Barbours Cut Terminal - Container Port Wharf Expansion Design

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Synopsis: This paper will present the challenges and unique aspects associated with increasing the capacity of one of the container wharves at Barbour's Cut Terminal to support new Ship-to-Shore (STS) container cranes with gage lengths of 100 ft. (30 m), which was an upgrade from the previous container cranes that featured 50-ft. (15 m) gage lengths. The design criteria included achieving an additional 50 years of service life from the existing elements and new elements; therefore, the assessment results and techniques used for service life modeling will be discussed. In the new structural elements, service life modeling was used to determine the necessary concrete mixture characteristics, including use of fly ash and corrosion-resistant reinforcement, to achieve the required service life.

This paper will also discuss the design approach, including the use of springs to represent the soil-structure interaction, for determining the demands on the various components. In addition, the interaction between the new structure and existing structure and the resulting torsion will be discussed. Finally, various lessons learned from using strut-and-tie modeling, including the relative stiffness of the chord elements and need for three-dimensional modeling, will be summarized.

Keywords: container crane, rehabilitation design, service life modeling, strut-and-tie modeling (STM), wharf

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INTRODUCTION

The expansion of the Panama Canal was completed in June 2016 after nine years of construction. With the capacity to accommodate larger ships carrying more cargo, the new locks caused a shift in world trade routes, prompting ports and shipping infrastructure upgrades in U.S. cities from New York to New Orleans. To date, the largest project investments have been along the Gulf Coast. These changes in shipping commerce have necessitated larger Ship-to-Shore (STS) cranes at terminals to unload and load container cargo with increased efficiency.

In preparation of the Panama Canal expansion, Port Houston began renovations in 2012 to re-develop and modernize the container wharves at Barbours Cut Terminal (BCT). The key modernization of the terminal was the installation of new STS container cranes to support the larger ships coming through the Gulf of Mexico (Figure 1). BCT was constructed in the mid- to late-1970s and consists of six, 1,000-ft. (305 m) long wharves. Each of the wharves are approximately 108 ft. (33 m) wide and are supported across their width by concrete drilled shafts. Due to their age, portions of the existing concrete structure have ongoing corrosion-related distress. While Wharf Nos. 1 and 2 at BCT were rehabilitated between 2012 and 2017, this paper will discuss the assessment and rehabilitation of Wharf No. 3, which has many unique and challenging aspects. The design of Wharf No. 3 differed from the other two wharves, namely that Wharf No. 3 features slightly higher design loads and was designed for an explicit 50-year service life. An overview of Wharf No. 3, including loading considerations, will be described in the next section, followed by a summary of the results from the field assessment and service life modeling tasks. The final sections of the paper provide a discussion of the design and a summary of the conclusions.

Figure 1—New STS container cranes at BCT.

STRUCTURE INFORMATION

BCT is located in Morgan's Point, Texas, on the northwestern end of Galveston Bay, just west of the Houston Ship Channel near Houston, Texas. The gulf coast climate is normally warm and humid for the majority of the year. Mean monthly temperatures range from 54 °F (12 °C) in January to 84 °F (29 °C) in August, and mean relative humidity exceeds 70 percent year round [1-2]. This equates to an Exposure Class F0 according to ACI 318 [3] for concrete not exposed to freezing and thawing cycles.

Galveston Bay is a large estuary and has a mix of seawater and fresh water that fluctuates in composition as tidal and stream flows vary. Data on the quality of the ship channel water are available through the state's surface water quality monitoring program by Texas Commission on Environmental Quality (TCEQ). Chloride data from a station close to BCT (segment ID:2436) is only available from 1973 to 1994. The data indicate annual averages in the range of 4,000 to 8,000 mg/l of chloride. For reference, typical seawater has a chloride content of 19,400 mg/l. The measured chloride levels indicate that channel water is about one-third to one-half the salinity of sea water and should be considered a corrosive environment. This equates to an Exposure Class C2 according to ACI 318 [3], or concrete exposed to both moisture and external chloride source in service.

Wharf No. 3 consists of three units, each 333 ft., 4 in. (102 m) long for a total berth length of 1,000 ft. (305 m). The original design of the wharf accommodated 50-ft. (15 m) gage container cranes, but the rehabilitation of the wharf replaced the existing crane rails with 100-ft. (30 m) gage STS cranes. The existing structural configuration of the wharf necessitated new structural elements to support the larger cranes and increased loads.

The original structure consisted of a flat slab system (typically 16 in. (41 cm) thick with drop panels), supported by 36-in. (91 cm) diameter reinforced concrete drilled shafts (piers) (Figure 2). The drilled shafts were generally spaced at 21 ft., 6 in. (6.6 m) along the length of the wharf, except under the original container crane rails where the spacing was 10 ft., 9 in. (3.3 m). A cement-stabilized fill was specified between the flat slab and wearing surface. The thickness of the fill varied from 2 ft., 3 in. (69 cm) to 2 ft., 9 in. (84 cm) over the width of the wharf. At the landside of the wharf, a steel sheet pile bulkhead wall, along with tie rods and a concrete wale beam, hold back the soil.

The top of the wharf is located approximately 20 ft. (6.1 m) above mean lower low water (MLLW) while the future design dredge depth is approximately -48 ft. (-15 m) MLLW. As such, the drilled shafts along the front of the wharf extend nearly 60 ft. (18 m) in the water unsupported. The new, conventionally-reinforced drilled shafts, with lengths varying up to 180 ft. (54.9 m) along the waterside and 130 feet (39.6 m) along the landside, will be constructed using both permanent and temporary steel casing. Temporary casing will be used to mitigate caving of the in-situ soil during drilling, whereas permanent casing will be used along the waterside to extend the service life of the drilled shafts (see Service Life Modeling section). To place concrete into the reinforced shafts during construction, deep termie and pump truck hosing will be employed. Where applicable, temporary casing will be extracted using large 100-ton track cranes after concrete has filled the lower portion of the shafts.

The rehabilitation concept at Wharf No. 3 was similar to that of Wharf Nos. 1 and 2—at the waterside, the existing crane beam was demolished along with the front portion of the wharf and was replaced by a new crane beam on new drilled shafts. The new crane beam and front portion of the wharf was integrated with the existing structure such that the new crane loads were shared with the existing structure for efficiency; without sharing some of the load with the existing structure, the extent of new concrete structure would have been much greater. At the landside, the existing crane beam was abandoned, and a new crane beam on new drilled shafts was placed behind the existing bulkhead wall. The landside crane beam was connected laterally to the existing wharf structure using new tie beams.

The new structural elements added along both the waterside and landside of the existing wharf are identified in Figure 3. Along the waterside, the new elements consist of a crane beam, drilled shafts, deck beams, a frontal beam, and a thickened deck. The framing was aligned with the new fender system and bollards to resist the higher berthing and mooring forces, respectively. The new drilled shafts are 48 in. (1.2 m) in diameter and spaced at 10 ft., 9 in. (3.3 m) to match the spacing of other drilled shafts in the wharf. Along the landside, a new crane beam, drilled shafts, and tie beams were added. The spacing of the drilled shafts varied, as needed, to avoid the existing tie rods that support the bulkhead wall. The new landside drilled shafts were either 36 in. (91 cm) or 48 in. (1.2 m) in diameter, depending on the varied crane beam span lengths and loading.

A variety of loads were considered for the rehabilitation of Wharf No. 3. In addition to the self-weight of the structural components, the wharf was also designed for a uniform distributed live load of 800 psf (38 kPa) as well as truck and mobile crane loads. Based on discussions with the owner, the uniform live load was not applied simultaneously with the vehicular live loads. The axle loads and impact factors associated with the vehicular live loads are summarized in Table 1.

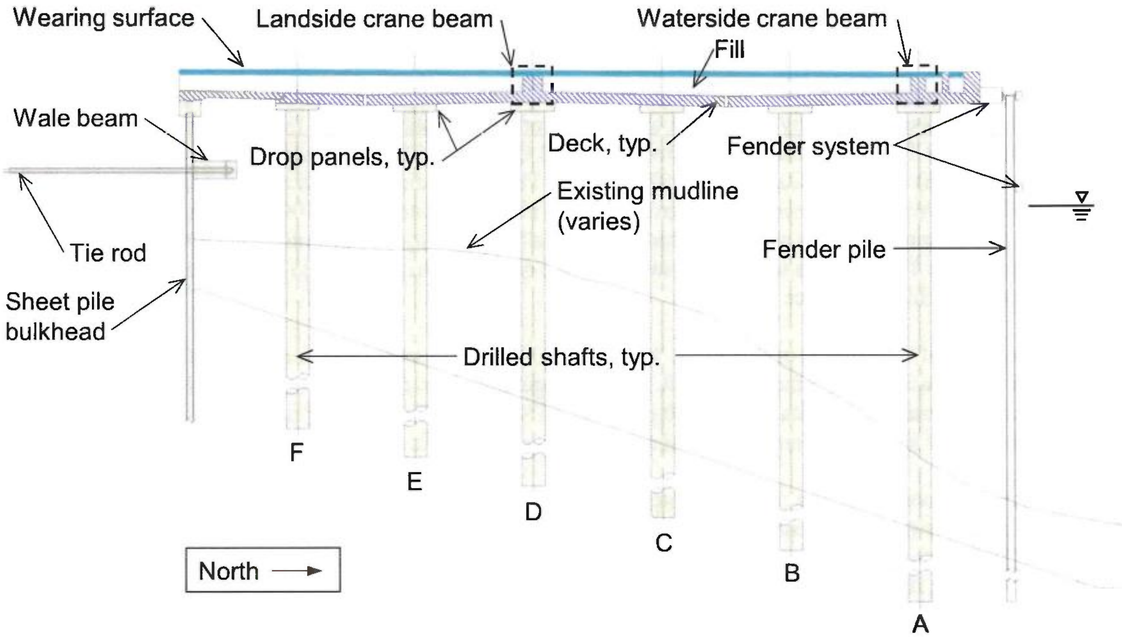

Figure 2—Typical section with original elements identified (Gridlines identified at tip of drilled shafts).

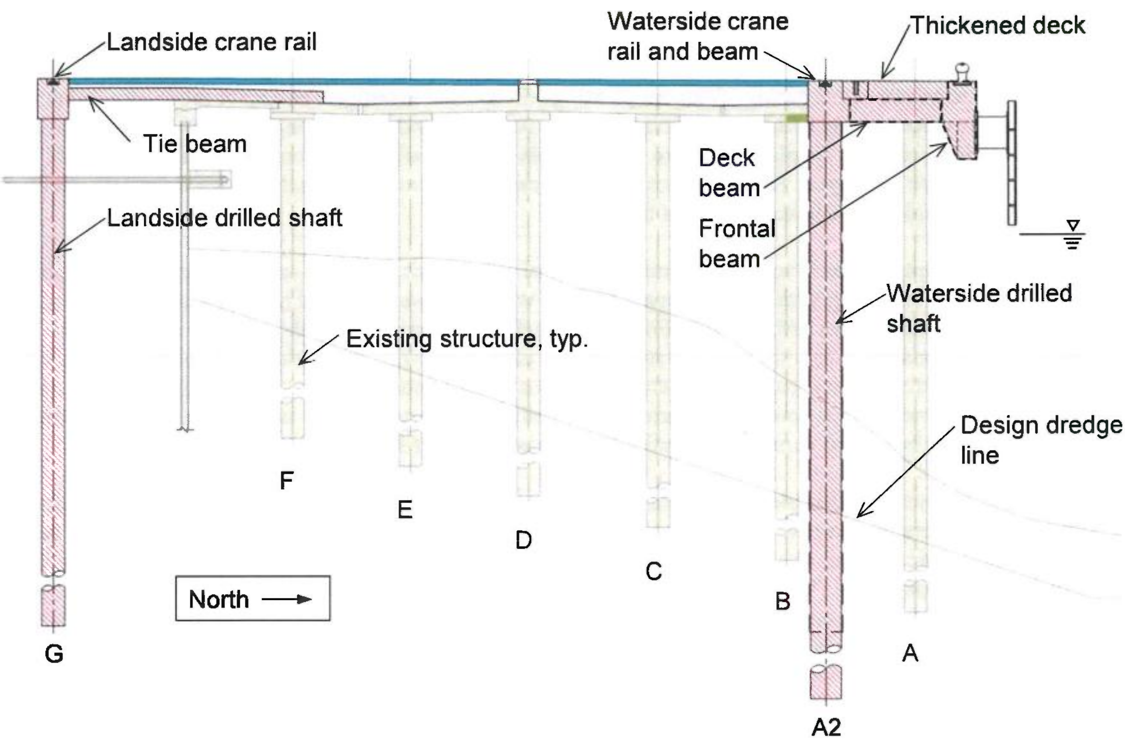

Figure 3—Typical section with new elements identified (Gridlines identified at tip of drilled shafts). Reference Figure 2 for original elements not identified.

The operating wheel loads from the new STS cranes are much larger than the loads associated with the original container cranes at the wharf. The design operating wind speed of the STS cranes is 55 mph (89 km/h). At higher wind speeds, the cranes are placed in a stowage condition where ships are required to leave port, and the cranes are connected to the wharf superstructure through steel tie-down anchorages. The vertical container crane wheel loads associated with normal operation and stowage condition, as well as the horizontal loads from the stowage pins and vertical uplift loads from the crane tie-downs during stowage conditions, are summarized in Table 2. The cranes also induce lateral loads on the crane rails during operation and stowage conditions, and the service-level lateral forces vary from 2.1 kip/ft. (31 kN/m) during operational conditions to 6.2 or 7.0 kip/ft. (90 to 102 kN/m) during stowage conditions.

Table 1—Point Live Loads

Type	Axle Loads		Impact Factor	Notes
	kips	kN		
HS 25	10 | 40 | 40	44 | 178 | 178	15%	(6) driving lanes between rails
Container Lift Truck	21.6 | 302.2 60.7 | 172.5	96 | 1,344 270 | 767	10% 10%	Steering | drive axles, loaded Steering | drive axles, unloaded
Roll Trailer	68 | 90 | 90	302 | 400 | 400	10%	Any location north of Gridline A2
200T Crane	126 | 360 265	560 | 1,601 1,179	10% 10%	Front | rear axles Maximum float reaction

Table 2—STS Container Crane Loads

Type	Waterside Rail•		Landside Rail•		Notes
	kips	MN	kips	MN	
Wheel - operating (vertical)	236 (295)	1.1 (1.3)	187 (259)	0.8 (1.2)	(8) wheels per corner
Wheel - stowage (vertical)	375 (558)	1.7 (2.5)	375 (531)	1.7 (2.4)	(8) wheels per corner
Stowage pins (lateral)	550 (880)	2.5 (3.9)	425 (675)	1.9 (3.0)	Total per rail
Crane tie-downs (vertical)	1,440 (2,300)	6.4 (10)	1,070 (1,700)	4.8 (7.6)	Total per corner

•Factored loads are shown in parentheses

In addition to the structural load requirements, Port Houston specified a design service life extension of 50 years for both the existing and new elements of the rehabilitated Wharf No. 3. The design process to meet a desired service life requires definition of the end-of-life criteria for all major component types in the structure. The end-of-life criteria for the project were developed in consultation with Port Houston, and are summarized in Table 3. These criteria define the amount of corrosion loss that is permitted in various elements before the service life was exhausted. In addition, as will be discussed later, service life modeling was used to define key performance metrics, such as repairs for elements with large crack widths, to achieve 50 years.

Table 3—Minimum Service Life for Components in Scope

Component	Minimum Service Life (Years)	End of Life Criteria
Non-Replaceable Components		
Structural Concrete Elements	50	New: Corrosion initiation of reinforcement on more than 10% of element
Structural Steel Elements	50	Section loss in excess of 10% of cross-section
Foundation Structural Steel (tie-backs, anchor rods, sheet pile wall, etc.)	50	Section loss in excess of 25% of cross-section
Replaceable Components		
Protective Coatings	15	Coating failure on more than 5% of element area
Personnel Access Systems	50	Loss in usability due to weathering

FIELD ASSESSMENT OF EXISTING STRUCTURE

A baseline inspection was performed in July 2017 of the existing wharf in accordance with the Port's Maritime Facilities Inspection and Condition Assessment Program (FICAP). FICAP bears similarities between ASCE 130 [4] and the Federal Highway Administration (FHWA) National Bridge Inspection Standards [5]. Priority and routine follow-up action items were identified during the baseline inspection, and an overall asset condition rating was developed prior to the rehabilitation work. Subsequent to the baseline inspection, an in-depth assessment was also performed, which included detailed mapping of distress by mechanically sounding the concrete surfaces to identify areas of delamination. Material sampling and testing was performed as part of the assessment and included concrete petrographic analysis to assess concrete quality and characteristics and laboratory testing to ascertain chloride profiles, carbonation depth, and compressive strength.

Visual and sounding surveys identified cracks and delamination of some of the concrete elements due to reinforcement corrosion, especially around the waterline on the drilled shafts and the bulkhead wale beam. Spalls that were correlated to construction defects were also identified on some of the drilled shafts. Efflorescence and moisture staining were observed on cracks visible on the deck underside and through the longitudinal expansion joint near the steel sheet pile bulkhead. Corrosion and coating degradation was detected on the bulkhead steel wall, especially around the waterline. Finally, the existing catwalk steel supports, wood decking, and guardrails were severely degraded with associated loss in capacity.

Fourteen locations were selected for an in-depth study to represent the range of elements, conditions, and exposures identified in the visual and sounding survey, and include areas of both sound and unsound concrete. These study locations included two areas on the bulkhead beam, three on the underside of the deck, seven drilled shafts, and two on the drilled shaft caps (caps were required during original construction when a few drilled shafts were abandoned during drilling and new shafts were added on either side of the abandoned shaft). Nondestructive testing was conducted to measure the concrete cover, assess the probability of active corrosion, measure corrosion rate, and evaluate the corrosion risk. Concrete cores and powder samples were also extracted for laboratory testing. Additionally, the thickness of the steel sheet pile bulkhead wall was measured at various locations using ultrasonic testing. Findings of the in-depth study areas included:

- **Drilled shafts:** Electrochemical measurements indicated high corrosion risk on four of the seven drilled shaft study areas. These four drilled shafts also exhibited either vertical cracks or spalls. Concrete cover measurements varied widely around the perimeter shafts, likely due to a combination of ellipticity and potential off-centering of the original spiral reinforcing cage.
- **Drilled shaft cap beams:** Minor visible damage was detected during the overall inspection. However, these study areas were included due to the elevated risk of corrosion identified by the lower concrete cover when compared to the drilled shafts. Moderate to high corrosion risk were identified for these elements, based on electrochemical measurements, with high probability of active corrosion in insolated spots.
- **Bulkhead wale beam:** In-depth assessment of bulkhead beam study areas concluded consistent concrete cover along the vertical and horizontal surfaces of the beam, with evident active corrosion on the underside of the beam and overall moderate risk of corrosion.
- **Deck underside:** Low risk of ongoing corrosion activity was identified generally for the deck study areas. The exception to this was a localized area at a crack with efflorescence near a drain. At this area, an extracted core found corrosion on a bar crossing the crack and the local area exhibited moderate risk for further corrosion.
- **Bulkhead sheet pile:** The bulkhead sheet pile survey determined consistent metal thickness with limited section loss above the wale beam. Below the wale beam and closer to the waterline, metal thickness loss was measured to be less than 10 percent. A cathodic protection system was reported to be installed but was not assessed as part of the project.

Laboratory testing was performed on thirty-five cores extracted from selected areas of the drilled shafts, bulkhead beams, and deck underside. The cores were taken at different heights above MLLW in these elements to investigate different exposure zones (e.g., tidal, splash, and atmospheric). Testing included chloride content, carbonation depth, petrographic examination, and compressive strength. Findings of the laboratory testing were as follows:

- **Chloride content:** The peak chloride contents in the drilled shafts generally varied from 0.3 to 0.5 percent by weight of concrete in the tidal zone, 0.1 to 0.2 percent by weight of concrete in the splash zone, and 0.04 to 0.09 percent by weight of concrete in the atmospheric zone. The profiles from the drilled shaft cap samples were within the drilled shaft profile envelopes. The bulkhead beams had peak chloride contents that varied from 0.04 to 0.1 percent by weight of concrete; whereas, the underside of the deck had peak chloride thresholds of less than 0.02

percent by weight of concrete. Considering normal-weight concrete with a six-sack mix, as determined by petrographic examination, the corrosion risk was determined to be negligible if the chloride content by weight of cement was less than 0.03 percent, low if the chloride content was between 0.03 to 0.06 percent, moderate if between 0.06 to 0.09 percent, and high if greater than 0.09 percent.

- **Carbonation:** Carbonation depths were much less than the cover depth for the investigated elements. As such, carbonation is not expected to be a cause of corrosion-related distress for the next 50 years.
- **Petrographic examination:** Petrographic examination was performed on selected cores. The examined cores were found to be different in composition and mix proportions. No cracks or other signs of distress were detected in the cores, with the exception of one core extracted from a drilled shaft, which had signs of ongoing alkali-silica reaction (ASR). The extent of the ASR was minor.
- **Compressive strength:** Nine core samples were tested for compressive strength, three each from drilled shafts, bulkhead beams, and deck underside. Considering all cores as a group, the equivalent specified compressive strength was 4,650 psi (32 MPa), as determined from [6], which compares favorably to the originally-specified 28-day compressive strength of 4,000 psi (27 MPa).

SERVICE LIFE MODELING

The in-depth field assessment was incorporated into a probabilistic service life model for each typical concrete element. The models for the existing elements were calibrated using the measured concrete properties and chloride profiles. The service life modeling was used to develop an in-depth corrosion protection plan to support the rehabilitation of Wharf No. 3, both for the repair of existing elements and for design of new structural components.

The focus of this paper is on the analysis and design of the new elements; however, the existing elements were also analyzed. Based on service life criteria established by the owner, the remaining service life of the deck exceeded 50 years; however, the drilled shafts had a predicted remaining service life of 37 years, the bulkhead beams had no service life remaining, the drilled shaft cap beams had a remaining service life of less than 5 years, and localized areas in the deck with cracks and/or leaks were not explicitly modeled. Based on those results, the owner elected to repair only the drilled shafts with existing deterioration (i.e., spalls, cracks, delaminations, etc.), which accounted for approximately 15 percent of the 373 existing drilled shafts. Repairs to the drilled shafts involved some limited concrete repair and installation of pile jackets to provide cathodic protection, whereas a fiberglass jacket was detailed for the drilled shaft cap beams. For the bulkhead beams, partial-depth to full-depth concrete repairs were specified along with a metalized coating around the surface of the beam. Finally, at the deck drains where efflorescence and cracks (i.e., leaks) had been identified, a metalizing coating was specified in the vicinity of the drain to mitigate future distress. For the remainder of this section, only the service life modeling related to the new structural components will be presented.

Concrete provides a highly alkaline environment (pH typically at 12-14) around the steel, which results in the formation of a protective passive oxide film on the steel surface. This film protects the steel from corrosion until it is de-passivated, most commonly by an excess of chloride ions or by carbonation of the concrete due to atmospheric carbon dioxide.

A basic approach for corrosion protection of embedded reinforcing steel is increasing the time needed for chloride ions and/or carbonation front to reach the steel depth and initiate the propagate corrosion. This can be attained using adequate concrete cover, good concrete quality, and limiting the frequency and width of cracks.

Specifically, the rate of carbonation and chloride ingress can be reduced using dense, low-permeability concrete, which can be achieved by reducing the water-to-cementitious materials (w/m) ratio and by adding supplementary cementitious materials (SCM), such as fly ash, slag, or silica fume. Reducing the concrete permeability reduces the rate of diffusion of chloride ions and carbon dioxide into the concrete, thereby increasing the time to corrosion initiation.

Another approach to corrosion protection in new construction is to use corrosion-resistant reinforcing materials, such as hot-dipped galvanized or stainless steel. These materials have an increased corrosion initiation threshold, and may also extend corrosion propagation time (due to reduced corrosion rate). Both of these factors will extend the service life of the reinforced concrete element. The use of these corrosion-resistant bars adds a significant initial cost; however, the increased construction cost may be justified depending on the required design service life and considering life-cycle costs for the project. A detailed discussion on reinforcing corrosion and service life modeling for reinforced concrete is provided in [7].

Modeling Approach

To determine the necessary concrete properties used in the design, service life modeling for chloride-induced corrosion was performed using an in-house, diffusion-based model. In consideration of the variability inherent in concrete elements, a full probabilistic modeling approach was used. This approach determines the amount of concrete surface area affected by corrosion-related damage based on statistical distributions of key parameters considered to govern corrosion. This approach recognizes that corrosion is a local process that can develop at multiple locations over time.

Parameters for the model can be conceptually separated into exposures (loads) and resistances to corrosion. "Exposure" parameters included chloride surface concentration and chloride build-up time, each specific to various exposure zones on the structure. Exposure input parameters were based on conditions observed from the field and laboratory testing of the existing structure. "Resistance" parameters included concrete cover; apparent diffusion coefficient; concrete ageing factor; chloride initiation threshold (different for carbon steel, galvanized, and stainless steel bars); and temperature. The analysis included optimizing four potentially different concrete mixes for the new elements, including drilled shafts in water, drilled shafts in soil (landside), deck beams, and the deck. The basis for evaluated parameters included:

- **Exposure zones:** Different exposure zones were considered, including atmospheric, splash, tidal, submerged, and soil. Exposure parameters for the atmospheric, splash, and tidal zones for the waterside drilled shafts were selected based on field and laboratory tests performed. The surface chloride concentrations for the submerged zone were estimated based on the chloride levels in the ship channel water close to BCT. For the soil exposure zone, surface chlorides were estimated based on the chloride ion concentrations detected in soil samples collected as part of a geotechnical investigation of the site.
- **Concrete cover:** Design concrete cover was defined for each element based on discussions with the structural design team. Aligning with the probabilistic approach adopted in the model, distribution parameters were calculated considering a standard deviation equal to the specified tolerance of 0.5 in. (1.3 cm), per ACI 117 [8], divided by 1.64 as recommended by fib Bulletin 76 [9] (5% quantile of the normally distributed quantity).
- **Cement and fly ash content:** Different cement content were investigated in the range of six to seven sacks per cubic yard. Replacement of 10 to 50 percent of the cement content with fly ash was also investigated to reduce concrete permeability at later ages. The chloride initiation threshold for each type of reinforcement varied based on the assumed cement content.
- **Apparent chloride diffusion coefficient:** A range of 28-day diffusion coefficient values were investigated for service life requirements. A distribution was defined for each investigated value assuming a standard deviation of 20 percent of the mean value.

Service Life Determination

Several concrete mixtures were developed considering the variables discussed above and were analyzed using the service life model to determine whether a 50-year design service life could be achieved based on the end-of-life criteria presented previously. For each element, the primary findings were as follows:

- For the waterside drilled shafts, the tidal zone represented the most severe exposure conditions and corrosion risk. Including a steel casing (designated to be permanently installed) provided a significant benefit for corrosion protection by increasing the time until initial chloride exposure.
- The new portions of the wharf superstructure included deck and beam elements. The deck elements were found to govern the corrosion protection requirements for the concrete because of the lower concrete cover (2 in. (5.1 cm) versus 3 in. (7.6 cm)) for the beams.
- The landside drilled shafts are embedded in soil over their entire length, and thus had a very low chloride exposure. For the range of w/cm considered for the project (up to 0.45), the apparent chloride diffusion coefficient did not control durability considerations.

The service life analysis was repeated for three reinforcement types to demonstrate the effectiveness of using corrosion-resistant reinforcement. The required diffusion coefficients and w/cm for the drilled shafts for various combinations of reinforcing material and fly ash content are listed in Table 4. For elements directly exposed to seawater, both ACI 318 [3] and ACI 357.3R [10] recommend a maximum w/cm ratio of 0.40. Considering this limit and replacing 25 percent of the cement with fly ash would meet the 50-year service life requirement for all three of the reinforcement materials evaluated (Table 4).

Table 4—Comparison of Types of Reinforcing Materials, Diffusion Coefficient, and w/cm needed for a 50-Year Service Life

Reinforcing bars type	**Drilled shafts (Tidal exposure zone)**					
	Ordinary Portland Cement (OPC) (ageing factor=0.2)(3)		**OPC with 20% FA (ageing factor=0.36)(3)**		**OPC with 25% FA (ageing factor=0.40)(3)**	
	D_{28} (m^2/s)(1)	**w/cm(2)**	**D_{28} (m^2/s)(1)**	**w/cm(2)**	**D_{28} (m^2/s)(1)**	**w/cm(2)**
Carbon steel	$2.9* 10^{-12}$	0.22	$6.3* 10^{-12}$	0.36	$7.8* 10^{-12}$	0.40
Hot-dipped galvanized steel	$3.7* 10^{-12}$	0.26	$8.0* 10^{-12}$	0.40	$10.0* 10^{-12}$	0.44
Stainless steel	$13.5* 10^{-12}$	0.50	$>16* 10^{-12}$	0.53	$>16* 10^{-12}$	0.53

(1)Maximum 28-day diffusion coefficient to achieve 50-year service life based on service life model simulations
(2)An estimate for the w/cm for the D_{28} value, based on correlation listed in [11]
(3)The ageing factor is a constant that depends on the percentage of fly ash and slag cement, as listed in [12]

In conclusion, it was determined that a 50-year service life can be reasonably attained using adequate cover, concrete with a maximum w/cm of 0.40 and 25 percent fly ash, and carbon steel reinforcement.

REHABILITATION DESIGN

One of the biggest components of modernizing Wharf No. 3 was the design of the structure to resist much larger STS container cranes. The increased gage and larger loads associated with the container crane necessitated the addition of new structural elements (e.g., drilled shafts, crane beams, and tie beams) integrated into the existing structure to accommodate the new cranes. WJE developed a three-dimensional structural model to analyze the rehabilitated wharf for the loads described previously. In addition to the overall structural model, multiple sub-system models were developed to design specific components (e.g., crane beams and bollard supports) accounting for their deep beam behavior. The supporting models allowed for a more rigorous structural design approach that took into account expected deep beam behavior and resulted in efficient reinforcement layouts.

The geotechnical properties of the underlying soils significantly influenced the design approach and analysis. From the geotechnical investigation, the project site was determined to generally consist of firm to very stiff, medium to highly plastic cohesive clay soils with an intermediate layer of granular soils. The geotechnical axial capacity was determined by considering both side friction and tip resistances; however, due to the properties of the soil, the side friction component contributed most of the resistance (minimum of 80 percent). As a result, to accommodate the large loads associated with the wharf function, the drilled shafts required long penetration depths. Due to the length of the penetration depths and long unsupported lengths of the drilled shafts along the front of the wharf, the interaction between the wharf structure and the soil was analyzed to justify the assumed load transfer. As such, soil springs were developed in the three principal directions at multiple depths to account for the vertical and lateral stiffness of the soil.

Global Model

A three-dimensional, global analysis model was developed in CSiBridge, which allowed for efficient modeling of moving loads on the wharf while allowing for a range of element types. The wharf deck, drop panels, and the sheet pile wall were all modeled as shell elements; whereas, the drilled shafts, crane beams, deck beams, frontal beams, tie beams, and tie rods were modeled as frame elements. Nearly 2,200 frame elements, 2,400 shell elements, and 2,200 spring elements were used to model a single wharf unit. Due to the moving load cases (gantry crane, HS 25, and mobile crane loading across the width of the wharf), the analysis took approximately 10 hours on a desktop computer capable of twenty computing threads at 2.3 GHz. Given the structural framing and element section types, the geometric centroids of the various elements were not coincident. As a result, the degrees of freedom at offset member ends were constrained using link elements to ensure the desired translational and rotational compatibility was correctly represented in the model. In order to correctly model the integration of the new and existing structural elements, loading stages were used to account for self-weight and earth pressures on the existing structure prior to introduction of the new structural elements and design loads. The requirement for load stages and the introduction of soil springs required a non-linear model, which, in combination with modeling of the moving vehicle and crane loads and the various load combinations considered for the project, resulted in model run times of several hours. A three-dimensional view of the model is provided in Figure 4.

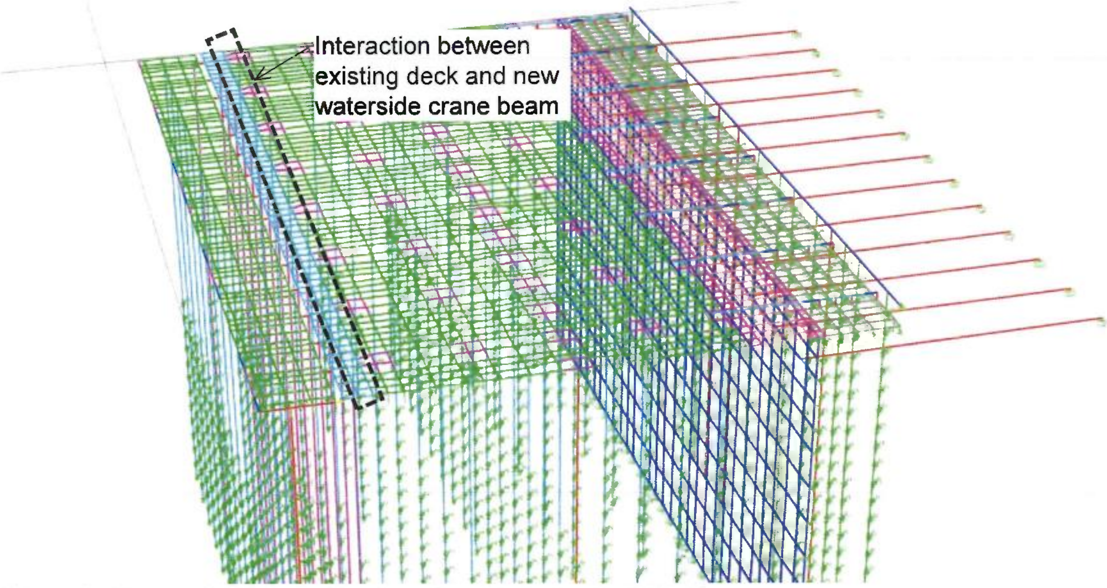

Figure 4—Representative isometric view of modeled wharf in CSiBridge.

Along the waterside, the connection between the existing wharf slab and the new crane beam was initially modeled through as shear transfer only (moments released) based on the design incorporating a single layer of reinforcing dowels (No. 10 at 6 in. (15 cm) spacing) between the two elements. Through the modeling and design process, the relative stiffness of the existing deck versus the existing drop panel and deck was better understood, and an additional layer of dowels was added in the drop panel regions due to increased shear demand. With the introduction of double layer of dowels near the drilled shafts (one in the drop panel and one in the deck), the model was modified to permit moment transfer at these locations.

The waterside crane beams were supported on 48-in. diameter (1.2 m) drilled shafts spaced at 10 ft., 9 in. (3.3 m) on-center. The free, or unsupported, length of the waterside drilled shafts was on the order of 60 ft. (18 m). This, in combination with the flexibility of the soil, resulted in relatively flexible support conditions in comparison to the existing wharf structure. Because the new crane beams were connected to the existing structure using the dowels described above, significant load sharing was developed between the new and existing structure. This was especially true at extreme loading events. The deflected shape of the deck is shown in Figure 5. As can be seen, deformation of the deck occurs well beyond the area where the tie-down force is applied on the crane beam. In this situation, the tensile load from the tie-down (uplift) force (at bottom right of figure) is shared amongst ten drilled shafts due to the flexibility of the structure and soil. The Gridline A drilled shafts, located 11 ft., 7 in. (3.5 m) away from the Gridline A2 shafts, contributes the least, at approximately 12 percent of the total tie-down load. Located only 5 ft. (1.5 m) away from Gridline A2, Gridline B drilled shafts resist nearly 20 percent of the applied force, leaving nearly 70 percent for the drilled shafts along Gridline A2. Due to relative stiffness of the new crane beam, only 50 percent of the total tie-down load is resisted by the two closest drilled shafts to the tie-down locations along Gridline A2.

The applied forces and wharf geometry result in high torsional forces in the various connecting elements, such as the deck beams and frontal beams. This is evident in the deflection shape shown in Figure 5. The torsional moments are primarily a result of compatibility between the elements and can be limited to the cracking torsional capacity, as suggested by ACI 318 [3], due to the potential for redistribution. Nonetheless, the torsion became an important design consideration that required careful reinforcement detailing.

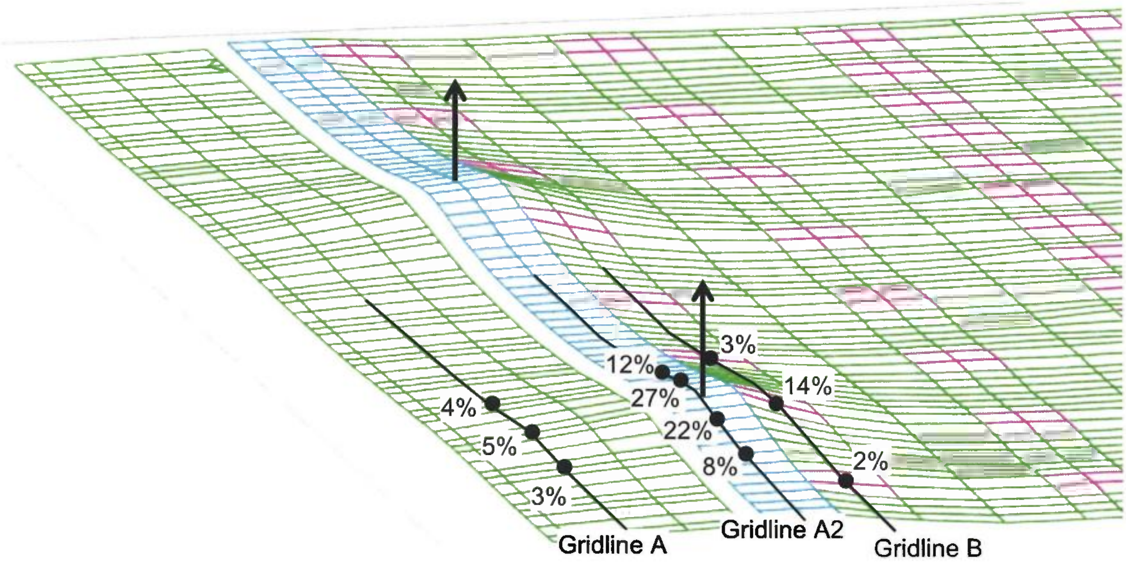

Figure 5—Deflected shape of deck due to tie-down force. For the tie-down force along the bottom of the figure, the relative load sharing of the drilled shafts immediately in the vicinity are shown.

Strut-and-Tie Modeling

The combination of a relatively short spacing between drilled shafts (10 ft., 9 in. (3.3 m)), relatively deep crane beams (4 ft., 6 in. (1.4 m)), and concentrated point loads from the wheels and tie-down locations of the container cranes, produces deep beam behavior in the crane beams. The deep beam behavior was represented using both strut-and-tie modeling (STM) as well as elastic modeling with shell elements. For design purposes, STM was used to proportion the reinforcing steel in both the longitudinal and transverse (vertical) directions of the crane beams, whereas the elastic modeling was used to confirm the longitudinal steel required and adjust the stiffness of the STM chords. STM simplifies complex stress states into simple load paths through a network of straight, pin-connected axially-loaded elements. The elements are typically arranged into various struts (compression elements), ties (tension elements), and nodes (concentrations of loads/elements) that overall form a truss that can be analyzed to satisfy external and internal equilibrium.

While relatively straightforward in concept, the application of STM in this project required addressing several challenges, namely the moving loads associated with the container cranes, the influence of soil springs on the distribution of forces in the model, and the three-dimensional behavior of the crane beam at the tie-down locations. STM is often used in bridges to evaluate and design the reinforcement necessary for bent caps, which are subjected to high concentrated loads from the bridge girders. However, in the example of bridge bent caps, the locations of the bridge girders are fixed, which simplifies the analysis. In contrast, at a wharf, the container crane is able to move along the crane rails, producing a variety of loading conditions that potentially need to be analyzed, wherein each loading condition requires a new truss model. As such, STM does not lend itself very well to performing moving load type analyses.

To evaluate the impact of the moving load, multiple load scenarios were considered along the waterside crane rail, which had uniform spacing of the new drilled shafts. For each load scenario, the load was moved between 6 to 12 in. (15 to 30 cm) along the crane rail and a new truss representation was created. Due to the uniform spacing of the drilled shafts and relative locations of the crane wheel loads, the range of forces was identified based on a limited number of models for the waterside crane beam (Figure 6). The results for the waterside crane beam helped guide the models development and load position cases for the landside crane beam, such that only a few additional models needed to be checked for the moving load condition. In addition, the specified longitudinal steel requirements from STM were compared to the elastic shell model (where moving load analyses can be performed), and the two models were found to be similar with only slight differences in the distribution of top steel to bottom steel.

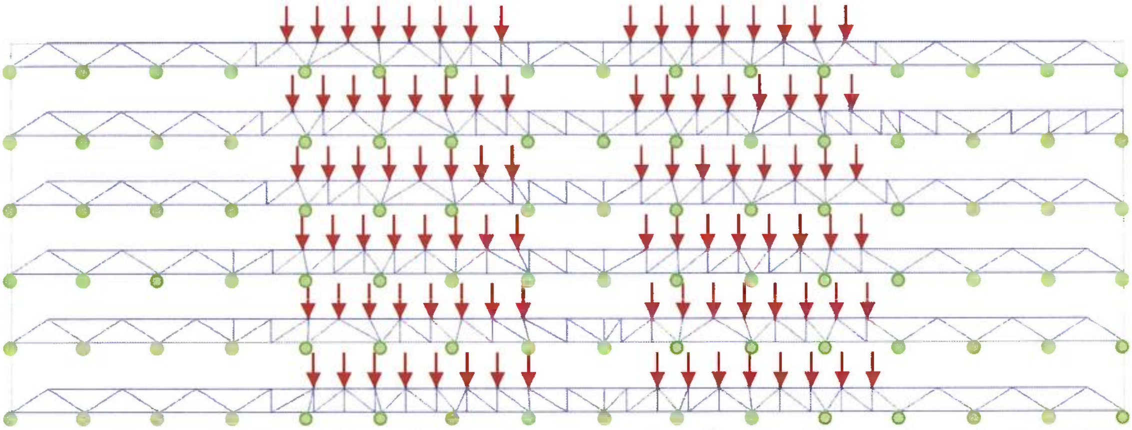

Figure 6—Variety of STM associated with the container crane along the waterside crane beam. Arrows represent container crane wheel loads, whereas circles represent drilled shaft locations.

The influence of soil springs (to represent the drilled shaft foundation conditions) on the analysis results can be evaluated by considering the deflected shape of the STM with pin supports (i.e., infinitely stiff drilled shafts and soil) and flexible supports (accounting for the flexibility of the drilled shafts and soil). As can be seen in Figure 7, the bottom of the crane beam does not deflect at the pin supports; whereas, the bottom of the crane beam does move down relative to the original location along most of the beam. The overall movement of the crane beam with flexible supports effectively increases the span length of the crane beam to something greater than the pin support spacing. As such, additional steel is required in the model that includes flexible supports as compared to the model with pinned supports. As a point of reference, the maximum force in the model with flexible supports was 40 to 100 percent greater than the model with pinned supports, thereby increasing the amount of reinforcement required for the section.

In general, the truss model at Wharf No. 3 represents the analysis of a continuous beam. As might be expected, the stiffness of the "beam" (i.e., the truss in the STM) will influence the predicted strut-and-tie forces. For the truss, the stiffness is mainly controlled by the choice of the chord members. For instance, in the Wharf No. 3 model with pinned supports, increasing the area of the chord members by a factor of ten (which influences the overall moment of inertia of the truss section) increased the maximum tensile forces by approximately 15 percent. Furthermore, the effect of the chord members was even more pronounced in the model with flexible supports due to the behavior described in the previous paragraph. As the truss member stiffness was increased, the truss (i.e., crane beam) tends to redistribute more load to adjacent drilled shafts to enforce compatibility. As such, moving to the model with the flexible supports, the maximum tensile forces increased by approximately 30 percent when the chord member stiffness was increased by a factor of ten. For the purposes of design on this project, the chord element properties were selected to approximate the transformed moment of inertia of the crane beam section.

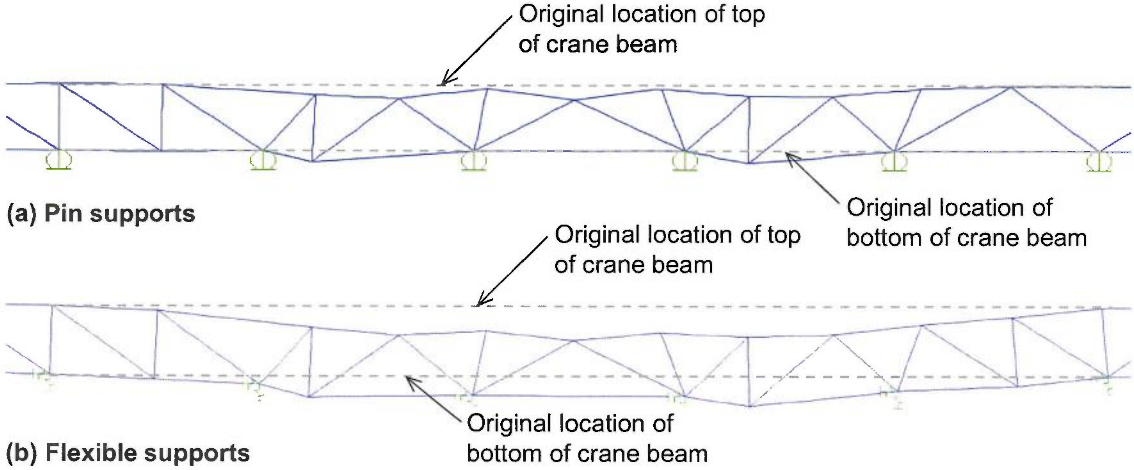

Figure 7—Comparison of deflected shape with (a) pin supports and (b) flexible supports.

As mentioned previously, during high-wind events, the STS cranes are tied down to the structure through clevises that are attached to an anchorage embedded in the crane beams. At each tie-down location, there are two anchorages located on either side of the rail for each corner of the crane (Figure 8). Assuming the tie-down force is distributed equally to each anchorage, a three-dimensional truss was needed to resolve the forces from the tie-down anchorages to the drilled shafts and appropriately proportion the reinforcement at the tie-down locations.

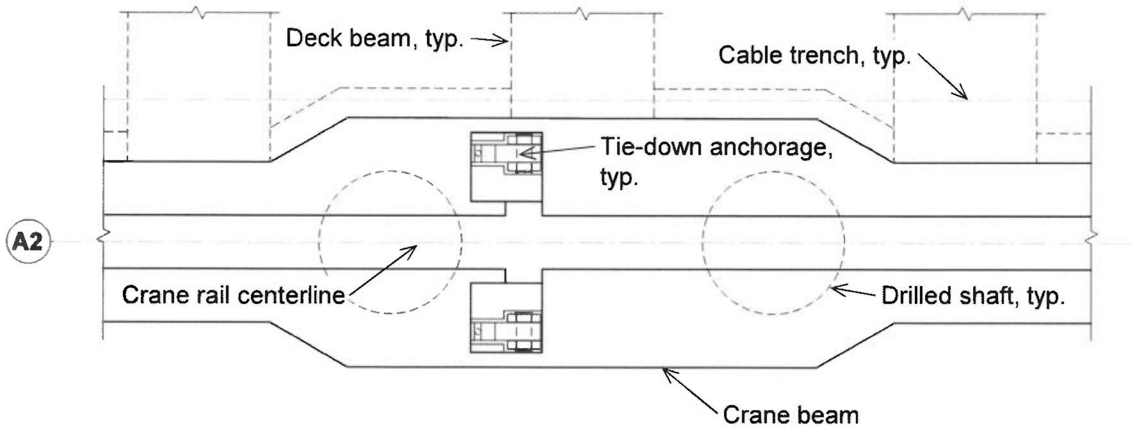

Figure 8—Plan view of the tie-down location at the waterside crane beam along Gridline A2.

A three-dimensional truss model of the tie-down location with flexible supports is shown in Figure 9. Due to the geometry of the tie-down location and the magnitude of the forces, the horizontal tie required around the anchorages is quite large. For this wharf, the force in the tension tie was approximately 430 kips (1.9 MN). To resist that force, hairpins were detailed in the tie-down section (Figure 10).

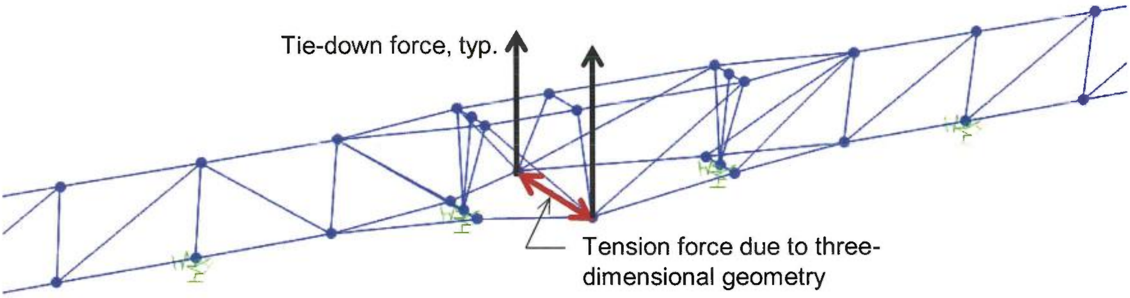

Figure 9—Partial view of the three-dimensional STM at the tie-down location of the waterside crane beam.

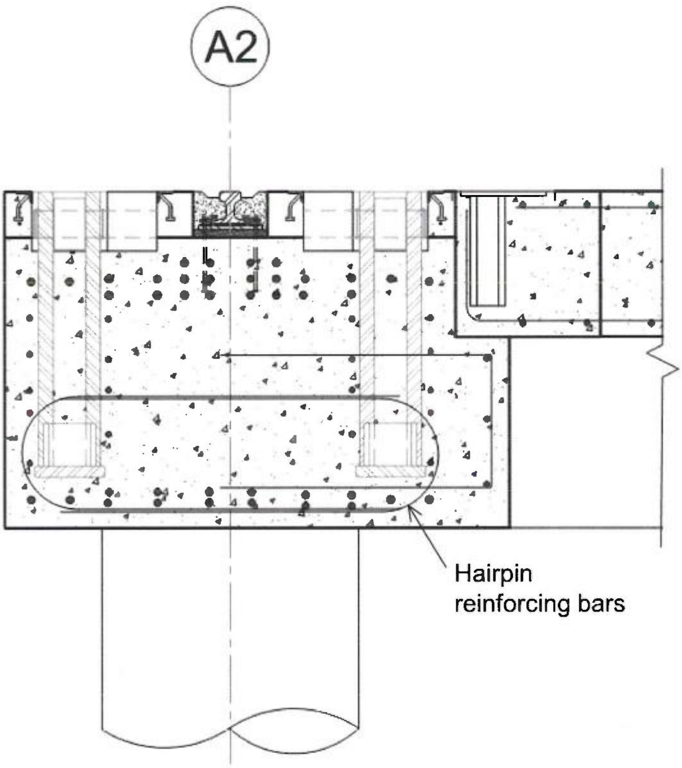

Figure 10—Section view of the reinforcing bars at the tie-down location (note hairpin reinforcing bars identified).

Corrosion Protection Plan and Additional Durability Considerations

As described previously, a diffusion-based corrosion model was used to determine concrete mixture requirements for all of the new concrete components to achieve a 50-year service life. In addition to mitigating corrosion-related distress, the concrete mixtures were designed for a variety of deterioration mechanisms (Table 5). Shrinkage and restraint cracking was identified as a concern given the size and lengths of many of the elements and because many of the new elements will be cast integrally with other large (i.e., rigid) concrete elements. In order to limit the occurrence of restraint cracking, isolation or control joints were placed at the one-third points of the 1,000 ft. (305 m) wharf length. As well, a drying shrinkage limit was specified for the deck and beam concrete to limit the total amount of shrinkage. This limit was set at 420 microstrain at 28 days when tested according to ASTM C157. Based on experience with local materials, it was determined that this performance requirement could be achieved for the project concrete mixtures without the need for shrinkage-reducing admixtures. For partial-depth repair placements in the existing wharf elements, which can be prone to restraint cracks, the shrinkage strain limit was reduced to 320 microstrain at 28 days. This stricter requirement is expected to require shrinkage-reducing admixtures to achieve. In addition, as noted in Table 5, if cracks greater than 0.010 in. (0.25 mm) are identified, the cracks are to be sealed to prevent these areas from becoming premature sources of deterioration.

Table 5—Summary of Concrete Mixture Durability Requirements

Degradation Mechanism	Protection Strategy	Requirement	Design Approach

ASR-related deterioration	▪ Provide aggregates not prone to ASR or mitigate with use of SCMs.
	▪ Test coarse and fine aggregates in accordance with ASTM C1260 (cement-only)
	▪ If aggregates fail the initial test, follow up with ASTM C1567 as a means of evaluating mitigation (cement and SCM combination)
		▪ Meet one of the following requirements:
		  ▪ ASTM C1260: maximum expansion less than 0.08% at 14 days
		  ▪ ASTM C1567: maximum expansion less than 0.08% at 14 days
		▪ Aggregates having deleterious reactions exceeding the above limits shall not be used.
			Avoidance of deterioration

Sulfate attack	▪ Provide low permeability concrete with Type II cement with Class F fly ash	▪ Concrete cover against soil not less than 3 in. (7.6 cm). ▪ Meet the requirements of ACI 301 [13] for ASTM C1012 testing.	Avoidance of deterioration
Cracking / DEF due to mass concrete placement	▪ Limit internal concrete temperatures to less than 160 °F (71 °C) and temperature differentials to less than 35 °F (19 °C) [13]	▪ For crane beams, concrete temperature at placement should be less than 80 °F (44 °C). ▪ Higher limits may be allowed if an element-specific thermal control plan is prepared and submitted.	Avoidance of deterioration
Cracking	▪ Seal or inject cracks with low-viscosity epoxy or methyl-methacrylate resin	▪ Seal cracks greater than 10 mils (0.010 in. (0.25 mm)), measured at surface	Deemed to satisfy
Excessive bleed water, settlement cracking	▪ Design mix with a lower water content ▪ Add fly ash to reduce bleeding ▪ Require testing for drilled shaft mixes	▪ Limit bleeding capacity of drilled shaft mixes to less than 1.5% when tested by ASTM C232	Deemed to satisfy
Chloride-induced reinforcement corrosion	▪ Provide sufficient reinforcement protection and concrete with resistance to chloride ingress to prevent corrosion	▪ Provide minimum concrete cover, reinforcement type, cement content, apparent diffusion coefficient, and ageing factor	Full probabilistic model

SUMMARY AND CONCLUDING REMARKS

The rehabilitation of wharves at BCT was in response to the opening of the Panama Canal expansion, which increased the size of vessels capable of visiting U.S. ports. Various wharves at BCT have been upgraded to include new STS container cranes that are capable of supporting the larger ships coming through the Panama Canal. A new concrete wharf structure was required to resist the much higher vertical and lateral loads associated with the new cranes. A variety of tasks were performed to modernize the structure for the higher loads, repair ongoing corrosion-related distress, and design the structure for another 50 years of service life. Those tasks included a visual and in-depth condition assessment, service life modeling, and rehabilitation design.

The visual and in-depth studies performed at the wharf and in the laboratory, along with service life analysis, helped to identify areas that required repair to attain an additional 50 years of service life. Service life modeling was performed using an in-house model for chloride-related corrosion. For the drilled shafts, the amount of corrosion-related damage was predicted to progressively increase through the next 50 years, especially for the tidal and splash exposure zones.

The model results correlated well with the percentage of distress observed during the overall field inspection, except for the drilled shaft caps where the predicted damage was higher than what was observed on the elements. The laboratory and nondestructive testing of these elements showed moderate risk for corrosion initiation, indicating that corrosion-related damage should be expected in the short term if left unmitigated. The bulkhead beam model correlated well with field results and predicted greater damage at the underside of the beam followed by the vertical and top surfaces over the next 50 years. Finally, away from leaks and cracks, chloride exposure on the deck elements was low and not expected to result in damage over the next 50 years. As a result, recoating the bulkhead wall below the wale beam and repair of expansion joint seals were specified in the repair drawings. In addition, cathodic protection jackets were designed for selected drilled shafts and drilled shaft caps and metalizing was specified for deck drain areas and surfaces of the bulkhead wale beam.

For the new elements, service life modeling was used to identify specific performance characteristics needed to achieve a 50-year service life. For instance, the drilled shafts on the waterside will be cast with a permanent steel casing, which will act as a sacrificial, non-structural element that can corrode over time in the marine environment. However, as long as the casing remains in place, it will act as a barrier to chloride ingress for the concrete on the interior. In addition, as shown in Table 4, the modeling determined that a w/cm of 0.40 with 25 percent fly ash replacement could provide sufficient performance with carbon steel reinforcement and a clear cover of 3 in. (7.6 cm). In contrast, the steel casing on the landside drilled shafts was assumed to be removed after construction, therefore, an alternative concrete mixture was specified for those shafts.

The design of the new structural components and the interaction to the existing elements was considered in both a global model of the wharf, as well as in multiple, sub-system models that accounted for the deep beam behavior of the crane beams. Due to the overall flexibility of the soil and drilled shafts, the various components were subjected to compatibility torsion and higher induced forces than might be expected in a similar structure with rigid foundation elements. Finally, consideration of the three-dimensional geometric constraints (as with the location of the tie-down anchorages) was needed to adequately reinforce the section.

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