Part 1 of this series of articles was published in the May 2011 issue of STRUCTURE® magazine. Visit ***www.STRUCTUREmag.org*** *for article archives.*

Rational Approach to Design and Analysis of Piers and Marginal Wharves

Part 2

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Loads Affecting Deck Design

Flexibility of pile bents in the direction of the deck span allows construction of piers without expansion joints. The coefficient of thermal expansion for reinforced concrete is equal to 6.5 x 10^{-6} units per inch per degree Fahrenheit. Pier length between expansion joints can be based on ½ of deck expansion. The total deck expansion between expansion joints shall be ≤ 0.2d, where (d) is pile bent diameter.

Forces acting on the deck include:

- static gravity forces
- dynamic gravity forces
- forces caused by temperature differential

Temperature Forces

For moderate climates, the temperature rise or fall is typically within the range of 40°F to 50°F. Therefore, it is not out of proportion to build a 1200 foot long deck supported on 24-inch ϕ diameter pile bents with no expansion joints: (6.5 x 10-6 x 1200 x 12 x 50 / 2= 2.34 inches < 0.1 x 24 inches = 2.4 inches). However, rigidity of the deck induces rotation on the laterally constrained pile caps (*Figure 4*) and produces additional forces on the deck itself. Therefore, the deck model shall be checked for the combination of gravity forces and forces induced by temperature differential.

Proper boundary conditions are absolutely necessary for analysis of temperature induced forces. Assumptions used for model building can significantly affect results. The location of the assumed point of fixity along the deck length shall be clearly visualized. The point on the elastic curve with "0" slope / "0" movement boundaries shall be used as a fixed support in the slab model. Frequently, this point is located in the middle of the pier length, (*Figure 4*).

Static Gravity Load

The majority of modern piers utilize precast prestressed planks.

Pier decks utilizing precast planks shall be designed as double span beams for superimposed live load, and as a simple span for self-weight and superimposed gravity dead load. By the time of live load application, the stresses induced by dead loads will be locked into the precast planks.

Gravity dead and live loads applied to the deck shall be separated in two different load categories:

- *Distributable*, and
- *Non-distributable* loads.

The PCI design manual gives good illustrations for load distribution in flat slabs. Failure to properly distribute loads results in overly conservative plank design and underestimated topping reinforcement. Pile cap rotation effect on the stability of the pile can be safely neglected. The rotational stiffness of the equivalent deck strip is 6 to 10 times higher than the rotational stiffness of the pile. Therefore, pile bending moment caused by pile cap rotation is negligible.

Dynamic Gravity Load-Vertical Impact

The following formulas explain why measured impact was much lower than what was predicted by classic impact formula, and significantly lower than impact allowance suggested by current practice.

Deflection predicted by classic formula for suddenly applied load:

$$\delta_{max} = \delta_{st}\,[1 + (1+2E_{kinetic}/W\delta_{st})^{½}] \quad (\textit{Equation 1})$$

Where,

δ_{st} = W/k–static deflection
W = static force
k = span spring constant
$E_{kinetic}$ = ½ (WV^2/g) = kinetic energy
g = 32.2 ft/sec^2

Therefore, in accordance with the above formula, at V = 0 ft/sec, dynamic amplification produces twice the stress and twice the displacement. However, impact produced by a vehicle is more accurately described by the rectangular impulse formula:

$$\delta_{max} = \delta_{st} \sin \{1- \cos [(k/m)^{½}\tau]\}$$

Where $(k/m)^{½}$ = τ = radial Natural Frequency in rad/sec.

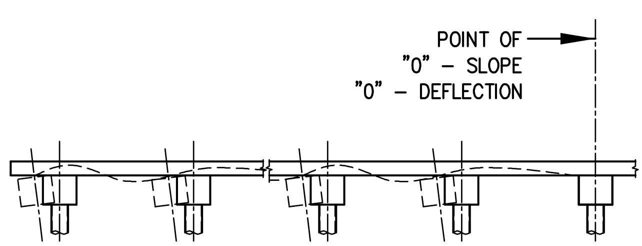

Figure 4 (continued from figures shown in the May article): Deck Temperature Deformations Model.

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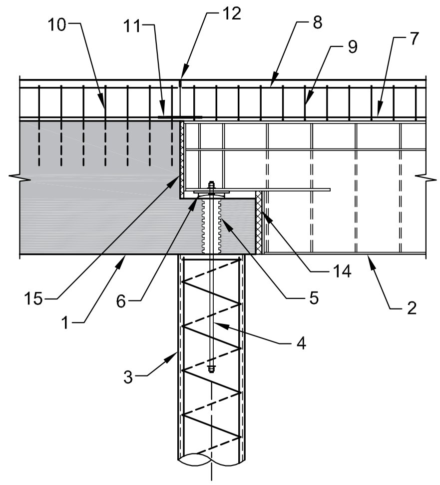
Table 1: Theoretical impact value. Rectangular impulse formula.

τ/T	$\delta_{max}=2\delta_{st}\sin(\pi\tau/T)$
0.500	$2.00\delta_{st}$
0.250	$1.41\delta_{st}$
0.215	$1.25\delta_{st}$
0.180	$1.07\delta_{st}$
0.166	$1.00\delta_{st}$



1. SHORT SPAN CONCRETE PILE CAP
2. LONG SPAN STEEL BOX GIRDER
3. RIGID PILE SUPPORT
4. THREADED ROD
5. CORRUGATED PVC SLEEVE AT GIRDER C.L.
6. S.S. SPHERICAL BEARING DETAIL WITH SHORT SLOTTED HOLE IN THE TOP SPHERICAL PLATE.
7. COMPOSITE POUR BOTTOM REINFORCEMENT
8. COMPOSITE POUR TOP REINFORCEMENT DISCONTINUED AT CROSS CUT.
9. COMPOSITE TIE WELDED TO THE STEEL BOX GIRDER
10. COMPOSITE POUR U–BAR
11. 24" LONG TEFLON TAPE WRAP AT EACH BOTTOM REBAR
12. 1/4" WIDE BY 1 3/4" DEEP CROSS CUT FILLED WITH CAULK
13. PRECAST PLANKS WITH HOOKED TAILS
14. RESILIENT FOAM PAD SUSPENDED FROM GIRDER FLANGE
15. RESILIENT FOAM PAD BETWEEN CONCRETE PILE CAP AND STEEL GIRDER.
16. PLANK COMPOSITE TIES

Figure 5: Special Movement Joint.

At $(\tau/T) \leq ½$ this formula is reduced to:

$$\delta_{max} = 2\delta_{st}\sin(\pi\tau/T) \qquad (Equation\ 2)$$

Where,
$2\sin(\pi\tau/T)$ is the dynamic amplification factor
τ is the duration of the impulse, and
T is the first mode, known as the fundamental mode, Natural Period.

τ naturally depends on span stiffness or span spring value.

Rectangular impulse, equal to half of the Fundamental Period, produces the same impact result as the classic formula for suddenly applied load (*Equation 1*). Naturally, response to the long step impulse depends on the pulse duration. Reverse analysis for different values of (τ/T) is presented in *Table 1*. As is evident from the formula for rectangular impulse, impact allowance depends on the ratio of (τ/T). *Equation 2* indicates that impact allowance grows directly proportional to Fundamental frequency, (1/T). However, the duration of impact impulse in stiffer spans is controlled by the softness of the vehicle suspension:

$$1/K_{system} = 1/K_{span} + 1/K_{v.\ susp.}$$

Obviously, a stiffer spring contributes very little to the softness of the system.

The following are factors that determine the length of impact impulse:

- Elastic spring of the span at the point under consideration, and
- Elastic spring characteristic of the vehicle suspension mechanism.

It is obvious that a softer system amplifies impulse load less than a stiffer one. Additional factors influencing dynamic amplification are caused by resonant excitation that depends upon:

- Length and composition of the traffic train, and
- Speed of the train traffic.

Vehicular Load on the Deck

Generally, vehicular traffic on the deck is very slow. Both AASHTO and UFC 4-152-01 recommend that 25% dynamic load allowance be added to the maximum listed wheel load. AASHTO recognizes that measured Dynamic Load allowance, or impact, does not exceed 25% of the static response to the vehicles. Furthermore, AASHTO indicates that such response is caused by a resonant excitation due to similar frequencies of vibration between the bridge and vehicle, or due to long undulations or surface discontinuities in the pavement. Resonant excitation of the short rigid pier deck under slow and irregular vehicular traffic is an unrealistic expectation. The softness of the system in that case depends not on the stiffness of the rigid deck but on the softness of the vehicle suspension system. It is virtually impossible to model the pier deck/vehicle impulse system with a (τ/T) ratio of 0.180 or greater. An impact factor of 10% is suggested as a more realistic and still conservative design assumption for pier traffic. A similar conclusion was reached by Gaythwaite.

Load from Fork Lift or Movable Crane

In accordance with above statement, the dynamic load allowance for a fork lift can be safely taken as 10% of the total wheel load. The dynamic load allowance for a movable crane can be taken as:

- 10% of wheel reaction from the crane dead weight as for vehicular traffic, or
- 10% of max reaction induced by lifted load on the wheel or crane outrigger

Dynamic amplification from the lifted load during crane lifting operation will be described by *Equation 3* (addressed in Part 3 of this article, to be published in a future issue of STRUCTURE), which was developed for ramped impulse.

Load from Stacked Containers

Besides the traditional load combinations, the decks of container terminals shall be designed for a load combination that includes unbalanced load equal to the weight of a single container <u>plus</u> 10% dynamic load allowance applied to the weight of one container. However, the superimposed load surface shall be held 6 feet away from the crane rail.

Support Detail for Long Flexible Box Girder

Development of some new waterfront properties required the introduction of long, relatively flexible steel box girders. The continuity between the long span girders and the short span concrete pile cap can be debated, but it is highly undesirable. Such continuity may result in extremely high negative moment $(^{-}M_{LL+SDL})$ induced by live and superimposed dead load. The resultant magnitude

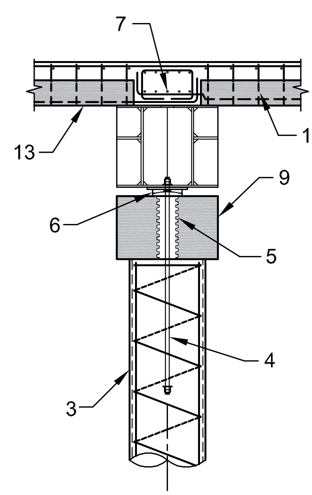


Figure 6: Section Through the Steel Girder Bearing Detail.

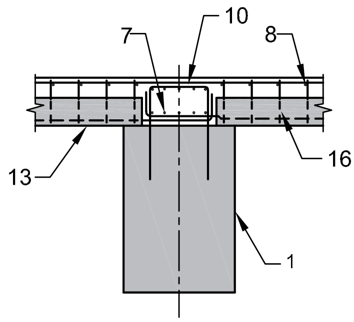


Figure 7: Section Through the Pile Cap.

of negative moment requires unreasonable reinforcement in that area, and often leads to development of random flexural cracks in the deck above the support. The supports of long span steel box girders shall have unrestrained rotational movement. The dictated solution can be found in the creation of a specially designed movement joint in the closure pour detail directly above the long span girder end, (*Figures 5, 6*, and *7*).

The movement joint running through the closure pour of the pile cap is provided by stopping all top longitudinal reinforcement at the face of the saw cut. The bottom reinforcement of the closure pour is run continuously through the joint; however, this reinforcement shall be wrapped with Teflon tape, 12 inches to each side of the saw cross cut. This suggested detail provides deck continuity for horizontal diaphragm action, but allows rebar protection when the closure pour reinforcement yields and stretches due to initial negative moment.

Example 1

The following example shows the design and justification of the proposed movement joint detail.

Live load + Superimposed Dead load = 1100 psf

(Structural topping and leveling concrete shall be treated as Superimposed Dead Load)

Girder span, L=36 feet

Tributary width, a =24 feet

b = 2 feet – vertical lever arm between sole plate of spherical bearing and bottom reinforcement of composite pour.

Drag Strut force, P drag =220 kip

Friction Coefficient between sole plate of spherical joint and concrete, $\mu = 0.70$

Maximum End Horizontal Reaction, R^{HORIZ} = 1100 x 0.5 x 36 x 24 x 0.7 = 332 kip

Maximum End Moment, M = R^{HORIZ} x b = 332 x 2 = 664 ft-k

Full Fixity Moment, M_{fix} = 332 x 36/6 = 1992 ft-k > 664 ft-k

The tension capacity of the composite pour bottom reinforcement shall not exceed the maximum end horizontal reaction.

Drag strut force transmitted through the pile cap, P drag =220 kip < 332 kip

$A_{s\ req}$ = 220 x 1.5 / 60 = 5.5 in^2, L.F. =1.5

The bottom reinforcement in the closure pour is designed for a force greater than the force required for the development of drag strut action. It is obvious that reinforcement yields and stretches at Maximum End Moment, but provides resistance to drag force. This condition requires creation of the slip zone around the bottom reinforcement, (*Figure 5*). The slip zone in concrete is provided by Teflon tape wrapped around each rebar.

An artificial crack in the pile cap closure pour shall be continued into the deck composite pour to eliminate the propagation of random cracks. A control joint in the deck can be achieved by cutting every other topping rebar at the control joint location. Note: Suggested solution can be used for gravity analysis only. Lateral force analysis shall not rely on friction components of the resisting system.▪

REFERENCES

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Gaythwaite J.W., *Design of Marine Facilities*, Van Nostrand Reinold, NY, 1990