Securing Cranes for Storm Wind: Uncertainties and Recommendations

Patrick McCarthy* and Feroze Vazifdar**

*PE, Associate, Liftech Consultants Inc.; 300 Lakeside Dr., 14th Floor, Oakland, CA 94612; Tel: 510-832-5606; pmccarthy@liftech.net
**SE, Vice President, Liftech Consultants Inc.; fvazifdar@liftech.net

Background

Storm wind is one of the few forces that, although considered in dockside container crane design, still causes significant damage—even collapse. See Figure 1. Depending on the geographic region, large tropical storms are called typhoons, cyclones, or hurricanes, even though the mechanism driving the storms is the same. This paper refers to all such storms as hurricanes, and although it focuses primarily on the design of systems to secure dockside container cranes against hurricanes, it also discusses differences in storm wind stowage requirements for cranes in regions not prone to hurricanes.

In almost every hurricane-related crane collapse that we have investigated, failure initiated in the tiedown and/or stowage pin systems. Several crane structures we have designed have experienced hurricanes exceeding the design wind speeds, yet have performed well. None of the over 1,000 cranes conforming to our force coefficients have failed due to an initial overstress in the frame structure. Of the crane failures we have investigated, the stowage pin system and tiedown wharf bracket, link plates, and turnbuckles are the weak links. In fact, the strength of the tiedown system is sometimes only a small fraction of the crane structure strength.

Figure 1. Hurricane-related crane collapse, 2003

Crane Stowage System Components

To secure a crane against storm wind, the crane must not be allowed to become unstable or run away, and must also be designed such that the securing devices do not fail.

Nomenclature

Figure 2 shows typical securing components and nomenclature used in container crane design.

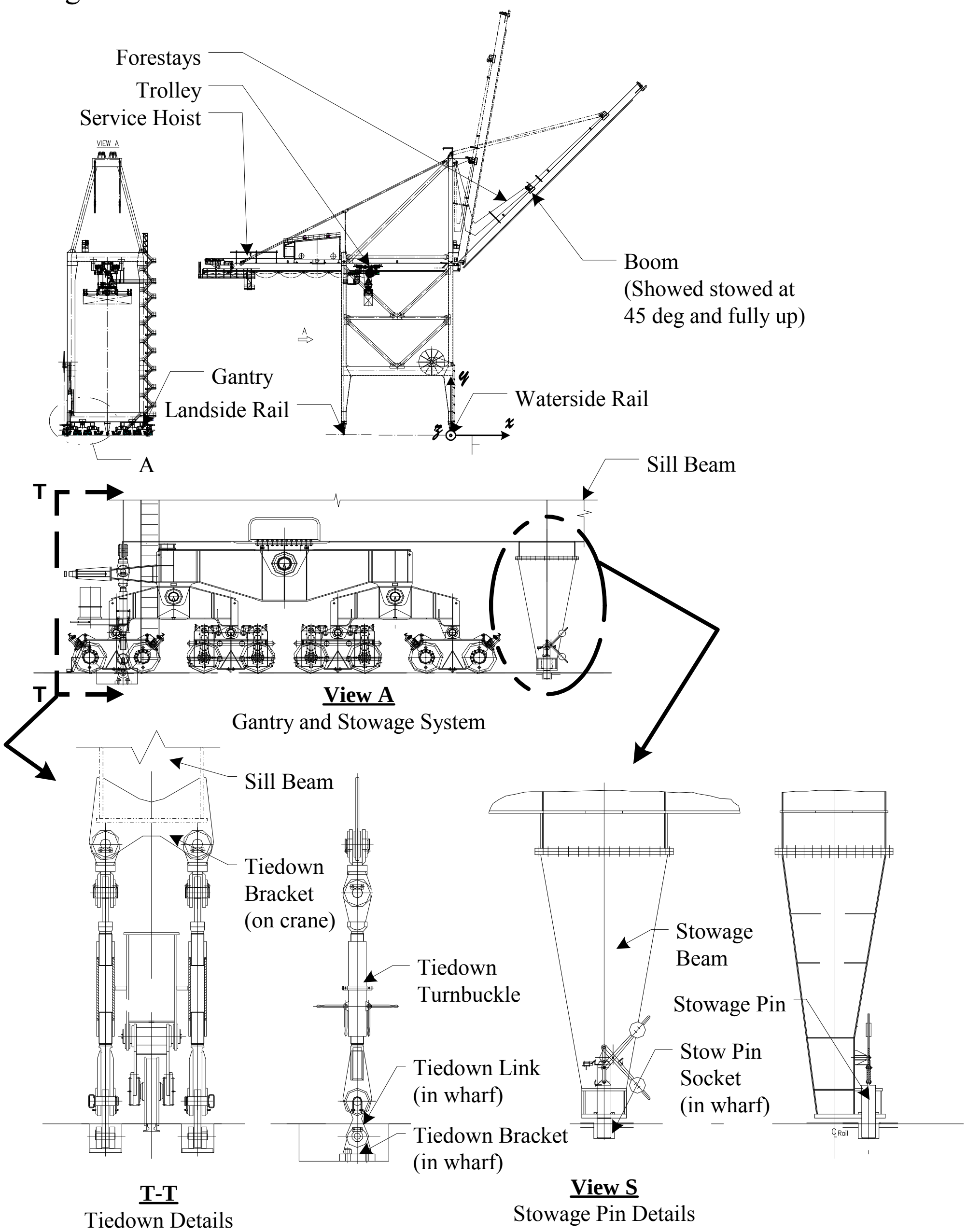

Figure 2. Crane nomenclature

Uplift Prevention System

In hurricane prone regions such as the East and Gulf coasts of the United States, unballasted cranes are unstable in the design hurricane wind. Cranes in non-hurricane-prone regions may or may not be stable in storm winds, but typically will not have tiedown systems, rather ballast will be added. For cranes in hurricane regions, adding ballast is not practical due to the amount of weight needed, which would result in high wheel loads and increased energy requirements for gantrying the crane. Instead, most cranes in hurricane regions use tiedown systems to prevent corner uplift when the crane is not in operation.

Tiedowns need to be tightened properly to prevent the gantry wheel flanges from lifting above and disengaging from the rail. If one corner lifts, the sill beam might move inboard, doubling the lateral load applied to the opposite rail gantry system.

Determining Corner Uplift Forces

Currently, there is no standard method for designing tiedown systems, or even calculating the "required" uplift force.

Wind Force Calculation

Depending on the geographic region, a specified wind code, such as ASCE 7-02, BS, DIN, FEM, or JIS, applies. Regardless of the code, however, the following equation, in some form, is used to calculate wind forces for each crane component:

$F = C_{fi} \times A \times (q_z G)$, where

C_{fi} = shape coefficient(s), where "i" is the direction,

A = projected area in the "i" direction, and

$(q_z G)$ = velocity pressure at height z, including gust, G.

$q_z = 0.5 \times \rho \times V_z^2$ (Bernoulli's equation), where

ρ = air density and

V_z = wind speed, which varies with probability of extreme wind at a site, height at which wind speed is measured, size of the components, and site exposure.. The variation of wind speed with height is referred to as the "wind profile." See Figure 3.

The shape coefficients tabulated in most wind codes are not meant for use with structures such as cranes. The normal industry standard for determining wind forces on container cranes is to compute wind forces on individual crane components. Wind force coefficients are derived from either engineering references or wind tunnel testing of a scaled crane model. It is neither economic nor practical to perform wind tunnel testing for every crane that is manufactured. Hence, the designer has to rely on results from previous wind tunnel studies on similar cranes. Base reactions calculated from the design force coefficients should be equal to or greater than reactions obtained from the wind tunnel test. Shape coefficients should take into account drag, shielding, and angled wind effects.

The appropriate maximum design wind speed at a given site is highly uncertain. In many parts of the world, the wind recording stations are too far apart and may not have sufficient time records to give an accurate portrayal of wind speed at the site of interest. Because wind pressure varies as the square of the speed, the effect of errors in wind speed are amplified. For example, a 10% error in wind speed results in a 21% error in wind pressure. However, this could result in an error in the tiedown uplift force of 100% or more! As an example, consider a crane "O-Frame" with a dead load, D, applied concentrically, a wind load, F_{wind}, applied at a height, h, a main equalizer spacing, B, and a distance between the tiedown and the opposite main equalizer pin, A, as shown in Figure 3. The ratio, γ, of the overturning moment to the righting moment is:

$$\gamma = \frac{F_{wind} h}{D \frac{B}{2}}.$$

Let "e" be the fraction error in the wind pressures. For instance, if the actual wind speed is 10% higher than the speed used in the calculations, "e" would be 1.21, or the ratio of the speeds squared.

The ratio of the actual tiedown force with respect to the calculated tiedown force is shown in Figure 3 and as follows:

$$\frac{F_{Tiedown,Actual}}{F_{Tiedown,Calculated}} = \frac{\frac{1}{A}\left[(1+e)F_{Wind} h - D\frac{B}{2}\right]}{\frac{1}{A}\left[F_{Wind} h - D\frac{B}{2}\right]} = \frac{(1+e)\gamma - 1}{\gamma - 1}$$

Of course, this is a simplified example, since it does not take into account frame stiffness or the affect of angled wind, but it shows how a slight error in the wind speed can amplify the calculated tiedown uplift forces. From this relationship, though, we can see that as the overturning moment becomes much greater than the righting moment, the error in the tiedown will approach the error in the wind pressure, "e." However, when the overturning moment is less than two times the righting moment, a slight error in the wind speed will result in a large error in the tiedown force. As an example, if the overturning moment is 40% greater than the righting moment (γ = 1.4), and the error in the wind pressure, "e," is 21% (10% error in V), the error in the calculated tiedown force is approximately 75%. Recent super-Panamax cranes designed for hurricane regions have overturning ratios, γ, ranging from 1 to 2.5 and 2 to 5 for the landside and waterside frames, respectively. The overturning ratio for the landside is typically small compared to that of the waterside and is in a range where it could be very sensitive to slight errors in the wind speed. We, therefore, recommend to design the landside tiedowns for a minimum uplift force of 50% of the waterside uplift force, even if the calculated uplift is much lower.

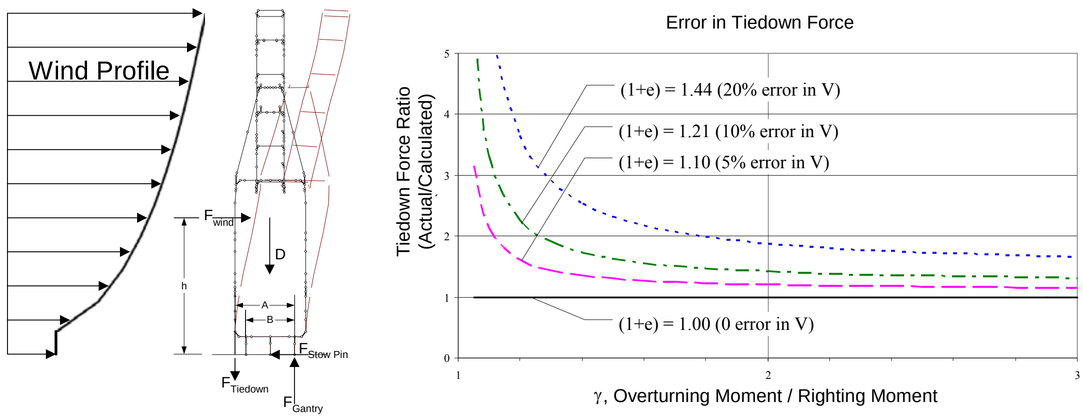

Figure 3. Error in Tiedown Force

An error in the dead load will have a similar effect. We recommend to weigh the completed crane, boom, and trolley to confirm the calculated tiedown and ballast requirements.

A probable maximum wind speed for a given time period, known as a "mean recurrence interval," or MRI, is used for the design of cranes. MRIs are based on statistical analyses of maximum wind speed records at certain weather stations. Typically, a 50-year MRI is used in container crane design. It should be noted that for a 50-year MRI, there is a 64% chance that the design wind speed will occur in 50 years, a 40% chance in 25 years, and even a 2% chance in the first, or any given, year. Therefore, is it likely that the crane will experience the design wind in it's lifetime.

Because wind is turbulent and fluctuates over time, the measure of wind speed depends on the method by which the wind speed is averaged. The relationship between wind speed and gust duration varies for different exposures. Averaging wind speed over shorter intervals results in a higher wind speed. Structures are sensitive to the smallest size gust that will envelop the structure. The larger the structure is, the less sensitive it is to gust. For the design of crane tiedowns, where the gust would have to envelop the entire crane, gust durations of approximately five to ten seconds are appropriate. For calculating the force applied to crane components such as the machinery house or boom, gust durations of one to five seconds may be more appropriate. The ASCE 7-02 code uses a "3-second gust" wind speed as the basis for the "basic wind speed," measured at 10 m above the ground. ASCE codes prior to 1995 used "fastest mile" or "mean-hourly" wind speeds. Since these definitions are based on longer averaging intervals, the speeds are lower. However, the "gust" and other factors tabulated in these codes were higher to compensate. Although the wind speeds appear to be widely different between different ASCE 7-xx wind codes, the calculated forces do not vary significantly in hurricane regions between the different code editions. It is important not to use load factors from one code with wind speeds from another code.

Calculation Method

Once the wind forces are calculated in each direction, the forces must be applied to the crane to calculate the overall force on the structure, and to calculate the corner uplift forces. The best way to calculate the uplift force is to use the output of a finite element analysis of the crane structure with the applied wind loading. The calculated uplift should be taken at the most adverse wind angle.

Oftentimes, manufacturers will use a simple grid based on the crane support geometry to calculate the corner reactions. This method is not accurate for forces in the gantry travel direction since it neglects the stiffness of the crane and the boundary conditions. The uplift forces calculated in this manner are often significantly lower than those obtained through the finite element analysis.

Load Combinations

Reasonable safety factors against failure must be specified. For strength design of the crane structure, the unfactored wind load is applied to the structure in the stowed position. An increase in the allowable stress of 1.4 to 1.5 over the operating condition is typically allowed. As mentioned earlier, however, most collapses are usually not caused by an initial failure of the crane structure itself, but rather by failure of the securing system.

As mentioned above, the maximum 50-year MRI storm wind is likely to occur during the life of the crane. In addition, the shape factors are approximate and, therefore, the calculated wind force is also approximate. For stability, then, it makes sense to factor the wind load. The storm wind combination load factors vary depending on the governing code. Table 1 shows our recommended minimum combination load factors for the storm condition. Since most ports do not secure the tiedowns unless a hurricane is projected, a lesser storm wind combination, SC1, is recommended in addition to the hurricane storm wind, to prevent instability and damage during a non-hurricane wind storm. Ballast may be required for this condition even if tiedowns are used for SC2.

The loads obtained from the Table 1 combinations are used to calculate the landside and waterside tiedown uplift with the wind applied at the most adverse angle. It is not reasonable to use unfactored wind loading for the design of these members, because an unfactored wind may not even produce uplift. This would result in a grossly underdesigned tiedown system.

The ASCE 7-02 code uses a 1.6 load factor for the storm wind, but has decreased the wind pressure by 15% to account for the unlikelihood of the design storm hitting the crane at the most adverse angle. Previous versions of the ASCE 7 code used a 1.3 load factor with no reduction for angled wind. The equivalent factor for the 2002 code is approximately $1.6*0.85 = 1.36$. One way of looking at this load factor is that the design wind speed at any angle is increased by a factor of $1.36^{0.5}$, or 17%, which effectively increases the wind from a 50-yr MRI to a 200-yr MRI. Most wind codes outside of the U.S. use a 1.2 or 1.3 load factor for the storm wind. It is important to consider the MRI of the design wind used in the code before determining the appropriate load combination factor. Engineering judgment is required.

Table 1. Recommended Minimum Stability Load Combinations

Load	SC1[1]	SC2[2]
Dead Load[3]	0.9	0.9
Stowed Moving Load[3]	0.9	0.9
Wind Load 20-year MRI	1.0	
Wind Load 50-year MRI		1.6[4]
Tiedowns secured?	No	Yes
Corners allowed to lift	1	0

Table Notes:

1. *For load combination SC1, the crane is recommended to remain stable without tiedowns for a 20-year MRI wind. For angled wind, one leg is allowed to lift off, but the stowage pin should be designed appropriately so as not to disengage from the wharf socket.*

2. *Used for calculating tiedown uplift forces and for ballast determination if tiedowns are not used.*

3. *May be increased if weighed. Engineering judgment is required.*

4. *ASCE 7-02 uses 1.6. For most codes outside the U.S., we recommend a 1.3 load factor. See load combinations discussion, above.*

Tiedown Strength Requirements

We recommend the breaking strength of the turnbuckle be designed for 2.5 times the tiedown uplift force calculated using load combination SC2 from Table 1. This in part compensates for possible uneven load distribution between tiedowns if there is more than one per corner. In addition, since the turnbuckle is a mechanical, high-strength, threaded component, it may fail in a brittle manner, unlike the main crane structural components. From our experience, the crane structure is not the "weak link." We recommend that the wharf attachment be designed to the same loading and safety factor as the crane tiedown components.

We recommend to consider proof testing the tiedown mechanical components to 125% of the calculated tiedown force. The turnbuckle should show no permanent deformation and the screw should turn freely after the test. Ideally, we would like to see a proof test with the tiedown and wharf attachment assembly, but this is not practical.

We, furthermore, recommend that structural components local to the tiedowns, such as the eye connection to the crane and the tiedown link bars be designed to an allowable stress of ($0.9*F_{yield}$), where F_{yield} is the yield stress, using the same tiedown force.

Stowage Pin System

The stowage pin system resists forces parallel to the gantry rails. If the stowage pin fails, the tiedowns may also fail, since tiedowns are only designed to take uplift loads. This typically results in a runaway crane, which can either collide with an adjacent crane or run into the bumpers at the end of the crane railway. Either scenario can result in significant damage or a total collapse of the crane.

Some manufacturers routinely attach the stowage pin under the main or intermediate equalizer beams. Attaching the stowage pins to the equalizer beams results in prying, which increases wheel loads. Placing the stowage pins at only one corner per rail also presents more problems for cranes without tiedowns, since uplift of only one corner can more easily result in the stowage pin disengaging from the wharf socket. We recommend placing the stowage pins under the center of the landside and waterside sill beams. This design necessitates the use of a stowage pin beam, as shown in Figure 2, but eliminates the disadvantages of prying and increases the likelihood of the pin staying in the socket. The stowage pin beam can be designed to incorporate a “checker’s cab” as well.

Cranes without tiedowns are usually designed to the same overturning safety factor as for the tiedown stability case. However, in this instance, they are checked for overall crane stability, with the wind blowing either parallel or perpendicular to the gantry rail, but not at the most adverse angle to cause corner uplift. To ensure that the stowage pin does not lift out of the socket, however, the corner uplift for the landside and waterside must be checked with the wind blowing at any angle. For hurricane-prone regions, a similar check should be made for the stability case SC1, as described in Table 1. The stowage pins should be designed to stay inside the sockets for these displacements. The stowage pins should also be designed so that they will not “ratchet” out of the socket if one corner repeatedly lifts off the rail.

To reduce the amount of ballast required, some designers will add “stability stools,” which are blocks between the sill beam lower flange and the main equalizer beam upper flange, directly above the centerline of the outer intermediate equalizer beam pin. This effectively increases the lever arm to resist overturning. To prevent binding due to unevenness in the crane rail and slight deflections in the structure during operation, a gap of 10 mm (0.4 in) is usually provided. Stability stools are only effective, however, if there is no gap. We recommend to consider adding a wedge or shim before a storm arrives, and to pay particular attention to designing the stowage pins against lifting out of the sockets.

Rail-to-Gantry Interface

The interface between the rail and gantry wheels resists forces perpendicular to the gantry rails and downward loads due to wind and dead load. Most gantry systems will adequately transfer the vertical loads to the rail beam. However, special attention must be paid to transmit the lateral forces to the gantry rails.

Expected Hurricane Cost vs. Stowage System Cost

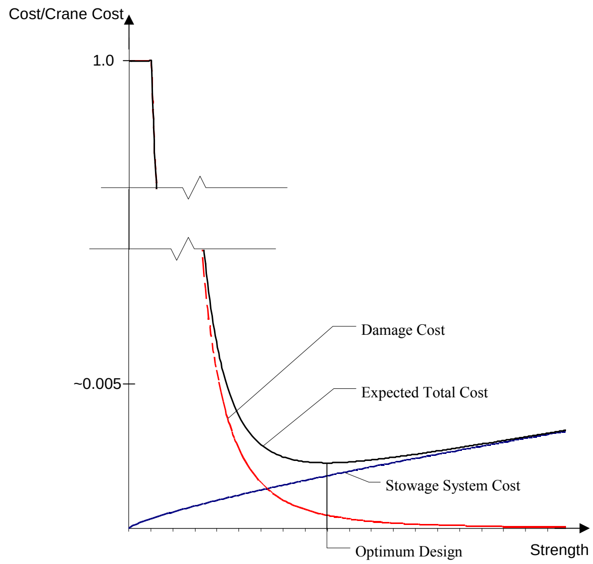

Figure 4. Expected hurricane costs vs. stowage system strength

The expected lifetime cost of a crane is the combination of the initial cost of purchase, maintenance costs, and the cost of repairing damage. As mentioned previously in this paper, the crane will likely experience the design storm wind load during the life of the crane. If the stowage system is not adequately designed to resist these forces, the entire crane could be lost. The expected cost due to a hurricane, then is a function of the likelihood of the crane receiving damage times the cost of that damage. Figure 4 demonstrates qualitatively how a small initial expenditure in the stowage system cost can reap huge rewards in reducing the total expected cost of hurricanes.

Typical Failure Modes

Figure 5 and Figure 6 illustrate typical failure modes for wharf tiedown brackets. Figure 5, discussed by C. Morris in her Port's 2001 presentation, schematically shows a failure due to bending of the bottom bracket plate, whereas Figure 6 shows a recent failure due to insufficient weld capacity in the wharf tiedown bracket.

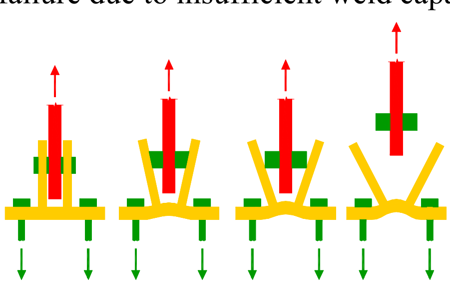

Figure 5. Wharf tiedown bracket failure due to bending

Figure 6. Wharf tiedown bracket failure due to insufficient weld capacity

Figure 7 and Figure 8 illustrate improper maintenance of stowage pin sockets. Figure 8 shows a stowage pin socket where the stowage pin has lifted and disengaged from the socket, resulting in a runaway crane. The initial failure was due to the failed wharf tiedown shown in Figure 6, causing uplift at that corner.

Figure 7. Wharf tiedown bracket failure due to bending

Figure 8. Wharf stowage pin socket failure

Miscellaneous Securing Considerations

In addition to the primary, wharf-level stowage system, it is important to properly design for the stowage of crane components such as the boom, forestays, trolley, and service hoists. Prior to the approach of a hurricane, port personnel should plan for and practice securing the cranes. A securing procedure should be firmly in place. The securing devices on the crane and in the wharf should also be periodically maintained to assure their proper functionality. Oftentimes, the stowage pin sockets are filled with dirt, concrete, and other garbage such that the proper stowage pin depth cannot be obtained. These sockets should be cleaned of all debris.

Conclusions

Storm wind is one of the few forces that still causes significant damage, even collapse, of dockside container cranes. Most collapses, however, are usually not caused by an initial failure of the crane structure itself, but rather by failure of the tiedown and/or stowage pin system securing the crane to the wharf.

This paper discussed the procedure for calculating tiedown uplift forces and the uncertainties involved. If the overturning moment is only slightly higher than the righting moment, a small error in the wind load can result in a large error in the calculated tiedown force. The cost of providing proper stowage systems is far less than the cost of providing inadequate systems, and will not significantly increase the initial crane cost. Typical failure modes and design recommendations for the proper design of tiedowns, stowage pins, and the rail-to-gantry interface were presented.

References

ASCE 7-02, *Minimum Design Loads for Buildings and Other Structures,* American Society of Civil Engineers, 2003

FEM 1.001 3rd Edition, 1987: *Rules for the Design of Hoisting Appliances,* Federation Europeene de la Manutention, Section I, Heavy Lifting Appliances, 1987

Morris, Catherine A. and McCarthy, Patrick W., *The Impact of Jumbo Cranes on Wharves,* Ports 2001 Conference, Norfolk, VA, May 2001