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MODELING AND ANALYSIS OF AGING BEHAVIOR OF CONCRETE STRUCTURES IN NUCLEAR POWER PLANTS

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INTRODUCTION

As nuclear power plants approach the end of their original design life and begin to transition to the life extension phase, consideration has to be given to the effects of structural aging when evaluating the extended operation of reinforced or pre-stressed concrete structures. Material aging of concrete is generally associated with changes over time in mechanical properties such as creep, modulus, and ultimate compressive and tensile strengths. Structural aging, on the other hand, is the combined effects of changes in the time-dependent material properties, the prior physical changes resulting from the structure's past operating history, and the structure's new loading environment. Because of the highly non-linear behavior of concrete structures, even under operating basis conditions, the structure's response to future operational and loading conditions will have to be performed using non-linear methods that consider the structure's changing physical conditions.

This paper focuses on three aspects of aging-dependent analytical modeling; 1) effects due to expected time-dependent changes in material properties, 2) effects due to unexpected degradation in material properties, and 3) effects due to actual environmental and loading conditions encountered. As to the first aspect of aging-dependence, certain characteristics of material aging are generally beneficial, such as the increase in time of the ultimate strength and modulus. However, this also means that the stiffness of the concrete structure also increases over time, and for loading conditions that are functions of stiffness, such as thermal and seismic loads, this could lead to some non-beneficial effects. For example, ambient temperature cycling or operationally-induced temperature changes could result in higher stresses, which in turn could cause further cracking and performance degradation. With respect to the second aspect of aging dependence, evidence of significant material property degradation is generally observed in unanticipated structural movement or unusual cracking trauma. Analytically simulating such material degradation is a necessary requirement for the correct prediction of the structure's future performance. The third category of aging dependence highlights the importance of treating the true environmental and loading conditions, rather than considering only assumed design conditions that may or may not be encountered over the life of the structure. For example, primary creep, which is the transient portion of the creep curve, can be activated with every change, increase or decrease, in the loading, and without considering the true creep behavior it would not be possible to correctly predict the effects of prestress retensioning or de-tensioning in PWR containments, as has recently occurred in containments subjected to hole-cutting processes to replace steam generators.

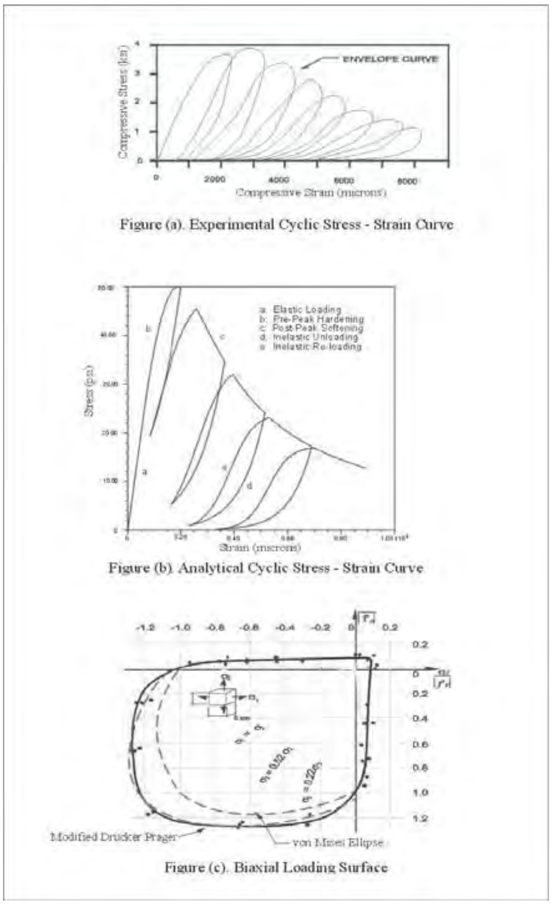
The capabilities needed in a concrete material model to capture the effects of aging on structural performance are discussed. Several example applications are presented to illustrate some engineering solutions for structures where modeling of aging effects was the key factor in the correct analytical prediction of structural performance. Finally, a discussion of issues for additional experimental testing and modeling that would help promote further understanding and quantification of structural aging is provided.

MATERIAL CONSTITUTIVE MODELING OF REINFORCED CONCRETE

The behavior of concrete is highly nonlinear, having low tensile strength, shear stiffness and strength that depend on crack widths, and a confinement-dependent compressive elasto-plasticity. A material constitutive model capable of simulating the behavior of reinforced concrete and capturing the effects of structural aging is provided through the ANACAP-U Material Model [1]. The model treats reinforced concrete as a three-phase composite: plain concrete material as a three-dimensional continuum phase, steel reinforecement (rebar) as a uni-directional phase, and a rebar-concrete interaction phase. The primary behavioral regimes considered for the concrete phase are: tensile cracking under multi-axial tensile stress field, compressive yielding and crushing, and post-cracking shear resistance that is a function of crack width. The steel reinforcement or rebar phase is treated as an elastic-plastic uni-axial bar that derives its local stress response from the surrounding strain field. The third, rebar-concrete interaction, phase modifies both the shear sub-model in the plain concrete phase and the elastic-plastic stress-strain curve in the rebar phase. This interaction phase is a distinguishing feature of the ANACAP-U Material Model, as it provides the distinction between treating reinforced concrete as a true composite versus the simple additive superposition of rebar and concrete stress-strain relationships. In addition, the material model capabilities include the modeling of primary and secondary creep, autogenous shrinkage, and the increase in elastic modulus and strength with time, all as function of temperature where appropriate. The material model's capabilities summarized above are described in some detail below.

Constitutive Modeling of the Concrete Phase

Addressing the compression regime first, the continuous stress-strain curve is defined from uniaxial compression data, which is then generalized to multi-axial stress/strain states using the uniaxial equivalence of the multi-axial state, namely, the effective stress and the effective strain. The uniaxial behavior is generalized to multi-axial behavior, within the analytical framework of isotropic hardening plasticity formulation, using a Drucker-Prager surface to represent the loading surface under multi-axial compression. In this formulation, the loading surface is a function of the hydrostatic pressure, the second invariant of the deviatoric stress tensor, and the yield strength. This type of formulation incorporates the effects of low to moderate confinement stress levels, which typifies the behavior of civil structures. These relations allow for linear behavior for compressive stresses below about 50% of the compressive strength, and then strain hardening behavior until the compressive strength is reached. Strain softening behavior develops for continued compressive loading past the strain capacity at the compressive strength. This behavior is illustrated in Figure 1, which depicts the uniaxial stress-strain curve in compression subjected to cyclic loading, Figure 1(a), showing continually degrading stiffness and strength with each cycle. Model simulation of this behavior is shown in Figure 1(b), and the biaxial loading surface is shown in Figure 1(c). The tensile behavior is illustrated in Figure 2, which is discussed in more detail below.



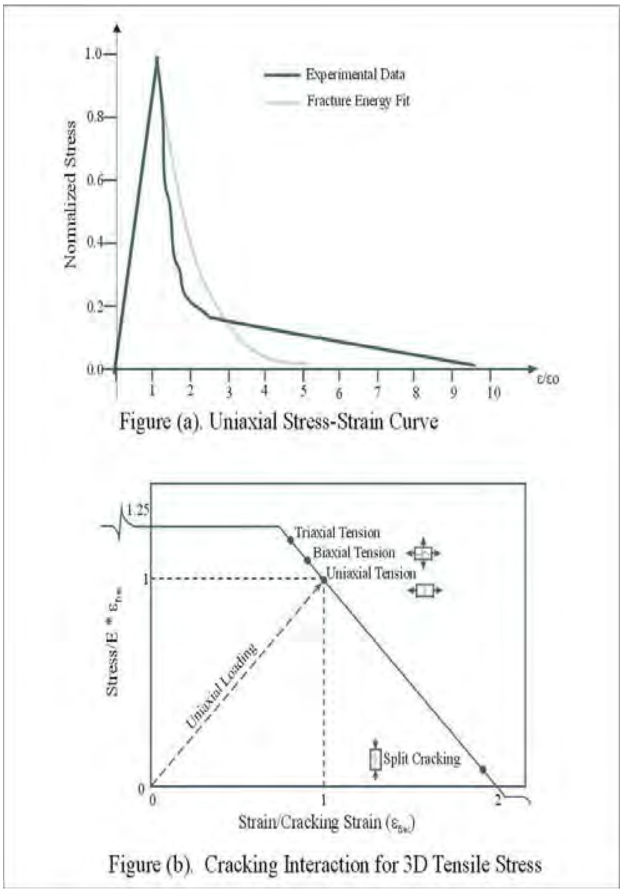


Figure-1 Behavior under Compression **Figure-2 Behavior under Tension**

Tensile cracking in the concrete phase is represented by the smeared-cracking model [2], in which the crack surface is taken to be the principal plane where the crack-surface normal prescribes the principal strain direction. Multiple cracks can form at each material point, but they are constrained to be mutually orthogonal. Once a crack forms, the direction of the crack remains fixed and can never heal. However, a crack can close, re-open under load reversals, and resist compression and shear. Tensile behavior is illustrated in Figure 2, which depicts a stress-strain in uniaxial tension in Figure 2(a) and is generalized to multiaxial tension through the interaction curve in Figure 2(b), where the stress and strain in the figure are the principal quantities. The model predicts cracking when the stress and strain state exceeds the limit state shown. Thus, under biaxial or tri-axial tension cracking occurs at a slightly higher stress and slightly lower strain than in the uniaxial case. The interaction curve models split cracking, which can occur in a parallel plane to a high compressive stress where low or near-zero tensile Poisson stress develops, generally near a free surface, by assigning a cracking strain limit of twice that under direct tension.

When cracking is initiated, the tensile stress normal to the crack surface, which is the principal plane, is reduced to zero and the stresses for the material point are recalculated in a subsequent iteration, thereby restoring equilibrium through stress redistribution to reinforcement and other points in the structure. Since the shear stress on the principal plane is zero by definition, the smeared-cracking model in its original form could not properly simulate the post-cracking shear behavior of concrete. This capability evolved later, through further research and experimental verification, from its initial form of a smooth-crack model to the development of the rough-crack model which is utilized in ANACAP-U. Figure 3 depicts the evolution of post-cracking shear stress with increasing shear strain acting over the crack surface, for two modes of crack-opening displacement. The development of shear stress, as shown in the figure, would not be possible without the presence of a compressive stress normal to the crack surface, which can be either a direct result of stress redistribution or by the presence of a rebar crossing the crack, where the relative motion of crack surfaces puts the rebar in tension and the concrete in compression. This behavior characterizes the rebar-concrete-interaction phase, as described below.

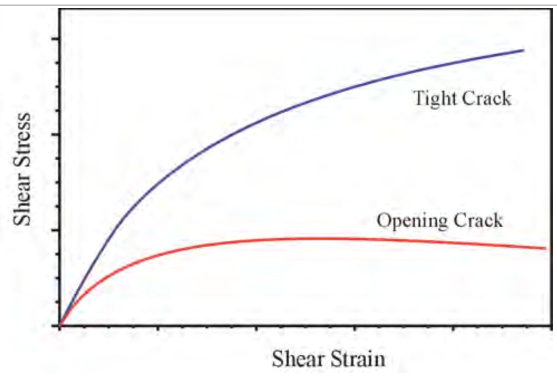


Figure-3 Example of Shear Stress Evolution on a Crack Plane

Constitutive Modeling of the Interaction Phase

The surfaces of cracks that develop in concrete are usually rough and irregular. When a shear force is applied parallel to the crack surface, tangential shear sliding occurs, and this induces displacements normal to the crack surface as the crack surfaces ride up on surface asperities. When this normal displacement is restrained by reinforcement crossing the crack, tensile stresses will develop in the steel bars, which will then induce compressive stresses across the crack in the concrete. The resistance to shear is provided by the frictional force generated by the compressive stress across the crack. The crack width is the primary variable affecting this mechanism of shear transfer. Smaller crack widths correspond to greater shear stiffness and strength. Aggregate size, reinforcement design, and concrete strength are other important factors. Two sub-models are developed to provide this shear resistance capability in the crack plane: the first operates on the shear stiffness by reducing the shear moduli in the plane of the crack, first by 60% when a crack forms followed by gradual reduction in proportion to the opening strain normal to the crack. The second sub-model operates on the stress through a shear-retention/shear-shedding feature that tracks the evolution of shear stress across an open crack. Following the formation of the crack, the shear retention feature allows the shear stress to be calculated in the usual way through the incremental stress-strain relations. However, if the strain increment normal to the crack plane begins to turn tensile, indicating a widening crack, the shear-shedding feature reduces the shear stresses previously supported across an open crack if the crack continues to open. This representation of post-cracking shear response is depicted in Figure 4, with Figure 3 illustrating the evolution of shear stress for two crack deformation modes. This loss of shear capacity as cracks open is a major contributor to structural failures in reinforced concrete structures.

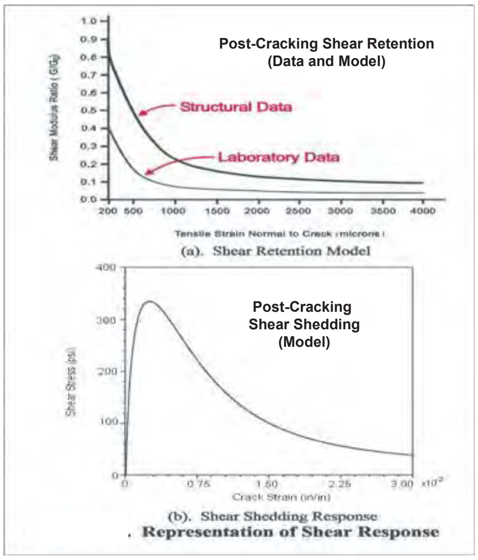
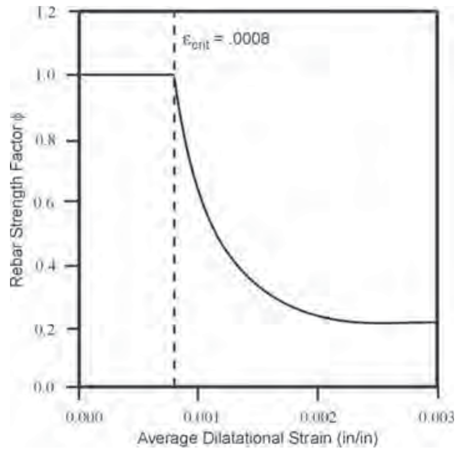


Figure-4 Modeling of Shear Behavior

Constitutive Modeling of the Rebar Phase

The rebar interaction with the surrounding concrete described above for the case of post-cracking shear behavior, is generalized further to consider the effects of rebar-concrete bond behavior. The rebar sub-element stiffness is first derived in the usual way employing standard J_2-plasticity formulation and modified by a bond-slip model. Bond slip is treated by monitoring the concrete dilatational strain normal to the rebar, and then degrading the rebar's effectiveness by introducing a rebar yield-strength reduction factor, Figure 5, as a function of the dilatational strain in the surrounding material; total bond loss occurs when the strain normal to the rebar reaches 0.1% or a dilatational strain of 0.3%. For cyclic loading, the bond-slip model is used to modify the rebar hysteresis loop, where the hysteresis loop is pinched as function of the dilatational strain, Figure 6, in the concrete material surrounding the rebar. The modified rebar model was empirically calibrated to observed material data.



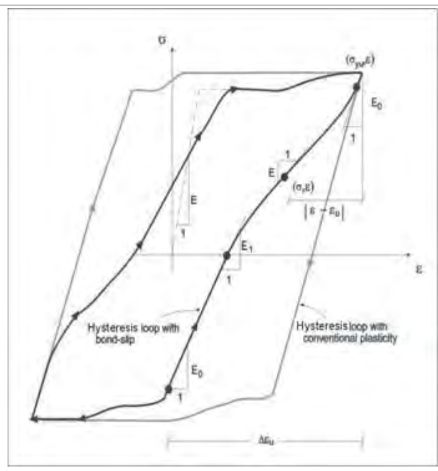


Figure-5 Rebar Yield Strength Factor

Figure-6 Pinched Rebar Hysteresis Loop

Constitutive Modeling of Energy Dissipation and Damping under Dynamic Loading

Under dynamic loading, the response of a concrete structure is strongly dependent on internal energy dissipation due to cracking, which is usually accounted for by using higher damping ratios. For uncracked concrete, where the material response can be assumed to be isotropic, a reasonable value is 2%. However, when cracks form, a large amount of energy loss occurs locally, anisotropically, at the crack locations. The use of a single damping value to simulate this energy-loss would not be valid, and using linear analysis with higher uniform damping can under-predicting the response. In order to simulate the cracking-induced energy loss, a crack-consistent damping model, which allows energy losses associated with open cracks to be modeled explicitly, is included in the material model. The development of the crack-consistent damping model was motivated by containment model experiments conducted by NUPEC at the Tadutsu shaking table [3], the results of which are summarized below.

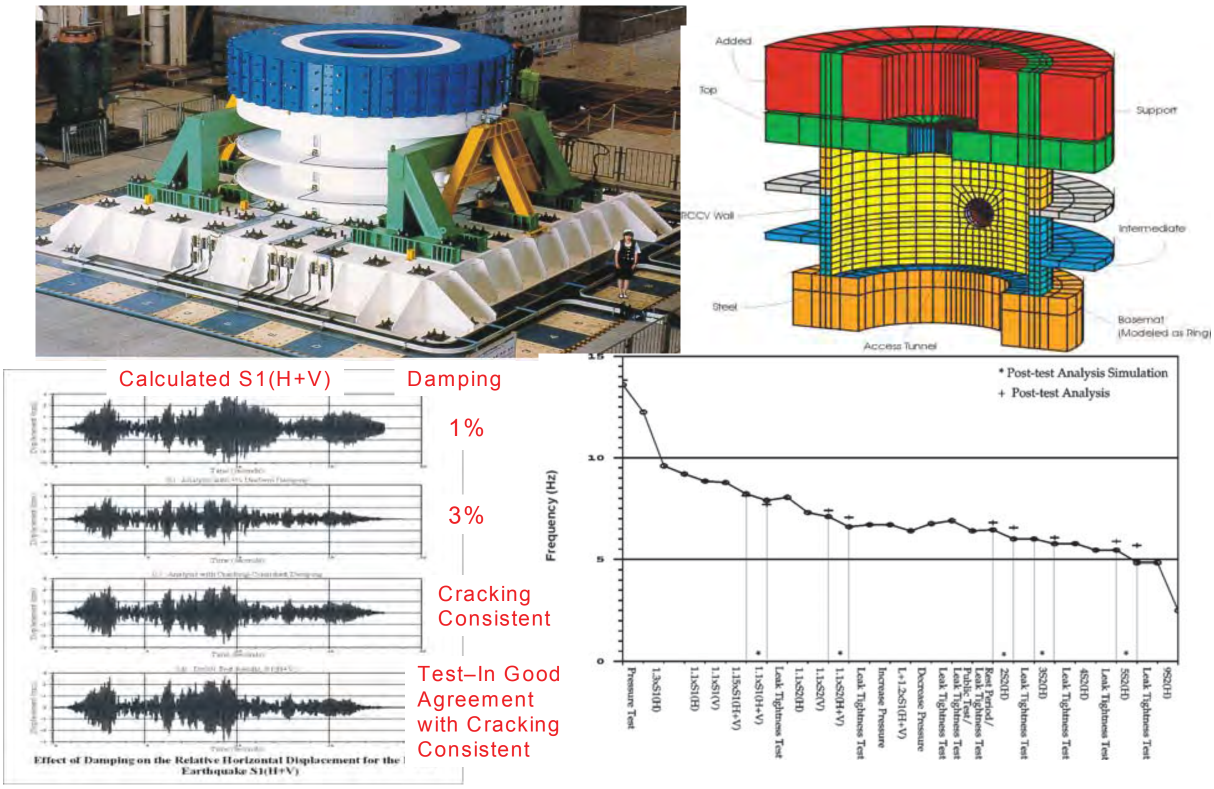


Figure-7 NUPEC's Seismic Test of a Model of Reinforced Concrete Containment Vessel (RCCV)

Figure 7 above depicts the RCCV/shaking table configuration juxtaposed with a finite element model of the RCCV. Three analyses were performed, using the ANACAP-U material model as described in this paper, for combined horizontal and vertical motions of a design-basis earthquake S1(H+V). Two of the analyses were performed with uniform damping of 1% and 3% respectively. The third analysis was performed with the cracking-consistent damping sub-model, which shows close agreement with the test results. The structural degradation with repeated shaking, represented by the continuous decrease of the fundamental frequency is shown in the lower right corner figure.

Constitutive Modelling of Creep

Creep is treated in the material model using a hereditary integral of the form,

$$\varepsilon^c(t) = \int_0^t J(\xi - \xi') \frac{d\sigma(\xi')}{d\xi'} d\xi, \quad (1)$$

where J is the uniaxial creep compliance, which is independent of stress, σ is the stress, and ξ and ξ' are equivalent times that account for temperature variation with time. An example of the creep relations is based on the data in [4]. The creep compliance is a function of temperature and time represented as,

$$J(T,t) = \phi(T) \cdot C(t), \quad (2)$$

where $C(t)$ is the specific creep with time, in units of micro-strain/psi, defined by

$$C(t) = 0.1936(1 - e^{-.069t}) + 0.280(1 - e^{-.0069t}) + 0.375(1 - e^{-.00069t}) + 0.348(1 - e^{-.000069t}) \quad (3)$$

and $\varphi(T)$ is a multiplier based on temperature, defined by

$$\phi(T) = 226.09 - 0.00429T + 147.52T^{-0.367} - 309.26T^{-0.044} \quad (4)$$

where t is time in hours and T is temperature in °F. The specific creep strain is illustrated in Figure 8 for 5 ksi nominal concrete held at various temperatures.

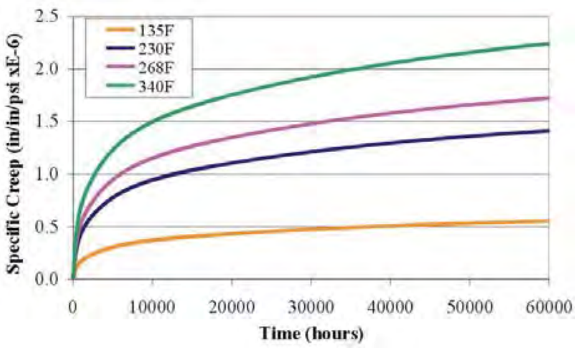


Figure-8 Specific Creep Strain at Elevated Temperatures

Creep, can promote structural degradation through prestress retensioning to compensate for tendon relaxation in PWR containments, and plays a significant role in containment re-licensing following structural modification for steam generator replacement, as will be discussed in an example analysis presented in a later section.

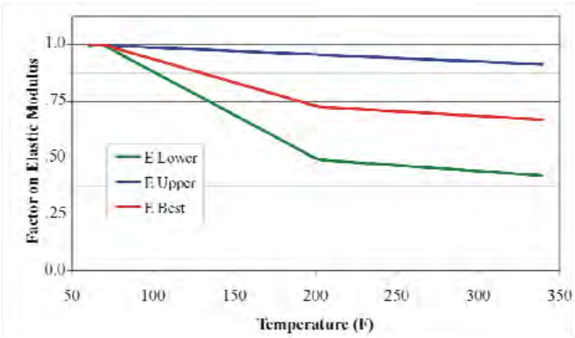
AGING OF CONCRETE STRUCTURES DUE TO MATERIAL DEGRADATION

Time-dependent material degradation mechanisms that can affect the long term performance of concrete structures include temperature and alkali aggregate reaction (AAR). Temperature effects can be service induced, or can be caused by climate changes. AAR, however, is an internal slow-evolving physico-chemical process that causes volumetric swelling akin to irradiation-induced void swelling in austenitic stainless steel. The treatment of temperature and AAR effects are discussed below.

Material Properties Degradation as Function of Temperature and Time-at-Temperature

Exposure to elevated temperatures is known to have a deteriorating effect on the physical and mechanical properties of concrete. At elevated temperatures, micro-cracking will develop between the aggregate and the cement paste due to differences in thermal expansion characteristics. At temperatures above 200°F, concrete properties degrade as function of time at constant temperature. This degradation is associated with the movement of free water and water of hydration within the hardened cement paste, which results in shrinkage of the paste. These changes cause degradation of the modulus, compressive strength, tensile strength, and creep compliance. Furthermore, this degradation of properties is non-recoverable upon return to lower temperatures since micro-cracking and moisture migration are irreversible.

The degradation of the concrete properties with temperature is separated into 2 parts, degradation as a function only of temperature, and degradation as a function of time held at elevated temperature. Degradation of concrete properties, modulus and compressive strength, with temperature is described in great detail in [5], based on a comprehensive review of test data that include a wide variety of test conditions, concrete mixes, and temperature conditions, and are recommended by the Department of Energy as a basis for the design of concrete structures at elevated temperatures. This characterization for modulus and compressive strength degradation as a function of temperature is illustrated graphically in Figure 9. Note that relations for temperature dependent tensile strength are also included, but not shown here.



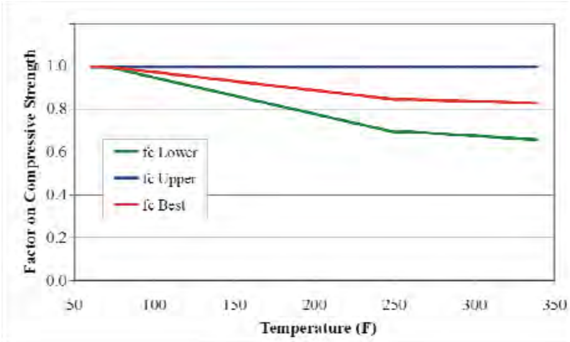
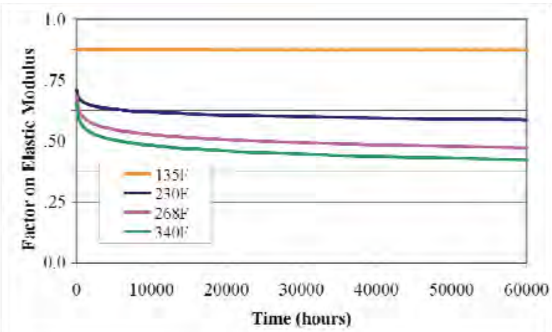


Figure-9. Property Degradation with Temperature

Time-dependent degradation of properties at constant temperatures is described in [6], based on test data developed by the Department of Energy for underground nuclear waste storage tanks at the Hanford Reserve. The tests involved concrete with 3.0 to 4.5 ksi nominal compressive strength and temperatures up to 450°F. Figure 10 illustrates the time-dependent component of degradation for modulus and compressive strength when held at various constant temperatures.



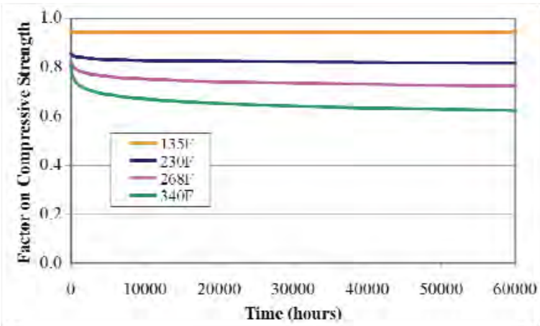


Figure 10. Property Degradation With Time at Constant Temperature

Structural Performance under Sevice-Induced Thermal Cycling

Changes in operational and loading environments experienced by NPP safety structures may cause structural degradation that require evaluation before they can be considered for extended operation. An example application, illustrated in Figure 11, is presented where aging effects due to thermal cycling at elevated temperatures are evaluated as part of the design process. The analysis is intended to demonstrate the structural integrity of the primary containment system for the GE-Hitachi ESBWR standard plant design for 60 years of thermal duty cycle operations [7]. The top slab of the containment spans the 36-meter diameter containment drywell with a central 10.5-meter diameter opening for the drywell head while supporting the water and equipment in large water pools. During normal operation, the water pool will undergo duty cycles where the water gets rapidly heated to boiling for some period of time and then cools down. This top slab structural system is subjected to the elevated temperatures that occur in these water pools and to thermal cycling due to temperature changes in the pools and in the drywell portion of the containment during shutdowns. These cyclic thermal demands interact with a changing structural condition because of concrete cracking, creep, and property degradation at elevated temperatures. Thus, there is a potential for structural ratchetting of the slab, with continually increasing deformations over time under the thermal cycling while supporting the pool loads. The long-term structural integrity of the top slab as a containment boundary must be verified for this duty cycle operation over the 60-year design life.

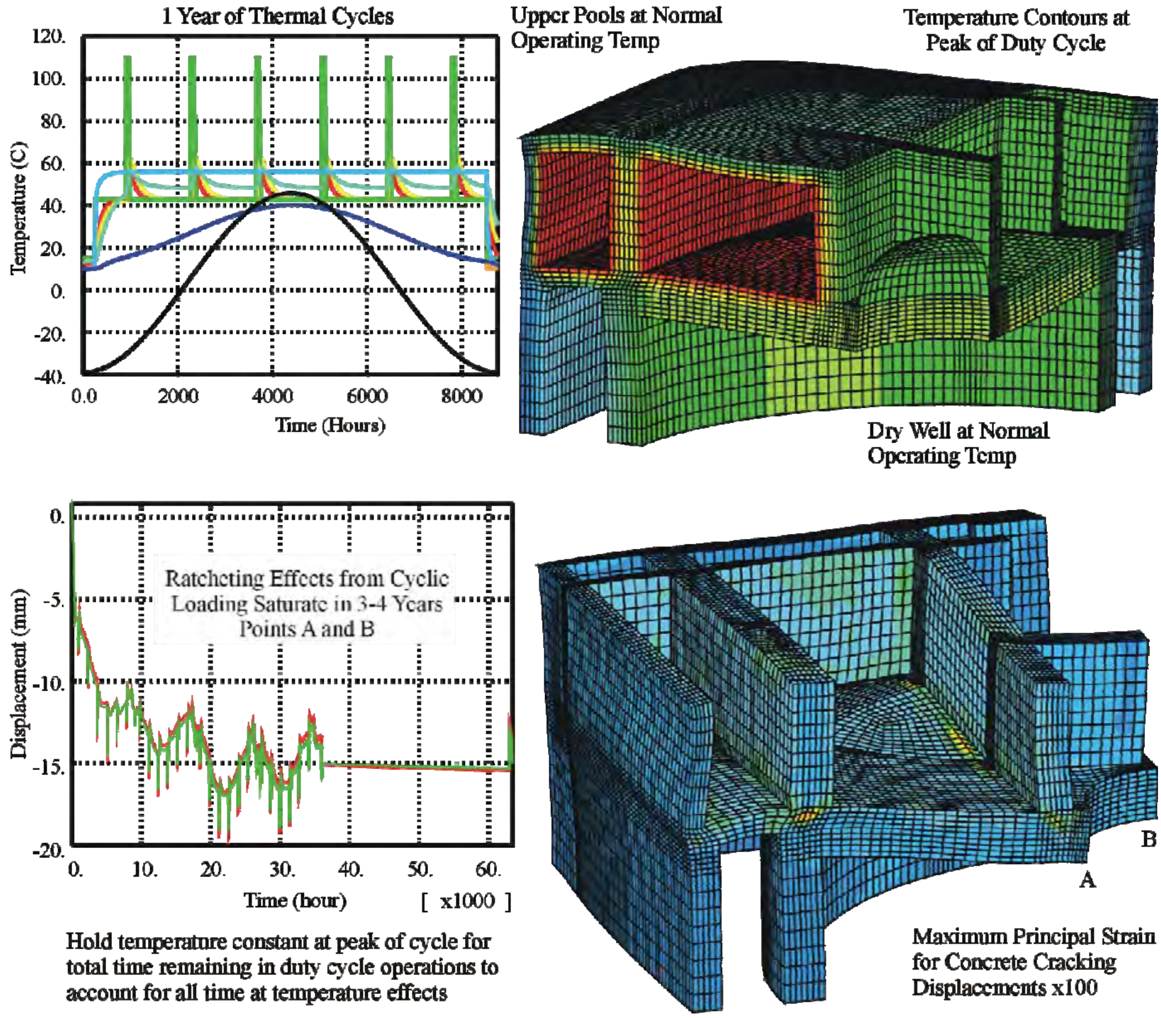


Figure 11. Response of ESBWR Containment System to Annual Thermal Cycles

Alkali Aggregate Reaction (AAR)

AAR is produced by a continual chemical reaction between the aggregate and the alkali in the cement, which causes volumetric expansion in the concrete over time. This expansive growth in the concrete, coupled with inherent restraints against movement, can cause severe concrete cracking. In addition, misalignment in mounted mechanical equipment can develop due to the relative movement of points on the structure, and this can lead to excessive component wear and eventual safety concerns. While the root cause of the AAR condition is well understood, the structural effects are very complex since the expansive growth is a function of the induced stress, as well as the environment and even the distribution of the constituents within the structure. What makes this mechanism of particular concern is the fact that it is an insitu mechanism that occurs late in time, which makes it an unrepairable condition. It can be avoided by conducting a′priori tests to help select the type of aggregate that is chemically compatible with the cement. This type of testing, however, is seldom performed, especially in NPP sites with limited sources of aggregate material.

AAR has been identified as a possible aging/damage mechanism for some reactor containments in Canada [8], but has not been considered an issue in the United States. However, it has been observed in waterway structures in the US, with a not-insignificant cracking damage. We have included in this paper, see discussion below, an analysis of a Lock, although a non-nuclear structure, to illustrate the extent of an AAR-induced damage.

A study performed to evaluate the future performance of a Lock wall subjected to AAR [9,10] is illustrated in Figure 12 as an example application considering the aging effects due to this type of material degradation. The effects of swelling due to AAR are simulated by modifying the time dependent shrinkage capability in the concrete model with a spatial distribution of expansion rate. The key is to benchmark the swelling effect with field data for both global movement of the lock wall and the local stress distributions as exhibited by the cracking history. The model is benchmarked to field data by simulating the structural history, including structural repairs that have been implemented, and matching measured global movement and observed cracking to the calculated results. The structure's future performance is then evaluated by extending the analysis in time, applying environmental and loading conditions based on past history. Using the results of this deterministic analysis, a probabilistic approach was also utilized to evaluate the structural reliability for future service.

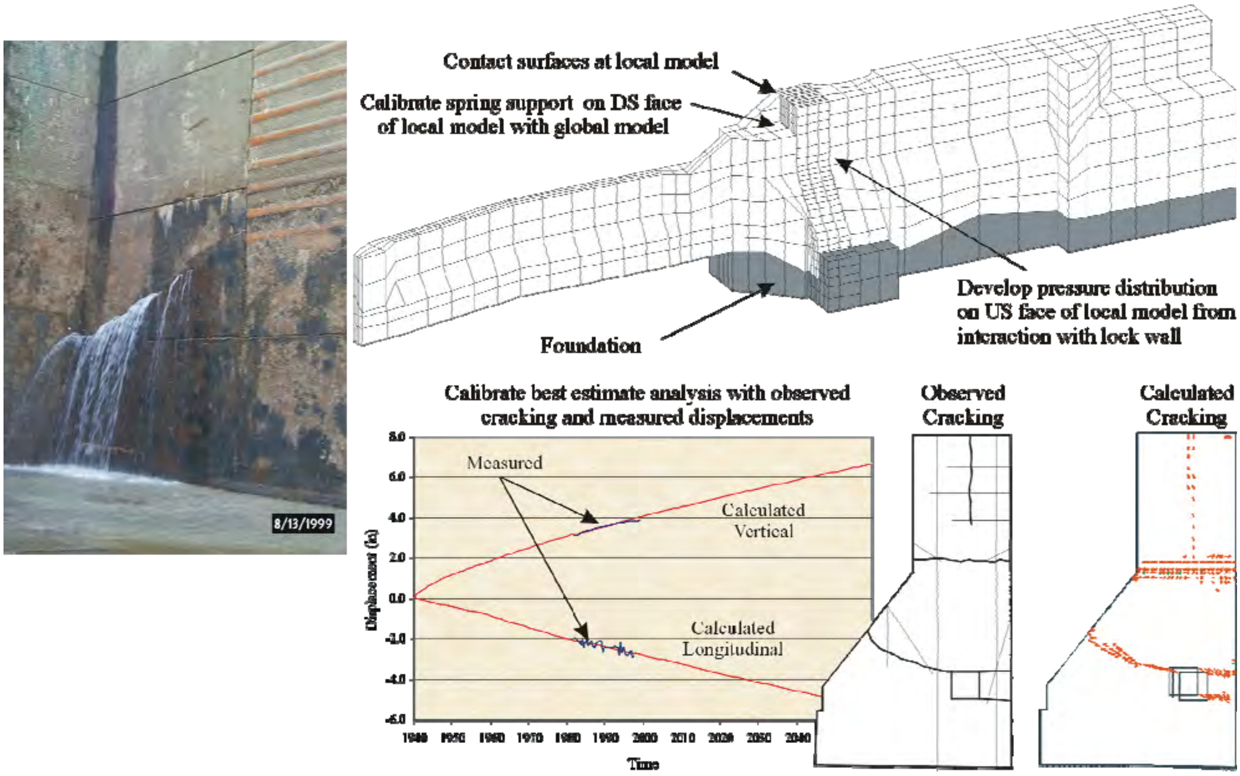


Figure 12. Example Application of the Effects of AAR in Large Concrete Structures

AGING OF CONCRETE STRUCTURES DUE TO CHANGES IN SERVICE AND LOADING CONDITIONS

A potential and often ignored structural damage mechanism is the effect of creep, particularly primary (recoverable) creep, on reactor containments, which are post-tensioned prestressed structures. Some of these structures have in recent years been subjected to structural modification involving the detensioning of prestressing tendons to create openings for the removal of large equipments, such as steam generators, that are too large to pass through the equipment hatch. The sudden removal of prestress loads from a quiescent structure creates two potential damage conditions: stress redistribution under reversed loading, possibly causing new cracking, and the reactivation of primary creep, which in turn activates new damage states as the structure tries to accommodate the changes in the loading with time. Because of possible impairment of structural performance after it is repaired, an evaluation of this condition may become a requirement for returning the structure to service. To this end, a demonstration analysis of a 60° sector of a post-tensioned cylindrical wall of a PWR containment was performed to evaluate the effects of primary creep recovery following a partial de-tensioning of the tendons in the wall.

The computational model consisted of a 3D finite element grid for a wall of typical old-vintage containment design was selected, having a thickness of 42 inches and an outside diameter of 137 ft. The height of the uniform 42" section of the wall was 102 ft. with a 4 ft. ring girder at the top. The left and right sides of the wall were bounded by 6 ft. wide buttresses. The wall was post-tensioned in both the vertical and circumferential (hoop) directions using un-grouted tendon strands contained within 5 in. tendon ducts. The hoop tendons were positioned 10 in. from the exterior of the wall and the vertical tendons were positioned immediately inside the vertical tendons, eccentric to the wall centerline. The area of the hoop tendons was 12 in^2 and the spacing was 39 in. The area of the hoop tendons was 12 in^2 and the spacing was 35 in. Only the hoop and vertical tendons in the center of the wall were de-tensioned. 8 hoop tendons were de-tensioned over a vertical distance of 20 ft. and 10 vertical tendons were de-tensioned over a horizontal distance of 23 ft. In addition to the hoop and vertical tendons, a grid of # 9 hoop and vertical rebars (area = 1 in^2) were placed 3 in. from the outside diameter at a horizontal and vertical spacing of 12 in. A 3/8 in. thick steel liner was bonded to the inner surface. The time-dependent analysis was performed using the following series of load steps: (1) dead load and tendon tensioning; (2) primary creep over 250 hours; (3) secondary creep over 30 years; (4) partial tendon de-tensioning; and (5) primary creep recovery over 250 hours.

The results of the analysis are depicted in Figure 13, whose most important feature is the occurance of wall delamination. This is depicted in a sequence of contour plots that describe the evolution of the split-cracking strain, which is the strain normal to the wall surface, and is a measure of the delamination. Figure 14 shows a plot of a ¼ symmetry section, (shaded portion), of the analysis model at 160 hours after detensioning showing a through-thickness dilation of about 17%, (see the strain scale in the figure), for the outer 10-inch concrete layer that forms the cover for the prestressing tendons, indicating that a dilamination width of 1.5 to 2 inches is possible. This analysis demonstrates the importance of modeling and simulation for root cause evaluation of unusual behavior.

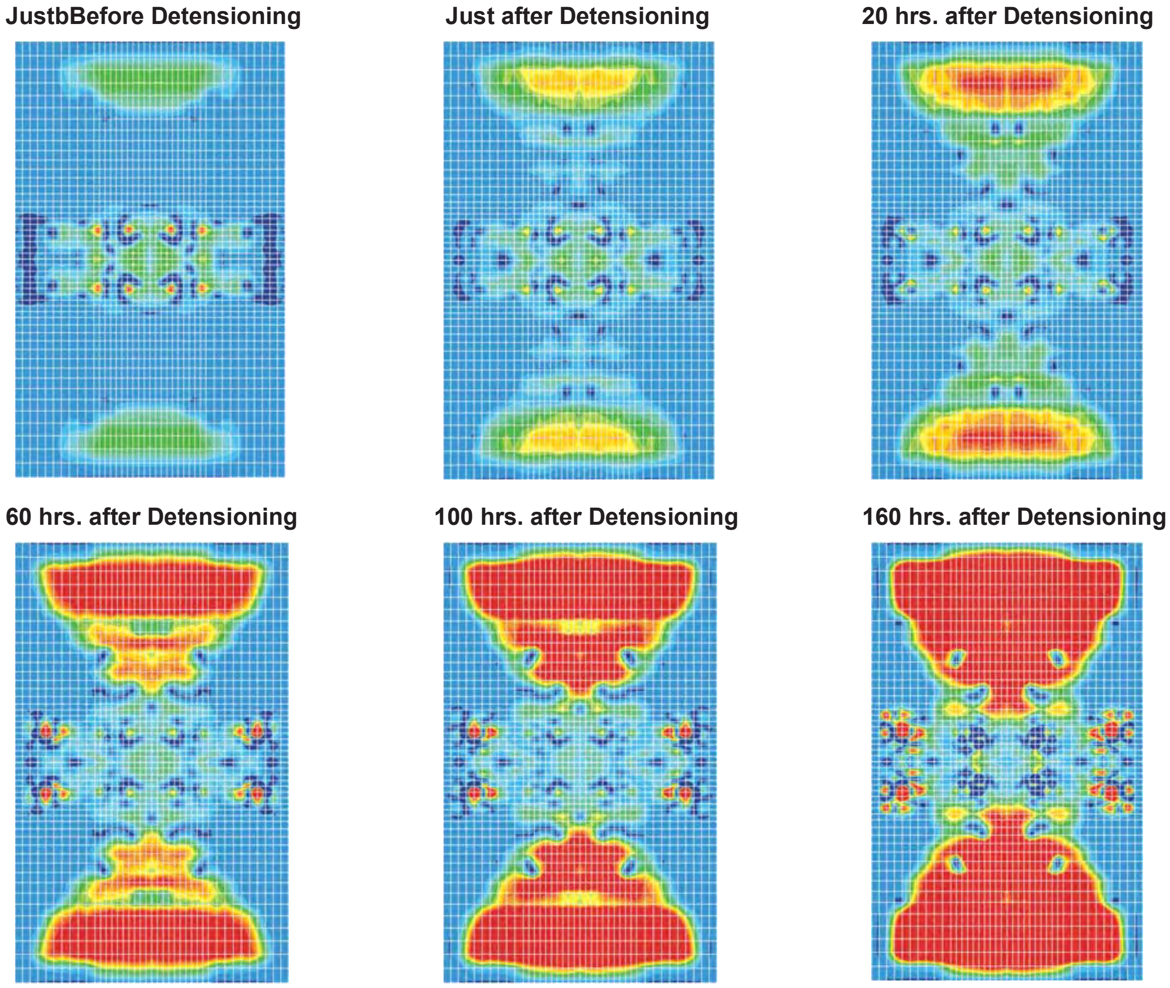


Figure-13 Contour Plots of the Evolution of the of the Delamination Surface

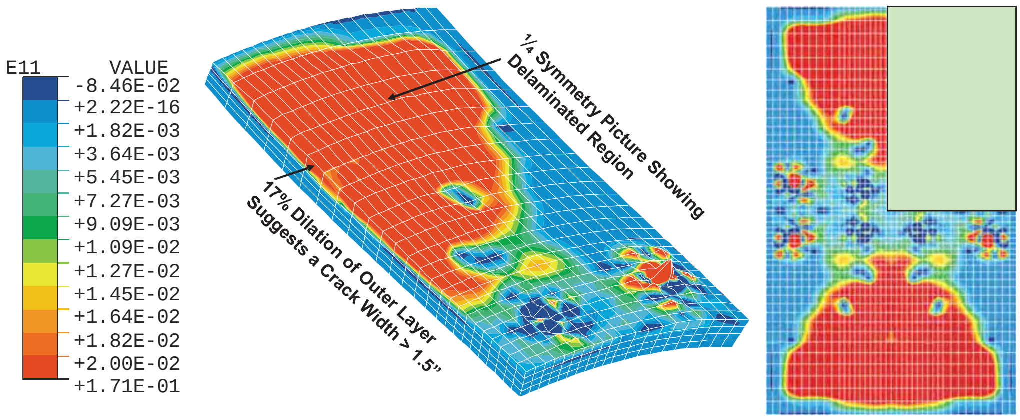


Figure-14 ¼ Symmetry Section of the Analysis Model Showing the Through-Thickness Dilation

SUMMARY AND CONCLUSIONS

A concrete material model is described having the appropriate capabilities required for evaluating structural aging. Structural aging is defined as the combined effects of time dependent material properties degradation and service-induced changes in loading and operational conditions. Three broad categories of structural aging, and the interaction between them, are considered: 1) Aging effects due to expected time dependent changes in material properties, 2) Aging effects due to unexpected time dependent material degradation, and 3) Aging effects due to operational environment and loading. Example analyses are presented which illustrate the value of using advanced modeling and simulation in evaluating expected and unusual structural behavior. This is particularly important for safety structures that are approaching the end of their design life and are facing the prospect of re-licensing for extended operation.