Part 1 of this series of articles ran in the May 2011 issue of STRUCTURE, and Part 2 in the November 2011 issue. Please note that the numbering of figures, equations and tables is continuous across all articles. Part 1 and 2 can be accessed in the Archives at ***www.STRUCTUREmag.org****.*

Rational Approach to Design and Analysis of Piers and Marginal Wharves

Design of Seaside and Land Side Crane Girders Supporting Portal Cranes Part 3

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Conventional crane girders are supported by transverse pile caps and intermediate piles spaced at 6 to 8 feet on center. Operation safety requires installation of crane girders at a distance of 6.0 to 7.5 feet from the pier face. The crane girder is designed as a beam on an elastic foundation. Finite Element analysis programs treat the pile as an elastic spring support. The pile length for that analysis is based on pile embedment adequate to develop the pile design load capacity at no appreciable vertical movement of the pile tip. Pile embedment length can be determined from analysis of two geotechnical curves:

- t-z curve, describes relationship between skin friction stress (t) and vertical displacement (z)
- q-z curve, establishes relationship between tip resistance stress (q) and vertical displacement (z)

Investigation of both curves is extremely important for profiles with weak or moderately stiff soils.

Each pile for a crane girder analysis should be modeled with two boundary conditions, based on:

- *Pile length to the point of "fixity"*. A pile is a vertical beam on an Elastic Foundation (EF). As such, the pile does not have a well defined fixity point. It is, rather, convenient approximated by the first "0" deflection point, and well developed shear and flexural forces along the pile elastic curve with at least two (2) "0" slope points.
  The definition of "pile fixity point" in reality describes partial fixity with linear rotational spring,

  $k_r = M / \Theta$

  where,
  Θ – is slope of the pile elastic curve and
  M – pile flexural moment at 0-deflection point, sometimes defined as "point of fixity"
- *Partial fixity modeled with partially restrained pile "tip" rotation, and vertical spring support*. Vertical spring support is based on the linear elastic part of the t-z or q-z curves developed for pile length below the partially restraint "fixity" point.

The pile tip in that model is defined not by the actual pile tip elevation, but the elevation of the 0-deflection point.

This simplified modeling technique neglects the t-z spring value developed within the top 5-7 feet below the mudline; however, it yields reasonably conservative results, making pile elastic supports only slightly softer.

Another point that is worth mentioning: Elastic Foundation for piles supporting crane beams, unlike EF for piles of the transverse or longitudinal bents, can be modeled with simplified soil springs based only on the linear part of the P-y curves.

The crane load for crane girder design should be modeled as a series of point loads from the wheels of two bogies. Load on the wheels of each bogie is based on several critical load cases, including the case of over the corner lift. All lift loads must incorporate an impact factor. Impact force, taken as a percent of the vertical force, is applied only to the crane wheel loads, and is considered only in the design of the crane girders and their connections. Impact is not considered in the pile analysis. The size of vertical impact force is a debatable issue. The primary reasons for vertical impact are:

- Vertical crane rail misalignment (≤ ¼-inch)
- Load lift-off and unloading

The paper presented by Griggs at the Canadian Structural Engineering Conference in 1976 indicated that the vertical impact force determined during tests have not exceeded 7% of the vertical static load on the crane wheel. The Whiting Crane Handbook further elaborates: "Actual tests have shown that impact on the crane girders rarely exceeds 5% to 7% of static load, even for relatively fast hoist speeds, due to cushioning effect resulting from the torsion – spring action of the ropes and leaf-spring action of the girders." The results, presented by Griggs and explained by Whiting Crane Handbook, are described by ramped impulse equation:

$$\delta_{max} = \delta_{st}\,[1 + T/\pi\tau * \sin(\pi\tau/T)] \qquad (\textit{Equation 3})$$

Where,
$[1 + T/\pi\tau * \sin(\pi\tau/T)]$ – is dynamic amplification factor (see Timoshento, et al)
τ – is the duration of the impulse, and
T – is the first mode, known as Fundamental mode, natural period.

Table 2 shows the dependence of dynamic amplification on the gradual rise of (τ).

The summary of suggested vertical impacts referenced by different sources is illustrated in *Table 3*. Vertical impact is taken as a percent of the total force from the dead weight of the container crane or percent of the reaction caused by over the corner lift.

The probability of over the corner lift and simultaneous crane run along the rail is next to zero. Modern high capacity container cranes do not have an over the corner lift option; therefore,

Table 2: Theoretical impact value based on ramped impulse formula.

τ/T	$\delta_{max}=\delta_{st}\ (1+T/\pi\tau*\sin(\pi\tau/T))$
1.00	$1.00\delta_{st}$
0.50	$1.63\delta_{st}$
0.25	$1.90\delta_{st}$
0.125	$1.97\delta_{st}$
0.00	$2.00\delta_{st}$

lift reaction attributed to the front and back bogies is equally shared by all wheels of the two bogies. Reduction in the impact load can significantly improve economical viability of the crane girder design. The load combinations suggested for design of waterfront crane runways are presented in *Tables 4* and *5 (page 19)*.

All piles supporting the front crane girder have to be designed for maximum gravity load produced by over the corner lift, or the most critical lift condition combined with downward wind reaction. No impact load is considered in the pile analysis. The back crane girder should be designed for loads that exclude uplift reaction. The Engineer shall consult with the local Port Authority on allowed operational wind magnitude. If such data does not exist, the maximum recommended operational wind speed magnitude is restricted to 25 mph.

Tie Downs, Stowage Pins, Crane Rail Stops

Omissions in design of these seemingly unimportant crane way elements frequently become the reason for catastrophic failures and expensive losses. Design criteria and design of these elements will be covered in a separate article scheduled to be published in a future issue of STRUCTURE® magazine.

Fascia Beam

It is very practical to install a continuous fascia beam along the pier edge. A fascia beam can prevent small craft from getting under the wharf deck at MLW or MLLW events. It also provides convenience for rubber fender installation. If the installation of a fascia beam is not feasible due to a high tide zone, the designer should consider the installation of discrete fascia panels and fender piles. The spacing of the fender piles should be adequate to prevent a small craft or tug boat from getting under the deck.

Natural Frequency, Seismic Loads and Load Combinations

Determination of the structure's Natural Frequency ($f_n = 1/T_n$) is based on the spring value of the combined resisting system. Seismic analysis will include a sufficient number of modes to obtain combined mass participation of at least 90% of the pier mass and attributed live load. The seismic response of the pier structure is dominated by the first

Table 3: Vertical impact on crane railway.

Reference	Description
Griggs	≤ 7% of the crane static load
Whiting Crane Handbook	15% from sum of hoist lifted load and weight of grappling device.
Russian Standard, SNIP 2.01.7-85	10% of the crane static load

or Fundamental Mode of response. Most of the pier mass is concentrated in the deck, and vibrates in that mode. The first mode Natural Period of the structure is determined using the well known expression:

$$T_n = 2\pi * [W/(g*K)]^{1/2}$$

Where,
(W) is the self-weight of the bent + 20% live load on the deck + 40% of crane self-weight
(g) is the gravity acceleration in ft/sec^2 or m/sec^2
(K) is the combined spring constant of the equivalent bent.

The first mode spectral analysis method, used for analysis of waterfront structures, is frequently reduced to the Uniform Load Method. The quasi-static seismic force determined from this method is equal to:

$$p_e = Csm * W$$

Where,
Csm – Elastic Seismic Response Coefficient.
The Elastic Seismic Response Coefficient (Csm) is determined from formulas described in Chapter 3.10.4.2 of AASHTO *LRFD Bridge Design Specifications*. It should be based on the point where the Fundamental Period of vibration, T_n, of the Pile Bent falls within the Excitation or Response Spectrum Period Chart.

The Modified (Plastic Response) Base Shear on the Pile Bent is determined using; V bs = Csm *W/R. R is the reduction in structural response due to plastic deformations, damping and over-strength. AASHTO Specifications provide reduction factors only for basic types of resisting systems. When the resisting system consists of two elastic resisting mechanisms, R analysis becomes identical to an analysis of springs in series. Both parameters are physically related.

However, analysis of frame ductility is based on Moment-curvature relationship and a reduced stiffness element placed within the length of the plastic hinge, while analysis of other participating systems are based on comparable plastic deformations within the plastic plateau of the material.

Ductility of the pile bent is based on the reduced effective stiffness of the concrete pile:

$$E_c I_{eff\ cr.} = M_y/k_{yi}$$

Where,
M_y – is the moment at the point where first tension rebar yields, and
k_{yi} – is the curvature corresponding to the first rebar yield point, where concrete strain reaches 0.002.

Similarly, equivalent stiffness of the plastic hinge section:

$$E_c I_{eff\ p.h.} = M_p/k_m$$

Where,
M_p – is the idealized plastic hinge moment, and
k_m – is the curvature at the strain limit corresponding to the investigated level of seismic event.

Notes on Corrosion Protection

The biggest enemies of structural steel in a marine environment are:

- corrosion
- abrasion
- ice

The first two items can be readily addressed by modern technology. However, even the best corrosion protection material will be eventually peeled off the steel by sheets of ice in a tidal zone.

Pile Protection

Frequently, epoxy coating applied to pipe piles subjected to ice action is more damaging to the piles than no coating at all. Salinity of the water in estuaries makes an excellent electrolyte. The smallest chip in the epoxy coating

Table 4: Suggested Load Combinations. Service Loads.

Mode	Operating			Stowed
	WOP1 *	WOP2**	WOP3***	
Crane Dead Load, **DL**	1.0	1.0	0.66	1.0 /(0.66)
Lift System, **LS**	1.0	1.0	0.66	1.0 /(0.66)
Lifted Load, **LL**	1.0	1.0		
Impact Load, **IL**	0.1(LS+LL)	0.1(DL+LS+LL)		
Operational Wind, **OWL**	1.0 / (0)	1.0 / (0)		
Storm Wind Load, **SWL**				1.0
Earthquake Load, **EQ**				
Collision Load, **CL**			0.66(DL+LS)	

Table 5: Suggested Load Combinations. Factored Loads.

Mode	Operating			Stowed
	WOP1 *	WOP2**	WOP3***	
Crane Dead Load, **DL**	1.2	1.2	1.0	1.2 / (1.0)
Lift System, **LS**	1.2	1.2	1.0	1.2 / (1.0)
Lifted Load, **LL**	1.6	1.6		
Impact Load, **IL**	0.1*1.6(LS+LL)	0.1*1.6(DL+LS+LL)		
Operational Wind, **OWL**	1.6 / (0)	1.6 / (0)		
Storm Wind Load, **SWL**				1.6
Earthquake Load, **EQ**				
Collision Load, **CL**			1.0(DL+LS)	

NOTES:
** Load Combination for load pick up*
*** Load Combination for load carried by crane along the crane way within the crane loading bay.*
**** Impact on Crane Stop*
Factors shown in parenthesis ( ) are given for case when wind load causes uplift.

may instantly initiate the corrosion process. A galvanic bridge establishes itself between the anode and cathode. Any source of potential difference can create the galvanic bridge:

- stress concentration
- proximity of the "new" and "old" metal
- sharp edges on flanges
- temperature difference (temperature difference is frequently coupled with difference in amount of dissolved oxygen)
- variation in oxygen content against the water depth
- mudline acidity
- metabolic activity of sulfate-reducing bacteria in low water
- metabolic activity of anaerobic sulfate-reducing bacteria at the mudline

Interesting phenomenon was reported by divers who investigated corrosion and deterioration of HP-sections, where accelerated corrosion had started at flange edges and gradually progressed towards the web. At some point, the web of the HP-section became the anode for the flanges and deteriorated at a much faster rate than the flanges themselves. Obviously, the section with rounded surfaces and no sharp transitions stands a better chance of long term survival in a saline and abrasive environment. However, pipe pile sections will stand a much better chance if their surfaces within the pile length, with boundaries 2 feet above MHW and 5 feet below MLW, were protected with HDPE or Fiberglass Jackets. This arrangement is a viable option in cold and moderate climates. An alternate design option is to increase the steel thickness to account for the annual corrosion rate. The corrosion rate in cold climates is much lower than in moderate climate zones. In climates where ice compression strength is low and solid ice sheets do not form around the pile, the best protection is provided by material produced by Flexcrete: Cemprotec E942. Kobayashi suggested that steel with traditional coal tar coating corrodes more severely in tidal and submerged zones than in a splash zone. Fast corrosion in the tidal zone can be attributed to mechanical abrasion and deterioration of the coating due to ice and wave activity. Corrosion in the zone of continuous immersion, however, cannot be easily explained. Corrosion mechanisms in that zone can be extremely complex. Even variation in the content of oxygen can create an initial galvanic bridge. In that case, the area with lower content of oxygen is anodic towards the area with higher oxygen content. Also, rust entraps oxygen, creating a never ending process. Deterioration, blistering and peel-off of coal tar epoxy coating within 2 to 5 feet below the MLW was frequently observed by divers during underwater inspection. Traditional coal tar epoxy coatings showed quick deterioration and peeling, initiated by mechanical abrasion, aggravated by corrosion and sun UV radiation. Restoration of traditional coal tar epoxy coating below the water is a technically impossible task. Such restoration requires application of a zinc phosphate primer, but the whole process has low dampness tolerance and cannot be used underwater. That makes the repair process of the surface applied corrosion protection a very complicated issue. A very thin layer of coal tar almost certainly guarantees abrasion damage, early corrosion in a tidal zone and deterioration of protection in a splash zone. An alternative coating, E942 offered by Flexcrete Technologies Ltd., makes corrosion protection much more durable, simplifies application and reduces maintenance cost. The coating is damp tolerant, tolerates early immersion, has superior abrasion resistance, and high alkalinity that passivates the steel. However, in areas subjected to heavy ice build-up, HDPE or Fiberglass-petrolatum jackets placed over zones of high abrasion should be considered.

Protection of Steel Girders

Structural steel can significantly increase the span of the pile cap and reduce the number of piles required. However, there are several issues that have to be addressed by the steel girder designer:

- Corrosion
- Abrasion
- Buoyancy
- Ice Crushing

Corrosion

A steel girder is most likely to be located in a splash zone, but it might be overlapped by a

tidal zone as well. The most dangerous type of corrosion for steel girders is Accelerated Low Water Corrosion (ALWC). This form of corrosion, also known as bacteriological corrosion, is frequently found in areas of sharp angular changes in the surface, and is characterized by soft orange corrosion residue, the life product of sulphate-reducing bacteria. Bacteria can reduce seawater sulphates into sulphuric acid. The annual rate of ALWC corrosion can be as high as 1 millimeter per annum. The best way to address this type of corrosion is not to use open sections and to reduce the number of sharp angles. This issue is of particular importance to girder bearing supports, where such deterioration should be prevented at any cost. The usage of Cemprotec E942 can solve the problem. Cemprotec E942 allows 5 times thicker coating than traditional coal tar epoxy coating, and tolerates much lower forms of surface preparation. The material also provides a quick fix maintenance solution. However, steel girders should not be the first choice in tidal zones where ice sheets can develop thickness in excess of 4 inches, and where girders can be partially submerged during high tides.

Abrasion

Abrasion resistance of structural steel girders can be improved by avoiding sharp angles and protection of the steel in area of ice fluctuation with HDPE or Fiberglass-petrolatum jackets.

Buoyancy

Buoyancy mostly affects structure during erection. A floating structure could be difficult to set in place. A contractor should proceed with erection during the low water tide, and the structure should be fully-anchored before high tide.

Ice Crushing Force

The ice crushing force acting on the box girder skin can be easily addressed by closely spaced internal diaphragms. The skin of the girder should be designed as a membrane supported by diaphragms. Skin deflection should be controlled between supporting diaphragms. Skin deflection between diaphragms creates an effect similar to "tension field action" in the plate girder, when the plate girder behaves like a truss. In that case, stiffeners behave like struts and a buckled web behaves as a tension diagonal. Deflection of such a truss is larger than the deflection of the original plate girder. Therefore, Serviceability of a girder with distorted webs and bottom flange should be closely investigated.

Welds and Fasteners

Selection of correct material for welds and fasteners in waterfront construction is frequently a neglected subject. However, the wrong selection of weld or bolt material can bear catastrophic consequences. Frequently observed extreme rust on welds and bolts in splash, tidal zone and zone of permanent submergence testifies to the significance of the problem. The root of the problem is the dissimilarity of weld and base metal. In the presence of oxygen and an electrolyte, galvanic bridge quickly establishes itself between dissimilar metals. If a weld, or even a fastener, becomes anodic to the base metal, the weld or fastener begins to quickly deteriorate. However, the rate of corrosion can slow down as newer, less corroded, steel is always anodic towards more corroded steel. Cathodic protection frequently favored by structural engineers for protection of submerged steel elements can also aggravate fatigue problems, as it increases production of hydrogen and encourages "hydrogen embrittlement" of the connection. The pros and cons of cathodic protection should be carefully reviewed for fully submerged structures, and particularly for *fracture critical elements* with stress ratio $-1 \leq R \leq 1$. For more information on that subject, interested readers are referred to *Survival of Long Span Crane Truss in Marine Environment*.

In addition, the designer should discuss with the owner all pros and cons of using spiral welded pipes for piles in a waterfront project. While use of spiral welded pipes is well justified in fresh water environments, their susceptibility to corrosion makes them less suitable for projects in brackish or sea water, unless they are filled with concrete and protected with zinc impregnated cementitious epoxy coating on exterior.

Additional provisions for protection of such piles in an ice fluctuation zone should also apply.

Sheet Pile Walls

Combined sheet pile walls utilizing soldier piles can stabilize the wall against backfill induced down drag. Using sheet piles in tension-compression elements of A-frames should be avoided for the following reasons:

- possible down drag forces, and
- weak uplift capacity of sheet piles.

If a sheet pile wall is used as a resisting mechanism of the wharf, consideration of a combination sheet pile wall is strongly recommended. Soldier piles of a combination wall can prevent settlement due to soil down drag, and enhance wall stability against circular slip failure.▪

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