Fishing Pier Design Guidance

Part 2:

Methodologies for Design and Construction

Ralph R. Clark
Florida Department of Environmental Protection
Bureau of Beaches and Coastal Systems

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Foreword

The Bureau of Beaches and Coastal Systems of the Florida Department of Environmental Protection is responsible for protection and management of the State's sandy beaches fronting the Gulf of Mexico, the Atlantic Ocean and the Straits of Florida, and the regulation of coastal development adjacent to those beaches. Pursuant to Chapter 161, Florida Statutes, the Bureau regulates coastal construction for the protection of the beach and dune system and adjacent properties, and for the protection of existing and proposed construction.

Among the various categories of major coastal structures regulated by the Bureau are gulf and ocean fishing piers, which extend across Florida's beaches and the nearshore littoral environment. Fishing piers along the State's beaches have unique design and construction challenges. This report is the second in a two-part series and provides technical guidance for the design and construction of gulf and ocean fishing piers.

STATE OF FLORIDA, DEPARTMENT
OF ENVIRONMENTAL PROTECTION

Michael R. Barnett, P.E., Chief
Bureau of Beaches and Coastal Systems

Acknowledgements

The inspiration of this document came from my childhood experiences in Port Aransas, Texas, living on a Gulf of Mexico fishing pier in the early to mid-1960's. Those adolescent days included working for the pier concessioner, and spending every free moment, including every night, watching the surf and fishing for speckled trout, redfish, drum, tarpon, sharks, jacks, and a multitude of finned creatures that filled me with amazement and incited my dreams of a working future in the coastal environment. Experiencing large gulf storms through the impacts of Hurricanes Carla (1961), Beulah (1967), and Celia (1970), and witnessing the damage inflicted on coastal properties and structures, particularly the large wooden gulf piers, gave me a sense of awe and planted the seed for my future career in coastal engineering. Admittedly, this book is a labor of love, and I am deeply thankful to the staff and supervisors of the Bureau of Beaches and Coastal Systems for allowing me the opportunity to utilize my "spare moments" over the past three years towards its preparation. During my 37 years of employment for the State of Florida as a coastal engineer, I had the privilege of inspecting each of the fishing piers listed in this document as well as reviewing and contributing to the design of roughly half of them. When I had to evaluate coastal impacts following over 40 hurricanes and a comparable number of tropical storms and major northeasters, the gulf and ocean fishing piers always provided me a venue for evaluating a storm's strength as determined from its tide level, wave heights, beach erosion and structural damage. I was always blessed for being employed to do what I enjoyed with so much passion.

There are too many "pier people" to acknowledge that I have encountered over the span of my life, including many names and faces forgotten. However, I would have to acknowledge those key individuals who developed my educational background and facilitated my employment, enabling me to work with fishing piers – Gus Mergle, who hired a Dallas teenager to work for him on the Texas pier he managed; Harry M. Coyle, who at Texas A&M University taught me the principles of marine pile foundation design; Dean Morrough P. O'Brien, who at the University of Florida taught me how to analyze coastal systems; Robert G. Dean, who taught me wave mechanics and sediment transport and whom I continue to seek advice; William T. "Bill" Carlton, who hired and mentored a graduate student out of the University of Florida in 1973; Gil Hill, who gave me all the pier projects to review; and Deborah Flack and Lonnie Ryder, who put their trust in me.

I have to provide special acknowledgement and thanks to unique individuals who have provided specific professional guidance of varying degree relating to fishing pier design over my career. Foremost among these are Robert M. Brantly and Michael R. Barnett. Their review, guidance, and encouragement in the preparation of this document is sincerely appreciated. I also acknowledge the professional guidance related to fishing pier design in the "early years" from James Purpura, T.Y. Chiu, Spencer Rogers, Chris Jones, James H. Balsillie, Mark E. Leadon and

Erik J. Olsen. During more recent years of dealing with fishing pier design problems, I gratefully acknowledge the input, guidance, and often long discussions with a number of design professionals, including Albert E. Browder, Kevin R. Bodge, Chris G. Creed, David L. Conrad, Brian C. Rheault, Stephen A. Nichols, Gregory S. Preble, Lee E. Harris, Michael R. Dombrowski, Chris Forehand, Dave K. Hemphill, Jeffrey R. Tabar, Gary L. Anderson, Wayne D. Lasch, James Patanio and Lizbeth Childs.

I would be remiss not to mention Bureau staff individuals who actually had hands on contributions towards the preparation of this document. Elizabeth Kromhout provided me technical support and converted my graphics into useable files. James Lagrone gave me assistance when I prepared the cover and pier table. Gene Chalecki gave me direction on matters involving the Florida Building Code, while Jamie Christoff provided guidance on environmental permitting. And Susan Eaton searched high and low to locate obscure and difficult-to-find essential books, codes and research papers that I needed to support the guidance given in this document. Various Bureau field engineers assisted by taking pier photos at my request, and their contributions are noted on the photos they provided. Credit is also cited for photos or figures provided by private engineering and construction firms that are currently building these new state-of-the-art fishing piers in Florida. Although I took many of the pier photos used in this document, a number were simply obtained from the Bureau's photo files that have been expertly archived by Gerri Krohn.

It is my hope that this document will provide guidance, interest, and appeal to not only the professionals dealing with the issues of fishing pier design, but will also be of interest to others who may be involved in the planning, permitting, management, operation, or maintenance of these piers. There may also be some interest to those who enjoy fishing piers, and solely wish to gain some insight into understanding what is involved in the design and construction of a fishing pier.

Ralph R. Clark, P.E., P.L.S.
Coastal Engineer
Bureau of Beaches & Coastal Systems

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Chapter 1 – Introduction

The nearshore waters of the Atlantic Ocean and Gulf of Mexico are host to a wide variety of fish species that are sought after by coastal recreational fishermen around the State of Florida. There are roughly 35 ocean and gulf fishing piers along the barrier beaches of Florida, which offer safe and stable recreational platforms to access this important coastal fishery (**Table 1**). Unfortunately, these fishing piers are subject to severe damage and total destruction during hurricanes and major storms that impact the coast (**Photo 1**). It has become increasingly important to construct these piers to resist, to the extent practical, the damages associated with major storms.

Photo 1. Okaloosa County Pier during the fringe impact of Hurricane Frances (2004) [Bureau of Beaches & Coastal Systems Photo Files]

The design and construction of ocean fishing piers has evolved with an increased understanding of the forces under which the structures are subjected. The most critical forces affecting the survivability of ocean fishing piers are caused by wind generated waves. Fishing piers should be designed to withstand the forces of breaking waves expected to occur at their location during major storms.

With the selection of a design storm event, it is important to determine the storm tide elevation across which the storm waves will propagate. Equally important as determining a design storm tide level is considering the beach and nearshore profile change caused by the erosion of the design storm event as well as the additional localized scour expected at the individual foundation piles. A geotechnical investigation with core borings is necessary for any pier construction in order to determine adequate pile penetration and breakout resistance resulting from the soil characteristics.

Pier construction techniques will likewise be important particularly when the dead loads of a construction crane need to be considered in the design of the foundation and structural members. Wind loads are specifically important for any canopies or concession buildings located on a pier. Pier decks and rails have additional design considerations.

Fishing Pier Design Guidance is divided into two documents. Its purpose is to identify and provide some basic guidance in the design of ocean and gulf fishing piers in Florida. Many of the same concepts are applicable for pier design in coastal inlets and estuaries or along the Florida Keys.

Part 1: Historical Pier Damage in Florida discusses the history of pier damages during recent coastal storm events in Florida. *Part 2: Methodologies for Design and Construction* discusses the major issues in fishing pier design and construction. An initial chapter discusses the selection of design storm conditions to determine the design storm tide and waves. There are both scientific and legal guidance that simplify this issue. The following chapter discusses the design consideration of erosion and scour. For projected design erosion and scour conditions, different acceptable methodologies have been employed for different fishing piers. Because wind generated water waves represent the greatest design challenge for fishing piers, separate chapters address the prediction of design wave heights and the evaluation of design wave loads. Subsequent chapters of *Part 2: Methodologies for Design and Construction* address various details of the structural design. A fishing pier's pile foundation is the most critical structural system to design. Different methods are discussed for determining adequate pile bearing capacity, including the static formula method, the dynamic formula method, the wave equation analysis, point bearing piles, and pile load tests. Also discussed are pile driving criteria and installation requirements, uplift capacity of piles, lateral resistance of piles and lateral support requirements. Various building code requirements are discussed for driven piles of concrete, steel, timber, and special materials.

A separate chapter discusses the non-water loads, including dead loads, live loads, construction loads, and wind loads. Because most new fishing piers in Florida are constructed of concrete, the applicable building code requirements for reinforced concrete structures are discussed. Various aspects of a pier's structural design follow conventional procedures employed in building construction today and it is not the intent of this document to discuss those typical design concepts or procedures but to elaborate on the issues considered fairly unique to the design of gulf and ocean fishing piers.

Additional discussion in *Part 2: Methodologies for Design and Construction* is provided on the design of breakaway decks and rails as well as the construction techniques typically employed in today's marine construction. Lastly, the final chapters provide discussions on a pier's anticipated effects on coastal processes and the various environmental considerations that affect pier construction and their subsequent operation.

Table 1. Ocean and Gulf Fishing Piers in Florida

Piers on the Gulf of Mexico

Pier	County	Location[1]	Year[2]	Length (ft)	Material[3]
Pensacola Beach	Escambia	R120.5	2001	1470	C
Navarre Beach	Santa Rosa	R209	1974 (2010)	1545	C
Okaloosa County	Okaloosa	R13.9	1998	1260	C
Russell-Fields,Panama City Beach	Bay	R40.5	1977 (2009)	1500	C
MB Miller, Bay County	Bay	R57.3	1968 (2010)	1500	C
St Andrews State Park	Bay	R92.8	1959	600	W
Mexico Beach	Bay	R129.2	1956 (2010)	816	W
Big Pier 60, Clearwater Beach	Pinellas	R44	1962	1116	C
Private Pier, Indian Shores	Pinellas	R98.4	n.a.	250	W
Redington Long	Pinellas	R104.2	1962	1021	W
Andrew Potter, Ft. Desoto Park	Pinellas	R179.8	1964 (2002)	1027	C
Manatee County	Manatee	R20.6	n.a.	300	C
Venice Municipal Pier	Sarasota	R131	1966	696	C
Ft Myers Beach	Lee	R180.7	1974	560	C
Naples	Collier	R74.4	1961	1000	W

Piers on the Atlantic Ocean

Pier	County	Location[1]	Year[2]	Length (ft)	Material[3]
Ft Clinch State Park	Nassau	R9	1980	2409	C
Amelia by the Sea Condominium	Nassau	R39	1948	775	W
Jacksonville Beach	Duval	R65.2	2004	1300	C
St Augustine Beach	St Johns	R142	1986	604	C
Flagler Beach	Flagler	R79	1928	800	W
Daytona Beach	Volusia	R84	1925	744	W
Sunglow, Daytona Beach Shores	Volusia	R117.7	1960	818	W
Lighthouse Point Park	Volusia	R147	1991	1056	C
Canaveral Inlet Jetty Park	Brevard	R1	1997	464	C
Canaveral Pier, Cocoa Beach	Brevard	R16	1963	706	W
Sebastian Inlet State Park	Brevard	R219	1970 (2003)	490	C
Sea Quay Condominium	Indian River	R75	2008	600	C
Juno Beach	Palm Beach	R31.2	1998	993	C
Lake Worth	Palm Beach	R128.5	1972 (2008)	940	C
South Lake Worth Inlet	Palm Beach	R151	1967	410	C
Deerfield Beach	Broward	R2.7	1963 (1992)	949	C
Pompano Beach Municipal Pier	Broward	R33.7	1963	829	C
Anglin’s, Lauderdale-by-the-Sea	Broward	R50.5	1963	876	C
Dania	Broward	R98.3	1962	928	C
Sunny Isles	Dade	R16.4	1936 (2012)	777	W

1. Location relative to the nearest FDEP reference monument established by the CCCL program.
2. Dates in parentheses are anticipated reconstruction dates or recently completed reconstruction dates.
3. C – Concrete; W – Wood

Chapter 2 – Selection of Design Storm Conditions: Tides and Waves

The discussion of the historical damages to Florida fishing piers in *Part 1: Historical Pier Damage in Florida* suggests that no region of the state is without storm damage potential. For each region of the state there have been several recent storms that have inflicted storm tides and wave loads that have had significant impacts on coastal structures, including fishing piers. Like the proverbial "canary in the mine", fishing piers are typically the first structures to experience the damaging impact of coastal storm events.

With an acceptance of the risk, the initial question of any pier designer then settles on the storm magnitude for the selected site for pier construction. For what magnitude storm event should a pier be designed? In reality, this question is not addressed by normal building codes.

In 1998, the Florida Legislature amended Chapter 553, Florida Statutes, Building Construction Standards, to create a single state building code that is enforced by local governments. The State of Florida has currently adopted the *Florida Building Code 2007* to govern construction throughout Florida; however, only Chapters 16, 18, and 31 of this code have any substantial relevance to the design and construction of ocean and gulf fishing piers. The *Florida Building Code* is updated every three years with the 2010 edition scheduled for 2011. While significant code requirements are not anticipated that would affect pier construction, any pier designer should review the latest version adopted by the State of Florida. Chapter 16 of the *Florida Building Code 2007* addresses structural design, Chapter 18 addresses soils and foundations, and Chapter 31 (specifically Section 3109) addresses structures seaward of a Coastal Construction Control Line (CCCL). CCCL's are established along most of the coast of Florida where coastal barrier beaches exist; therefore, most ocean and gulf fishing piers, except in the Florida Keys and Big Bend regions of Florida, are located seaward of CCCL's.

It should, however, be noted that the *Florida Building Code 2007* in Section 3109 addresses primarily habitable major structures, which are defined as structures designed primarily for human occupancy and are potential locations for shelter from storms. Typically these structures are residences, hotels, and restaurants, and the building code specifically requires that they be designed for the erosion, scour, and loads of a 100-year return interval storm event, including wind, wave, hydrostatic, and hydrodynamic forces acting simultaneously with typical structural (live and dead) loads. All habitable major structures are required to be elevated on, and securely anchored to, an adequate pile foundation. (Reference – *Florida Building Code 2007*)

Fishing piers have not been identified as habitable major structures. Although their function is definitely for human use, that use is not intended for shelter, particularly from storm events. Chapter 62B-33, Florida Administrative Code (*Rules and Procedures for Coastal Construction and Excavation*) has specifically identified the minimum design storm event for pier construction. Rule 62B-33.007 (4) (k), Florida Administrative Code, states –

> *Fishing or ocean piers or the extension of existing fishing or ocean piers shall be designed to withstand at a minimum the erosion, scour, and loads accompanying a twenty (20)-year storm event. Pier decking and rails may be designed to be an expendable structure. Major structures constructed on the pier shall be designed for the wind loads as set forth in the Florida Building Code. Pile foundations shall not obstruct the longshore sediment transport and shall be designed to minimize any impact to the shoreline or coastal processes.*

The 20-year return interval storm event is therefore the *minimum* design storm for which ocean and gulf fishing piers are required to be constructed in Florida. Public structures, including fishing piers, are typically designed for a 50-year life span. The probability of occurrence of a storm tide exceeding a certain elevation during a specified time period may be determined mathematically by a binomial theorem. Walton (1976) plots encounter probability versus encounter period for use in coastal construction economics of repair or replacement. The probability of occurrence for a minimum design event, a 20-year storm, during a 10-year period is about 42 percent. For a design life of 50 years, the encounter probability would be 94 percent. When considering the risk of an extreme event, the probability of having a 100-year storm during a 50-year design life would be about 40 percent.

While some piers may economically be constructed for the impact of a 20-year storm event, some piers or their foundations are designed for conditions exceeding the 20-year storm. For example, the Juno Beach Fishing Pier (**Photo 2**), which was constructed in 1998 in northern Palm Beach County, had its superstructure designed for a 40-year storm wave event and its substructure (foundation piles and pile caps) designed for a 100-year storm wave event (Buckingham and Olsen, 1991).

To completely avoid the effects of waves on the superstructure of a pier constructed on the coast of Florida, the pier deck would likely have to be constructed to an elevation exceeding +30 feet NAVD88 (North America Vertical Datum). Above +30 feet, such elevations become excessively too high for the intended function of recreational fishing. Therefore, the economics of repair or replacement need to be considered on an individual project basis as it relates to the potential risk from an extreme storm event. There are, however, various design measures that may reduce the risk of damage and improve the performance of a pier structure during storm events exceeding the 20-year recurrence interval. Such measures will be discussed in detail in subsequent sections of this document.

Selection of design storm event and associated storm tide level, leads to the determination of wave characteristics, and erosion conditions for the site of a proposed fishing pier. Design erosion conditions and design wave conditions are discussed in the following sections of this report.

Storm tide data may be determined by independent analyses or from the results of studies already conducted. The Bureau of Beaches and Coastal Systems has sponsored studies conducted by the Beaches and Shores Resource Center (BSRC), Florida State University, to determine the combined total storm tide frequency for each coastal county with an established CCCL. The "combined total storm tide" is the storm surge that occurs due to the drop in atmospheric barometric pressure, the wind stress over the water surface, the rise in the astronomical tide, and the dynamic wave set-up due to the momentum of repetitive shore-breaking waves. The results of these storm tide studies may be obtained at the BSRC web address – http://beach10.beaches.fsu.edu/. A CCCL is based on the damage potential of a 100-year return interval storm tide, and new studies have recently been conducted as part of the process to reestablish CCCL's in northwest Florida.

In these storm tide studies, the BSRC evaluates the historical probabilities of hurricane characteristics utilizing the data of hurricanes impacting an area since 1900. A more detailed study for the purpose of determining higher frequency storm tide conditions (e.g., 5-, 10-, 20-years) would necessitate evaluating the characteristics of all storm events as well as hurricanes, to include tropical storms, subtropical storms, and extratropical storms (northeasters). Absent a study of high frequency storm events, for the purpose of fishing pier design, the storm tide level determined in the CCCL studies by the BSRC would appear sufficient. Such data would provide the water level of a chosen storm event and could be employed in the calculations of erosion and breaking wave heights.

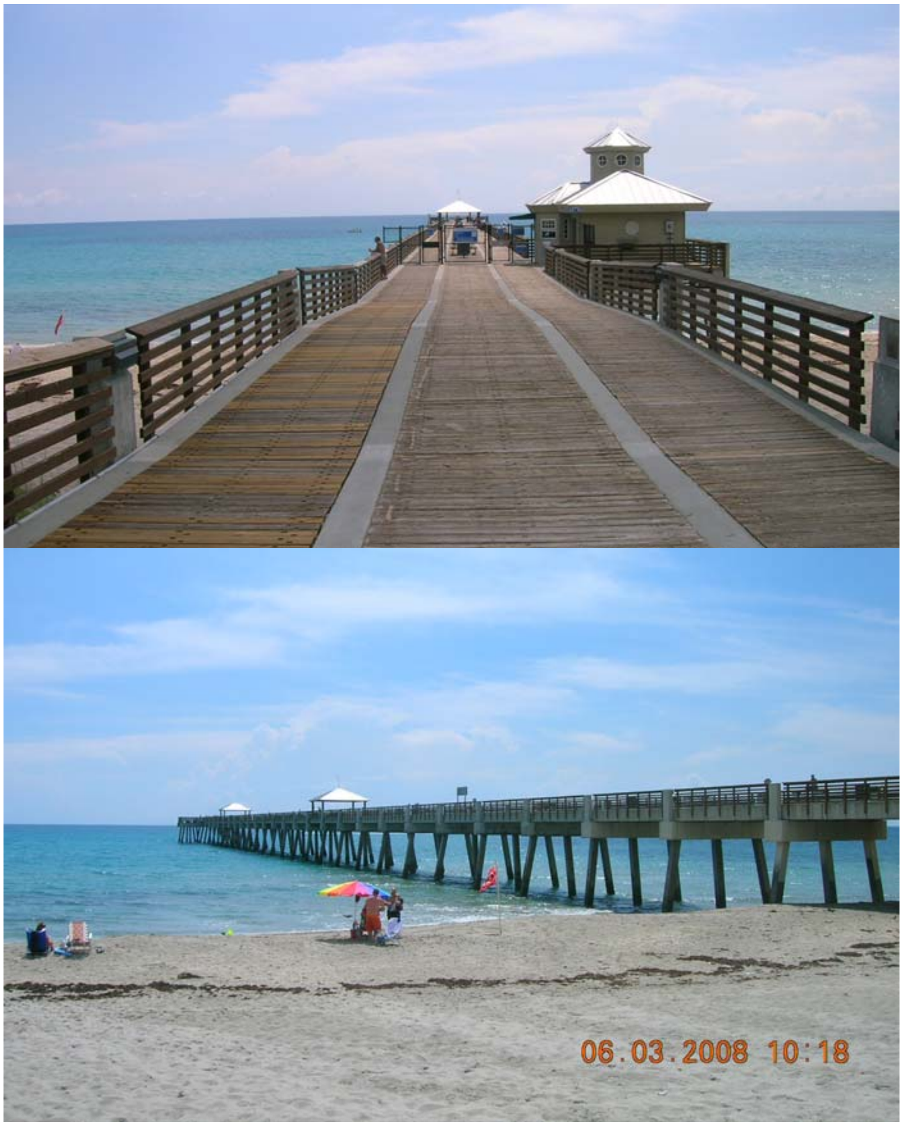

Photo 2. Juno Beach Fishing Pier [Photos by Mark Taynton, Bureau of Beaches and Coastal Systems]

Chapter 3 – Erosion and Scour

The beach and nearshore profile at the site selected for a new ocean or gulf pier is subject to fluctuations with respect to time. On the Florida coast, there are long-term erosion trends as well as seasonal fluctuations which need to be considered when designing a pier. Fortunately, around the coast of Florida there is substantial profile data available, which may be obtained from the Bureau of Beaches and Coastal Systems at the web address – http://www.dep.state.fl.us/beaches/data/data.htm.

Areas with a relatively high long-term shoreline change rate (e.g., greater than -5 feet/year) may not be desirable sites for pier construction. This horizontal recession of the entire profile may be problematic for the landward terminus of the fishing pier as well as any concession or restaurant building that may be attached. It is generally wise to look at the most recent approximately 30 years of historical shoreline change data along with longer term data from the past 100 or so years. Recent accelerated shoreline change rates may be higher than the long-term trend and therefore a better projection of profile change during the design life of a pier. It is reasonable to simply translate the entire equilibrium profile the same horizontal distance landward as projected with the change in shoreline position.

Conversely to the trend of an eroding shoreline and nearshore profile, there are some areas of the coast, often located at island ends, where substantial accretion is occurring. These areas would likewise be poor sites for fishing piers due to the decreasing depths for fishermen as the pier becomes increasingly landlocked. Such natural accretional areas should generally be avoided; however, sometimes the accretion is a result of man-made influences.

3.1 – St. Augustine Beach Pier Example

From the designer's perspective, an unexpected circumstance has occurred at a couple fishing piers in Florida due to the results of beach nourishment activities. On the Atlantic coast, the St. Augustine Beach Fishing Pier, constructed in 1986, was rendered completely landlocked during a federal beach restoration project in January 2003 with the placement of 4.2 million cubic yards of sand fill (**Photo 3**). In 2004, severe erosion of this beach fill took place with the greatest sediment losses occurring during Hurricanes Frances and Jeanne, which resulted in the pier becoming "fishable" again (**Photo 4**). To mitigate the hurricane losses to the beach fill, another 2.8 million cubic yards of sand fill was placed as nourishment in September 2005 resulting in the pier becoming landlocked once again. Subsequently, Tropical Storm Tammy and Hurricane Wilma (October 2005), followed by Subtropical Storm Andrea (May 2007) and a series of strong northeasters (September and October, 2007), caused significant erosion of the beach fill returning the pier to its pre-beach restoration state.

Photo 3. St. Augustine Beach Fishing Pier substantially landlocked following the federal beach restoration project [BBCS Photo Files]

Photo 4. St. Augustine Beach Fishing Pier “fishable” again following the erosion of 2004 storms [BBCS Photo Files]

3.2 – St. Andrews State Park Pier Example

Another example of beach nourishment activity affecting a fishing pier has been seen at St. Andrews State Park near Panama City Beach. The park's gulf pier has been rendered too shallow to catch most species of fish due to recent nearshore placement of dredge material from the federal navigation channel at St. Andrews Inlet. Immediately following the nearshore disposal operation, beach visitors could wade around the seaward end of the pier. Over time, the nearshore may be expected to deepen at the end of the pier and therefore result in an improvement in the fishing conditions.

3.3 – Storm Induced Erosion

As mentioned previously, the beach and nearshore profile at a site selected for a new ocean or gulf pier is subject to fluctuations with respect to time. Along with considering the long-term trends of erosion or accretion as well as seasonal or other short term fluctuations in the profile, the anticipated erosion effects of major storms should be considered. Rule 62B-33.007 (4) (k), Florida Administrative Code, requires designing piers "to withstand at a minimum the <u>erosion</u>, <u>scour</u> … accompanying a twenty (20)-year storm event." Many pier designers, however, prefer to consider a worst case situation by projecting the erosion of a 100-year storm event. This level of erosion provides a safer design grade for calculating the pile penetration in the foundation analysis, particularly within the dynamic beach zone.

Different methodologies are available for determining the erosion of a major storm event. Among the beach and dune erosion models typically employed are the EDUNE model developed at the University of Florida (Kriebel and Dean, 1985; Kriebel, D.L., 1994), the High Frequency Dune Erosion Model developed cooperatively at the University of Florida and Florida State University (Dean, Chiu, and Wang, 1993), and the SBEACH model of the U.S. Army Corps of Engineers (Larson and Kraus, 1989).

3.4 – Juno Beach Pier Example

The designers of the Juno Beach fishing pier (**Photo 2**), constructed in northern Palm Beach County in 1998, utilized the EDUNE model to evaluate the vertical profile change associated with various return interval storm events (Buckingham and Olsen, 1991). Actually, seven different return interval storm events were analyzed between a 5-year storm and a 100-year storm. That analysis determined a closure depth of about -12 feet (NGVD) for a 100-year storm extending roughly 700 feet offshore for the pier that extended a total length of 993 feet.

3.5 – Mexico Beach Pier Example

The designers of the 2003 Mexico Beach Pier extension, located in northwest Florida east of Panama City, utilized the SBEACH model to evaluate the profile change for a 20-year storm (Dombrowski, 2003). The minimum design conditions employed for this 90-foot extension to a 550-foot long wooden pier built in 1956 was economically justified. Water depths are relatively shallow at this pier and the model predicted a design depth of only -6.5 feet (NGVD) at the terminal end of the pier. Another extension of the pier to 816 feet was completed in May 2010.

3.6 – Profile Comparison Method

Aside from numerical modeling the response of beach and nearshore profiles to storm events, a designer may evaluate the site specific conditions following an extreme storm event when it has occurred. Instead of predicting conditions with a simulated storm, a designer may utilize actual storm data when it is available. This methodology involves plotting historical profiles for the pier site including data obtained soon after the impact of major storms and following post-storm recovery. The migration of the nearshore bar and trough due to storms as well as due to post-storm recovery can be compared. This methodology allows for the consideration of a worse case "envelope" of data, that is, the highest and lowest eroded elevation along the length of the profile. While this method requires actual post-storm profiles, such data is readily available for the coast of northwest Florida following Hurricanes Opal (1995), Georges (1998), Ivan (2004), and Dennis (2005).

3.7 – Panama City Beach Pier Example

The method of post-storm profile comparisons has been employed in recent pier project designs for Panama City Beach and Navarre Beach. In the design of the new Russell-Fields Pier for the City of Panama City Beach, Olsen Associates, Inc., compared all available beach profile data between 1971 and 2006 (Olsen Associates, 2006). In comparing shoreline change, Olsen Associates (2006) concluded,

> *The most landward documented position of the MHWL over the 32-year record is represented by the January 1973, post-Hurricane Agnes profile. The most seaward position of the MHWL was measured immediately following the 1998/1999 Panama City Beach Nourishment Project (May 1999). Since construction of the 1998/1999 project, the most eroded condition occurred post-Hurricane Ivan (October 2004) with the beach berm (+7 ft) receding roughly 100 feet relative to the pre-storm position (June 2004).*

Olsen Associates (2006) outlined the minimum and maximum profile elevations to develop the profile change envelope for the 32-year period of record (**Figure 1**) and noted,

> *During Hurricane Ivan, the nearshore bar moved seaward approximately 400 feet and localized areas of the seabed experienced elevation changes of up to six feet. The net result of the bar migration resulted in a temporary decrease of roughly 5.5 feet in the still water depth (i.e. shallowing) at the proposed seaward end of the pier.*

It is recommended that the decreased depth resulting from a bar migration to deeper water not be considered for a design depth. As a practical matter, the physical process of bar migration would lag the storm tide hydrograph and peak breaking wave activity anyway, and therefore, the decreased depths due to bar migration would not be fully developed or near an equilibrium until after the structure has already been subjected to the storm's maximum surge and wave conditions.

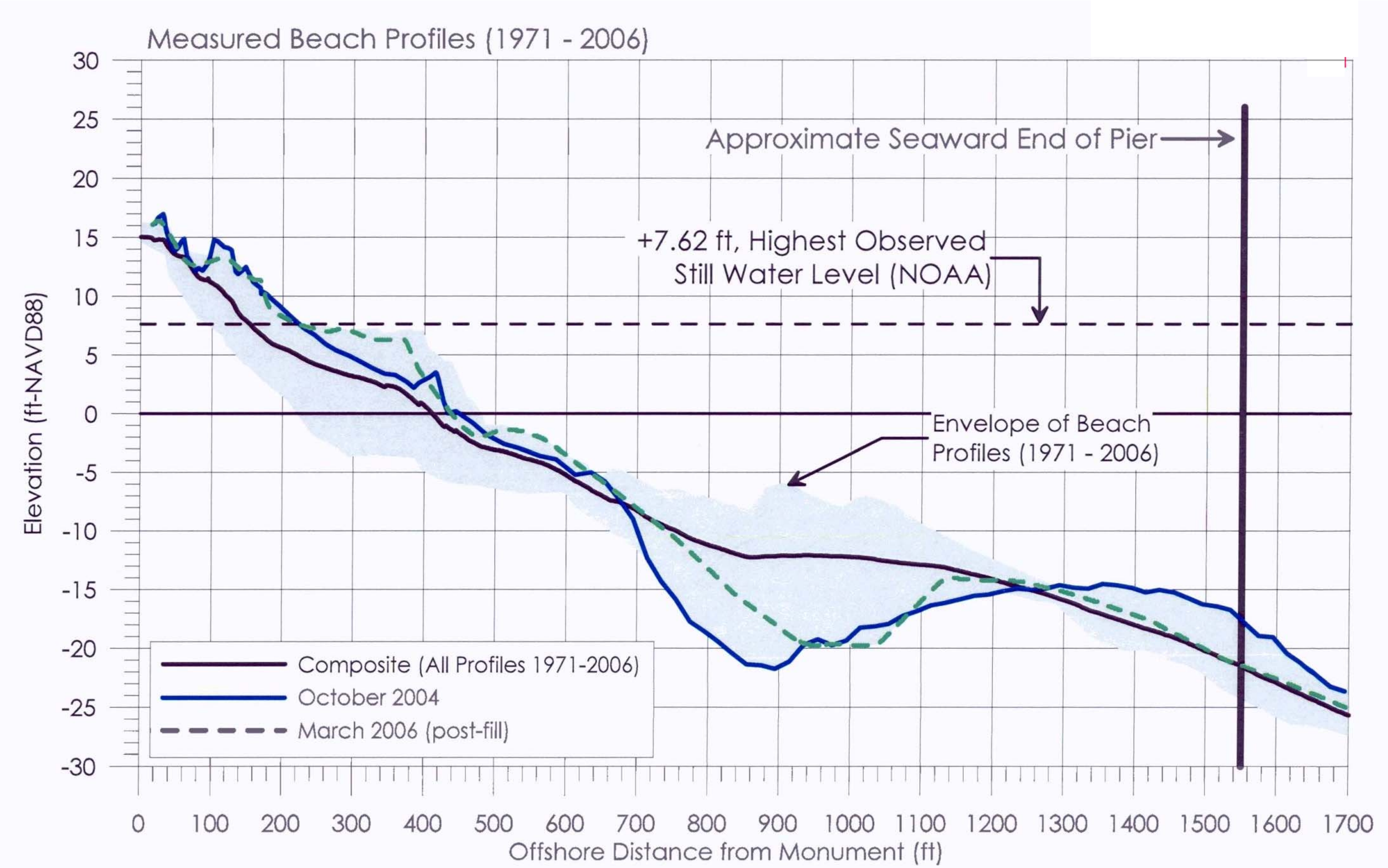

Figure 1. Design profile change envelope from historical record for Russell-Fields Pier from Olsen Associates (2006).

3.8 – Navarre Beach Pier Example

A beach and nearshore profile comparison method was also employed in the design of the new fishing pier for Navarre Beach by the design team of PBS&J (Conrad et al., 2007). Profiles from a period of record between 1996 and 2006 were compared, and maximum elevation differences were determined along the shore-normal length of the profile. A baseline profile of December 2006 was selected because it generally represented the worse case profile following Hurricanes Ivan (2004) and Dennis, Katrina, and Rita (2005) while still preceding the Navarre Beach Restoration Project, which substantially filled the eroded profile. Conrad et al. (2007) reports that:

> *Based on the maximum difference between historic beach profiles at the location, a scour value was determined and subtracted from the December 2006 bed elevation value. To be conservative, areas with less than five feet of observed scour were given a scour value of five feet.*

3.9 – Localized Scour for Vertical Piles

An important factor in designing a fishing pier's pile penetration is to determine the maximum expected localized scour around individual piles (**Photo 5**).

Photo 5. Typical localized pile scour [Photo from Chris Jones]

The most important factors resulting in scour around fishing pier piles are the wave orbital velocity, the bottom current, and the diameter of the pile. Other important factors are the grain size of the bottom sediments and the shape of the pile (e.g. round, square, or octagonal). Niedoroda and Dalton (1986) provide a detailed description of the physical processes of scour around a vertical pile. Localized scour at vertical piles for fishing piers may be calculated by several methods; however, for most cases of combined waves and currents, the “rule of thumb” recommended by the Coastal Engineering Manual (USACOE, 2008) is the maximum depth of scour at a vertical pile is equivalent to twice the diameter of the pile. This rule would be applicable to any shape pile commonly used in pier construction. For example, for either a two-foot square pile or for a two-foot diameter circular pile, the maximum localized scour would be expected to be at least four feet below the predicted storm eroded profile or the minimum historical profile elevation.

3.10 – Pile Group Scour Effects

Research at the Coastal Engineering Research Center (CERC) Pier at Duck, North Carolina, revealed that localized scour due to a group of piles observed at the seaward terminus of the pier can be caused by the interaction of waves and currents on the pier (Miller et al., 1983). This pile group scour effect would be in excess of that localized scour about individual piles. Miller et al. (1983) measured scour depths of between three and 10 feet, with an average of six feet below the average surrounding grade.

3.11 – M.B. Miller Pier, Bay County Example

In the design of a new concrete Bay County pier located in Panama City Beach to replace the 40-year old storm damaged wooden pier, Olsen Associates (2007) compared historical beach profiles for a period of record between 1971 and 2004. At the proposed seaward terminus of the pier located about 1625 feet seaward of the FDEP reference monument R57, a minimum historical elevation was determined to be -27.2 feet NAVD. For the proposed two-foot octagonal piles, an additional four feet of vertical scour was included to obtain a recommended design grade elevation of -31.2 feet NAVD for the seaward piles. Using the profile comparison method and the rule of thumb for vertical scour, a design grade was determined along the shore-normal axis of the pier. This design grade could then be used in the calculations for wave heights, wave loads, and pile embedment.

Chapter 4 – Wave Height Prediction

The principal causes of damage to ocean and gulf fishing piers are the effects of storm waves. A successful pier design requires both an understanding of the wave climate in the region and a projection of an extreme storm wave event that may reasonably be expected to occur at the pier site. The cover photo shows the extreme wave conditions that affected the Atlantic coast of Florida during the Halloween northeaster in 1991. During this extreme wave event the Lake Worth Pier in Palm Beach County, Florida, sustained major damage and lost 200 feet off its seaward end.

4.1 – Wave Climate Data

The U.S. Army Corps of Engineers, Coastal & Hydraulics Laboratory (CHL) has developed a program known as the Wave Information Studies (WIS). Early wave data reports are provided by Jensen (1983) for the Atlantic coast and by Hubertz and Brooks (1989) for the Gulf of Mexico. WIS provides more recent wave climate information around the United States coastline including the Atlantic Ocean and Gulf of Mexico coasts of Florida. The wave data is developed by a process known as hindcasting where numerical modeling of known wave and wind data simulates wave climate data at numerous stations along the coast. The CHL generates WIS hindcast data using the WISWAVE model and publishes the data on its web site – http://chl.erdc.usace.mil/.

Another major source of wave data is the National Oceanic and Atmospheric Administration's National Data Buoy Center (NDBC). Off Florida, the NDBC maintains the moored buoys listed in **Table 2**.

Table 2. National Data Buoy Center Stations Offshore Florida

Station	Location	Lat.(N)-Long.(W)	Depth (m)
41012	40 nm ENE of St. Augustine	30.04N – 80.55W	38.4
41009	20 nm East of Cape Canaveral	28.50N – 80.17W	41.5
41010	120 nm East of Cape Canaveral	28.95N – 78.47W	872.6
42080	offshore Sand Key (Key West)	24.39N – 81.95W	142.6
42003*	262 nm South of Panama City	25.74N – 85.73W	3233
42036	106 nm WNW of Tampa	28.50N – 84.52W	54.5
42039	115 nm ESE of Pensacola	28.79N – 86.02W	291.4
42040	64 nm South of Dauphin Is., AL	29.18N – 88.21W	443.6

* - NDBC reports the station went adrift twice during 2007. See NDBC web site for latest location.

The NDBC archives the buoy data at their web site – http://seaboard.ndbc.noaa.gov/. Particularly valuable for pier design, recent extreme wave events were measured by NDBC buoys during Hurricanes Ivan (2004), Dennis (2005), and Wilma (2005). **Figure 2** provides the significant wave heights for Station 42039 during Hurricane Dennis reaching 34.8 feet. **Figures 3 and 4** graph significant wave heights respectively for Stations 42039 and 42040 during Hurricane Ivan that reached 40 and 50 feet. **Figure 5** graphs significant wave heights off St. Augustine for Station 41010 during Hurricane Wilma (2005).

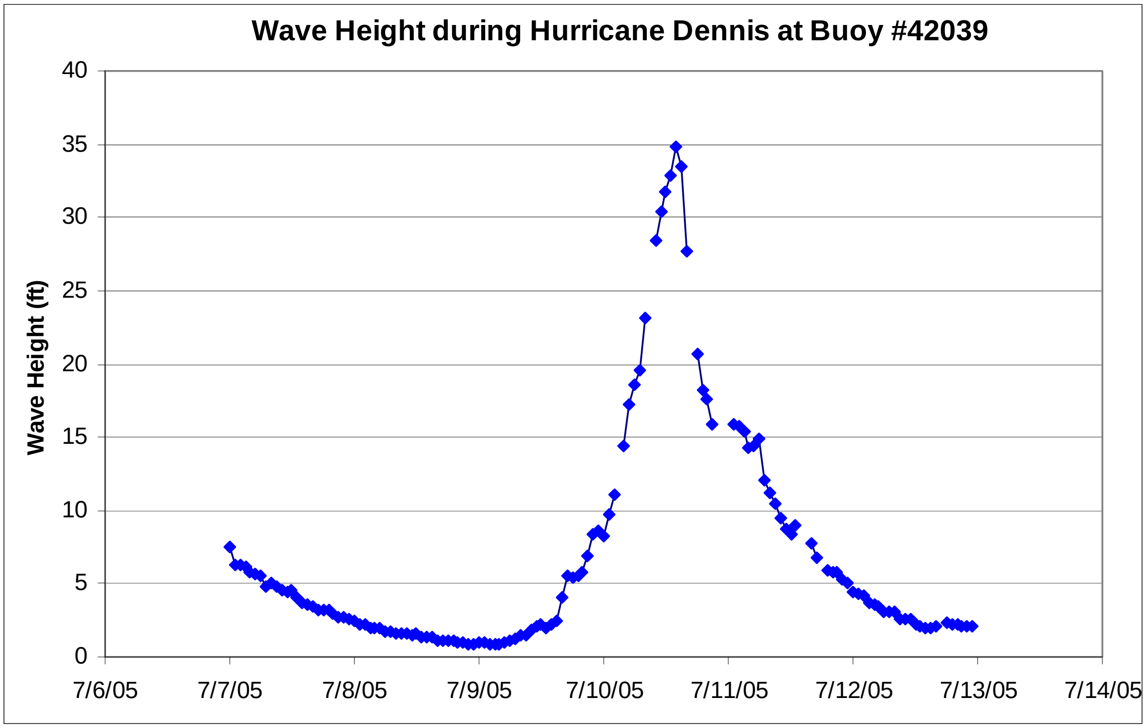

Figure 2. Hurricane Dennis wave heights from the National Data Buoy Center, NOAA

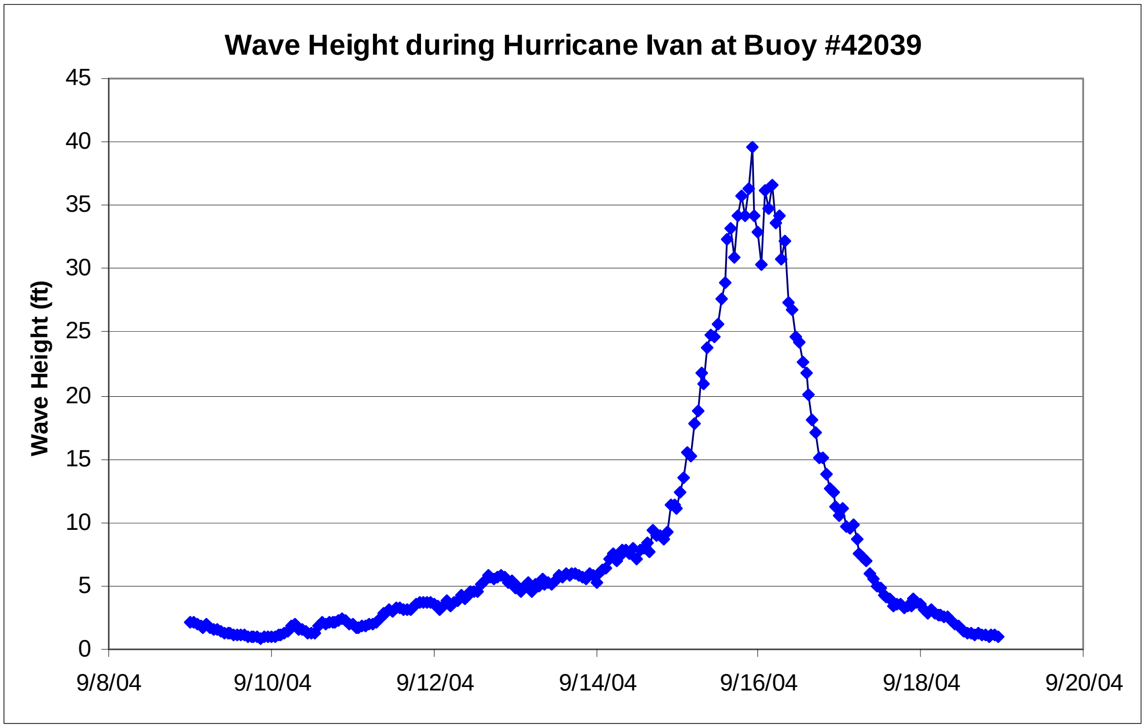

Figure 3. Hurricane Ivan wave heights from the National Data Buoy Center, NOAA

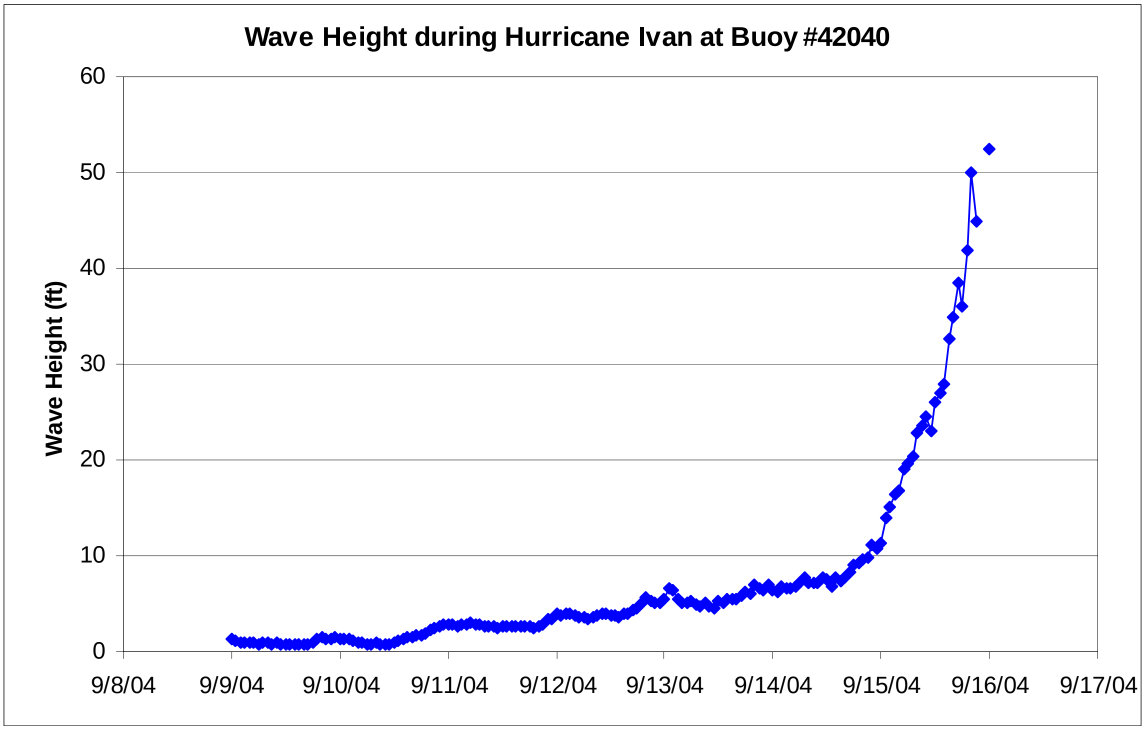

Figure 4. Hurricane Ivan wave heights from the National Data Buoy Center, NOAA

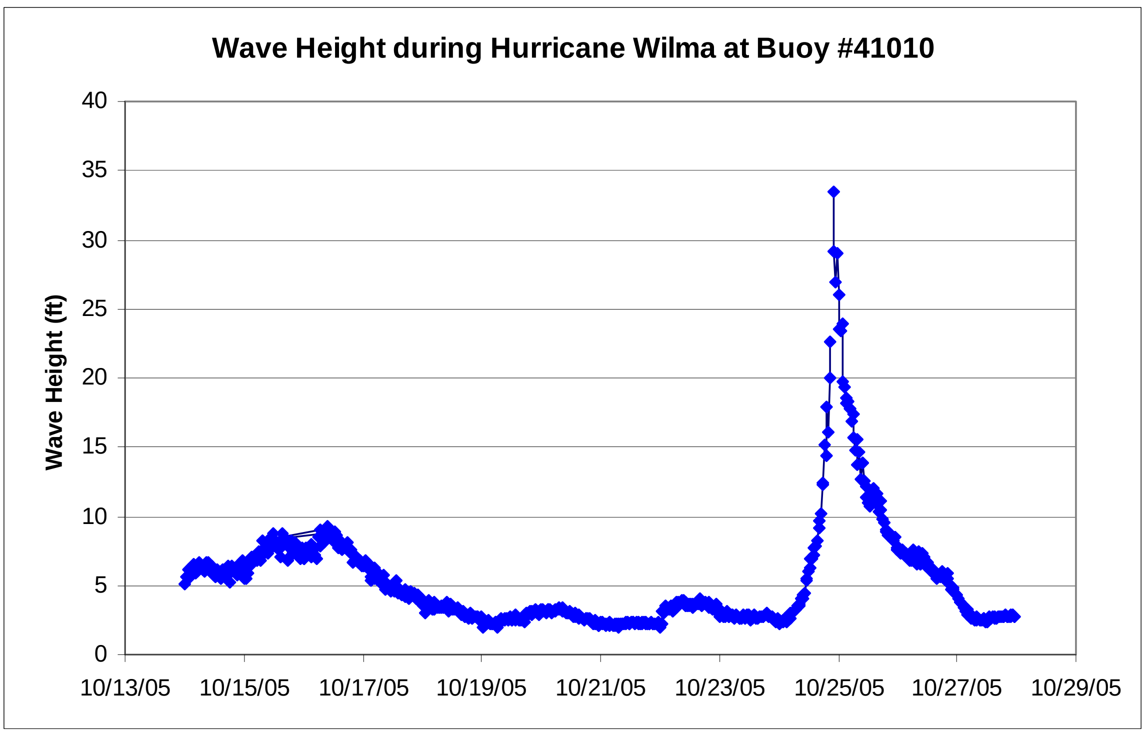

Figure 5. Hurricane Wilma wave heights from the National Data Buoy Center, NOAA

4.2 – Depth Limited Wave Heights

Shore-propagating deep water waves undergo a transformation in wave height, period, and steepness, as they enter shallow water. Shore-breaking has been extensively described by Galvin (1968) and others. A plunging type breaking wave reaches its shore-breaking point when the front face of the wave becomes vertical (Figure 6).

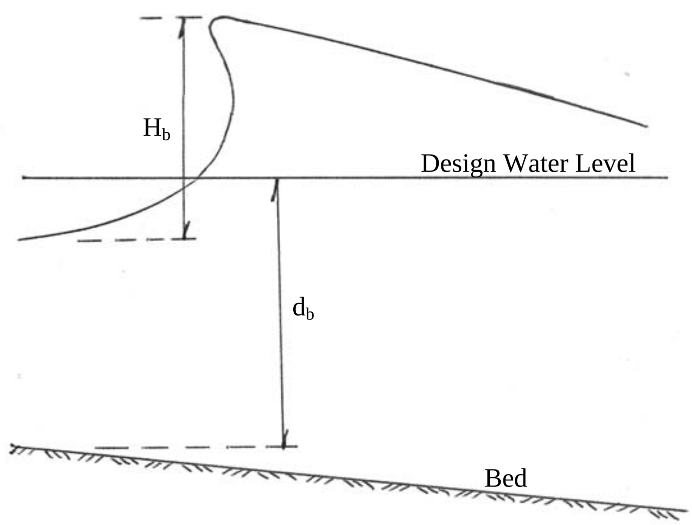

Figure 6. Plunging type breaking wave at shore-breaking point

The height of a shore-breaking wave is related to the water depth by Equation 1 (McCowan, 1894; Munk, 1949; Weggel, 1972; Balsillie, 1983):

$$\mathbf{d_b \;/\; H_b = 1.28} \qquad (1)$$

where H_b is the shore-breaking wave height and d_b is the shore-breaking water depth. Depending upon whether a wave is non-breaking, breaking, or broken, the amount of the wave crest lying above the still water level will range between 0.5 and 0.84, with a maximum value of 0.84 at the point of shore-breaking.

Given the depth limitation on breaking wave heights, the extreme hurricane generated wave heights seen in **Figures 2-5** during recent events will break in water depths greater than the depths to which ocean and gulf fishing piers will extend. Olsen Associates (2006; 2007) computed wave conditions at the end of the new Russell-Fields Pier and M.B. Miller Pier at Panama City Beach. They computed wave transformations across the nearshore region in the vicinity of both piers using a spectral wave breaking model with a cross-shore sand transport model (SBEACH). Even though offshore significant wave heights were varied between 14 and 40 feet, the breaking depth at the ends of the piers limited the maximum computed breaking wave height, $\mathbf{H_b}$, to that which the breaking depth would allow ($\mathbf{H_b = 0.78\, d_b}$). For the seaward ends of the Russell-Fields Pier and M.B. Miller Pier, Olsen Associates (2006; 2007) computed maximum expected wave heights, respectively, of 21.5 and 21.3 feet.

Conrad et al. (2007) computed wave propagation from 100 miles offshore to the proposed Navarre Beach Pier location using the STWAVE model. For a case involving an 8-meter wave height and a 12-second wave period with no wind or storm surge effects, the STWAVE model showed significant wave attenuation resulting in a 4.5-meter wave height near the shore. For another case involving an 8-meter wave height, a 12-second wave period, an 80-mph on-shore wind, and a 2-meter storm surge, the STWAVE model showed an increase in wave height near shore due to the transfer of energy from the wind to the water surface. This exercise simply provided further validation of using the extreme offshore wave heights in an analysis.

Notwithstanding the 35 to 55-foot significant wave heights measured during major hurricanes by NDBC buoys off the northwest coast of Florida during the past decade, the design breaking wave projected at the end of a fishing pier will ultimately be limited in height by the water depth at the pier's seaward end. For that reason, it is completely acceptable for a designer to employ Equation (1) and determine the design wave based upon its depth limited height. The designer may likewise use $\mathbf{0.84\ H_b}$ as the amount of wave crest above still water in anticipation of calculating breaking wave loads as discussed in the next chapter.

Chapter 5 – Wave Load Analysis

As previously mentioned, the principal causes of damage to ocean and gulf fishing piers are the effects of storm waves, which specifically inflict extremely high forces on the structural members of the pier. With the projection of an extreme storm wave event that may reasonably be expected to occur at the pier site, wave forces may be calculated. The wave forces act on a pier's structural members on both a horizontal and vertical plane; therefore, it is necessary to conduct separate computations for both the lateral wave forces as well as the vertical uplift forces.

The following discussion is intended to be an introduction in the methodologies for predicting design wave forces. The reader is referred to more detailed analyses discussed throughout the literature for the design of pile-supported offshore structures.

5.1 – Horizontal Wave Forces

Morrison et al (1950) developed Equation (2) to describe the horizontal force per unit length of a vertical pile:

$$\mathbf{F_h = F_i + F_d = C_m \rho \frac{\pi D^2}{4} \frac{du}{dt} + C_d \frac{1}{2} \rho D u |u|} \qquad (2)$$

where, $\mathbf{F_i}$ = inertial force per unit length of pile

$\mathbf{F_d}$ = drag force per unit length of pile

$\boldsymbol{\rho}$ = density of sea water (64 lbs/ft.^{3})

$\mathbf{D}$ = diameter of pile (ft.)

$\mathbf{u}$ = horizontal water particle velocity at the axis of the pile

$\mathbf{\frac{du}{dt}}$ = total horizontal water particle acceleration at the axis of the pile

$\mathbf{C_d}$ = hydrodynamic force coefficient (the drag coefficient)

and $\mathbf{C_m}$ = hydrodynamic force coefficient (the inertia or mass coefficient)

The Coastal Engineering Manual (CEM) notes that for Equation (2) to be valid, the pile diameter, **D**, must be small with respect to the wave length, **L**. The following restriction is established:

$$\mathbf{D / L < 0.05} \qquad (3)$$

For typical fishing pier designs in Florida, Equation (3) will generally hold true. The CEM (2008) also advises that it is necessary to choose an appropriate wave theory in order to estimate the water particle velocity, **u** , and acceleration, **du/dt**, from the values of wave height, **H**, wave period, **T**, and water depth, **d**. In the design of fishing piers projecting from a shoreline, as opposed to designing offshore platforms, the waves are shoaling in progressively shallower water and are nonlinear in their form with steep crests and shallow troughs. For this reason, it is more appropriate to utilize a nonlinear wave theory as opposed to a linear wave theory.

Wave inertial forces and drag forces on vertical piles may be determined by Stoke's fifth order wave theory (Skjelbriea et al., 1960) and the stream-function wave theory (Dean, 1974). In addition to drag and inertial forces that act in the same direction as the wave's direction of travel, there are transverse forces due to vortex or eddy shedding in the lee of the pile that result in a lateral oscillatory force. These transverse forces act perpendicular or normal to the direction of wave approach. The CEM (2008) recommends a maximum transverse force equal to the drag force for rigid pier structures.

Wave force relationships are described by dimensionless wave force coefficients, $\mathbf{C_d}$ and $\mathbf{C_m}$. These coefficients have been evaluated by different researchers conducting experiments measuring forces. For non-breaking waves on square piles subject to high Reynold's number flow ($R_e > 1 \times 10^4$), researchers have concluded that $\mathbf{C_d}$ should equal 2.0 (Keulegan and Carpenter, 1956; Sarpkaya, 1976). For non-breaking waves on cylindrical piles $\mathbf{C_d}$ should equal 0.8. It is noteworthy that cylindrical piles have less resistance to flow than square or other shaped piles, and therefore the drag forces are lower for comparably dimensioned cylindrical piles. Sarpkaya (1976) also showed that the coefficient of inertia, $\mathbf{C_m}$, should equal 1.5 for non-breaking waves on cylindrical piles in high Reynold's number flow. McConnell et al. (2004) recommends a $\mathbf{C_m}$ equal to 2.5 for non-breaking waves on square piles. Thus, inertial forces as well as drag forces will be less for cylindrical piles than square or other shaped piles. Square concrete piles may be formed with rounded or smoothed off corners to bring the drag coefficient down to values consistent with cylindrical piles (*British Standard Code 6349*, 2000).

For breaking waves on cylindrical and square piles, the wave forces have been shown to be substantially higher than for non-breaking wave conditions (Weggel, 1968). The CEM (2008) refers to these breaking wave forces as "slamming" forces. USACE (1973) recommends increasing the drag coefficient by a factor of 2.5 to account for breaking waves in shallow water. Also, for breaking waves in shallow water, the inertia force component is very small relative to the drag force component. Therefore, the "slamming" forces of shallow water breaking waves, $\mathbf{F_{max}}$, on a unit length of vertical pile may be determined by Equation (4) as follows,

$$\mathbf{F_{max} = 2.5\, C_d \frac{1}{2} \rho\, g\, D\, H_b^2} \qquad (4)$$

where, $\mathbf{H_b}$ = breaking wave height (ft.)

and $\mathbf{g}$ = acceleration of gravity (32.2 ft./sec.^{2})

Small-scale experiments by Hall (1958) in high Reynold's number flow ($R_e \approx 5 \times 10^4$) determined that the maximum moment, $\mathbf{M_{max}}$, about the mudline (or sand/water interface) was approximately equal to the maximum wave force, $\mathbf{F_{max}}$, times the breaking wave height, $\mathbf{H_b}$, or

$$\mathbf{M_{max} \approx F_{max}\ H_b} \tag{5}$$

Large-scale experiments by Ross (1959) determined a maximum breaking wave force per unit length of pile near the breaker crest to be equivalent to

$$\mathbf{F_{max} \approx 0.88\ \rho\ g\ D\ H_b} \tag{6}$$

Horizontal wave forces also need to be calculated for non-vertical piles and other structural members than the foundation piles, such as pier beams, pile caps, and bents. It is recommended that maximum breaking wave forces be applied (i.e. slamming forces) to the non-pile members as well as the foundation piles for fishing piers.

5.2 – Safety Factors in the Design of Foundation Piles

Before applying calculated wave forces in the design of foundation piles, a factor of safety is generally applied. There are valid reasons to justify adding a factor of safety, which include potential exceeding of the design wave, uncertainties in the selected wave theory or data, and approximations of wave force coefficients.

The CEM (2008) recommends a factor of safety of 2.0 be applied to forces and moments when the design wave may occur frequently. Because of the depth limited wave height typically employed in the design of fishing piers and the potential for being affected by numerous storm events of varying magnitudes and durations, it is recommended that a factor of safety of 2.0 be used in the design of ocean and gulf fishing piers in Florida.

5.3 – Vertical Wave Uplift Forces

Foundation pile design in the surf zone is dependent upon calculating the vertical loads acting in alignment with the pile axis. Vertical loads on all horizontal members need to be calculated to determine the total uplift that needs to be resisted by the piles (i.e., the breakout resistance).

For piers constructed with fixed or rigid decks such as the second Navarre Beach Fishing Pier built in 1974 (**Photo 6**), the underside of the deck will be subject to wave uplift forces from breaking and non-breaking waves that have heights exceeding the deck elevation. The Shore Protection Manual (1973) states, "For design computations, uplift forces should be considered as full hydrostatic force for walls whose bases are below design water level." For piers constructed with wood decks fastened to the pier's structural members, wave uplift damage typically occurs when the vertical forces of waves exceed the resistance of the fasteners connecting the decking to the stringers or the stringers to the pile caps. **Photos 7** and **8** provide examples of wave uplift damage to wood decking that is directly connected to a pier's superstructure.

Photo 6. Solid concrete deck, Navarre Pier (1974-2004)

Photo 7. Pier decking damage, Lake Worth Pier (1972-2004) [BBCS Photo Files]

Photo 8. Pier decking damage, Lake Worth Pier (1972-2004) [BBCS Photo Files]

The vertical uplift force per unit width of pier deck due to the passage of a wave crest that exceeds the level of the pier deck (see **Figure 7**) may be determined by the relationship as follows,

$$\mathbf{F_v = \rho\, h} \tag{7}$$

where, $\boldsymbol{\rho}$ = density of sea water (64 lbs/ft.^{3})

and $\mathbf{h}$ = height of the wave crest above the underside of the pier deck

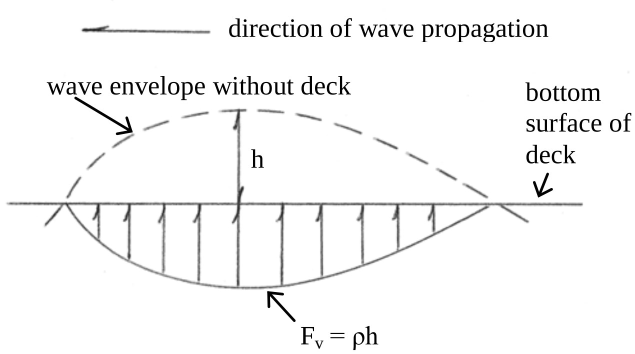

Figure 7. Vertical uplift forces on pier decks

When wave crests impact horizontal structural members such as pile caps and beams, a significant vertical uplift force may result from the upward deflection of the wave. **Photo 9** shows selective deck damage due to the vertical deflection of wave crests impacting each bent on the old Lake Worth Pier following a northeaster in 1989.

Photo 9. Selective deck damage at each bent, Lake Worth Pier [BBCS Photo Files]

Recently, there have been some breakaway deck sections dislodged from a couple new fishing piers during tropical storm conditions that were substantially below the design storm tide elevation and wave conditions. These dislodged deck sections were located above and immediately seaward of the pile caps of those piers. It is believed that the best strategy to account for this upward wave reflection effect is to include breakaway deck sections in lieu of raising the pier deck any higher than the normal design would require. In doing so, the problem only becomes a periodic nuisance to reset the dislodged deck sections while maintaining the integrity of the structure.

5.4 – Wave Peaking Damage and Wave Attenuation

While most storm wave damage occurs at the terminal ends of fishing piers due to the greater water depth, and corresponding higher depth-limited wave heights, there are often damaged mid-sections of piers. This seemingly anomalous intermediate pier damage often results in the complete truncation of pier sections (see **Photo 10**).

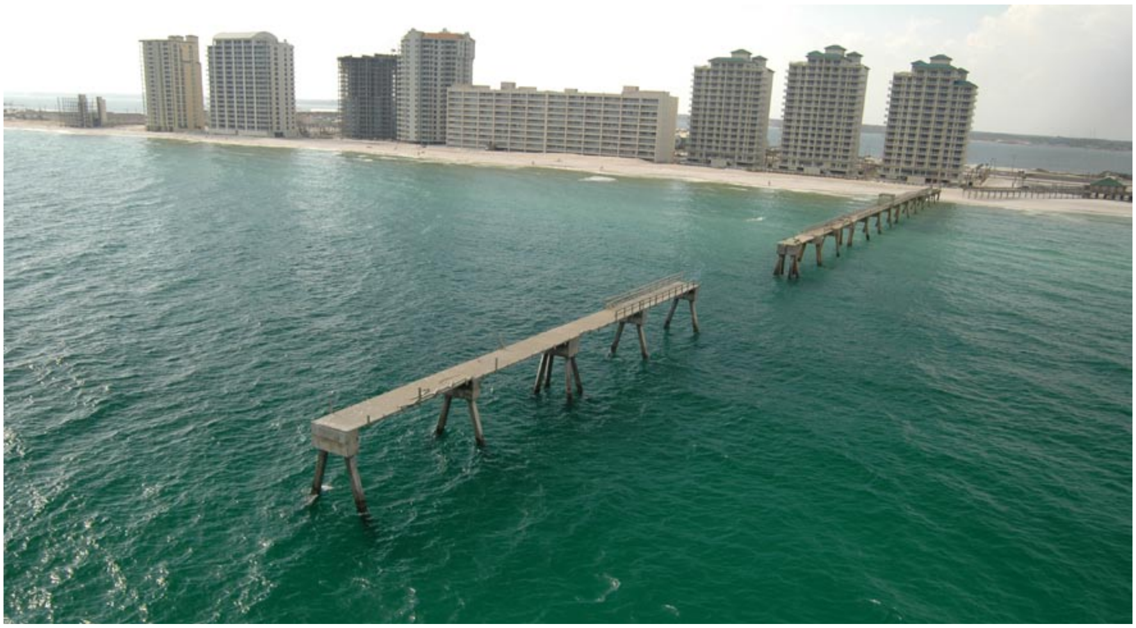

Photo 10. Hurricane Dennis destroyed three sections of the solid concrete Navarre Beach Fishing Pier at the seaward end, a mid-section, and at the shoreline [BBCS Photo Files]

The intermediate pier section damage typically occurs due to the wave peaking process during wave breaking caused by the existence of a shore-parallel sand bar. The location of deck failure is controlled by the location of the sand bar. Generally, the incipient storm waves will break and transform to smaller waves as they propagate shoreward. This wave peaking and attenuation process is well documented and has direct application to the prediction of surf zone wave conditions in the design of fishing piers (Balsillie, 1994).

Balsillie (1985) verified the Multiple Shore Breaking Wave Transformation (MSBWT) numerical model by comparing wave height attenuation results to field data for three storm damaged fishing piers (**Figure 8**). The St. Augustine Beach Fishing Pier and the Flagler Beach Fishing Pier lost 600 and 432 feet, respectively, from their terminal ends during the Thanksgiving Northeaster of 1984, which had a storm tide level of about +6 feet NGVD (see *Fishing Pier Design Guidance, Part 1*). The San Clemente Pier in California was likewise damaged by a 1983 El Niño storm that had a storm tide level of about +8 feet NGVD (Walker et al., 1984).

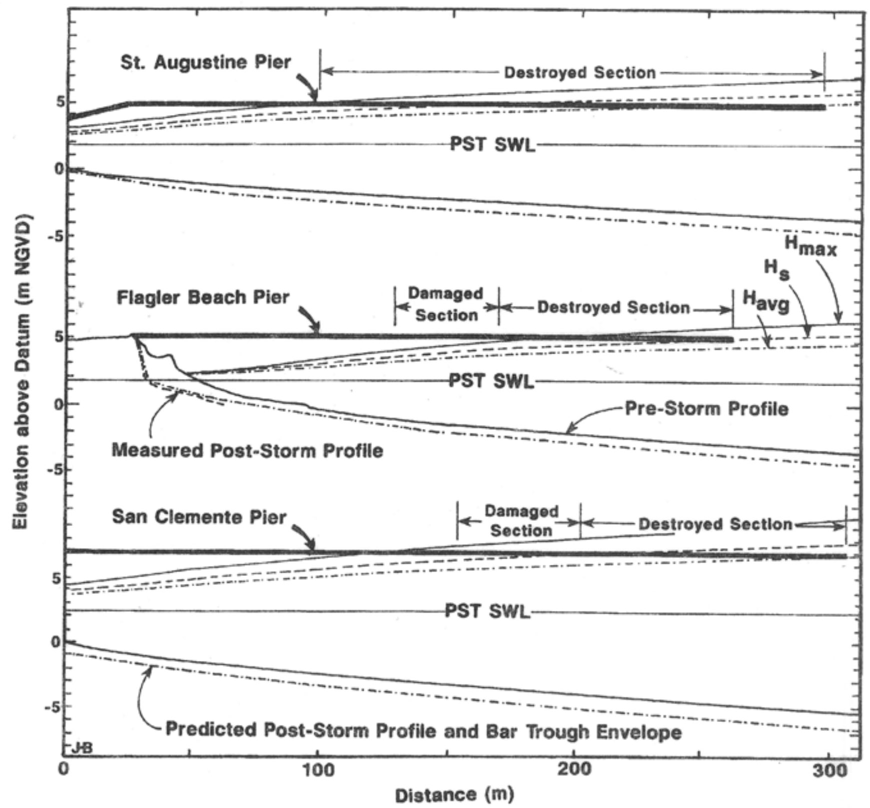

Figure 8. Illustration of the correlation between actual storm impact damage and littoral wave activity from Balsillie (1985)

In **Figure 8**, the breaker height envelopes represent the crest elevation of the waves propagating on the peak storm tide. $\mathbf{H_{ave}}$ is the average breaker height, $\mathbf{H_s}$ is the significant breaker height represented by the average of the highest one-third of the waves, and $\mathbf{H_{max}}$ is the maximum breaker height. Only the bar trough envelope is shown, which provides the minimum profile elevations below the pre-storm profile. It is noteworthy that the destroyed sections of the two Florida fishing piers correlate directly with the maximum breaking wave height envelope.

Chapter 6 – Foundation Design Analysis

Fishing pier design depends upon an engineering analysis of a deep pile foundation. Piles are structural members of timber, concrete, or steel used to transmit surface loads to lower levels within a soil mass. For ocean and gulf fishing piers in Florida, this load transfer is typically by a combination of friction and end bearing. Friction is generated along the surface of the pile and end bearing is determined by the sediment stratum at the pile's terminal tip. Section 1808.2.10 of the *Florida Building Code* requires –

> *The design and installation of the pier or pile foundation shall be under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pier or pile foundations who shall certify to the building official that the piers or piles as installed satisfy the design criteria.*

Pile driving is employed in pile-supported structures to increase the density of the sediment. Piles are driven by a succession of blows either by a drop hammer or by a diesel, steam, or compressed-air-powered hammer. Diesel powered hammers and diesel vibratory hammers are most common. With vibratory hammers, a variable-speed oscillator is attached to the top of the pile, consisting of two counter-rotating eccentric weights that are in phase twice per cycle in the vertical direction. This introduces a pulsation or vibration through the pile that can be made to coincide with the resonance frequency of the pile, which creates a push-pull effect at the pile tip to disturb the soil structure, and thus improves the rate of pile driving.

In some cases, pile driving alone will not achieve the design embedment of a pier's piles. In such cases, a supplemental method may be chosen to install piles to their design embedment depth. Perhaps the most common supplemental procedure employed in Florida pier construction has been driving piles with the aid of water jets, which displace the soil at the pile tip by using a stream of water under high pressure. It should be noted that the jetting process adversely affects the properties of the supporting soils to an unpredictable degree. It is usually not possible to predict with reasonable accuracy the actual capacity of a pile installed with the aid of jetting. Unfortunately, a pile embedded with the assistance of jetting may have less capacity than a pile driven to refusal at a higher elevation without jetting. Professional judgment of an experienced marine contractor in the installation of piles plays an important role in determining the appropriate circumstances for employing supplemental jetting.

Traditionally, in the pile foundation design for fishing piers the prediction of pile capacity has been determined by computations using the static method and by load tests. Most geotechnical engineers would agree that a field load test is the only "fool-proof" method that can be used to determine bearing capacity at a particular site. Following completion of a structure's design as determined by computations that utilize core borings with laboratory or in-situ tests, one or more load tests are often conducted at the commencement of the construction project.

6.1 – Prediction of Pile Bearing Capacity – Static Formula Method

As represented in **Figure 9**, the static method of predicting the ultimate capacity, $\mathbf{Q_u}$, of a pile driven to a specific depth of penetration, **d**, relies on empirical data derived from model studies and full-scale load tests. Ultimate capacity may be represented by the static bearing capacity Equation (8) (McClelland et al, 1967; Coyle and Sulaiman, 1970) as follows,

$$\mathbf{Q_u = Q_p + Q_s = q\ A_p + f\ A_s} \qquad (8)$$

where, $\mathbf{Q_p}$ = pile tip bearing

$\mathbf{q}$ = unit bearing capacity

$\mathbf{A_p}$ = pile end area

$\mathbf{Q_s}$ = frictional resistance

$\mathbf{f}$ = unit friction or soil-pile adhesion

and $\mathbf{A_s}$ = embedded pile surface area

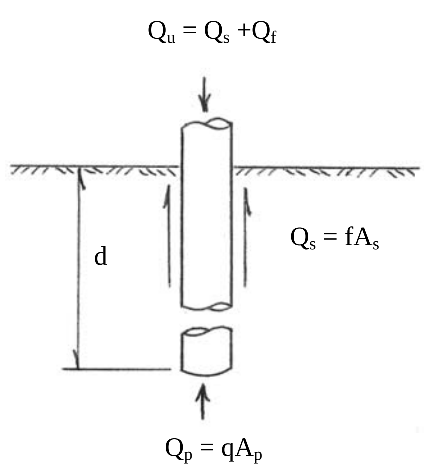

Figure 9. Ultimate capacity of a pile driven to a specific depth of penetration

Successful application of this analysis depends on the selection of values for **q** and **f**. Consideration must be given to the combined factors of pile dimensions, pile type, soil conditions, pile installation and loading. It is common practice for computing the capacity of piles in sand to express the unit skin friction, **f**, as a coefficient **k** times the overburden pressure, $\mathbf{p_o}$, and the tangent of the angle of skin friction, $\boldsymbol{\delta}$, as in Equation (9).

$$\mathbf{f = k\ p_o \tan \delta} \qquad (9)$$

The overburden pressure, $\mathbf{p_o}$, is equal to the unit weight of sand, $\boldsymbol{\gamma}$, times the depth of embedment. That is,

$$\mathbf{p_o = \gamma\, d} \tag{10}$$

The unit bearing capacity, **q**, is expressed as a bearing capacity factor, $\mathbf{N'_q}$, times the overburden pressure, $\mathbf{p_o}$ (Terzaghi and Peck, 1967). That is,

$$\mathbf{q = p_o\, N'_q} \tag{11}$$

The bearing capacity factor, $\mathbf{N'_q}$, for a deep circular bearing area is typically considered to be a function of the angle of internal friction, $\boldsymbol{\varphi}$. It has been suggested that the total capacity of a pile in sand has a near-linear increase with depth of pile penetration, because both **q** and **f** have been shown to be directly proportional to the overburden pressure, $\mathbf{p_o}$. However, pile tests by Kerisel (1961) and Vesíc (1965) reveal **q** and **f** reach maximum limiting values and then remain essentially constant with increasing depths.

In Equation (9), the coefficient of earth pressure, **k**, is the most difficult factor to determine. The overburden pressure, $\mathbf{p_o}$, is multiplied times the coefficient of earth pressure to obtain the intensity of earth pressure pushing against the side of the pile. This intensity, and therefore the magnitude of **k**, is influenced by the initial state of stress in the sand deposit, the initial density of the sand, the displacement volume of the driven pile, the pile shape, installation methods other than pile driving, and the load direction (upward tension or downward compression). When a pile is driven into sand and earth pressure intensity increases, **k** will typically range from 0.7 to over 3.0. When piles are jetted into sand, **k** may typically range from 0.1 to 0.4. Thus, not only the condition of the sand deposit, but the type of pile and method of installation need be considered.

For rule of thumb design guidance when driving piles in medium-dense sand, the following typical but conservative values are suggested (Terzaghi, 1943) –

Angle of skin friction, $\boldsymbol{\delta} = 30^{o}$
Angle of internal friction, $\boldsymbol{\varphi} = 35^{o}$
$\mathbf{k} = 0.7$, $\mathbf{f_{max}} = 2$ kips/ft.^{2}
$\mathbf{N'_q} = 41$, $\mathbf{q_{max}} = 200$ kips/ft.^{2}

These limiting values for unit friction and end bearing in medium-dense sand will typically be reached at about 80 ft. of depth. For other non-cohesive materials, including shelly material, lower friction angles and smaller limiting values of friction and end bearing may be considered.

6.2 – Prediction of Pile Bearing Capacity – Dynamic Formula Method

In some areas, pier designers have pile driving records from numerous projects and may elect to use a dynamic formula for the initial foundation design. There are at least a dozen different dynamic formulas with widely varying recommended values of safety factors. It is beyond the scope of this document to present and compare the different dynamic formulas employed to predict pile bearing capacity. The dynamic formula method relates the resistance to penetration during pile driving to the static bearing capacity (Coyle and Sulaiman, 1970). As with the static formula method previously discussed, the dynamic formula method may be considered conservative and the designer may favor a field load test for final design. An inherent problem with the use of the dynamic formula method is the difficulty in determining the energy lost during pile driving and the difficulty in relating resistance during pile driving to the static capacity of the pile. For piles driven by vibratory hammers, the dynamic equations will not apply, and static equations must be used to estimate load capacity.

6.3 – Wave Equation Analysis

An advanced dynamic formula method has been developed, termed the wave equation (Smith, 1962). It is beyond the scope of this document to present this methodology, but some discussion is worthy given its wide acceptance since the advent of high speed computers. As the name implies, this equation describes the movement of an impulse or wave down a pile as it is being driven. The solution of the wave equation consists of reducing the pile, the driving system, and the soil mass to a set of weights, springs, and side and point resistances. The impacts from the pile driving hammer travel through short pile segments and return in a small time interval that depends upon the type of material and the length of the chosen pile segments. For concrete piles, Smith (1962) recommends pile segment lengths of 8-10 feet and time intervals of 1/3,000 second. The wave equation analysis can be used to estimate the bearing capacity of a pile that would be obtained if a pile were field tested immediately after driving. In Florida's coastal sand environment, this estimate should be consistent with the actual ultimate bearing capacity of a pile.

6.4 – Point Bearing Piles

When the tip of a pile is resting on a solid rock stratum, the pile is assumed to derive its load capacity from point bearing. While not common in the construction of fishing piers on the barrier beach coast of Florida fronting the Atlantic Ocean or Gulf of Mexico, this situation does occasionally occur, particularly with a few piles that may be driven to a submerged geologic reef formation. This condition is most common fronting on the Straits of Florida, particularly in the Florida Keys. The bearing resistance of rock is typically obtained from triaxial testing of rock cores. For the case of point bearing on rock, the ultimate pile capacity, $\mathbf{Q_u}$, may be computed by Equation (12).

$$\mathbf{Q_u = q_r\, A_p} \tag{12}$$

where, $\mathbf{q_r}$ = ultimate bearing resistance of rock
and $\mathbf{A_p}$ = pile end area

6.5 – Load-Bearing Capacity Requirements

The *Florida Building Code* sets criteria for load-bearing capacity in Section 1808.2.8.6, which states –

> *Piers, individual piles, and groups of piles shall develop ultimate load capacities of at least twice the design working loads in the designated*

load-bearing layers. Analysis shall show that no soil layer underlying the designated load-bearing layers causes the load-bearing capacity safety factor to be less than two.

6.6 – Pile Load Tests

The most acceptable method to determine the load bearing capacity of a pile is to test-load it. This procedure involves driving a pile and applying a compressive load in increments while recording the pile movements. Although the compression test is most common, load testing may also determine lateral load capacity or resistance to uplift loading from tensile loads. Uplift loading is particularly important in the construction of fishing piers in order to determine the pile's breakout resistance when subjected to the uplift forces from the design wave conditions. The data obtained from compression load tests are plotted as load vs. deformation. The ultimate pile load is taken as that value corresponding to the deflection curve becoming nearly vertical. The allowable pile load is determined to be a percentage of the ultimate load or as the load that causes a specified amount of deflection.

The test pile should be located near a core boring obtained to determine the soil conditions at the site. Most often in pier construction as well as other coastal construction of major structures, the test pile is one of the piles being used in the project. ASTM Standards provide the procedures for conducting a pile load test. An important factor to consider when conducting a load test for a fishing pier is recognition that the load test is only yielding data for in-situ conditions and not the design conditions with erosion and scour having occurred, as discussed in Chapter 3. Depending upon the depth of vertical profile scour that would occur during the design storm event plus the additional local scour caused by the structural elements, a significant reduction will be seen in the embedded pile surface area and the overburden pressure. This will result in lower frictional resistance and therefore lower ultimate bearing capacity during design erosion conditions than will be observed during load tests.

6.7 – Load Tests Requirements

The *Florida Building Code* sets load test criteria in Section 1808.2.8.3, as follows –

Where design compressive loads per pier or pile are greater than those permitted by Section 1808.2.10 or where the design load for any pier or pile foundation is in doubt, control test piers or piles shall be tested in accordance with ASTM D 1143 or ASTM D 4945. At least one pier or pile shall be test loaded in each area of uniform subsoil conditions. Where required by the building official, additional piers or piles shall be load tested where necessary to establish the safe design capacity. The resulting allowable loads shall not be more than one-half of the ultimate axial load capacity of the test pier or pile as assessed by one of the published methods listed in section 1808.2.8.3.1 with consideration for the test type, duration, and subsoil. The ultimate axial load capacity shall be determined by a registered design professional with consideration given to tolerable total and differential settlements at design load in accordance with Section 1808.2.12. In subsequent installation of the balance of *foundation piles, all piles shall be deemed to have a supporting capacity equal to the control pile where such piles are of the same type, size, and relative length as the test pile; are installed using the same or comparable methods and equipment as the test pile; are installed in similar subsoil conditions as the test pile; and, for driven piles, where the rate of penetration (e.g., net displacement per blow) of such piles is equal to or less than that of the test pile driven with the same hammer through a comparable driving distance.*

Section 1808.2.8.3.1 provides the following acceptable load test evaluation –

It shall be permitted to evaluate pile load tests with any of the following methods:

1. *Davisson Offset Limit.*
2. *Brinch-Hansen 90% Criterion.*
3. *Butler-Hoy Criterion.*
4. *Other methods approved by the building official.*

6.8 – Pile Driving Criteria and Installation Requirements

The *Florida Building Code* sets pile driving criteria in Section 1808.2.8.2, stating –

The allowable compressive load on any pile where determined by the application of an approved driving formula shall not exceed 40 tons. For allowable loads above 40 tons, the wave equation method of analysis shall be used to estimate pile drivability of both driving stresses and net displacement per blow at the ultimate load. Allowable loads shall be verified by load tests in accordance with Section 1808.2.8.3. The formula or wave equation load shall be determined for gravity-drop or power-actuated hammers, and the hammer energy used shall be the maximum consistent with the size, strength, and weight of the driven piles.

Section 1808.2.16 addresses pile drivability as follows –

Pile cross sections shall be of sufficient size and strength to withstand driving stresses without damage to the pile, and to provide sufficient stiffness to transmit the required driving forces.

Section 1808.2.14 addresses pile installation sequence as follows –

Piles shall be installed in such sequence as to avoid compacting the surrounding soil to the extent that other piles cannot be installed properly, and to prevent ground movements that are capable of damaging adjacent structures.

Section 1808.2.6 further addresses pile structural integrity as follows –

Piers or piles shall be installed in such a manner and sequence as to prevent distortion or damage that may adversely affect the structural integrity of piles being installed or already in place.

Also, Section 1808.2.15 addresses the use of vibratory hammers as follows –

> *Vibratory drivers shall only be used to install piles where the pile load capacity is verified by load tests in accordance with Section 1808.2.8.3. The installation of production piles shall be controlled according to power consumption, rate of penetration, or other approved means that ensure pile capacities equal or exceed those of the test piles.*

6.9 – Uplift Capacity of Piles

The *Florida Building Code* sets the following uplift load capacity criteria in Section 1808.2.8.5, stating –

> *Where required by the design, the uplift capacity of a single pier or pile shall be determined by an approved method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 1808.2.8.3 divided by a factor of safety of two. For pile groups subjected to uplift, the allowable working uplift load for the group shall be the lesser of:*
>
> 1. *The proposed individual pile uplift working load times the number of piles in the group.*
> 2. *Two-thirds of the effective weight of the pile group and the soil contained within a block defined by the perimeter of the group and the length of the pile.*

6.10 – Lateral Resistance of Piles

The design of a pier's foundation piles is integrally related to the design of a pier's superstructure. As design wave forces are applied to the pier, the designer must consider the problem of the laterally loaded pile. To achieve a rational solution for a laterally loaded pile, a deflected shape must be computed for the pile that is compatible with the characteristics of the superstructure and with the force-deformation characteristics predicted for the soil (Matlock and Reese, 1961).

For improved lateral resistance against wave attack, pier designers are aligning exterior piles at a small angle off of vertical (**Photo 11**). These "battered" piles are calculated to provide greater lateral resistance under the applied design breaking wave forces. The wave forces produce bending between the support points and introduce rotational restraints from connecting members of the structure at these support points. An axial load is included in the analysis, and soil

Photo 11. Installation of battered piles.

characteristics are described by a series of linear springs that increase in stiffness in proportion to the depth of pile penetration. The resulting structural analysis will determine a deflection curve along the pile and corresponding bending moments. A complete solution may be made directly for bending in the structural members of the superstructure by a frame analysis approach. The general beam-column representation may be used as an element in the solution of pier frames.

The problem of determining the moments, shears, and reactions of laterally loaded piles has two distinct elements. There has to be a determination of soil stress-strain characteristics pertaining to the laterally loaded pile and there also has to be a mathematical determination of the pile deflection curve once the soil characteristics are known. There are various mathematical procedures available for computing the deflection curve and its accompanying moment and deflection diagrams. Standard practice typically employs the solution of a beam on an elastic foundation using the differential equation (Palmer and Thompson, 1948; Gleser, 1954) –

$$\mathbf{P = E\ I\ \frac{d^4y}{dx^4}} \tag{13}$$

where, E = modulus of elasticity
I = pile moment of inertia
y = pile deflection
and x = depth below soil surface

McClelland and Focht (1956) have modified the differential equation from the theory of beams to determine the unit soil reaction, p, by the equation –

$$\mathbf{p = -E_s\ y} \tag{14}$$

where, E_s = soil modulus of pile reaction (or modulus of elasticity)

The soil modulus of pile reaction is by definition the ratio between the soil reaction at any point and the pile deflection at that point, and may be best determined by pile load tests. Terzaghi (1955) observed that this soil modulus of pile reaction will decrease in time with soils as consolidation takes place. Also, the lateral deflection of the pile will increase with consolidation. He noted that the lateral modulus of subgrade reaction and deflection are independent of time in sands because the deflection is relatively instantaneous. The modulus of subgrade reaction of sands typically increases in proportion to depth.

6.11 – Lateral Support Requirements

The *Florida Building Code* sets criteria for the lateral support of piles. Section 1808.2.9.2 addresses lateral support for unbraced piles, stating –

> *Piles standing unbraced in air, water, or in fluid soils shall be designed as columns in accordance to the provisions of this code. Such piles driven into firm ground can be considered fixed and laterally supported at 5 feet*

below the ground surface and in soft material at 10 feet below the ground surface unless otherwise prescribe by the building official after a foundation investigation by an approved agency.

Section 1808.2.9.3 addresses the allowable lateral load for piles, stating –

Where required by the design, the lateral load capacity of a pier, a single pile, or a pile group shall be determined by an approved method of analysis or by lateral load tests to at least twice the proposed design working load. The resulting allowable load shall not be more than one-half that test load that produces a gross lateral movement of one inch at the ground surface.

6.12 – Structural Design Requirements of Reinforced Concrete Piles

The load that a single pile may bear depends both on the supporting values of the soil (Equation 8) as well as the structural strength of the pile. Chapter 1809 of the *Florida Building Code* provides allowable stresses for piles of different materials, including concrete, steel, and timber piles.

Most new fishing piers in Florida are constructed with reinforced concrete piles (**Photo 12**) and are required to conform to the standards set forth in ACI 318 *Building Code Requirements for Structural Concrete* (American Concrete Institute, 2005). As most fishing pier designs involve precast concrete piles, they must conform to the requirements of Section 1809.2 of the *Florida Building Code*. Section 1809.2.1 sets criteria for the design and manufacture, the minimum lateral dimension, the reinforcement, and the installation of precast concrete piles. The materials strength requirements, minimum reinforcement, allowable stresses, and concrete cover vary for precast nonprestressed piles and for precast prestressed piles.

Photo 12. Concrete pile bent Russell-Fields Pier, Panama City Beach

Section 1809.2.2 sets the standards for precast nonprestressed concrete piles. Section 1809.2.2.1 provides a material strength requirement stating –

> *Concrete shall have a 28-day specified compressive strength, f'_c, of not less than 3,000 psi.*

However, ACI 318, Section 4.2.2 recommends a minimum compressive strength of 4,000 psi be used in coastal environments. Section 1809.2.2.2 sets forth the minimum reinforcement criteria as follows –

> *The minimum amount of longitudinal reinforcement shall be 0.8 percent of the concrete section and shall consist of at least four bars.*

Section 1809.2.2.3 specifies the following allowable stresses for precast nonprestressed concrete piles stating –

> *The allowable compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength, f'_c, applied to the gross cross-sectional area of the pile. The allowable compressive stress in the reinforcing steel shall not exceed 40 percent of the yield strength of the steel, f_y, or a maximum of 30,000 psi. The allowable tensile stress in the reinforcing steel shall not exceed 50 percent of the yield strength of the steel, f_y, or a maximum of 24,000 psi.*

Section 1809.2.2.5 sets forth the concrete cover requirements for precast nonprestressed concrete piles; however, as would be specifically applicable in the construction of gulf and ocean front fishing piers, there are special requirements for such piles exposed to seawater. For this environment, Section 1809.2.2.5 states –

> *Reinforcement for piles exposed to seawater shall have a concrete cover of not less than 3 inches.*

Section 1809.2.3 sets the standards for precast prestressed concrete piles. Section 1809.2.3.1 provides the material strength requirement, which differs from those of the nonprestressed piles stating –

> *Prestressing steel shall conform to ASTM A 416. Concrete shall have a 28-day specified compressive strength, f'_c, of not less than 5,000 psi.*

Section 1809.2.3.2 addresses the following design requirements for precast prestressed piles stating-

> *Precast prestressed piles shall be designed to resist stresses induced by handling and driving as well as by loads. The effective prestress in the pile shall not be less than 400 psi for piles up to 30 feet in length, 550 psi for piles up to 50 feet in length, and 700 psi for piles greater than 50 feet in length. Effective prestress shall be based on an assumed loss of 30,000 psi in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values in ACI 318.*

Section 1809.2.3.3 specifies the following allowable stresses for precast prestressed concrete piles stating –

The allowable design compressive stress, f_c, in concrete shall be determined as follows:

$$f_c = 0.33\, f'_c - 0.27 f_{pc} \qquad (15)$$

where, f'_c = the 28-day specified compressive strength of the concrete
f_{pc} = the effective prestress stress on the gross section

Section 1809.2.3.5 sets forth the concrete cover requirements for precast prestressed concrete piles, and as was considered for precast nonprestressed piles, there are special requirements for such piles exposed to seawater. For this environment, Section 1809.2.3.5 states –

For piles exposed to seawater, the minimum protective concrete cover shall not be less than 2.5 inches.

The installation requirements set forth for nonprestressed and prestressed concrete piles are the same. Sections 1809.2.2.4 and 1809.2.3.4 both address installation, stating –

A precast concrete pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the 28-day specified compressive strength, f'_c, but not less than the strength sufficient to withstand handling and driving forces.

There are also certain requirements for reinforced concrete pile caps that are applicable to fishing pier design. Section 1808.2.4 includes the following –

The tops of piles shall be embedded not less than 3 inches into pile caps and the caps shall extend at least 4 inches beyond the edges of piles.

6.13 – Structural Steel Pile Design Requirements

Gulf or ocean fishing piers may be designed with structural steel piles, although no such structures exist on the Florida coast. Steel piles have been occasionally used as replacement piles for damaged wooden piers (**Photo 13**).

Photo 13. Corroded steel replacement pile, Sunglow Pier, Daytona Beach Shores

Corrosion is a significant factor with steel piles in the marine environment. Pier construction with structural steel piles must conform to Section 1809.3 of the *Florida Building Code*. Section 1809.3.1 addresses materials for steel pile construction stating –

> *Structural steel piles, steel pipe, and fully welded steel piles fabricated from plates shall conform to ASTM A 36, ASTM A 252, ASTM A 283, ASTM A 572, ASTM A 588, ASTM A 690, ASTM A 913 or ASTM A 992.*

Section 1809.3.2 provides for the allowable stresses in structural steel piles stating –

> *The allowable axial stresses shall not exceed 35 percent of the minimum specified yield strength, F_y.*

Section 1809.3.3 provides various web and flange dimension restrictions on H-piles and Section 1809.3.4 provides outside diameter, minimum cross-section, and wall thickness requirements for steel pipe piles.

6.14 – Timber Pile Design Requirements

At least 10 existing piers on Florida's gulf and ocean beaches are constructed with timber piles (see Table 1); however, all these piers pre-date the current 20-year storm design requirements of Chapter 62B-33, Florida Administrative Code (**Photo 14**). The one exception is the recent Mexico Beach Pier extension that is designed for a 20-year storm with piles along its shore-normal length being timber and the piles for the outer T-section comprised of fiberglass pipe piles.

Photo 14. Timber Mexico Beach Pier prior to 2010 extension

Where gulf and ocean fishing pier construction with timber piles is acceptable and feasible, round timber piles are recommended in lieu of sawn timber piles. Section 1809.1 of the *Florida Building Code* sets criteria for timber piles stating –

> *Timber piles shall be designed in accordance with the AF&PA NDS.*

Section 1809.1.1 sets forth the required timber materials standards stating –

Round timber piles shall conform to ASTM D 25.

Section 1809.1.2 sets forth the required standards for preservative treatment stating –

Preservative and minimum final retention shall be in accordance with AWPA U1 (Commodity Specification E, Use Category 4C) for round timber piles. Preservative-treated timber piles shall be subject to a quality control program administered by an approved agency. Pile cutoffs shall be treated in accordance with AWPA M4.

Section 1809.1.3 provides standards for defective timber piles stating –

Any substantial sudden increase in rate of penetration of a timber pile shall be investigated for possible damage. If the sudden increase in rate of penetration cannot be correlated to soil strata, the pile shall be removed for inspection or rejected.

Section 1809.1.4 provides for the allowable stresses in timber piles stating –

The allowable stresses shall be in accordance with the AF&PA NDS.

6.15 – Special Pile Type Design Requirements

In the design of the new Mexico Beach Pier extension (**Photo 15**), it was determined that the anticipated design depth with scour at the end of the pier precluded the use of timber piles.

Photo 15. Mexico Beach Pier extension with T-section under construction using fiberglass piles

The design engineer incorporated the use of 12-inch fiberglass composite pipe piles (**Photo 16**) with lengths of at least 58 feet (Preble, 2008). Section 1808.2.3 of the *Florida Building Code* sets criteria for special types of piles stating –

> *The use of types of piles not specifically mentioned herein is permitted, subject to the approval of the building official, upon the submission of acceptable test data, calculations and other information relating to the structural properties and load capacity of such piles. The allowable stresses shall not in any case exceed the limitations specified herein.*

The 12-inch fiberglass composite piles used in the Mexico Beach Pier's T-section have the following mechanical properties –

Axial Tensile Strength	65,000 psi
Axial Tensile Modulus	4.3×10^5 psi
Axial Flexural Strength	64,000 psi
Axial Compressive Strength	55,000 psi
Transverse Tensile Strength	25,000 psi
Effective Bending Stiffness	1.014×10^9 psi
Young's Modulus	4.3×10^5 psi
Poisson's Ratio	0.30
Allowable Bending Moment	113 kips-ft
Allowable Axial Load	281 kips
Moment of Inertia	236 in^4
Section Modulus	39.33 in^3
Barcol Hardness	>50
Glass to Resin Ratio (by weight)	60:40
Wall Thickness	0.375 in
Weight	10.7 lbs/ft
Density	112.6 lbs/ft^3
Cross-sectional Area	13.68 in^2

Photo 16. Fiberglass composite piles

Chapter 7 – Additional Structural Design Considerations

Like other major structures, ocean and gulf fishing piers should be designed and constructed to safely support any anticipated normal loads without exceeding the appropriate specified allowable stresses for the materials used in the construction. The structural design of fishing piers requires the consideration of all appropriate design loads acting in combination, to include normal dead loads, live loads, construction loads, wind loads, hydrostatic loads, hydrodynamic loads, and wave loads. As previously discussed in Chapter 5, the depth limited breaking wave loads for the selected design storm event are the greatest forces to be considered in the pier's design. However, the complete structural design also includes the other various loads that may reasonably be expected.

As discussed on page 3, the *Florida Building Code 2007* has been adopted to govern construction throughout the State of Florida. Chapter 16 of the *Florida Building Code* specifically addresses structural design.

7.1 – Non-Water Loads: Dead Loads, Live Loads, Construction Loads, and Wind Loads

The computation of most of a pier's non-water loads is relatively typical for the construction of major structures. The guidance provided in ASCE 7, *Minimum Design Loads for Buildings and Other Structures* (2002), should be employed.

A pier's dead loads include the weight of the materials of construction for all the pier's structural elements. For example, in the design of the new fishing pier for Navarre Beach, the designers selected the following typical dead loads for various pier or deck components (Conrad et al., 2007) –

Concrete Beams, 36" deep sections:	895.31 plf
Concrete Beams, 45" deep sections:	1,120.31 plf
Concrete Pile Cap:	2,025 plf
24" Concrete Pile:	600 plf
Decking:	6.25 psf
Crane Mats:	30.67 psf

[plf - pounds per linear foot; psf - pounds per square foot]

A pier's live loads include the loads produced by the use and occupancy of a pier. Example live loads selected in two recent pier designs, the concrete Navarre Beach Pier and the timber Mexico Beach Pier, have been 125 psf (Conrad et al., 2007; Preble, 2008). In addition to the live load employed as a blanket uniform load, both piers were also designed for a vehicle load of 6,000 pounds per tire. This allowed for vehicle maintenance access via the operation of light equipment, pick-up trucks, or even an ambulance.

As concrete piers are typically constructed with much heavier equipment and materials than timber piers, construction loads are important loads to consider (**Photo 17**). In fact, the weight of a crane operating in the construction of a pier is generally the greatest point load that need be considered. The *Florida Building Code* specifies crane loads in Section 1607.12, which says –

> *The crane live load shall be the rated capacity of the crane. Design loads for the runway beams, including connections and support brackets, of moving bridge*

cranes and monorail cranes shall include the maximum wheel loads of the crane and the vertical impact, lateral and longitudinal forces induced by the moving crane.

The *Florida Building Code* also provides for maximum crane wheel loads, vertical impact or vibration forces, and lateral and longitudinal forces on crane runway beams.

Photo 17. Construction crane and vehicle, Jacksonville Beach Pier [Photo from PBS&J]

Wind loads are particularly important where buildings are constructed on piers. Most piers include the construction of a concession building where fishing permits, tackle, bait, snacks and drinks are purchased (**Photo 18**). These buildings also typically include restrooms. The recently constructed public piers at Pensacola Beach, Navarre Beach, Panama City Beach, Jacksonville Beach, and Juno Beach, all have concession buildings. As mentioned on page 3, Chapter 62B-33, Florida Administrative Code (*Rules and Procedures for Coastal Construction and Excavation*) sets the minimum design storm event for pier construction. The 20-year return interval storm event is the design storm for which ocean and gulf fishing piers are required to be constructed in Florida. However, the chapter specifically states –

Major structures constructed on the pier shall be designed for the wind loads as set forth in the Florida Building Code.

The *Florida Building Code* specifies wind loads that are more closely identified with a 100-year return interval storm event. Section 1609 of the *Florida Building Code* provides the following requirements for the determination of wind loads (1609.1.1) –

Wind loads on every building or structure shall be determined in accordance with Chapter 6 of ASCE 7. The type of opening protection required, the basic wind speed, and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE 7. Wind shall be assumed to come from any horizontal direction to act normal to the surface considered.

Photo 18. Canaveral Pier concession and restaurant buildings

The *Florida Building Code* provides a basic wind speed map (figure 1609 of the *Florida Building Code*) that provides the required wind speed in values for a 3-second gust measured in miles per hour at a height of 33 feet above ground. To convert a 3-second gust, $\boldsymbol{V}$, to fastest-mile wind speeds, $\boldsymbol{V_{fm}}$, the following Equation (16) is used –

$$\boldsymbol{V_{fm}} = \frac{(\boldsymbol{V}-\mathbf{10.5})}{\mathbf{1.05}} \qquad (16)$$

The basic wind speed requirement for pier structures around the gulf coast and northeast coast of Florida is 130 mph. This 3-second gust basic wind speed was used in the design of the recent fishing piers at Navarre Beach, Panama City Beach, Mexico Beach, and Jacksonville Beach (Conrad et al, 2007; Nichols, 2007; Preble, 2008; Rheault, 2003). The wind speed requirement increases in southeast Florida. The basic wind speed used in the design of the recently constructed Lake Worth Pier was 140 mph (Rheault, 2006).

7.2 – Requirements for Reinforced Concrete Construction

Concrete is one of the most durable materials used in coastal construction. As discussed in Section 6.12, most new fishing piers in Florida are constructed of reinforced concrete and are required to conform to the standards set forth in ACI 318. Chapter 19 of the *Florida Building Code* sets forth the requirements for concrete construction in Florida. Section 1901.2 states –

> *Structural concrete shall be designed and constructed in accordance with the requirements of this chapter and ACI 318…*

In some requirements, the *Florida Building Code* differs from ACI 318. Section 1903, Specifications for Tests and Materials, and Section 1904, Durability Requirements, set forth the standards specified in ACI 318.

Of particular relevance to reinforced concrete fishing piers is the issue of corrosion protection. The *Coastal Engineering Manual* (USACE, 2008) notes that, "Corrosion rates for carbon steel exposed to the air at the shoreline are 10 times greater than rates at locations 500 meters inland from the shoreline." Many concrete piers may be of structurally adequate strength when they are constructed or even after a number of years of service, but long-term exposure to the corrosive salt-water environment will substantially weaken even the strongest of concrete structures. Reinforcing steel corrosion is a major problem for structures exposed to salt-water, which leads to spalling of the concrete and exposure of the steel reinforcement (**Photo 19**). Mehta (1991) discusses the use and deterioration of concrete in the marine environment. Section1904.4 of the *Florida Building Code* provides for corrosion protection of reinforcement stating –

> *Reinforcement in concrete shall be protected from corrosion and exposure to chlorides in accordance with ACI 318, Section 4.4.*

For coastal and offshore structures, steel reinforcing bars should have a minimum concrete cover of 2.0 inches for portions submerged or exposed to the atmosphere; however, the cover should be increased to 2.5 inches for portions of the structure in the splash zone or exposed to salt spray. Ocean and gulf fishing piers in Florida should be considered within the salt spray or splash zone. Submerged prestressed members should have a minimum cover of 3.0 inches, with 3.5 inches of cover in the splash zone.

Photo 19. Concrete pile damage exposing deteriorated reinforcing steel [BBCS Photo Files]

Section 1904.2.2 of the *Florida Building Code* requires that concrete subject to seawater exposure shall conform to the corresponding maximum water-cementitious materials ratio and minimum specified concrete compressive strength requirements of ACI 318, Section 4.2.2. ACI 318 recommends using a maximum water-cement ratio by weight of 0.40 and a minimum compressive strength of 4,000 psi in coastal environments.

Section 1905 of the *Florida Building Code*, sets forth the requirements for concrete quality, mixing, and placing. Section 1905.1.1 addresses strength saying –

> *Concrete shall be proportioned to provide an average compressive strength as prescribed in Section 1905.3 and shall satisfy the durability criteria of Section 1904. Concrete shall be produced to minimize the frequency of strengths below*

f'_c as prescribed in Section 1905.6.3. For concrete designed and constructed in accordance with this chapter, f'_c shall not be less than 2,500 psi. No maximum specified compressive strength shall apply unless restricted by a specific provision of this code or ACI 318.

However, for ocean and gulf fishing piers, a 4,000 psi <u>minimum</u> compressive strength needs to be used for f'_c. Coastal construction in Florida typically uses 5,000 psi compressive strength concrete.

The remainder of Section 1905 of the *Florida Building Code* follows closely the requirements of ACI 318 and addresses selection of concrete proportions (1905.2), proportioning on the basis of field experience and/or trial mixtures (1905.3), proportioning without field experience or trial mixtures (1905.4), average strength reduction (1905.5), evaluation and acceptance of concrete (1905.6), preparation of equipment and place of deposit (1905.7), mixing (1905.8), conveying (1905.9), depositing (1905.10), curing (1905.11), cold weather requirements (1905.12), and hot weather requirements (1905.13) (**Photo 20**). FEMA (2006) recommends the addition of admixtures such as pozzolans (fly ash) for coastal construction stating, "Fly ash when introduced in concrete mix has benefits such as better workability and increased resistance to sulfates and chlorates, thus reducing corrosion from attacking the steel reinforcing."

Photo 20. New concrete work on M.B. Miller Pier under construction

Section 1906 of the *Florida Building Code* addresses formwork (1906.1), removal of forms, shores, and reshores (1906.2), conduits and pipes embedded in concrete (1906.3), and construction joints (1906.4). And Section 1907, Details of Reinforcement (**Photo 21**), addresses hooks (1907.1), minimum bend diameters (1907.2), bending (1907.3), surface conditions of reinforcement (1907.4), placing reinforcement (1907.5), spacing limits for reinforcement (1907.6), concrete protection for reinforcement (1907.7), special reinforcement details for columns (1907.8), connections (1907.9), lateral reinforcement for compression members

(1907.10), lateral reinforcement for flexural members (1907.11), shrinkage and temperature reinforcement (1907.12), and requirements for structural integrity (1907.13). Specifically applicable to gulf and ocean fishing piers, Section 1907.7.5 states –

> *In corrosive environments or other severe exposure conditions, prestressed and nonprestressed reinforcement shall be provided with additional protection in accordance with ACI 318, Section 7.7.5.*

Photo 21. Steel reinforcement on M.B. Miller Pier under construction

Other factors in constructing concrete fishing piers include particle abrasion and attachment by marine organisms. Those portions of the pier's foundation that are in proximity to dynamic sediment transport are susceptible to wear by abrasion. Also, floating debris and wave impact loads can chip concrete surfaces and either weaken the structure or expose the steel reinforcement as was seen in **Photo 19**. On the other hand, marine organisms will not typically have any adverse effect on a well mixed concrete with strong aggregates. Concrete has no food value to marine organisms, and even though marine plants, crustaceans, and marine worms will attach to the foundation piles, they provide little effect other than the increased drag resistance to waves and currents.

7.3 – Structural Steel Construction

Steel has been employed in Florida marine construction since the late 1800's. Although more commonly used in seawall and bulkhead construction, an important marine application of steel is pilings. H-piles and pipe piles are used to support foundations and to support coastal structures. Older timber fishing piers in Florida have been repaired using driven steel H-piles as replacement piles or as additional piles as was seen in **Photo 13**. Steel H-piles can be driven into hard strata, such as coral, or through rocky sediments. Steel piles can also be encased in concrete for protection from salt water corrosion. Because steel piles can usually be easily extracted from sandy sediments using high pressure hydraulic jets, they are often employed in the construction of temporary piers, such as those used to support equipment in nearshore construction activities. Page 37 addresses the building code requirements for structural steel piles in pier construction.

Steel bolts are typically used as the connectors of wood decks and handrails of concrete or timber fishing piers as well as the principle structural connectors of timber piers. Corrosion of steel bolts can substantially weaken the structural integrity of a pier (**Photo 22**). Historic practice allowed for the use of oversized bolts in the marine environment to compensate for excessive corrosion. Bolts are made of carbon structural steel or high-strength steel. The resistance of high-strength steel bolts to atmospheric corrosion is about twice that of carbon structural steel, and would be the better choice of the two in the marine environment. Allowable stresses for standard and high-strength steel bolts are provided in AISC (1980). However, in today's marine construction, stainless steel is commonly used for components openly exposed to salt water (**Photo 22**). Exposed steel elements must be covered with a protective coating. Painting, galvanizing, or applying a thick tar coating may provide adequate protection from corrosion.

Photo 22. Corrosion of steel bolt, left; stainless steel bolts, right.

Like concrete pier pilings, steel piles and connections can suffer abrasion by hydrodynamically and aerodynamically propelled sand particles. While loss of metal to direct abrasion will be minor, the loss of protective coatings leading to metal exposure may result in rapid corrosion. ASTM A-690 (2000) notes that pilings located in the splash zone can achieve two to three times the corrosion resistance of carbon steel if they are fabricated of high-copper-bearing, high-strength, low-alloy steel conforming to ASTM Standards.

Unlike concrete piles, the attachment or bio-fouling of marine plants and organisms increases the corrosion rate of steel piles. Copper and copper-nickel alloys have the best resistance to bio-fouling. Bio-fouling can be decreased by application of antifouling paints.

7.4 – Timber Pier Construction

Timber fishing pier construction in Florida has slowly been replaced by concrete construction. As mentioned in Section 6.14, only the recent Mexico Beach fishing pier extension has timber pile and frame construction (**Photo 23**). Wood has historically been used to construct a large variety of marine structures in Florida and elsewhere, but most specifically wharves, bulkheads, pile dolphins, channel markers, and piers. Untreated wood is not recommended for the coastal zone because it will soon decay if it comes in direct contact with seawater. In marine applications, timber is attacked by marine borers, insects, fungus, and rot. Typically, timber elements that are directly subject to the marine environment are pressure-treated with coal-tar creosote or a similar protective treatment. Marine plants, algae, crustaceans, and marine worms attach to treated timber piles; however, these do not appear to harm the strength characteristics of the wood.

Photo 23. Timber pile and frame

Marine engineering design of timber structures needs to consider the mechanical properties of wood, which are determined for the longitudinal axis (parallel to the wood grain), the radial axis (perpendicular to the wood grain and growth rings), and the tangential axis (perpendicular to the wood grain and tangential to the growth rings). There is little variance between the tangential and radial stresses, so only the stresses perpendicular and parallel to the grain are generally considered in engineering design applications. Timber piles, beams, bracing, etc., will have the grain parallel to the longest dimension of the structural element, because wood has its greatest strength when loaded for tension or compression parallel to the grain. Tensile stresses perpendicular to the grain have the least strength. But wood components have substantial strength from shear stresses applied perpendicular to the grain. Imperfections in the wood as well as the presence of knots will reduce compression, tension, and shear stress design limits. AF&PA (2005) provides the *National Design Specification for Wood Construction* (*NDS*). The *NDS* provides lumber engineering parameters used in design.

Over time, most old timber pile construction will deteriorate in the marine environment. Old timber piers in Florida are often restored by the addition of replacement piles or by encapsulating piles in concrete. **Photo 13** showed a steel replacement pile on the Sunglow Pier, Daytona Beach Shores, which was originally constructed in 1960. **Photo 18** showed concrete encapsulated piles on the Canaveral Pier, constructed in 1963.

The oldest existing pier on the Florida coast is the Daytona Beach Pier, constructed in 1925. **Photo 24** shows the encapsulation of many of this pier's piles with concrete collars. In this photo taken at low tide, also seen is the encrustation of marine algae, crustaceans, and marine worms on each of the piles.

Photo 24. Concrete collars with bio-fouling, Daytona Beach Pier [Photo by John McDowell, BBCS]

7.5 – Structural Design – Frame Analysis

In the design of all complex structures, including fishing piers, the structural engineer needs to employ the use of mathematical models to analyze the structure. These structural models use matrix math solutions for a complex system with multiple load combinations. Most models employed today are typically linear and elastic models, and some allow second order effects. Among the models used in fishing pier design include STAAD, RISA, GTSTRUDL, ETABS, SAP, TEKLA, and others. While it is not the intent of this document to describe these models or to discuss and compare their capabilities and merits, a simple general discussion is warranted. These models are complete structural analysis and design software packages. With comprehensive structural engineering software, the structural engineer can build a model, verify it graphically, perform analysis and design, and review the results in a graphics based environment. A model considers the structure's geometry, the structural member and material properties, the design loads and their different load combinations, and the soil structure interaction. The basic analysis typically assumes linearity of the results allowing a simple summation of all the load cases, and thus, the conclusions are the result of first order effects.

Fishing piers have long slender structural elements in the piles, which introduces non-linear second order effects. A model will evaluate the non-linearity in the materials, the structural members, and the entire structural frame. When a structure undergoes displacement from the primary loads, the displacement creates secondary loads. Typically, structural members will soften and the computations will have to solve the load combinations through an iterative process. The resulting second order analysis reveals the design loads to be greater than determined in the linear or first order analysis. The first and second order loads may then be compared to determine how much amplification will occur and whether the structure will potentially reach elastic instability. The structural engineer may use the second order loads where amplification is not excessive to size the piles, beams, caps, and connections, as well as add bracing as necessary.

Frame analysis is particularly important in fishing pier design. Individual pile bents (pile cap and pile combinations) standing unsupported cannot typically resist the lateral loads of the design breaking waves. For this reason, a pier is designed with multiple bays (segments between pile bents) tied together. The frame action of multiple bays is designed to resist the design breaking wave impact loads. The grouping of multiple bays are held to a length less than the design wave length so that only one wave crest acts on the frame section at any given moment.

Chapter 8 – Breakaway Decks and Rails

As discussed in the chapter on “Selection of Design Storm Conditions: Tides and Waves”, the 20-year return interval storm event is the minimum design storm for which ocean and gulf fishing piers are to be constructed in Florida. Along with specifying this performance requirement, Rule 62B-33.007 (4) (k), Florida Administrative Code, also states –

Pier decking and rails may be designed to be an expendable structure.

In interpreting this rule, it is understood that the *Florida Building Code* requirements for design must be satisfied. In other words, the pier decking and rails would have to be designed for the normal loads anticipated, including live loads, dead loads, construction loads, and wind loads. However, the pier decking and rails would not have to be able to withstand the extreme loads of breaking wave forces. As discussed in the chapter on “Wave Load Analysis”, storm waves inflict extremely high forces on a pier’s structural members, and both horizontal and vertical wave forces need to be considered. **Figure 7** on page 24, diagrams the vertical wave force on a pier’s deck, which may be determined by the relationship in Equation (7), $\mathbf{F_v = \rho\ h}$.

8.1 – Deck Design

A solid concrete deck, like that shown in **Photo 6** of the old Navarre Pier, will be impacted by all storm waves exceeding the bottom elevation of the deck. These forces may ultimately cause the pier to sustain major structural damage or its complete loss. A dramatic reduction in these loads can be designed into a pier by utilizing a breakaway deck and rails design (**Photo 25**).

Photo 25. Breakaway wood deck at outer T-section, Russell-Fields Pier

Most of the recently constructed concrete piers in Florida have utilized a breakaway deck design feature that involves the construction of individual wooden plank deck panels that simply rest on top of the concrete beams. The beams have a design notch running parallel with the length of the pier upon which the wood deck panels are placed (**Photo 26**).

Photo 26. Notched beams to support breakaway deck panels, M.B. Miller Pier

Typically, the wood deck panels are sized about 4.5 to 5.0 feet in width and length and consist of pressure treated wood planks connected by wood runners or diagonal bracing along their underside using steel carriage bolts. **Photo 27** shows a breakaway deck panel on the Pensacola Beach Fishing Pier that dislodged due to wave uplift forces during Tropical Storm Isidore in 2002. The pier deck elevation is +26 feet NAVD88.

Photo 27. Breakaway pier deck section, Pensacola Beach Fishing Pier

Typically, these breakaway wooden deck panels weigh approximately 300 pounds. This design concept met its ultimate test at the Pensacola Beach Fishing Pier during Hurricane Ivan in 2004, which was considered a 200-year storm event for its extreme storm tide on Santa Rosa Island in northwest Florida. Browder and Norton (2005) reported 206 of the wooden panels were dislodged out of the total 1,495 breakaway deck panels on the pier; however, the pier sustained no structural damage. Of interest, about 100 of the dislodged panels were transported overboard and about 80 of those were subsequently recovered including one that traveled across Santa Rosa Sound (Al Browder, personal communication). In 2005, the Pensacola Beach Fishing Pier was once again impacted by a major hurricane, as Hurricane Dennis made landfall at Pensacola Beach, causing severe damage along Santa Rosa Island. Once again, the pier lost many of its deck panels to wave uplift forces exceeding the deck elevation of +26 feet NAVD88, but again no structural damage was incurred by the pier (**Photo 28**).

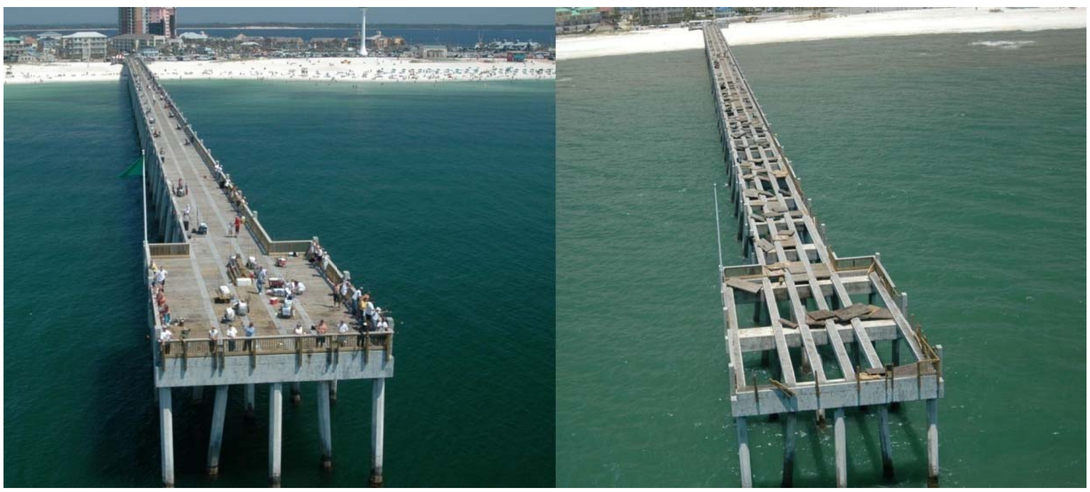

Photo 28. Pensacola Beach Fishing Pier before and after Hurricane Dennis' wave uplift forces dislodged breakaway deck panels and rails without structural damage to the pier [BBCS Photo Files]

Another breakaway design concept uses steel grate deck panels. This is the design used on the fishing pier at Sebastian Inlet State Park (**Photo 29**). This pier is subject to the up-rush of breaking waves on the inlet's north jetty and therefore the design allows substantial flow through the steel grating. Deck grates may breakaway completely during an extreme event, such as during Hurricane Frances in 2004, when the pier lost 40 to 50 grates with no major damage (Clark et al, 2004).

Photo 29. Sebastian Inlet Fishing Pier after Frances

Typically for wooden piers, instead of special deck panels, the wood planks are simply nailed to the beams and the deck will subsequently disintegrate when subject to wave up-lift forces. Some of the older concrete piers are also designed with wood planks attached to the beams with steel bolts (**Photo 30**). Pier decking damage was shown previously in **Photos 7** through **9**.

Photo 30. Wood deck planks on concrete St. Augustine Beach Fishing Pier

8.2 – Rail Design

Although not as important to reducing structural resistance to wave forces as breakaway decks, pier rails may also be designed as breakaway features. Most of the older and recently constructed piers have wooden rails of various designs (**Photos 30-31**). As with deck design, the rails are required to be designed for the normal loads anticipated, including live loads, dead loads, and wind loads, but would not have to be able to withstand the extreme loads of breaking wave forces.

Photo 31. New rails of recently constructed Russell-Fields Pier, Panama City Beach

As was observed in **Photo 28**, the Pensacola Beach Fishing Pier sustained rail damage in addition to the dislodging of breakaway deck panels. Some pier designs have incorporated a more substantial steel or aluminum rail design due to their greater exposure to breaking wave forces. Such is the case with the Sebastian Inlet Fishing Pier as was seen in **Photo 29**. The old Manatee County Pier on Anna Maria Island south of Tampa Bay in southwest Florida employed a substantial aluminum rail design (**Photo 32**). This 300-foot monolithic concrete pier was constructed at a low deck elevation, well below the current design requirements, and subsequently was condemned due to structural deterioration resulting in a public safety hazard. A new replacement pier has been designed, and a permit was issued by the Department in December 2010.

Photo 32. Aluminum rails on old Manatee County Pier, Anna Maria Island

An additional issue regarding rail design is the maximum height recommended for anglers in wheelchairs. Most recently constructed fishing piers set the top or cap elevation of the pier rail at 42 inches above the deck elevation for public user safety. However, to facilitate fishing access by anglers in wheelchairs a maximum rail elevation above the deck elevation is recommended to be 34 inches. Some piers, such as the Mexico Beach Pier, provide segments of rails that facilitate this wheelchair accessibility (**Photo 33**).

Photo 33. Wheelchair accessible rail segments, Mexico Beach Pier

8.3 – ADA Standards for Accessible Design

A discussion of fishing pier deck and rail design would not be complete without acknowledgement of the requirements set forth in the federal regulations resulting from the Americans with Disabilities Act of 1990 (ADA). The Department of Justice (1994) has adopted in the Code of Federal Regulations (28 CFR Part 36) the access guidelines for individuals with disabilities. All new public buildings, including fishing piers, have to meet these federal regulations. The most important issue in these ADA requirements that is applicable to the construction of fishing piers is seen in the design of access ramps. Rule 4.8.2 states,

> *The maximum slope of a ramp in new construction shall be 1:12. The maximum rise for any run shall be 30 inches.*

It is important to note that these requirements are the maximum allowable slope and rise. The federal guidelines note that slopes between 1:16 and 1:20 are preferred. The ability of a disabled person to manage an incline on foot or in a wheelchair is related to a ramp's slope and its length. A wheelchair user that also has disabilities that affect their arms or have low stamina will have a very difficult time with steep or lengthy inclines. The federal guidelines note that, "Most ambulatory people and most people who use wheelchairs can manage a slope of 1:16. Many people cannot manage a slope of 1:12 for 30 feet."

Other applicable rules affect handrails and ramp design for outdoor conditions. Rule 4.8.5 states,

> *If a ramp run has a rise greater than six inches or a horizontal projection greater than 72 inches, then it shall have handrails on both sides…Top of handrail gripping surfaces shall be mounted between 34 inches and 38 inches above ramp surfaces.*

And to minimize ponding on ramp surfaces, Rule 4.8.8 states,

> *Outdoor ramps and their approaches shall be designed so that water will not accumulate on walking surfaces.*

8.4 – Revised ADA Standards for Fishing Piers

In 2008, the Department of Justice went through a formal procedure with public comment and agency review to propose major amendments to the ADA requirements. On July 23, 2010, the United States Attorney General signed final regulations revising the Department's ADA requirements, including its *ADA Standards for Accessible Design*. The official text was published in the *Federal Register* on September 15, 2010.

These final rules will take effect on March 15, 2011. The new rules set forth specific new requirements for public fishing piers in Rule 1005, *Fishing Piers and Platforms*. Future fishing pier designers should determine all applicable ADA requirements in developing a pier design.

The following is quoted from *2010 ADA Standards for Accessible Design* as published in the *Federal Register*:

1005 Fishing Piers and Platforms

1005.1 Accessible Routes. Accessible routes serving fishing piers and platforms, including gangways and floating piers, shall comply with Chapter 4.

EXCEPTIONS: 1. Accessible routes serving floating fishing piers and platforms shall be permitted to use Exceptions 1, 2, 5, 6, 7 and 8 in 1003.2.1.

2. Where the total length of the gangway or series of gangways serving as part of a required accessible route is 30 feet (9145 mm) minimum, gangways shall not be required to comply with 405.2.

1005.2 Railings. Where provided, railings, guards, or handrails shall comply with 1005.2.

1005.2.1 Height. At least 25 percent of the railings, guards, or handrails shall be 34 inches (865 mm) maximum above the ground or deck surface.

EXCEPTION: Where a guard complying with sections 1003.2.12.1 and 1003.2.12.2 of the International Building Code (2000 edition) or sections 1012.2 and 1012.3 of the International Building Code (2003 edition) (incorporated by reference, see "Referenced Standards" in Chapter 1) is provided, the guard shall not be required to comply with 1005.2.1.

1005.2.1.1 Dispersion. Railings, guards, or handrails required to comply with 1005.2.1 shall be dispersed throughout the fishing pier or platform.

> Advisory 1005.2.1.1 Dispersion. Portions of the railings that are lowered to provide fishing opportunities for persons with disabilities must be located in a variety of locations on the fishing pier or platform to give people a variety of locations to fish. Different fishing locations may provide varying water depths, shade (at certain times of the day), vegetation, and proximity to the shoreline or bank.

1005.3 Edge Protection. Where railings, guards, or handrails complying with 1005.2 are provided, edge protection complying with 1005.3.1 or 1005.3.2 shall be provided.

> Advisory 1005.3 Edge Protection. Edge protection is required only where railings, guards, or handrails are provided on a fishing pier or platform. Edge protection will prevent wheelchairs or other mobility devices from slipping off the fishing pier or platform. Extending the deck of the fishing pier or platform 12 inches (305 mm) where the 34 inch (865 mm) high railing is provided is an alternative design, permitting individuals using wheelchairs or other mobility devices to pull into a clear space and move beyond the face of the railing. In such a design, curbs or barriers are not required.

1005.3.1 Curb or Barrier. Curbs or barriers shall extend 2 inches (51 mm) minimum above the surface of the fishing pier or platform.

1005.3.2 Extended Ground or Deck Surface. The ground or deck surface shall extend 12 inches (305 mm) minimum beyond the inside face of the railing. Toe clearance shall be provided and shall be 30 inches (760 mm) wide minimum and 9 inches (230 mm) minimum above the ground or deck surface beyond the railing.

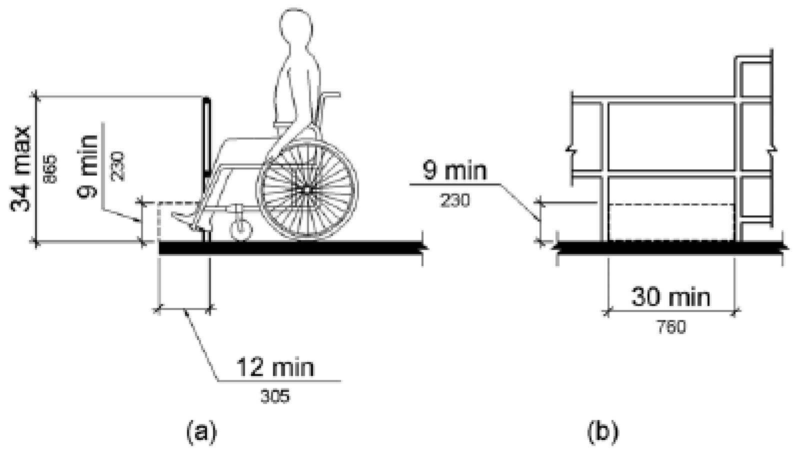

Figure 1005.3.2 Extended Ground or Deck Surface at Fishing Piers and Platforms

1005.4 Clear Floor or Ground Space. At each location where there are railings, guards, or handrails complying with 1005.2.1, a clear floor or ground space complying with 305 shall be provided. Where there are no railings, guards, or handrails, at least one clear floor or ground space complying with 305 shall be provided on the fishing pier or platform.

1005.5 Turning Space. At least one turning space complying with 304.3 shall be provided on fishing piers and platforms.

Chapter 9 – Construction Techniques

There are three general methodologies in the construction of gulf and ocean fishing piers. These three methodologies may be categorized as follows –

1. Barge mounted crane
2. Temporary access trestle
3. Top-down construction

9.1 – Barge Mounted Crane

In the marine construction field, the need to construct large structures in the marine environment presents unique problems and issues not dealt with during normal terrestrial construction. Having to construct within not only an aquatic system, but within a corrosive marine environment creates unique challenges. The construction of many marine structures has by necessity been dependent on mounting construction equipment, particularly construction cranes, onto a stable floating vessel or barge (**Photo 34**).

Photo 34. Barge and construction crane [BBCS Photo Files]

Historically most of the early wooden fishing piers, constructed during the 20th century, were constructed using cranes and other equipment mounted on a floating barge. Using a barge mounted crane continues to remain a cost effective option for specific types of pier construction activities. Often after hurricanes or other major storms, there is limited damage to fishing piers that requires the repair or replacement of foundation piles as seen by numerous examples in *Part 1: Historical Pier Damage in Florida*. To conduct piling repairs or replacements, a barge mounted crane may be the only viable option for most existing piers constructed before 2000.

In addition, demolition and removal of storm damaged piers often requires a barge mounted crane (**Photo 35**). A second barge could be employed to transport the derelict pier members to a disposal site, which is typically an offshore artificial reef site located in the vicinity.

Photo 35. Demolition of old Navarre Pier by barge mounted crane

9.2 – Temporary Access Trestle

The shore-breaking wave environment creates hazardous conditions for floating platforms in marine construction. Another methodology employed for fishing pier construction utilizes a temporary access trestle, which is constructed adjacent to and parallel with the pier being constructed (**Photo 36**). The Sea Quay Pier, constructed in the winter of 2008 in Vero Beach, provides a recent example of this method.

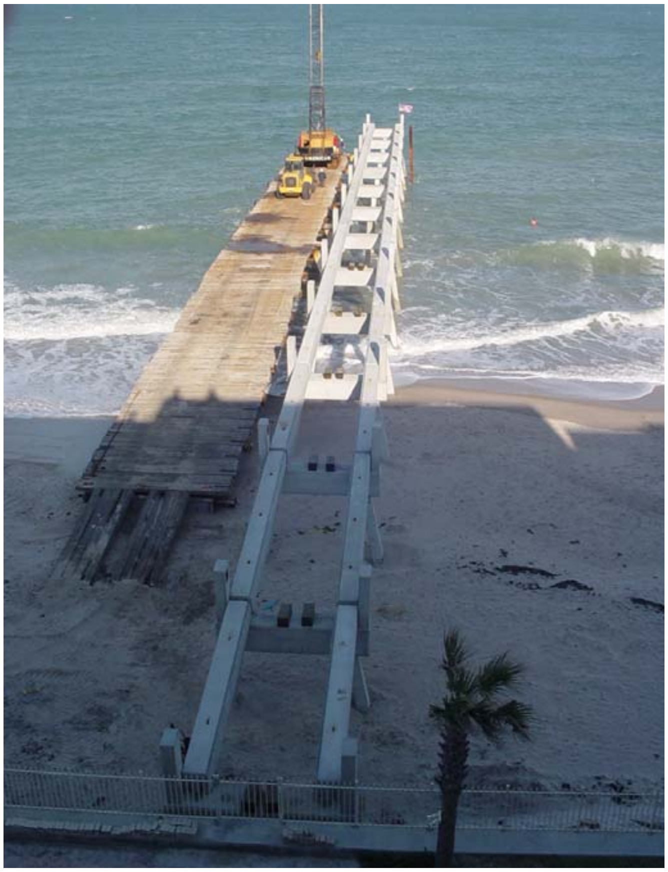

Photo 36. Temporary access trestle at Sea Quay Pier, Vero Beach
[Photo from Bridge Design Associates, Inc.]

The access trestle is constructed of steel H-piles and supporting I-beams and is overlaid with a heavy wooden work deck to serve as the construction platform. The trestle provides a stable platform for the construction crane and the vehicle to transport the beams and piles (**Photo 37**).

Photo 37. Sea Quay Pier
[Bridge Design Associates, Inc.]

With an access trestle the fishing pier's piles may be installed using a pile placement template of steel I-beams supported by temporary steel H-piles (**Photos 38-39**).

Photo 38. Pile placement with hammer from access trestle [Bridge Design Associates, Inc.]

Photo 39. Pile placement template [Bridge Design Associates, Inc.]

Following pile embedment, the pile caps are placed, which cap and connect the piles and provide vertical support for the beams that span between each pile bent or group of piles. The pile caps also support the bollards that house the pier lights and provide lateral support for the beams (**Photo 40**).

Photo 40. Pile cap with bollards [Bridge Design Associates, Inc.]

With pile bents in place, the pier's beams may be placed onto the pile caps (**Photo 41**).

Photo 41. Beam placed on pile caps [Bridge Design Associates, Inc.]

Upon completion of beam placement, the fishing pier's superstructure is essentially intact and the finishing tasks of installing utilities, rails, and deck remain. Prior to installation of the deck, the electrical lines and railing posts are installed (**Photo 42**).

Photo 42. Electrical lines and railing posts [Bridge Design Associates, Inc.]

The last phase of construction includes the placement of the wood deck and the side rails (**Photo 43**).

Photo 43. Wood deck and side rails [Bridge Design Associates, Inc.]

9.3 – Top-Down Construction

Most new gulf and ocean fishing pier construction in Florida employ the top-down construction method. With this methodology the construction crane and other equipment is supported by the pier structure that is under construction (**Photo 44**). The most recent structures constructed by this methodology are the concrete piers at Pensacola Beach (2001), Jacksonville Beach (2004), Lake Worth (2008), Panama City Beach (2009), Navarre Beach (2010), and Bay County (2010). While the designs of each of these piers had significant differences (except for the twin county and city piers at Panama City Beach), the top-down construction methodology varied little.

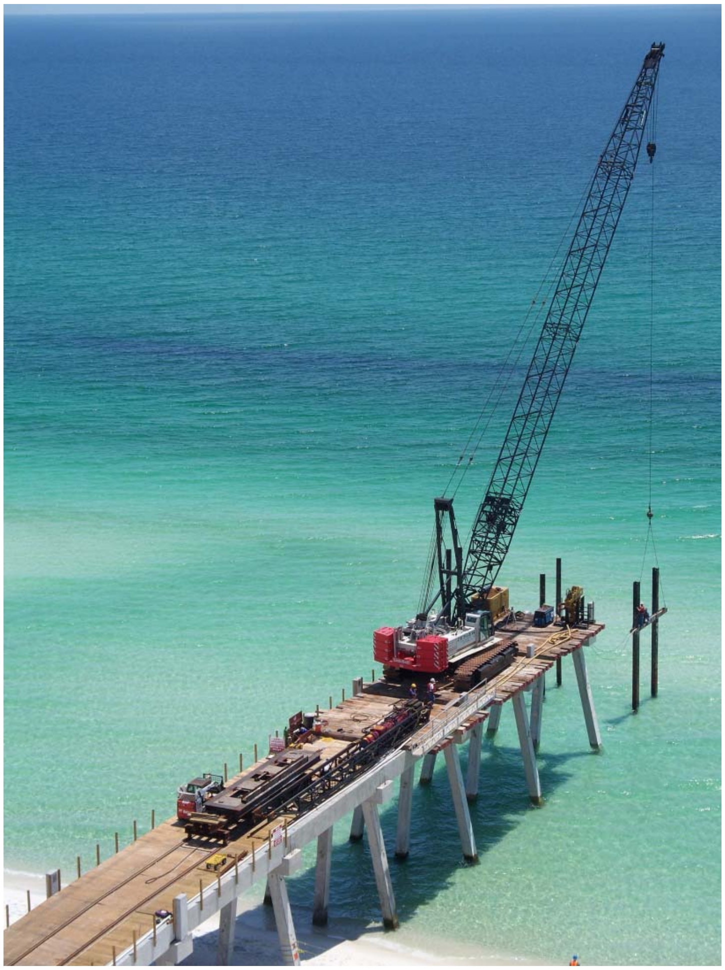

Photo 44. Crane and equipment supported by Navarre Beach Pier under construction [Photo by PBS&J]

The procedure typically involves commencing at the pier's landward-most pile bent and working seaward. Each pile in a pile bent is embedded and the pile cap is placed before proceeding to the next pile bent seaward (**Photo 45**). Steel H-piles are employed as temporary piles supporting the pile placement template (**Photo 46**).

Photo 45. Pile placement by hydraulic jetting, Navarre Beach Pier [PBS&J]

Photo 46. Pile placement template, Navarre Beach Pier [PBS&J]

With each group of piles properly embedded, the pile cap, with bollards cast into them, is placed to connect the piles and provide beam support (**Photo 47**).

Photo 47. Cap placement on piles to construct pile bent [PBS&J]

Following construction of each pile bent, the connecting beams are placed (**Photo 48**).

Photo 48. Beam placement by pier mounted crane [PBS&J]

The fishing pier's connecting beams are typically formed and precast off-site at a concrete yard and truck hauled to the construction site (**Photo 49**).

Photo 49. Forming a precast concrete beam off-site [PBS&J]

The precast concrete beams as well as piles are typically carried the length of the pier by a special transporter (**Photo 50**).

Photo 50. Trolley of steel casters and rails transporting precast piles [PBS&J]

Some of the large public fishing piers of greater width require one or two regular (un-notched) interior beams as well as two notched exterior beams. **Photo 51** shows the placement of an exterior notched beam adjacent an interior beam already set in place.

Photo 51. Notched exterior beam placed adjacent interior beam [PBS&J]

On top of each pile bent and between the beams are cast in place diaphragms, which are poured reinforced concrete connections that create a continuous beam effect (**Photo 52**).

Photo 52. Diaphragm form, left, and casting in place the diaphragm, right [PBS&J]

On piers where the construction crane and support system require a greater width than the pier allows, the bollards are typically cast in place on the pier following completion of setting all the beams. These bollards would be cast onto the pier commencing with the seaward terminus of the structure and working landward (**Photo 53**).

Photo 53. Casting in place the bollards, M.B. Miller Pier

The terminal platforms of concrete piers require longer pile caps and more rows of support beams (**Photo 54**). Many piers are designed with a T-section at their seaward terminus; however, some piers are designed with a square platform at their terminus. The Navarre Pier is designed with an octagon shaped terminal platform.

Photo 54. Terminal platform of M.B. Miller Pier prior to deck placement.

Chapter 10 – Effects on Coastal Processes

Historically, fishing piers have been constructed with little understanding about their effects on the beach and nearshore processes. The general contention has been that piers have little effect on these processes given their wide pile spacing and open trestle design (**Photo 55**). Undoubtedly, little immediate impact to the adjoining shoreline is seen at most any given new pier site. However, long term effects have been witnessed at numerous piers. More often these effects have been viewed as being more favorable as opposed to negative.

Photo 55. Sinusoidal beach cusps along Pensacola Beach seen relatively unaffected by the Pensacola Beach Pier [Photo by Jill Hubbs, WSRE, PBS]

Rule 62B-33.007(4) (k), Florida Administrative Code, requires for fishing pier design –

> *Pile foundations shall not obstruct the longshore sediment transport and shall be designed to minimize any impact to the shoreline or coastal processes.*

Several of the post-storm investigations cited in *Part 1: Historical Pier Damage in Florida* have noted beach erosion conditions adjacent to surviving fishing piers. The erosion effects are typically "downdrift" of the incident storm wave direction of attack.

A longer term trend is often seen adjacent piers that have existed for decades. At many open coast piers (not significantly affected by an inlet) there exists a relatively stable salient or advanced shoreline position beyond the average updrift or downdrift shoreline. This salient typically forms after many years of a pier's effects on wave dampening and sediment accumulation. Given the permeable nature of a pier's foundation design with wide pile spacing, littoral sediment has virtually no structural impediment. Even the pilings, which are subject to localized scour, present little significant impediment to longshore sediment transport.

This long-term salient effect has typically resulted in stable beach conditions adjacent numerous piers in Florida. Over the past nearly 40 years, this author has observed this beach stability effect at a number of Florida piers, but most notably at the Okaloosa County Pier and Panama City Beach Pier in the northwest, the Manatee County Pier and Naples Pier in the southwest, the Flagler Beach Pier and Daytona Beach Pier in the northeast, and the Lake Worth Pier and Dania Pier in the southeast. With the benefit of the salient or stable beach conditions near these piers, significant storm-induced erosion and damages to uplands have generally been lacking or minimized.

Studies of the effects of piers on adjacent shorelines have been conducted in California and North Carolina. In a beach survey in North Carolina, Everts and DeWall (1975) investigated five piers and found no effects on the adjacent shorelines. Likewise, Noble (1978) examined 20 piers along the Southern California Bight and determined negligible effects on adjacent shorelines. Follow-up observations of southern California piers in this region continue to suggest that these piers still have had a negligible effect on the shorelines (Leadon, 2009). The author has likewise observed little effect on adjacent beaches at fishing piers located on North Padre Island and Port Aransas, Texas, dating from the mid-1950's to present.

Wave tank studies have been conducted by various researchers to evaluate the use of closely spaced piles as breakwaters and wave transmission losses through pile arrays. Van Weele and Herbich (1972) determined that the transmissibility of different pile arrays depends considerably on the spacings between the piles and the corresponding combinations of reflection loss and eddy loss. They also determined that staggering the piles did not decrease transmissibility nor significantly affect the reflection coefficient. The reflection coefficient does decrease with an increase in the longitudinal and transverse spacing between piles. Noble (1978) reports that when pile spacing exceeds four times the pile diameter, reflection and eddy losses become insignificant, and the ratio of the transmitted wave height to incident wave height approaches unity.

As discussed, laboratory and field studies have shown little effect from piers on adjacent beaches. In 1977, a pier was constructed at the Coastal Engineering Research Center's Field Research Facility (FRF) at Duck, North Carolina. This pier is 1,840 feet long and has 30-inch and 36-inch diameter concrete piles and a 20-foot wide deck with an elevation of +25.4 feet NGVD. There are 108 piles with 15-foot shore-parallel spacing and 40-foot shore-normal spacing. Miller, Birkemeier, and DeWall (1983) report on an extensive data acquisition effort from the FRF pier in which wave data and bathymetric data was obtained and analyzed. They determined that wave data measured from the pier compared favorably with data collected away from the pier. However, notwithstanding the results of earlier studies that report negligible effects on coastal processes, this study identified significant effects in both the shore-normal and shore-parallel (alongshore) directions. The researchers discovered a long shallow depression under much of the pier including a scour hole near the pier's seaward end. This scour hole averaged over six feet deeper than the surrounding contours.

Miller, Birkemeier, and DeWall (1983) also identified significant effects on shore-parallel contours at distances approaching 1,000 feet from the FRF pier and to a depth of -23 feet. During major storms, they measured changes due to the pier's influence as far as 1,150 feet from the pier. During extended periods of predominantly northeast wave conditions, a material effect was seen where sediment accumulated to the updrift of the pier and eroded to the downdrift, similar to the effects of a permeable groin.

In summary, the effects of fishing piers on beach and nearshore processes are normally minimized by the wide spacing of the foundation piles. Observations at numerous piers along the coasts of California, Texas, North Carolina, as well as Florida, would indicate that significant impacts should not be expected with piers designed with wide pile spacing. Some piers may have a long-term accretional influence on an adjacent beach that may facilitate the implementation of a beach management strategy.

Chapter 11 – Environmental Considerations

Most engineering design documents that endeavor to discuss environmental issues typically do so as an after-thought with only a brief and generalized discussion of the issues involved. The discussion that follows is not intended to adequately address all the potential environmental effects of gulf and ocean fishing piers, but is intended to disclose enough of the environmental issues typically encountered with piers in Florida to raise awareness of the factors that will most likely affect pier design and construction.

The State of Florida regulates coastal construction, including all gulf and ocean fishing piers. On October 13, 1995, the Florida Department of Environmental Protection (FDEP) implemented Section 161.055, Florida Statutes, by initiating concurrent processing of applications for coastal construction permits, environmental resource permits, wetland resource (dredge and fill) permits, and sovereign submerged lands authorizations. These permits and authorizations, which were previously issued separately and by different state agencies, have now been consolidated into a Joint Coastal Permit (JCP). The consolidation of the prior environmental regulatory programs and the assignment of responsibility to a single state agency (FDEP's Bureau of Beaches and Coastal Systems) has eliminated the potential for conflict between permitting agencies and helped ensure that reviews are conducted in a timely manner. A copy of the JCP permit application is forwarded to the United States Army Corps of Engineers for separate processing of the federal dredge and fill permit.

When the Bureau issues a JCP for fishing pier construction, it does so under Chapter 161, Florida Statutes, Part IV of Chapter 373, Florida Statutes, and Title 62, Florida Administrative Code for an activity specifically described in the approved plans and specifications. Issuance of the JCP also constitutes certification of compliance with state water quality standards pursuant to Section 401 of the Clean Water Act, 33 United States Code 1341. And concurrent with issuance of the JCP, the Department also grants a lease to use sovereign submerged lands for the fishing pier, under Article X, Section 11 of the Florida Constitution, Chapter 253, Florida Statutes, Title 18, Florida Administrative Code, and the policies of the Florida Board of Trustees. Issuance of the JCP also constitutes a finding of consistency with Florida's Coastal Zone Management Program, as required by Section 307 of the Coastal Zone Management Act.

Activities that require a JCP include the construction of public and private fishing piers. Specifically, a JCP is required for activities that meet all of the following criteria:

1. Located on Florida's natural sandy beaches facing the Atlantic Ocean, the Gulf of Mexico, the Straits of Florida or associated inlets;
2. Activities that extend seaward of the mean high water line;
3. Activities that extend onto sovereign submerged lands; and
4. Activities that are likely to affect the distribution of sand along the beach.

This state environmental regulatory program ensures that any fishing pier construction does not degrade water quality, such as through the loss of wetlands, through improper in-water construction techniques, or through the creation of excessive turbidity. This regulatory program also ensures that fishing pier construction and operation causes no harm or damage to protected wildlife species or important marine resources, including corals, seagrasses, mangroves, or habitat for manatees or marine turtles. When a JCP is issued for the construction and operation of a gulf or ocean fishing pier, various general and specific permit conditions are imposed to ensure the protection of water quality, and to ensure the protection of marine resources. Each JCP includes specific wildlife protection measures.

11.1 – Reefs, Hard Bottom, and Seagrasses

The beach and nearshore profile adjacent Florida's gulf and ocean fishing piers is subject to fluctuation as the sandy sediment erodes and accretes. This constant state of sediment flux observed along the beach and throughout the littoral zone creates a marine environment that is not susceptible to significant impact by pier construction. However, in some areas of the Florida coastline there are marine features and resources that are protected and need to be avoided by pier construction. Most important among these are coral reefs, worm reefs, exposed hard bottom substrate, and seagrass beds. Wetlands, such as intertidal grasses and mangrove, are typically resources to be avoided on interior tidal shorelines, but these habitats are generally not present for gulf and ocean front pier construction along Florida's barrier island coast. Wetland avoidance is, however, an important factor for consideration along the Florida Keys, the Big Bend Coast between Tallahassee and Tampa, and along some of the coastal inlets' shorelines.

Coral reefs, and isolated patches of live coral, may be encountered along the Florida Keys, which is an elongate, arcuate archipelago over 220 miles in length from Soldier Key at its northeast end near Miami, southwest to the Dry Tortugas. Various piers and docks exist along the Keys shorelines fronting the Straits of Florida, and although this document does not specifically address the design of these structures, they typically have the greatest potential for impacting resources such as corals or seagrasses (**Photo 56**).

Photo 56. White Street Pier, Key West [BBCS Photo Files]

As seen by the dark patches in the waters adjacent the White Street Pier in Key West (**Photo 56**), seagrasses may be encountered along the Straits of Florida shoreline. Seagrasses may also be encountered along some of the low energy barrier beaches, such as Key Biscayne on the southeast Atlantic Ocean coast, along the southern Collier County beaches in southwest Florida, along the mainland coast of Gulf County in northwest Florida, and along Mashes Sands and Shell Point in Wakulla County. Two gulf fishing piers were authorized by the State of Florida in Gulf County between Port St. Joe and Beacon Hill in 2006, but they have not yet been constructed. One of these piers had to be sited with a dogleg in the alignment to avoid seagrasses growing in the nearshore.

As with Florida Keys piers, this document does not specifically address pier construction along the Big Bend Coast of Florida between Tallahassee and Tampa. While the design factors for open gulf piers on the Big Bend Coast should follow the general guidance discussed in this document, nearshore seagrasses as well as shoreline wetlands should be avoided. Examples of piers on this coast are seen by the old (currently closed) timber pier in the Town of Cedar Key, and the more recent concrete pier at Keaton Beach (**Photo 57**).

Photo 57. Cedar Key Pier (left) and Keaton Beach Pier (right), Big Bend Coast

Another protected resource to be avoided by fishing pier construction is Sabellariid worm reef. Sabellariid worm reefs are found at several locations along the Atlantic Ocean coast of Florida (Kirtley and Tanner, 1968). Sabellariids are tiny marine worms that depend on large quantities of sand sized particles suspended in the water to build protective tubes and form large colonies over any hard substrate, natural or man-made, in turbulent, sediment-laden shallow coastal waters. At one of these locations at the south end of Hutchinson Island in Martin County, Florida, the former Seminole Shores Pier (now gone) extended across a Sabellariid worm reef known as the Bathtub Reef (**Photo 58**).

Photo 58. Exposed Sabellariid worm rock, Bathtub Reef, Hutchinson Island [BBCS Photo Files]

Even though fishing pier construction should avoid Sabelleriid reefs, it is of interest that the tiny marine worms that construct this rock will also attach and form large clumps of worm rock around the base of fishing pier piles (**Photo 59**).

Photo 59. Sabellariid worm rock on pile, Sunny Isles Pier [BBCS Photo Files]

11.2 – Water Quality and Turbidity

Water quality issues rarely become a problem with gulf and ocean fishing piers. Even the discharge of fish carcasses into the gulf or ocean in the limited amount that occurs on a public fishing pier has not been shown to cause violations of water quality standards for dissolved oxygen or nutrients. One potential problem, turbidity (reduced water clarity due to sediment suspension) can be elevated with improper construction techniques. During construction, a pier's concrete or timber piles will typically be jetted into the seafloor to at least within a couple feet of the design pile embedment and then driven with a pile driver to meet the design load-bearing criteria (**Photo 60**). Core borings obtained beyond the depth anticipated for pile embedment may provide essential geotechnical data to assess the potential for generated turbidity during pile jetting. This construction technique does not typically cause turbidity plumes that would exceed Florida's turbidity standard of 29 NTUs beyond the 150-meter mixing zone (Chapter 62-302, Florida Administrative Code). JCP's routinely require the following specific permit condition to address turbidity:

> *Best management practices for turbidity control shall be implemented at all times during construction and piling installation to prevent turbidity in excess of 29 NTU's above background levels beyond the edge of a 150-meter mixing zone, pursuant to Chapter 62-302, F.A.C. Methods may include, but are not limited to, the use of turbidity curtains around the immediate project area and staged construction (breaks) to allow turbidity to remain at acceptable levels.*

Photo 60. Pile installation operations, Navarre Beach Pier (left) and Jacksonville Beach Pier (right) [Photos from PBS&J]

11.3 – Debris Issues, Fish Cleaning Stations, and Trash Receptacles

The construction of a large structure intended for human activity over ocean or gulf waters raises numerous issues related to debris that might enter the water from a pier. JCP's carry the construction requirement –

> *During pier construction, there shall be no construction debris discarded into the Gulf of Mexico (or Atlantic Ocean).*

As discussed in the chapter on "Breakaway Decks and Rails", fishing piers are typically designed to allow their decks and rails to be dislodged by extreme breaking wave forces that exceed the basic design conditions of the pier. In anticipation of such conditions that would cause the design breakaway features to become dislodged as well as any other pier damage, JCP's routinely require the following specific permit condition:

> *The permittee shall expeditiously recover any breakaway debris, such as pier deck sections or railing, dislodged from the pier following the impact of major storms. Any storm damage that occurs shall be reported to the Coastal Engineering Section of the Bureau in writing or by email and be supported by photo documentation.*

Following construction, when a pier is open to the public for fishing, the major debris issue becomes the indiscriminant discarding of trash, fishing tackle, and particularly fishing line into the ocean or gulf. Waste receptacles and signs to discourage this activity are required ancillary features to all fishing piers. Trash receptacles and fishing line recycling bins should be strategically located along every pier (**Photo 61**).

JCP's for new fishing piers require the following specific permit condition:

> *During pier operations (for the life of the structure), there shall be no trash, tackle, or fishing line discarded into the Gulf of Mexico (or Atlantic Ocean) from any part of the pier. Large trash and recycling receptacles (including receptacles for recycling of fishing line), with associated signs, shall be installed and maintained at key points along the pier to ensure adequate collection and removal to approved upland disposal or recycling sites.*

In recognition that debris may ultimately be discarded notwithstanding the availability of receptacles and all efforts to prevent it from occurring, JCP's require:

> *All debris on the pier, suspended in the water and on the floor of the Gulf of Mexico within 50 yards around the pier shall be cleaned once per quarter (or every three months). The amounts and types of debris collected shall be reported quarterly to the JCP Compliance Officer.*

A typical feature of fishing piers is a facility to allow anglers to clean their catch. It is understood that the Florida Fish and Wildlife Conservation Commission regulates the size limits on numerous popular saltwater game fish and restricts the cleaning of these regulated species until the angler reaches his/her final destination for storage or consumption, and therefore, these regulated species may not be cleaned on the pier. The rationale for this is that once cleaned, particularly with the head and tail removed, state wildlife officers are unable to ascertain the length and therefore the legality of a fish caught. There are, however, a greater number of species routinely caught by pier anglers that do not have regulated size limits.

Fish cleaning stations are an acceptable amenity with public piers and their use is preferable to indiscriminant fish cleaning on the deck or rails where blood stains and fish debris become attached and attract sea birds. Attracting sea birds such as gulls, terns, and pelicans to the pier deck during fishing activity endangers them to the potential of fishing line entanglement. Fish cleaning stations are typically simple sturdy tables constructed and attached to the pier and having running water and PVC drain pipes that will discharge directly into the gulf or ocean (**Photo 62**). There have been reports indicating that feeding fish-cleaning debris to birds can be hazardous to the health of the birds. The exposed bones on fish carcasses discharged from fish cleaning tables can become lodged in the throats of birds that ingest these carcasses. Therefore, in order to minimize this hazard, the drain pipes from the fish cleaning tables should terminate below the surface of the water. JCP's include the following conditions on fish cleaning tables:

> *The fish cleaning station(s) shall include 6-inch PVC drainage pipes that extend at least two feet below MLW. Each fish cleaning station shall include a sign directing users to dispose of fish parts (especially bones) through the drainpipe rather than throwing this material into the open water. These drains shall only be used for fish cleaning debris and unused bait. All other waste material shall be placed in the trash receptacles located on the pier.*

Photo 62. Fish cleaning station, Jacksonville Beach Pier [Photo by Trey Hatch, BBCS]

11.4 – Protected Wildlife Species

The construction and operation of fishing piers has the potential to impact protected wildlife species. Pier contractors need to take special precautions to prevent any adverse impacts to protected species during construction. During pier operation specific measures are required to ensure protected wildlife species are not harmed. JCP's include the following specific conditions:

The permittee shall install and maintain informational displays on the pier that list the appropriate procedures and wildlife rescue/rehabilitation contact(s) in the event that protected species (marine turtles, pelicans, etc.) are hooked or entangled in fishing line. The following shall be made available to the public all times:

1. *Assistance from a qualified pier attendant to retrieve and safely release animals that are not intended to be caught (i.e. turtles, birds, and restricted fish).*
2. *Nets capable of lifting unintended catches to the deck of the pier to facilitate release, and then to lower the animals for safe release.*
3. *De-hooking devices to aid in the safe release of unintentionally hooked animals.*

11.5 – Sea Turtle Protection

Florida's beaches are the primary nesting habitat for endangered marine turtles (**Photo 63**). Sea turtles are protected by the Endangered Species Act and harming or disturbing them in any way is prohibited. A sea turtle "take" is defined in the Marine Turtle Protection Act, Section 370.12, Florida Statues, as any act that actually kills or injures marine turtles by significantly impairing essential behavior patterns such as breeding, feeding, or sheltering. Any sea turtle take is required to be reported to the Florida Fish and Wildlife Commission by the permittee or pier attendant within 14 days of the incident. This report shall contain the cause of take, location, species and final disposition of the turtle.

Photo 63. Sea turtle, Navarre Pier

If pier construction occurs during the period from May 1st through October 31st(March 1st through October 31st in southeast and southwest Florida), early morning surveys for sea turtle nests shall be conducted daily from May 1st through September 1st. JCP's require the following:

- *No equipment or materials shall be stored seaward of the dune crest or rigid coastal structure in marine turtle nesting habitat during the marine turtle nesting season, May 1st through October 31st.*
- *It is the responsibility of the permittee to ensure that the project area and access sites are surveyed for marine turtle nesting activity. All nesting surveys, nest relocations screening or caging activities, etc., shall be conducted only by persons with prior experience and training in these*

activities and who is duly authorized to conduct such activities through a valid permit issued by the Florida Fish and Wildlife Conservation Commission (FWC), pursuant to Florida Administrative Code 68E-1.

- *Nesting surveys shall be conducted daily between sunrise and 9 a.m.*
- *From May 1st through September 1st, the contractor shall not initiate work until daily notice has been received from the sea turtle permit holder that the morning survey has been completed and all marine turtle nest protection measures have been completed.*
- *Nests deposited in the project area shall be marked and left in situ unless other factors threaten the success of the nest. The turtle permit holder shall install an on-beach marker at the nest site and a secondary marker at a point as far landward as possible to assure that future location of the nest will be possible should the on-beach marker be lost. A series of stakes and highly visible survey ribbon or string shall be installed to establish an area of 10 feet radius surrounding the nest. No activity shall occur within this area nor shall any adjacent construction activity occur that could result in impacts to the nest. Nest sites shall be inspected daily to assure nest markers remain in place and the nest has not been disturbed by the construction activity.*

Prior to completion of construction, a permittee is required to submit a turtle protection plan to the FWC for review and approval. At all times during pier operation, a pier attendant is required to be present who is familiar with the approved turtle protection plan and available to implement the plan.

The permittee is required to post at least four signs in prominent areas of the pier stating that patrons shall notify the pier attendant if a turtle is caught. The signs are also required to detail the National Oceanic and Atmospheric Administration's Fisheries safe fishing practice guidelines for sea turtle protection. These guidelines are as follows:

1. *Do not cast your line where sea turtles are surfacing to breathe.*
2. *If you hook or entangle a sea turtle on your line, contact the pier attendant immediately.*
3. *The pier attendant shall gently attempt to retrieve the turtle in accordance with the approved marine turtle protection plan. If required, cut the line close to the hook and remove line that has become entangled around the turtle. Avoid the turtle's mouth and flipper claws; use blunt scissors/knife to cut line.*
4. *Do not lift the turtle above the water by pulling the line (this will result in further injury). If the distance to the pier from the water is too large to bring the turtle up safely using a lift net, cut the line as short as possible to release the turtle.*
5. *Turtles with serious cuts and or ingested or deeply imbedded hooks need veterinary care.*
6. *Injured sea turtles shall be kept wet and in the shade and a qualified veterinarian (or other permitted and qualified person as determined by the FWC) shall be immediately notified. The person who will be responsible for the veterinary care should be able to be on site within 30 minutes.*

Bright lights along the beach at night adversely impact the nesting habits of marine turtles and their hatchlings. Fishing pier lights are permitted to provide light to the pier's deck area without casting their beams in a manner that would disrupt sea turtle nesting or disorient hatchlings when they emerge from their nests. **Photo 64** shows the newly constructed Navarre Pier at night with lights illuminating only the pier's deck in contrast to the ambient moonlight.

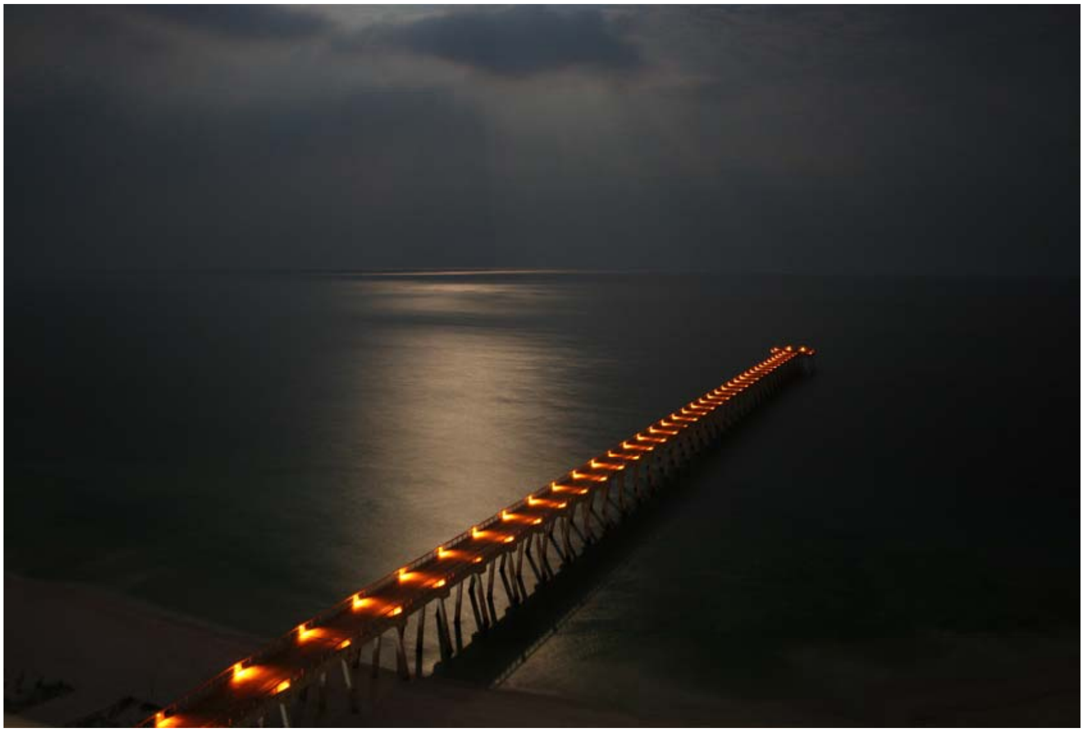

Photo 64. Turtle friendly lights on the new Navarre Pier [Photo by PBS&J]

The installation of special low pressure sodium light fixtures with a maximum of 18 watts on a pier is recommended to help reduce the chances of marine turtles and their hatchlings from becoming disoriented. Typically, new fishing piers are constructed with concrete bollards with seaside shields and low pressure sodium or light emitting diode lamps with a maximum of 18 watts (**Photo 65**). Stanchion or pole-mounted light fixtures are generally not authorized.

Photo 65. Concrete bollard with turtle friendly light, Russell-Fields Pier, Panama City Beach

Special permit conditions are included on all JCP's to regulate pier lighting during construction and operation as follows:

- *All project lighting during construction shall be limited to the immediate area of active construction only and shall be the minimal lighting necessary to comply with U.S. Coast Guard and/or OSHA requirements.*
- *Lighting on the pier (following construction) shall be limited to bollards with seaside shields and low pressure sodium (LPS) with a maximum of 18 watts or amber light emitting diode (LED) lamps (> 580 nanometers).*
- *All permanent exterior lighting shall be installed and maintained as depicted on the approved lighting fixture schedule and cut sheets stamped "FISH AND WILDLIFE CONSERVATION COMMISSION APPROVED LIGHTING PLAN". The lighting plan must be approved by FWC prior to construction.*
- *No additional exterior lighting is authorized on any structure or in the landscape in the project area or adjacent upland areas such as parking lots.*
- *There may be a decrease in the wattage of each approved lamp and a decrease in the total number of each fixture without submitting a modified lighting plan for review and approval. However, if for any reason a fixture or lamp is changed to a different type, manufacturer or catalog number, or if the location of any fixture is changed, an amended lighting plan shall be submitted for review and approval by FWC prior to installation.*

- *The permittee shall update any existing noncompliant exterior lighting associated with the pier (including the pier parking lot and upland building) so it complies with FWC sea turtle friendly lighting guidelines.*
- *If any of the lights from the pier, parking lot or upland buildings become visible from the beach or disorient nesting or hatchling sea turtles at any time, they shall be modified such that they are no longer visible from the beach.*

Marine turtle nesting activity is required to be monitored along the beach within a quarter-mile radius of the pier for three years following completion of pier construction. Any nest within this radius is required to be monitored during emergence, with the hatchlings being monitored in accordance with approved FWC protocol for disorientation events in order to determine if they are attracted to the pier (**Photo 66**). If this monitoring shows a correlation between the pier lighting and hatchling disorientation, modifications to the pier lighting may be required, per FWC's instruction.

Photo 66. Emergent sea turtle hatchlings [BBCS Photo Files]

11.6 – Other Environmental Permit Conditions

There is always the remote possibility that the site of a proposed new fishing pier is located over the sunken remains of a historically important ship wreck or other archeological artifacts. This is an inherent possibility with any coastal construction, and therefore all JCP's are provided with the following General Permit Condition:

> *If historic or archaeological artifacts, such as, but not limited to, Indian canoes, arrow heads, pottery or physical remains, are discovered at any time on the project site, the permittee shall immediately stop all activities in the immediate area that disturb the soil in the immediate locale and notify the State Historic Preservation Officer and the Bureau of Beaches and Coastal Systems (JCP Compliance Officer). In the event that*

unmarked human remains are encountered during permitted activities, all work shall stop in the immediate area and the proper authorities notified in accordance with Section 872.02, Florida Statutes.

The State of Florida recognizes the need for coastal scientific data which is used by the public and various government agencies for a vast array of reasons including weather forecasting, wave conditions for swimming and surfing, rip tide and health advisories, water quality conditions, beach erosion studies, ocean tide conditions, storm tide studies, and numerous other uses. Because fishing piers provide a stable platform projecting across the coastal littoral zone on sovereign submerged lands of the State of Florida, they are ideal structures from which to mount scientific instrumentation.

Photo 67. NOAA tide recorder and weather station

All JCP's for fishing pier construction have the following Specific Permit Condition:

Following completion of construction, the permittee shall provide access on or about the pier to Department employees for the purpose of conducting compliance inspections, post-storm damage assessments of the pier and beach or data acquisition. Sufficient space shall be provided for the installation and maintenance of scientific instrumentation such as those used to record tides, waves, sediment, temperature, turbidity, water quality, meteorology, hydrology, and hydrographics.

The environmental effects of pier construction are site specific and depend upon the construction methods employed. Environmental permit conditions are subject to change, so any pier designer, permit applicant, or pier construction contractor should consult with the Florida Fish and Wildlife Conservation Commission and the Florida Department of Environmental Protection, Bureau of Beaches and Coastal Systems for the latest guidance.

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