour at Bridge Piers.* Boulder, Colorado State University.

FOWLER, J.E., 1992. Scour problems and methods for prediction of maximum scour at vertical seawalls. *Coastal Engineering Research Center, Technical Report CERC-92-16,* December 1992.

FOWLER, J.E., 1993. Coastal scour problems and methods for prediction of maximum scour. *Coastal Engineering Research Center, Technical Report CERC-93-8,* May 1993.

FROEHLICH, D.C., Analysis of onsite measurements of scour at piers. *Journal of Hydraulic Engineering,* (source unknown).

HANCU, D.I.S., 1971. Sur le Calcul des Affouillements Lacaux dans la Zone des Piles du Pont. *Intern. Assoc. for Hydraulic Research and Its Impact on the Environment,* V.3 paper C36.

HERBICH, J.B. and KO, S.C., 1968. Scour of sand beaches in front of seawalls. *Proceedings, Eleventh Coastal Engineering Conference,*

HERBICH, J.B., 1981. Scour around pipelines and other objects. *Offshore Pipeline Design Elements,* Chap.3, 1981.

HERBICH, J.B.; SCHILLER, R.E.; DUNLAP, W.A., and WATANABE, R.K., 1984. *Seafloor Scour: Design Guidelines for Ocean-Found Structures.* New York and Basel, Switz: Marcel Dekker

IBRAHIM, A. and NALLURI, C., *Scour Prediction Around Marine Pipelines,* (source unknown).

JAIN, S.C. and FISCHER, E.E., 1979. Scour around circular bridge piers at high froude numbers. *National Technical Information Service, Document No. PB 80-139322,* U.S. Dept. of Commerce.

KADIB, A., 1963. Beach profiles as affected by vertical walls. *Beach Erosion Board Technical Memo No. 134.*

KAWATA, Y. and TSUCHIYA, Y., 1988. Local scour around cylindrical piles due to wave and currents combined. *Proceedings, Twenty-first Coastal Engineering Conference, Spain,* V.2, Chap. 97.

KHANBILVARDI, R.M.; AKHTAR, M.W., and ROGOWSKI, A.S., 1988. Local scour around cylindrical objects. *Water Resources Bulletin,* American Water Resources Association, 24(4).

POWELL, K.A., 1987. Toe scour at sea walls subject to wave action. *Hydraulics Research, Wallingford, Report SR 119.*

RANCE, P.J., 1980. The potential for scour around large objects. *Scour Prevention Techniques Around Offshore Structures.* The Society for Underwater Technology, Seminar in London, 16 December, 1980.

RICHARDSON, D., 1990. Evaluating scour at bridges. *Federal Highway Administration, Report No. FHWA-IP-90-017,* HEC 18.

SHEPPARD, D.M. and NIEDORODA, A.W., 1990. *Local Structure Induced Sediment Scour.* Department of Coastal and Oceanographic Engineering, University of Florida.

U.S. ARMY, 1984. *Shore Protection Manual.* Washington: Coastal Engineering Research Center, Department of the Army, Volume II, 4th ed.

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