Material Modeling Related to Underground Nuclear Waste Structures

R. J. James, Y. R. Rashid
ANATECH Research Corporation, La Jolla, CA USA

INTRODUCTION

At temperatures above 200°F, concrete exhibits a significant departure in its elastic and viscoelastic behavior from that observed at lower temperatures. This occurs because of thermally activated damage that is evidenced by the degradation of the elastic modulus during hold time at constant temperatures. Furthermore, the loss in modulus is not recoverable upon return to lower temperatures. In addition, the compressive strength, tensile strength, and creep compliance degrade with time under increasing and constant temperatures so that the material response is highly nonlinear and exhibits a complex dependence on the temperature history. Thus, the response of a concrete structure at elevated temperatures is not only a function of the temperature but also of the prior thermal history of the structure. This behavior is very important for underground storage structures for heat generating wastes and spent nuclear fuel storage systems where concrete temperatures near 400°F may not be uncommon. A proper analytical representation of this complex behavior is of critical importance in capturing the correct response of concrete structures subjected to sustained elevated temperatures and thermal cycling.

The impetus for this work was the need for an evaluation of the structural integrity of underground, reinforced concrete nuclear waste containments which may be subjected to high temperature thermal cycles over a 50-year design life (James, 1988). Previous computational models were developed in the early 1970's when very little data for these effects were available (Rashid, 1974 and Rashid, 1985). An accepted analysis practice for these structures at elevated temperatures is to estimate the long term, degraded value of the concrete properties based on a postulated temperature history and maintain these values constant throughout the analysis. Based on the current model for concrete response at elevated temperatures, it will be demonstrated that this procedure is not necessarily conservative.

METHODOLOGY

The concrete constitutive model is designed to interface to an implicit finite element program equipped with iteration algorithms for each load increment to handle nonlinear material response. The concrete module accepts increments in strain and temperature at each material point, computes increments in creep strain and thermal loads, evaluates current material properties, and constructs the constitutive matrix. The stresses are then computed and modified for internal damage due to cracking or crushing and the constitutive matrix adjusted accordingly. The consistent stress vector and constitutive matrix are returned by the module to the analysis code for integration and subsequent iteration on the displacement solution to redistribute loads in excess of the material capacity.

Material Property Degradation

It has been recognized for many years that exposure to elevated temperatures has a deteriorating effect on the physical properties of concrete. This is due to both chemical and mechanical mechanisms. At temperatures above 200°F some of the water of hydration is removed from the hardened paste as well as evaporable water required by the chemical process for hardening. At these higher temperatures the differences in the coefficients of thermal expansion between the aggregate and paste cause separation of these components. This effect is also magnified by the removal of the water of hydration since this causes shrinkage of the paste. The reverse is also recognized; concrete kept at lower temperatures with continual moisture for cure will continue to gain strength indefinitely, as demonstrated by the footing samples from the Golden Gate Bridge.

In the mid 1970's, the Portland Cement Association (PCA) conducted a long term test program (PCA, 1981) at their Construction Technology Laboratory (CTL) in Skokie, Illinois, for the Department of Energy (DOE), to quantify the effects of elevated temperatures on concrete. Standard 6 x 12 inch cylindrical specimens were prepared for mixtures of minimum 3000 psi and for 4500 psi concrete (compressive strength at 28 days). Specimens were moist cured and then continuously heated at 250°F, 350°F, and 450°F for up to 920 days. In 1986-1987, a statistical analysis (Henager, 1988) of this data was conducted by Pacific Northwest Laboratory (PNL) of Battelle Labs to establish concrete property equations (and uncertainty bands) for Hanford concrete. The resulting fitted equations have the form:

$$E = a_1 + b_1 S - c_1 T - d_1 \log_e (t + 1) \quad (1)$$
$$f_c = a_2 + b_2 S - c_2 T - (d_2 + e_2 T) \log_e (t + 1) \quad (2)$$
$$f_t = a_3 + b_3 S - c_3 T - d_3 \log_e (t + 1) \quad (3)$$

where E is the modulus of elasticity, f_c is the compressive strength, f_t is the splitting tensile strength, S is the nominal initial compressive strength, T is the constant value of temperature, t is the hold time at constant temperature, and a_i, b_i, c_i, d_i, and e_2 are the data fitting variables.

Note that these expressions are derived from specimen tests held at constant and discrete temperatures. In adapting this empirical data to a general constitutive model, a complete range of temperatures and temperature cycles must be accommodated. Thus, several assumptions must be employed to implement these material property equations for practical applications. First, it is widely accepted that exposures at nominal (less than 200°F) temperatures cause insignificant loss in material properties so that a cutoff for material property degradation with time must be implemented. This cutoff region is defined to be from 200 to 250°F. No property degradation due to hold time is allowed below temperatures of 200°F. Full degradation based on the above equations occurs for temperatures greater that 250°F. A linear ramp for the hold time degradation terms is used between these temperature values to smooth this discontinuity.

The next assumption involves applying these property equations to variable temperature histories. Since the mechanisms causing deterioration in properties discussed above are not reversible; i.e. return to a lower temperature cannot "heal" microcracks and does not replace lost moisture, the loss in strength is assumed not to be recoverable upon return to lower temperatures. Also, it is assumed that new microcracks will not form during the next temperature rise until the current temperature exceeds the maximum temperature that has been seen by the material. Thus, no recovery of property values is allowed during cool down periods, and no further degradation is allowed during subsequent heating until the maximum temperature seen at a material point is reached.

Finally, since the concrete model is formulated for incremental analysis, the fitted property equations must be recast in differential form. In addition, because time in the analysis is measured from an arbitrary reference point, while time in these equations is measured from the instant the specimen reaches the constant temperature, the time variable t cannot be explicitly evaluated and must not appear directly in the equations. A general expression for any of the three properties can be written as,

$$P = A_1 + A_2\ S + A_3\ T + (A_4 + A_5\ T)\ \log_e(t+1) \qquad (4)$$

where now A_4 and A_5 are functions of temperature. Using a change of variable, $t^* = \log_e(t+1)$ and differentiating equation (4) gives,

$$dP = A_3\ dT + (A_4 + A_5\ T)\ dt^* + (dA_4 + dA_5\ T + A_5\ dT)\ t^* \qquad (5)$$

where the pseudo time, t^*, is stored as a state variable and used to calculate an equivalent hold time at any step. During a step where degradation occurs, the equivalent hold time is updated. The increment in the property value, dP, is then calculated from increments in temperature and the pseudo time, and the current property value is updated by the current increment.

Figure 1 illustrates the degradation history for the elastic modulus for an arbitrary temperature history. Note the first temperature cycle stays below 200°F and thus the modulus degrades linearly with temperature on the temperature rise and remains constant during the temperature hold since the temperature is below the cutoff for time degradation. The modulus remains constant during the cool down and only continues degrading during the next cycle after the temperature reaches the value of the first cycle. During temperature hold of the second cycle, the exponential decay with hold time occurs since the temperature is above 200°F.

<u>Creep</u>

Data for creep of concrete at temperatures above 200°F is very sparse. Browne (1968), and Hannant (1968) have reported data in the temperature range from 68° to 200°F, and for hold times up to about 1000 days. Marechal (1969) reports data in the temperature range of 68°F to 482°F for 100 days. There is fair agreement in these data for some temperatures and significant differences at other temperatures. Browne and Marechal show relative agreement for temperatures of 68°F and 158°F, while Browne and Hannant show better agreement around 122°F. However, Marechal's 100-day creep strain is about twice that of Hannant for this temperature. Marechal shows higher creep strains at 100 days for 122°F than for 158°F or 482°F and higher creep for 68°F than for 302°F or 221°F indicating that the creep rate is not monotonic with temperature. This result is also indicated by Neville (1973). However, the data reported by Browne and Hannant contradict this non-monotonic behavior of the creep rate with temperature. There are many factors in the experiments for these differences, but the major reasons appear to be mix designs and sealed vs unsealed specimens.

In addition to the material property measurements, the PCA program discussed above tested six 4500 psi cylinders to obtain creep strain data at 250°F and 350°F for 650 days. However, this data appears inconsistent with all the other data when extrapolated to the lower temperatures. In an effort to be conservative, the approach adopted here is to assume a monotonic behavior with temperature of the creep rate and fit the specific creep function to the Hannant data, which appears to be the most consistent qualitatively. Hannant's data at 167°F is fitted with a six term exponential series and extrapolated to 50 years. To introduce temperature dependence, a curve fit for a temperature multiplier is constructed and extrapolated to 450°F. The specific creep function is then the product of the time and temperature functions.

<u>Internal Damage</u>

The concrete model accounts for cracking and crushing of the material at each material point. Crushing occurs when the effective compressive stress exceeds the compressive strength of the material and is handled by the usual methods for plasticity. Cracking is more algorithmic in nature and must allow cracks to open and close. Cracks are assumed to form in the principal planes. When the stress/strain state in the maximum principal direction reaches the cracking criteria, a crack develops in the plane perpendicular to this maximum principal direction. The normal modulus across the crack is dropped to zero and the shear modulus is reduced by 1/2 to include the effect of reinforcement and aggregate interlock during crack formation. The local stresses normal to a crack are set to zero. Multiple cracks are allowed to form at the same material point but they are assumed to be mutually orthogonal. Once a crack forms, its direction is fixed and it can never "heal"; however, it can close and reopen. Crack formation is governed by the fracture strain limit which is the tensile strength divided by the modulus (initial values) and remains constant during an analysis. An existing crack closes if the total normal strain across the crack is compressive or if the current increment of strain normal to the crack is negative. When an existing crack starts to close, a gradual increase in stiffness takes place to recognize that the rubble in the crack transmits some compressive stress during closure. Once the normal strain across a crack becomes negative, the stiffness is returned to its full normal value. If the closed crack experiences positive total normal strain, the crack immediately reopens.

DEMONSTRATION CASE

To illustrate the effects of elevated temperatures on concrete constitutive behavior, a numerical experiment was conducted to examine the effects of material property degradation in the calculated response of a reinforced concrete structure. A model of an underground nuclear waste storage tank was constructed using representative dimensions and reinforcement ratios. This model is axisymmetric with roller boundary conditions along the bottom edge to simulate a sliding condition on a rigid foundation. The mechanical loads on the structure, which are constant during the analysis, include the dead weight of the structure and the overburden pressure due to the weight of the soil. Figure 2 illustrates this model. The thermal load on the structure is applied by defining a temperature profile along the inside wall and a corresponding profile along the outside wall that gives a 5°F temperature drop across the wall. A heat transfer solution from these boundary conditions then defines the temperature distribution throughout the structure. The inside temperature profile, representing the peak temperature distribution, is shown in Figure 3. The shape of this profile and the 5°F temperature drop is based on previous experience in solving for the complete thermal response of these structures (James, 1988). These peak temperatures represent the tank under full heat generation conditions, and thermal cycles are defined as fill from a reference temperature over one year, hold at capacity for one year, and drain over one year. Thus, the point of peak temperature has a linear rise from 50°F to 400°F over 360 days or a heating rate of about 1°F / day.

The example calculations consider a fill, hold, drain, and refill cycle (4 years) where the overburden and dead weight are applied and the thermal load ramped in 15 day increments. At each load increment, iteration on the displacement/stress solution is used to balance concrete cracking and load redistribution to reinforcement. The first analysis considers constant material properties for concrete based on the 10 year (held at 250°F) fully degraded values. Figure 4 shows the cracking pattern from this analysis at the top of the first fill cycle. The horizontal lines at mid wall represent meridional cracks caused by tensile stress in the meridional direction. This stress is caused by outward bending in the wall induced from the thermal gradient along the wall.

The comparison analysis used the measured initial values of modulus, compressive strength, and ultimate tensile strength and degraded these values according to the empirical data fits as described above. This analysis shows additional cracking during the fill and hold cycle compared to the constant property analysis, as seen in Figure 5. The circles in the haunch region represent hoop cracking caused by hoop tension due to radial expansion beyond the soil prestress. This additional cracking occurs because the stress due to thermal load is proportional to the modulus, so that this stiffer structure develops more stress. In addition, for the constant property analysis, the initial degraded values of tensile strength and modulus represent a fictitiously high value of fracture strain. This, coupled with the lower stresses for this analysis, predict much less cracking in the concrete. For the degrading property analysis, the modulus degrades during the temperature rise and hold cycle (with different rates at different points in the structure), and this stress must be redistributed.

Figure 6 compares the vertical displacement at the apex of the dome for the two demonstration analyses. The constant property results show a constant vertical displacement during the temperature hold and a return to this displacement magnitude during the subsequent fill cycle. The degrading property analysis shows a decay in the apex displacement during temperature hold due to continued cracking and load redistribution. During the drain cycle, the apex drops significantly below the original dishing caused by the soil overburden, and the subsequent fill cycle produces apex displacement significantly less than the first cycle. This "ratcheting" of the structure due to the additional cracking and loss in modulus indicates a need to critically examine subsequent thermal cycles to determine the true life of the structure. At the top of the second fill cycle, the peak rebar stress for the constant property analysis is 17.3 ksi compared to 23.4 ksi for the degrading property analysis. This also indicates that the constant property analysis tends to overestimate the margin of safety for the structure.

SUMMARY AND CONCLUSIONS

In summary, an analysis using fully degraded but constant material properties for concrete is compared to a degrading property analysis under the same loading. The results indicate that using degraded but constant properties for an analysis of reinforced concrete structures subjected to elevated temperatures may not be conservative and may substantially under estimate margins of safety or structural life. This occurs since the stresses due to thermal loads are proportional to the modulus, and because the initial degraded value of modulus dictates a higher fracture strain criteria. Using fully degraded properties generates less stress and less cracking. In addition, if the properties remain constant, then load redistribution under constant load is not required during the analysis.

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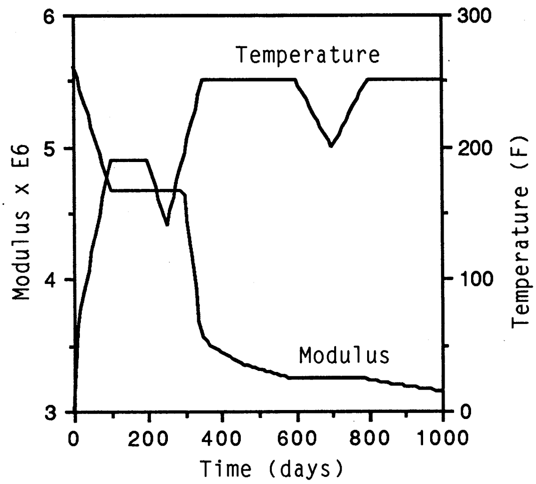

Figure 1. Modulus Degradation for Variable Temperature History Example

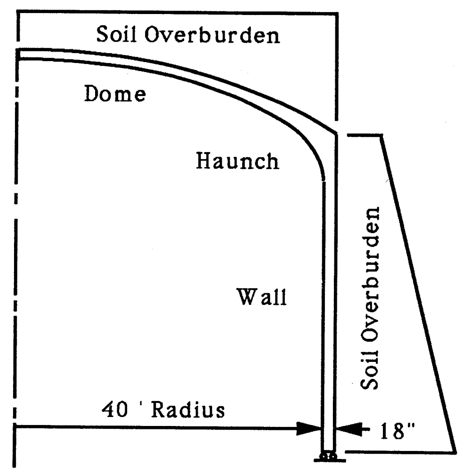

Figure 2. Underground Tank Example Model

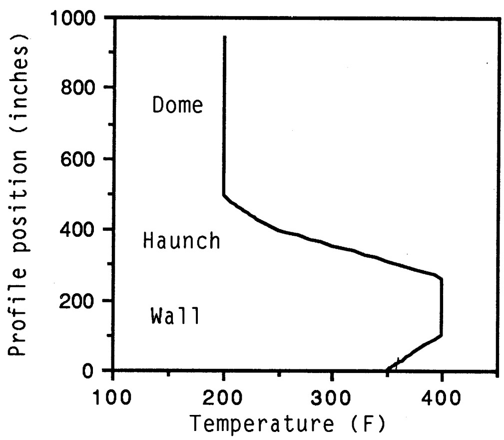

Figure 3. Peak Temperature Profile

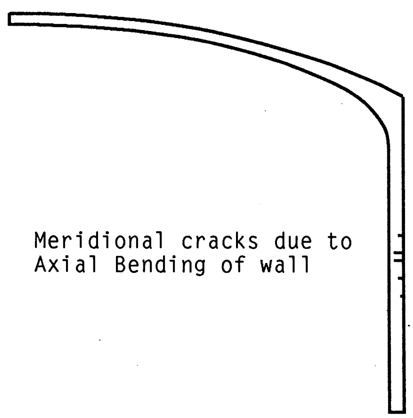

Figure 4. Cracking Pattern for Constant Property Analysis

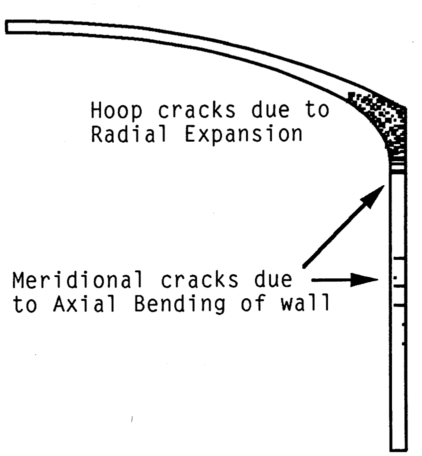

Figure 5. Cracking Pattern for Degrading Property Analysis

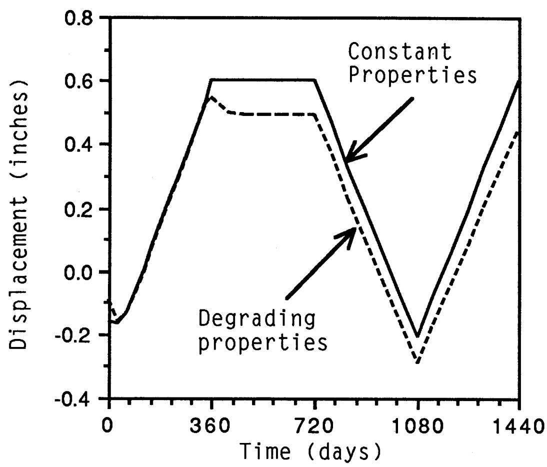

Figure 6. Dome Apex Displacement History