EFFECT OF DIFFERENTIAL CREEP AND SHRINKAGE ON PRESTRESSED COMPOSITE CONCRETE SECTIONS

Markus Wernli, PE, PhD, BergerABAM, Federal Way, WA
Iman Ghorbani, EIT, PhD, BergerABAM, Federal Way, WA
Michael Oesterle, PE, PhD, NAVFAC EXWC, Port Hueneme, CA

ABSTRACT

Differential creep and shrinkage in composite decks made of precast panels with cast-in-place topping concrete can yield to unacceptable cracking and adversely affect durability of a structure. The common design procedure to include creep and shrinkage is to consider the differential strain between the time of construction and the life time of the structure as a single step analysis. The paper shows that this simplified assumption can significantly overestimate the stresses as it does not consider stress redistribution due to creep over time. This paper discusses time dependent viscoelastic procedures that consider stress redistribution over time, updating the creep behavior according to the stress state at each time step. The analysis has been performed on typical berthing pier decks following the definitions of the creep and shrinkage behavior per American Concrete Institute (ACI) 209R-92 and Model Code 90. It was found that the results between the two code approaches are also significantly different and that the change in the parameters, such as the age of precast elements and concrete curing time, significantly affect the stresses from differential creep and shrinkage. The paper suggests using such design approaches that are used in standard software with caution.

Keywords: Precast, Concrete, Substructure, Creep, Shrinkage, Design

INTRODUCTION

The use of precast/prestressed concrete elements has the potential to increase durability of concrete structures in corrosive environments as the controlled element fabrication in a precast yard typically leads to higher concrete quality compared to in-situ concrete construction. This advantage has long been acknowledged by owners of marine structures, such as ports and the U.S. Navy. Nowadays, the use of precast concrete piles and deck panels is quite common for piers and wharfs.

In 2011, the U.S. Navy conducted a study that investigated methods for the use of prestressing or post-tensioning of concrete to minimize cracking in piers[1]. In this study, it was recognized that the internal stresses caused from differential creep and shrinkage between precast deck elements and cast-in-place topping concrete can be significant and, thus, should be considered in the design of cracking moment and nominal crack width for the purpose of controlling cracking.

Creep and shrinkage behavior of concrete is dependent on the stress level and specific material properties, as well as the shape of the concrete element, the climate at fabrication and installation site, the curing methods, and the age of concrete. Several codes provide elaborate design provisions for the calculation of strains and stresses as a result of creep and shrinkage behavior of concrete. The codes also provide means on how to apply these design provisions to evaluate the internal stress state in composite elements that results from differential creep and shrinkage behavior, i.e., from the different creep and shrinkage behavior between its components.

The study evaluated the differential creep and shrinkage stresses between precast panels and cast-in-place concrete using ACI 209R-92[2], “Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures,” and compared it to the analysis per Model Code 90[3] (MC 90), which is commonly used by standard software like LARSA 4D. This paper explains the analysis approach and discusses the results.

TIME DEPENDENT VISCOELASTIC FORMULATION

The highest internal stresses from differential creep and shrinkage in a pier deck can be expected at a location where a pier deck has the highest level of constraint from adjacent elements. This condition would be met by a deck span in the center of the pier. For simplified analysis, it can be assumed that such a center span acts like a beam that is fixed in rotation at both ends. The fixity reflects an axis of symmetry and is justified as the adjacent deck spans are of the same size and experience the same internal stress state. Furthermore, it can be assumed that axial forces within the beam are not restrained as a typical pier deck experiences only marginal lateral constraint from bending of the piles. For the evaluation of the internal stress state from differential creep and shrinkage, the pier can be reduced to a one-dimensional (1-D) or single-degree-of-freedom model that consists of multiple parallel springs representing the individual components of the pier deck (Fig. 1). The springs shown in the figure represent the axial stiffnesses of the different sections for a composite element, such as a cast-in-place toping concrete pour, a precast deck panel, and a pile cap beam.

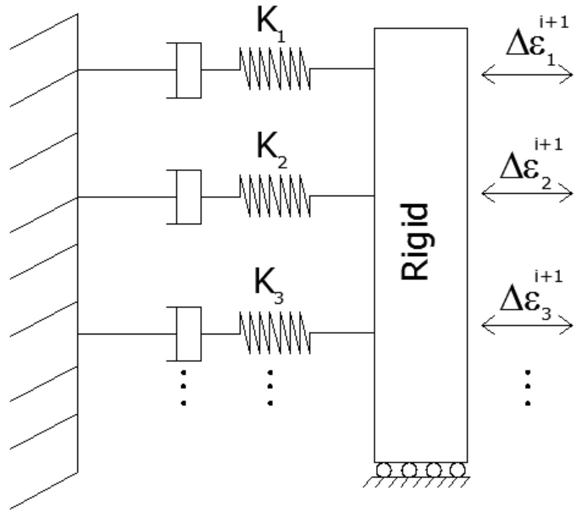

Fig. 1 - 1-D Parallel Spring Model

Stress redistribution on the sections due to creep and shrinkage strain for this model is based on the strain compatibility between the layers and force equilibrium on all the sections. The development of creep and shrinkage within the individual layers can now be analyzed by conducting a time-step analysis. At each time step, the internal stress state can be evaluated by meeting force equilibrium and strain compatibility. Based on this internal stress state, the incremental creep and shrinkage strains for each layer can be calculated and applied at the next time step. The incremental creep and shrinkage strain at each time step is given in Eq. (1).

$$\{\Delta\varepsilon^{i+1}\} = [\{\varepsilon_c{}^{i+1}\}-\{\varepsilon_c{}^{i}\}]+[\{\varepsilon_{sh}{}^{i+1}\}-\{\varepsilon_{sh}{}^{i}\}] \quad (1)$$

Where:

$\{\Delta\varepsilon^{i+1}\}$: Differential strain vector for the layers at time step i+1

$\{\varepsilon_c{}^{i}\}$: Absolute creep strain vector for the layers at time step i

$\{\varepsilon_{sh}{}^{i}\}$: Absolute shrinkage strain vector for the layers at time step i

As shrinkage is independent of the stress level, the incremental shrinkage strain from one time step to the next can be obtained from the shrinkage strain rate of concrete proposed in different codes, such as ACI 209R and MC 90. Creep is stress dependent; and, by variation of stress over time, the creep strain in Eq. (1) should be updated reflecting the stress level at the considered time step. This absolute strain can be calculated from the time integration of the constitutive equation for concrete [Eq. (2)].

$$\varepsilon_c^{i+1} = \sum_{j=0}^{i} \frac{\emptyset\ t_i, t_j}{E_c} . \ \Delta\sigma^j \tag{2}$$

Where:

ε_c^{i+1} :	Absolute strain due to creep of each layer at time step *i+1*
$\Delta\sigma^j$:	Differential stress on the section of each layer at time step *j*
$\phi(t,\tau)$:	Individual creep function for each layer at age *t* for a load increment applied at age τ
E_c :	Modulus of elasticity at 28 days

Eq. (2) is representing the absolute creep strain at time *t* due to the variation of stress on the section. In Eq. (2), the creep is always with a function of the modulus of elasticity at 28 days. But to calculate elastic deformations, the time dependent modulus of elasticity should be taken into account (MC 90). Therefore, the updated modulus of elasticity is used in the compatibility and equilibrium equations. In the proposed model in this study, time dependency of the modulus of elasticity has been considered according to MC 90. The creep function (ϕ) is defined in the literature and by different codes. For this study, the creep and shrinkage definitions from ACI 209R and MC 90 have been used and compared to each other.

The force equilibrium and strain compatibility equations to calculate the differential stress vector for the next time step can be formulated per Eq. (3) (shown for 3 layers). The analysis approach is referred to as the Time Integration Model in the remaining paper.

$$\begin{matrix} -\frac{1}{E_1^{i+1}} & \frac{1}{E_2^{i+1}} & 0 \\ 0 & -\frac{1}{E_2^{i+1}} & \frac{1}{E_3^{i+1}} \\ A_1 & A_2 & A_3 \end{matrix} \cdot \ \Delta\sigma^{i+1} \ = \ \begin{matrix} \Delta\varepsilon_1^{i+1} - \Delta\varepsilon_2^{i+1} \\ \Delta\varepsilon_2^{i+1} - \Delta\varepsilon_3^{i+1} \\ \Delta P^{i+1} \end{matrix} \tag{3}$$

Where:

$\Delta\sigma^{i+1}$:	Differential stress vector for the layers at time step *i*+1
$\Delta\varepsilon_s^{i+1}$:	Differential strain of layer *s* at time step *i*+1 per Eq. (1)
E_s^{i+1} :	Elastic modulus of layer *s* at time step *i*+1
A_s :	Concrete section area of layer *s*
ΔP^{i+1} :	Incremental increase of external long-term load at time step *i*+1 (If post-tensioning is added after integration of the layers)

VALIDATION OF THE TIME INTEGRATION MODEL

The Time Integration Model was validated using LARSA 4D, a commercial finite element analysis program that has the ability to perform time-step analyses for construction sequence investigations allowing consideration of creep and shrinkage effects. The program models creep and shrinkage behavior of concrete per MC 90. For comparison, a typical deck section of a pier was considered. It was assumed that the pier deck section consists of a top layer of cast-in-place topping concrete (TC), a middle layer of precast deck panels (DP), and a bottom layer of a precast cap beam (CB). A tributary width of the deck was assumed for the cap beam. Each layer is defined as a set of frame elements. The layers are connected by linking their corresponding nodes along the axis. As boundary condition, at one side, all frame nodes were fixed, while at the other side, the nodes of the layers were slaved to move together in the axial direction. The physical geometry of the composite section is shown in Fig. 2.

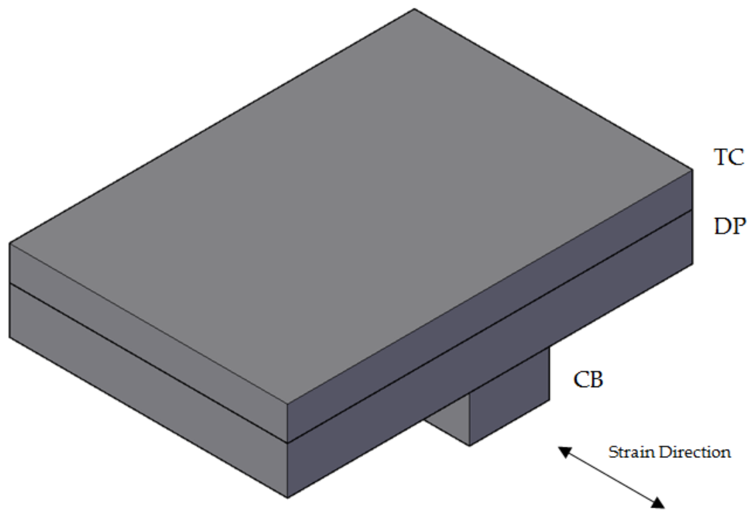

Fig. 2 - Physical Modeling of the Studied Superstructure

It was assumed that the precast deck panels and pile cap beams were 90 days old by the time the topping concrete was poured and, thus, integrating the layers to function as one composite deck. It was assumed that the precast pile cap beam was prestressed to a stressing level of 670 psi (4.6 MPa) one day after it was cast. The precast panels were not prestressed in the analyzed direction. Prestress losses from tendon relaxation were neglected. No wet curing was considered. The cross section area of the cap beam, the deck panel, and the topping concrete were 13 ft^2 (1.2 m^2), 26 ft^2 (2.44 m^2), and 19.7 ft^2 (1.83 m^2), respectively. The modulus of elasticity and compressive strength of concrete at 28 days were considered to be 4,350 ksi (30,000 MPa) and 4.35 ksi (30 MPa), respectively.

The comparison between the simplified model and the LARSA 4D analysis of the stress behavior over the service time of the deck can be seen in Fig. 3, Fig. 4, and Fig. 5 for the pile cap beam, precast deck panels, and topping concrete, respectively. Zero in the time scale of the figures represents the time of concreting of the topping concrete, hence the integration of the components.

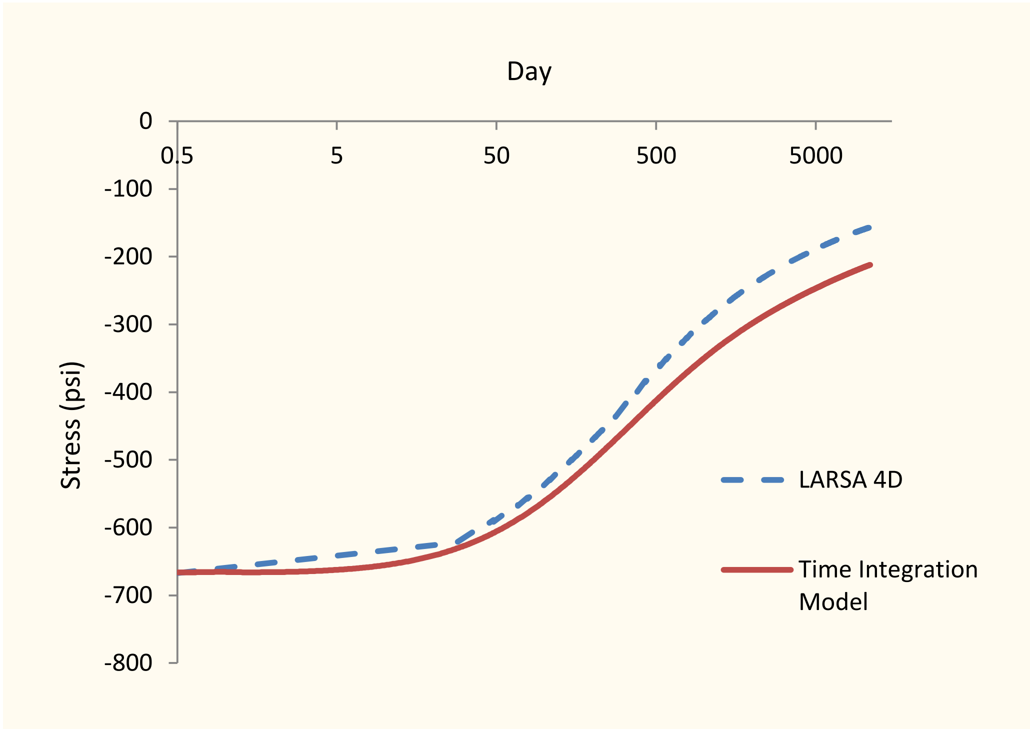

Fig. 3 - Comparison of Stress Development over Time in Cap Beam between LARSA 4D and Time Integration Model (Positive is Concrete Tension)

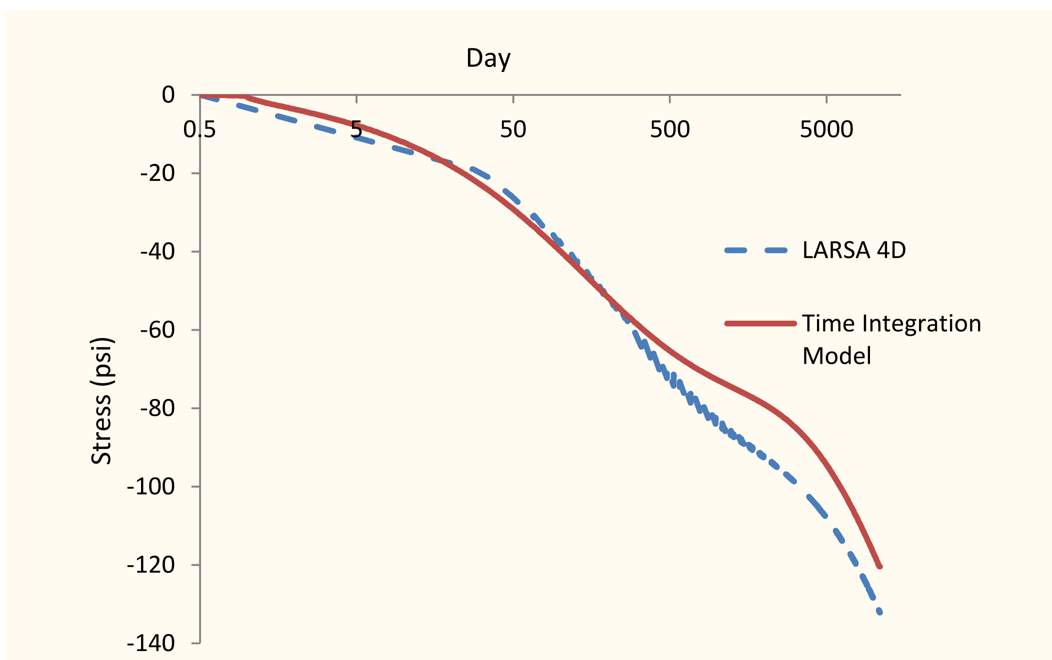

Fig. 4 - Comparison of Stress Development over Time in Deck Panel between LARSA 4D and Time Integration Model (Positive is Concrete Tension)

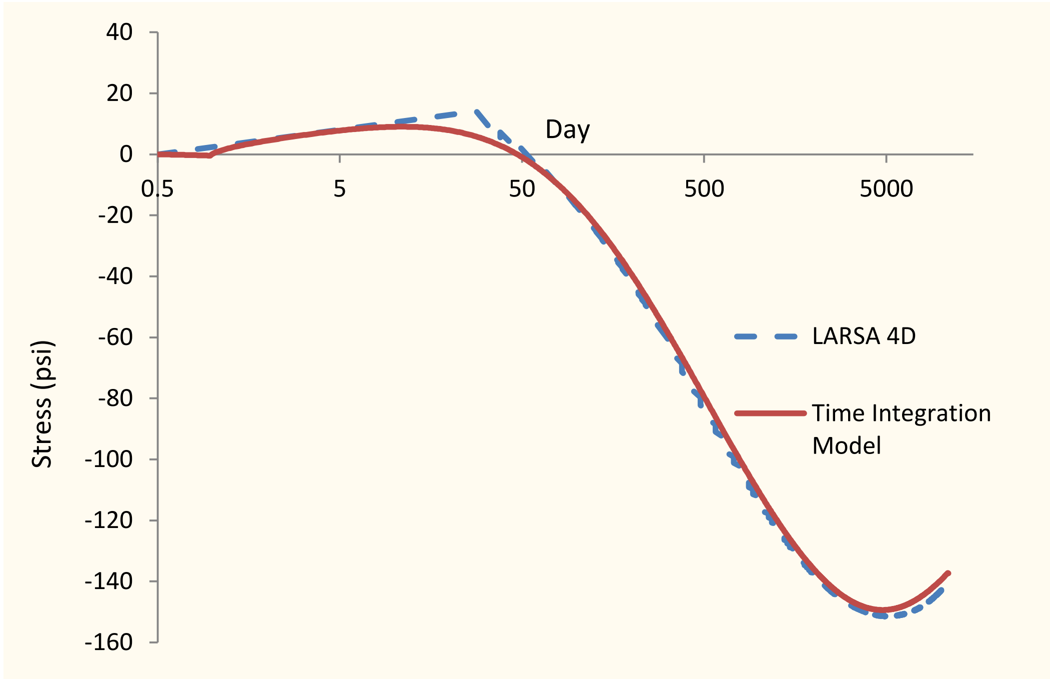

Fig. 5 - Comparison of Stress Development over Time in Topping Concrete between LARSA 4D and Time Integration Model (Positive is Concrete Tension)

From these figures, excellent agreement can be observed between LARSA 4D and the Time Integration Model developed in this paper. It shows that both analysis approaches use the same creep and shrinkage models per MC 90 and yield similar results. Note that Fig. 3 is indicating that the cap beam is losing most of its initial post-tensioning over time as it bleeds into the deck panel and topping concrete, increasing the compression in these two components (Figs. 4 and 5).

COMPARISON OF ACI 209R AND MC 90 CODE APPROACHES

The Time Integration Model was used to compare the differential creep/shrinkage analysis approaches per ACI 209R and MC 90. For the comparison, a slightly different deck geometry was assumed with a cross-section area of cap beam, deck panel, and topping concrete of 11.1 ft^2 (1.03 m^2), 39.3 ft^2 (3.65 m^2), and 29.5 ft^2 (2.74 m^2), respectively. It was assumed that the precast pile cap and deck panels were 90 days old by the time the topping slab was poured, and that the topping slab was wet cured for 14 days while the cap beam and deck panels were cured for only 1 and 3 days, respectively. The pile cap beam was prestressed to 670 psi (4.6 MPa).

Fig. 6, Fig. 7, and Fig. 8 compare the stress development over time between ACI and MC 90 on the pile cap beam, deck panel, and topping concrete, respectively. Again, zero in the time line marks the time of integration of the components.

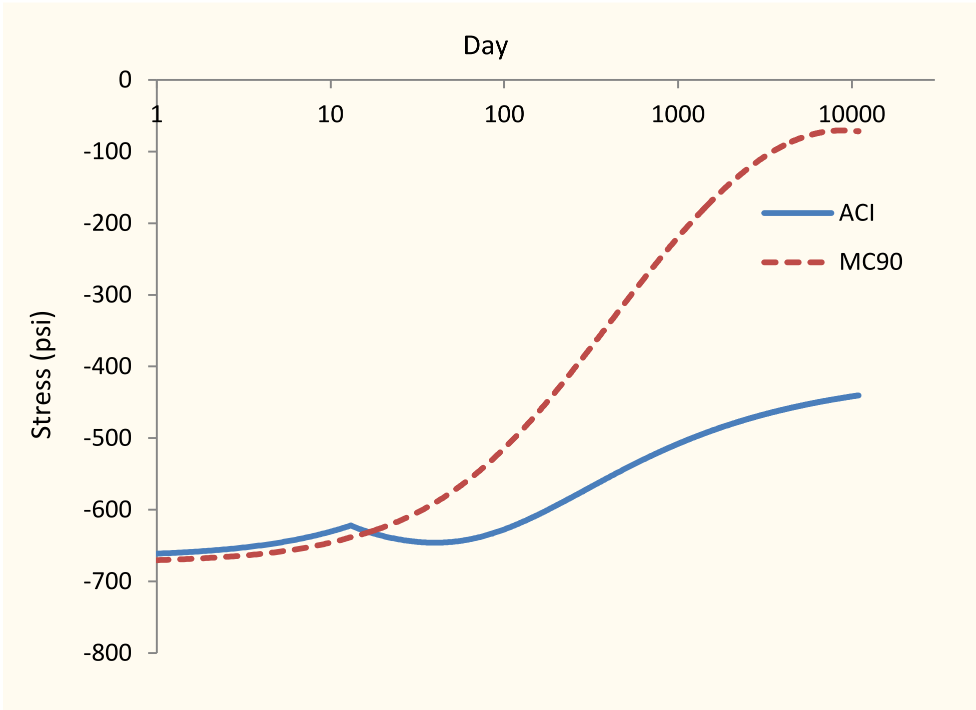

Fig. 6 - Comparison of Stress Development over Time in Pile Cap Beam between ACI 209R and MC 90 (Positive is Concrete Tension)

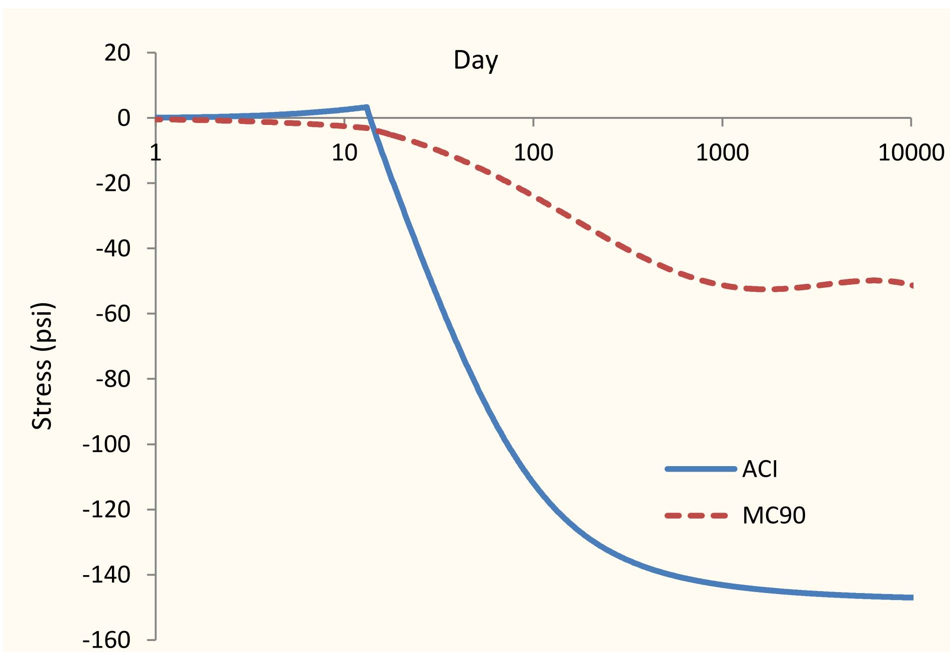

Fig. 7 - Comparison of Stress Development over Time in Deck Panel between ACI 209R and MC 90 (Positive is Concrete Tension)

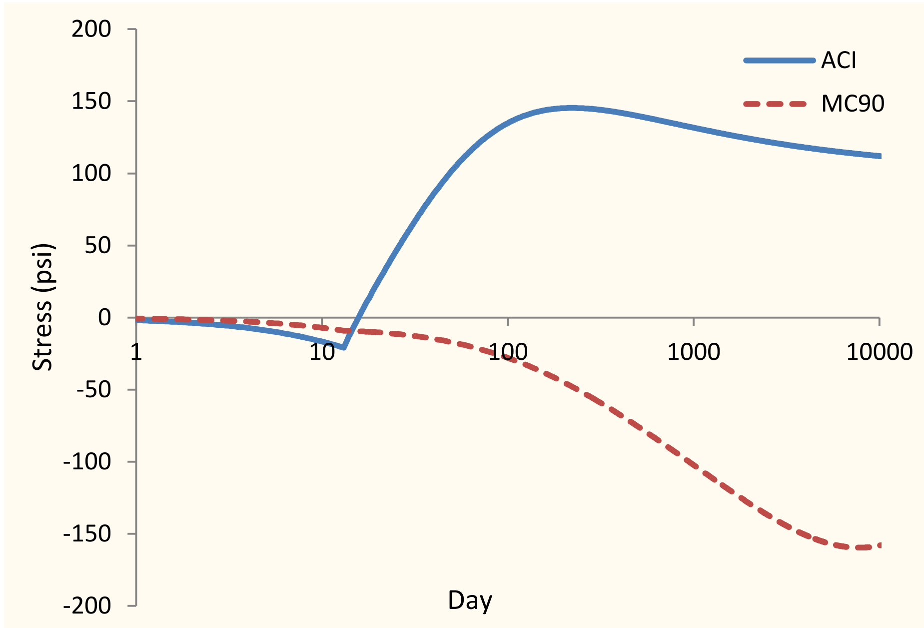

Fig. 8 - Comparison of Stress Development over Time in Topping Concrete between ACI 209R and MC 90 (Positive is Concrete Tension)

Significant difference in the stress development over time can be observed between ACI and MC 90. This difference is mostly due to the different definition for shrinkage behaviors over time defined in MC 90 and ACI. Fig. 9 compares the absolute shrinkage strain development over time evaluated using the two codes. It is obvious that the shrinkage strain rate within the first year is much higher per ACI than per MC 90. According to the definition per ACI 209, the ultimate shrinkage strain is almost reached by the time the concrete reaches an age of 400 days, while in MC 90 for that same period of time the strain is still less than a quarter of its ultimate shrinkage strain. It gets clear that if differential creep and shrinkage is evaluated within the first months after concreting, the ACI model is much more sensitive.

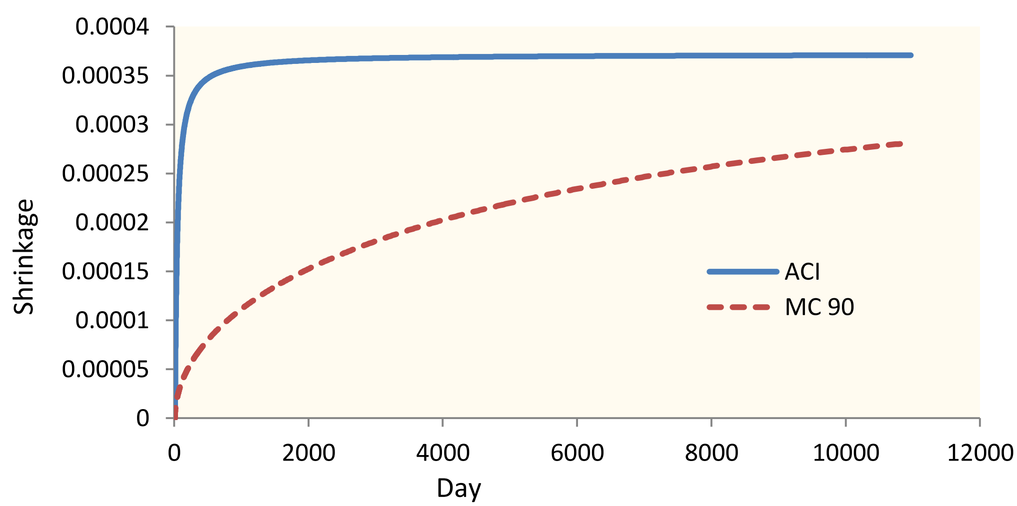

Fig. 9 - Comparison of Shrinkage Strain Development over Time per ACI 209 and MC 90

The ACI 209 creep and shrinkage model was introduced in 1972 based on research from the 1960s and has been reapproved by ACI in 2008. The MC 90 creep and shrinkage model was developed 1990 and has been adopted by Eurocode in 2002 after a model update in 1999[5]. Differences in the creep and shrinkage models have been discussed for years. A new shrinkage/creep model has been developed at Northwestern University under Zdeněk Bažant et al. and has been compared to other existing models in 2008[5]. Bažant suggested that, compared to other models, shrinkage and creep predictions with the current ACI 209 and MC 90 model provide the largest scatter if compared to existing creep and shrinkage testing data. No new creep and shrinkage model has been adopted by ACI, MC 90, or Eurocode since. However, ACI Committee 209 published ACI 209.2-08[6], a guide for modeling and calculating shrinkage and creep in hardened concrete that discusses several prediction models. The guide recommends that for structures sensitive to creep and shrinkage, test data on actual concrete mixes should be used to improve the accuracy of the prediction.

SINGLE STEP ANALYSIS OF DIFFERENTIAL CREEP AND SHRINKAGE

For further simplification of the creep and shrinkage behavior, the stress redistribution over time is often neglected. The differential creep and shrinkage strain is calculated for each individual component by simply subtracting the creep/shrinkage strain experienced at integration from the end creep/shrinkage strain. The stresses are then evaluated by meeting the force equilibrium and section compatibility under the shrinkage strain of the individual components. This single step approach, in essence, involves only one time step of the Time Integration Method. Table 1 compares the differential creep and shrinkage strain for the above-discussed pier deck using the time integration approach according to ACI and MC 90 with the single step procedure.

Table 1 - Comparison of Stress between Time Integration and Single Step Analysis

	Per ACI 209 in psi (MPa)			Per MC 90 in psi (MPa)		
Approach	Topping	Deck Panel	Pile Cap	Topping	Deck Panel	Pile Cap
Time Integration	111 (0.76)	-147 (-1.01)	-440 (-3.03)	-156 (-1.07)	-51 (-0.35)	-70 (-0.48)
Single Step	506 (3.5)	-483 (-3.33)	-300 (-2.07)	-213 (-1.46)	-127 (-0.87)	351 (2.42)

Note: Positive stress is tension in concrete

The results of Table 1 indicate that the single step approach overestimates the tension and compression stress on the sections. This is because the single step method does not rely on the stress redistribution with time; hence, the creep is overestimated for the pile cap beam and no creep effects are considered in the deck panel and topping concrete. For example, the single step approach per ACI generates tension in the topping concrete that is around four times the tension generated by the time integration approach. Likewise, the single step approach of MC 90 generates tension on the pile cap beam, which is unrealistic. Table 1 also shows that there is a significant difference between the stress obtained from ACI and MC 90. As was mentioned before, this difference is mostly due to the difference in shrinkage strain rates between the two codes.

SENSITIVITY OF CREEP AND SHRINKAGE BEHAVIOR PER ACI 209

The sensitivity of the creep and shrinkage behavior was studied using the Time Integration Method per ACI 209. The parameters under consideration were the age of precast concrete elements at time of structural integration, the time of wet curing of the topping concrete, the level of post-tensioning of the pile cap beam, and the use of a post-tensioned cast-in-place cap beam instead of the pre-tensioned precast cap beam. The sensitivity analysis has been performed on the above-defined deck section. Five different construction conditions with different model parameter sets have been considered.

1. **Standard Condition**
   This construction condition is the one that has been considered in the previous study comparing the two code approaches. Assumed is that the precast cap beam is prestressed to 670 psi (4.6 MPa) one day after casting and that the cap beam and deck panels are cured for 1 and 3 days, respectively, and structurally integrated at an age of 90 days. The topping concrete was assumed to be wet cured for 14 days.

2. **Precast Elements at 21 Days (21-Day PC)**
   This condition is the same as the standard condition but considering that the precast elements are only 21 days old instead of 90 days old at structural integration.

3. **Reduced Curing Time (3-Day Cure)**
   This condition is the same as the standard condition but considers that the topping concrete is cured only 3 days instead of 14 days.

4. **No Prestressing (NP)**
   This condition is the same as the standard condition but considers that the precast cap beam is not prestressed.

5. **Cast-in-Place Cap Beam (CIP Cap Beam)**
   This condition is the same as the standard condition but considers that the cap beam is not precast but constructed of cast-in-place concrete. The cap beam is assumed to be cured for one day, post-tensioned to 670 psi (4.6 MPa) three days after concreting, and structurally integrated thereafter.

Fig. 10, Fig. 11, and Fig. 12 present the development of the internal stress state from differential creep and shrinkage over time for the cap beam, the precast deck panels, and the topping concrete, respectively. Again, zero in the timeline marks the time of integration of the components. It gets obvious that the internal stress state is largely dependent on the considered parameters.

Fig. 10 suggests that the duration of concrete curing is the most important parameter to be considered for the internal stress state of the topping concrete. Per analysis, the lack of curing elevates the tensile stresses in the topping concrete by 200 psi (1.4 MPa). This does not come as a surprise, and it is common practice in marine construction to require 14 days of concrete curing of decks.

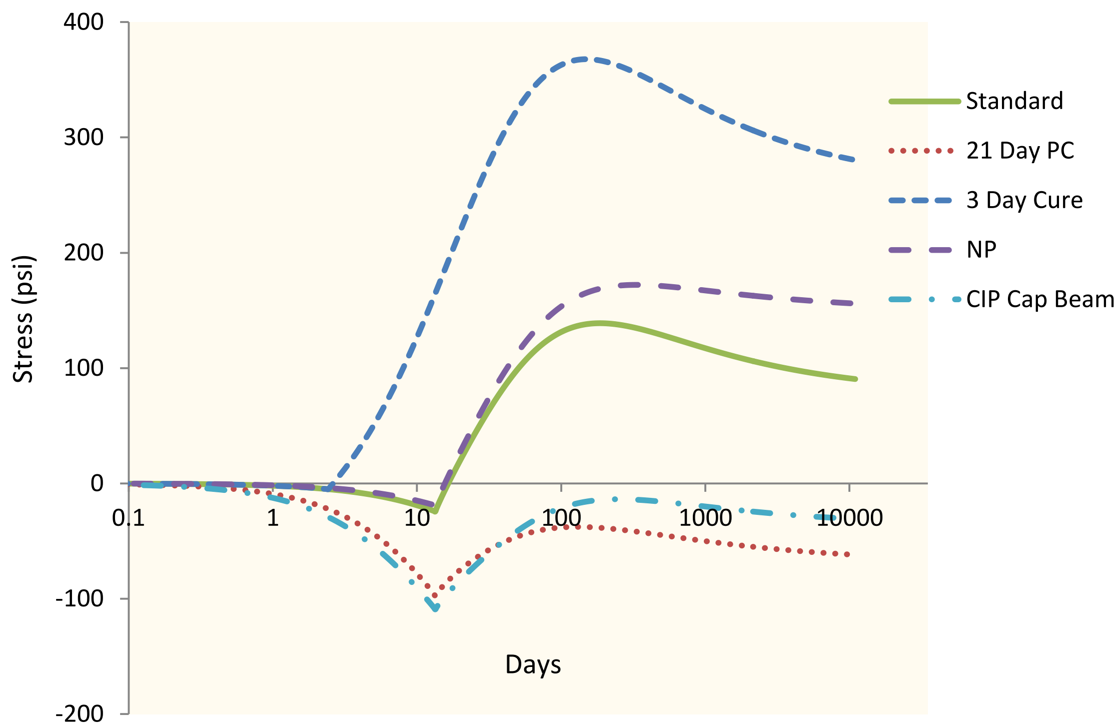

Fig. 10 - Stress Development over Time in Topping Concrete for Different Construction Conditions (Positive is Concrete Tension)

More surprising, though, is that the analysis suggests that integrating precast elements at a younger age dramatically changes the stress response in the topping concrete. Using a cast-in-place cap beam seems to have a similar effect. In particular with the ACI model, the majority of creep and shrinkage strain is developing at early concrete age, making stresses from differential creep and shrinkage very sensitive to the age of the precast components at structural integration. The younger the precast components are at integration, the more similar is their creep and shrinkage behavior to the cast-in-place concrete. However, precast concrete age at integration is a parameter over which the designer has only limited control. The designer, thus, would have to assume a range of potential ages to account for its influence on the internal stress state.

Fig. 11 suggests that the precast deck panel is mostly under compression. Shrinkage in the topping concrete imposes compression on the deck panel and so does some of the post-tensioning in the cap beam that bleeds into the deck. A cast-in-place cap beam is aggravating this effect.

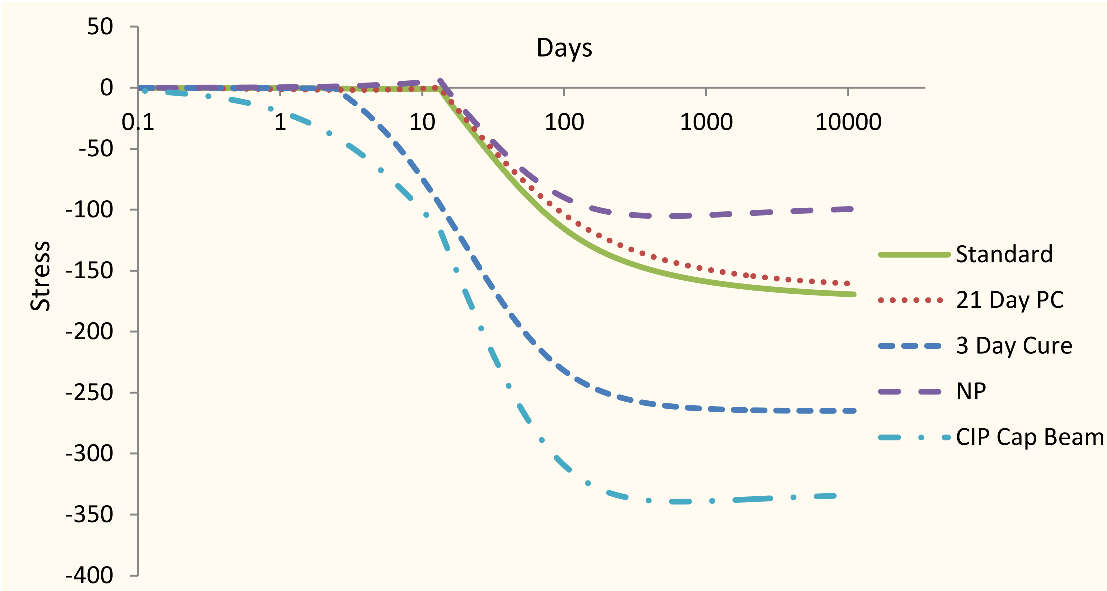

Fig. 11 - Stress Development over Time in Deck Panel for Different Construction Conditions (Positive is Concrete Tension)

Fig. 12 suggests that potentially most of the post-tensioning force of the cap beam will bleed into the deck structure due to differential creep and shrinkage. It even suggests that the cast-in-place cap beam may end up in tension if the precast panel constrains it from shrinking. A designer, thus, has to carefully consider how much of the prestress in a cap beam is available to be applied to crack control.

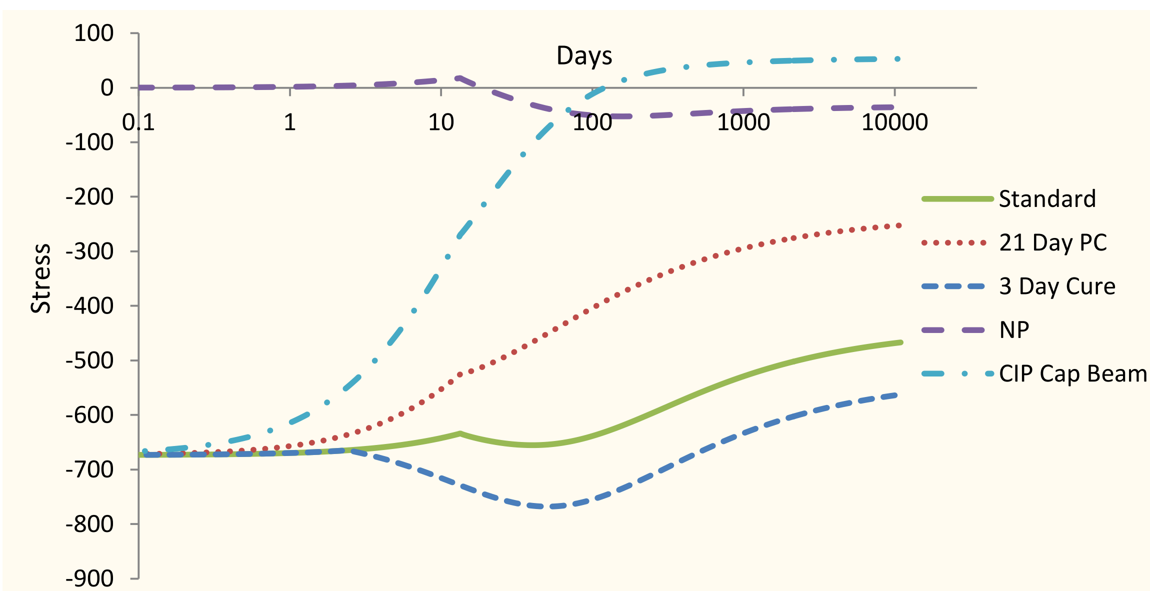

Fig. 12 - Stress Development over Time in Cap Beam for Different Construction Conditions (Positive is Concrete Tension)

CONCLUSION

The internal stress state in a pile supported deck made of precast elements and cast-in-place concrete topping can be evaluated by a time integration of a 1-D parallel spring model. However, large differences can be seen between calculating the differential shrinkage and creep stresses using MC 90 and ACI 209. The large difference in results comes mostly from the different assumptions of initial shrinkage strain rates. ACI assumes that most of the shrinkage is happening during the first months after concreting, while MC 90 spreads shrinkage more over time. The difference between the codes gets amplified as the first few weeks determine the development of the differential creep and shrinkage behavior over the lifetime of the structure. Furthermore, the creep and shrinkage behavior of the structural components are dependent on a number of factors that are often beyond the control of the designer, such as the construction sequence, the age and temperature of the precast concrete pieces at integration, concrete mix characteristics that influence creep and shrinkage, and curing and handling practices. The designer, thus, has to make assumptions that might not reflect the actual field conditions. There seems to be no satisfying current design approach that could evaluate the differential creep and shrinkage stress sufficiently enough to meaningfully consider the approach for crack control. The designer should use such approaches with caution and should use design and construction strategies that make the structure less vulnerable to differences in creep and shrinkage stresses, such as through the use of fiber-reinforced or shrinkage-compensated concrete mixes. It will be up to the research community to investigate the discrepancy of the shrinkage behavior between the two codes.

ACKNOWLEDGEMENTS

The study was part of the research topic, “Applications of Technologies/Methodologies to Advance the Use of Prestressing or Post-Tensioned Concrete in Design/Construction of Piers to Minimize/Eliminate Cracking under Load,” funded by Naval Facilities Engineering Command (NAVFAC) and the Department of Defense (DoD) through the Corrosion Prevention and Control (CPC) Program. Their support is greatly appreciated.

REFERENCES

1. BergerABAM, “Application of Technologies to Advance the Use of Prestressing or Post-Tensioned Concrete in Design of Pier to Minimize Cracking under Load,” Final Report, Contract 62583-11-C-0626, Naval Facilities Engineering Service Center, Port Hueneme, CA, April 12, 2012.
2. ACI Committee 209, “Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures (209R-92 Reapproved 2008),” American Concrete Institute, Farmington Hill, MI, 1992, 47 pp.
3. Comité Euro-International du Béton, “CEB-FIP Model Code 1990: Design Code,” Thomas Telford Ltd., London, UK, 1993, 464 pp.

4. Bažant Z.P., Li G.H., “Unbiased Statistical Comparison of Creep and Shrinkage Prediction Models,” *ACI Materials Journal*, V. 105, No. 6, November-December 2008, pp. 610-621.
5. ACI Committee 209, “Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete (209.2R-08)” American Concrete Institute, Farmington Hill, MI, 2008, 45 pp.