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# STATE OF THE ART OF PRESTRESSED CONCRETE PRESSURE VESSELS FOR NUCLEAR POWER REACTORS, A CRITICAL REVIEW OF THE LITERATURE

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# 1. INTRODUCTION

## 1.1 The Idea

The prestressed concrete pressure vessel (PCPV) promises advantages over a conventional steel vessel in economy and safety for large reactor application. The steel vessel for a power reactor needs a substantial external biological shield; concrete has met the requirements for the least cost. This concrete shield appears amply massive to support a system of prestressing tendons capable of withstanding the pressure loading and a considerable thermal loading on the concrete as well.[1,2] In Europe the limit of practical one-piece steel-vessel construction was already being approached in long range gas-cooled reactor planning of the mid-fifties. PCPV programs in France and England have already resulted in the Marcoule G-2 vessel,[3] operational in 1958, and the large vessels now under construction at Oldbury and Chinon.[2,4] In these vessels only a vestige of the steel vessel remains as a thin, gastight, lining membrane deriving its mechanical support from the prestressed concrete shell.

## 1.2 Steel

As plate or forging thickness for a steel vessel increases the quality obtainable decreases. Further, the percentage weight of weld-deposited metal increases more rapidly than thickness from two causes, first, the increasing width of the weld-preparation openings at joints, and second, the increasing length of joint per unit area of shell, because of the limits on the weight of an ingot. Radiographic inspection, stress relief, transportability and production time are unfavorably affected. Multilayer construction may offer relief, but the problems of heads and head-to-shell connections appear formidable in very large multilayer vessels. The use of multiple concentric steel vessels with controlled pressure in the intermediate annuli does not seem safe or economical.

## 1.3 Concrete

On the other hand, the wall of a concrete vessel may be thickened and additional layers of prestressing tendons may be disposed therein with no particular penalty for size other than the provision of support framing and access for the increased number of anchors. High-carbon steel wire is probably the most readily produced and fabricated form of high-strength, ductile, structural material. From a safety standpoint, the great multiplicity of independent tensile members opposing the working loads on the concrete insures that excessive loading will produce only the typical pattern of gradual failure of prestressed concrete. Although concrete cracks at a low level of tensile stress, reinforcing and prestressing steel are ductile and may be expected to absorb considerable energy plastically during a period of overpressure or overheating. The vessel's ability to contain coolant and control radiation may be greatly impaired through cracking of the concrete, but catastrophic rupture is believed to be improbable.[5,6]

Since concrete is able to withstand relatively high compressive stress, the prestressing steel may be placed near the outside of the vessel wall. Here the attenuation of the nuclear radiation flux traversing the inner concrete is favorable to the avoidance of brittleness or relaxation in the steel.

### 1.4 Fundamental Issues

Granted that prestressed-concrete is already well established in much of the field of civil engineering construction, special problem areas remain in extending this technology to pressure vessels for nuclear power reactors. Some of the more critical of these are:

1. The thermal problem: What are the temperature limits for Portland cement concretes? How can optimal balancing of internal shielding, thermal insulation, and separate cooling system for the concrete vessel be approached? The influence of temperature on the shrinkage and creep of concrete as well as on the relaxation of highly-stressed steel must be taken into account.

2. The prestressing problem: The materials and geometry of the tensile steel must be chosen to meet operating requirements safely and economically. Systems adaptable to large, high-pressure vessels, convenient to construct and maintain, are needed, beyond the present fluid storage practice.

3. Penetrations for coolant ducts will constitute either large or numerous interruptions in the prestressing system, and will induce complicated systems of triaxial stress concentration. Ingenious means for reinforcing these discontinuities will need to be developed unless the complete primary coolant circuit is to be accommodated within a correspondingly-enlarged pressure vessel, as at Oldbury.

4. The modes of failure of PCPV's in the new environment must be investigated to establish a realistic basis for design.

5. Quality control during acquisition of materials, fabrication of the components, construction, and acceptance testing, and the surveillance to

establish continued adequacy of the vessel during its service life will require solutions different from those acceptable for steel pressure vessels or for familiar types of prestressed-concrete structures. Extrapolation of short-time tests of time-dependent behavior of materials and components to long periods is inherent in an emergent technology for long-lived components. Obviously intuition must be reinforced by the most ingenious and wise planning and interpretation of thoroughgoing experimental programs. The evidence of progress to be found in the literature of concrete technology ought to be thoughtfully exploited, neither neglected nor mis-used in the development of PCPV.

6. The cleanliness problem: The designer must prevent leakage of coolant through the concrete and conversely contamination of the coolant with dust from the concrete. The design, erection, monitoring, and maintenance of an impervious membrane are possible sources of difficulties.

Although the extensive existing technology of prestressed concrete cannot supply complete means of designing and constructing nuclear reactor vessels, it would make little sense to ignore rational principles of structural resistance and knowledge of the behavior of materials that have been painstakingly built up. It is true that columns, beams, slabs, buildings, bridges, dams, and tanks are not reactor vessels. Yet they all must, in non-ideal environments and with reasonable success, withstand in varying degree diverse spectra of mechanical and thermal loading, body forces, and external reactions. This resistance must be supplied by the same sorts of internal stress fields, and by the same principles of mechanics, all in spite of the shortcomings of elasticity and plasticity when applied to real steel and real concrete in usefully-sophisticated configurations. That in a strictly literal sense the literature of prestressed concrete vessels for gas-cooled nuclear reactors is scarcely nascent does not signify that the whole body of design principles, choice of materials, and construction methods must be newly created. Such creative labor would better be directed into the critical areas where prestressed-concrete practice lacks a basis for solving the problems, unique in degree rather than in kind, inherent in a pressure vessel for gas-cooled reactor application. This review attempts to single out these critical problem areas.

## 2. The Nature of Prestressed Concrete Structures

### 2.1 Prestress Behavior

The interaction of prestress, coolant pressure and thermal stress in the concrete vessel must be examined to arrive at a basis for choosing the amount of prestress. Although the axial prestressing must be accomplished first to enable the shell to withstand the axial bending stresses induced by the various stages of the tangential prestressing procedure, the latter is dominant and critical to effective design. A look at the internal radial reactions in a cylindrical shell "remote" from discontinuities will serve to illustrate the principles that establish the amount of prestress. These reactions are indeterminate by static equilibrium, so that the spring rates of the concrete shell and the prestressing tendons must be investigated.

For simplicity, the influence of axial prestress and of the pressure load on the ends of the vessel is neglected in the following discussion. The stresses and radial displacements of a thick, prestressed, concrete shell will be discussed by means of the solution of Lame's equation for a thick, isotropic, open-ended hollow cylinder subjected to uniform internal and external pressures. From Timoshenko's "Strength of Materials,"[7] the radial stress at any radius r is

$$\sigma_r = \frac{a^2 p_i - b^2 p_o}{b^2 - a^2} - \frac{(p_i - p_o)\, a^2 b^2}{r^2 (b^2 - a^2)} ,$$

the tangential stress is

$$\sigma_t = \frac{a^2 p_i - b^2 p_o}{b^2 - a^2} + \frac{(p_i - p_o)\, a^2 b^2}{r^2 (b^2 - a^2)} ,$$

and the radial displacement is

$$\Delta r = \frac{1 - \nu_c}{E_c} \frac{a^2 p_i - b^2 p_o}{b^2 - a^2} r + \frac{1 + \nu_c}{E_c} \frac{a^2 b^2 (p_i - p_o)}{(b^2 - a^2)\, r} ,$$

where

$a$ = internal radius of shell

$b$ = external radius of shell

$p_i$ = internal pressure

$p_o$ = outside pressure

$E_c$ = Young's modulus of concrete

$\nu_c$ = Poisson's ratio of concrete

For the considerations that follow, the displacement at the outside radius where the prestressing steel bears and the maximum stresses in the concrete are of interest. The latter will be found at the inner or outer surface. The expressions for these quantities are shown in Table 2.1-1. These stresses and deformations may be validly superimposed at regions remote from discontinuities on those of an isothermal thick cylinder already subjected to uniform meridional strain.

Although the circumferential prestressing steel might be distributed in the outer part of the concrete in practice, its nominal radius for the purposes of this discussion may also be taken as b and its area denoted by $A_s$ per unit axial length of shell. Since the steel is the source of the external pressure $p_o$, the hoop stress in the steel will be

$$\sigma_{ts} = \frac{p_o\, b}{A_s} .$$

The corresponding unit strain will cause a proportional change in radius

$$\Delta b = \frac{b\sigma_s}{E_s} = \frac{p_o b^2}{E_s A_s}$$

An example may clarify interplay of certain of the factors in the choice of amount of prestress for a prestressed concrete reactor vessel. Assume that a gas-cooled reactor and its associated components are to be housed within a cylindrical space 80 ft in diameter and that 11 ft of concrete of the type planned are needed for the biological shield. What working pressure of coolant might be practicable for a PCPV of 480 inches inner and 612 inches outer radius of concrete if the pressure and stiffness of prestressing means are suitably chosen?

It is assumed that the concrete has an elastic modulus $E_c$ of 5 $(10)^6$ psi and Poisson's ratio $\nu_c = 0.15$. The displacements of the outer surface and associated maximum stresses in this concrete shell are shown in Table 2.1-2 in terms of pressure of coolant $p_i$ and prestressing tendon pressure $p_o$. It is worth noting the similarity of certain ratios displayed by this thick cylindrical shell:

Wall ratio $\frac{b}{a} = \frac{612}{480} = 1.275$

Displacement ratio at $r = b$

$$\frac{\Delta b_{p_o}}{\Delta b_{p_i}} = \frac{-495(10)^{-6}\, p_o}{391(10)^{-6}\, p_i} = -\,1.266\,\frac{p_o}{p_i}$$

Max circumferential stress ratio at inner radius, $r = a$

Table 2.1-1. Displacements and Stresses Caused by Uniform Radial Load on a Thick-Walled Hollow Cylinder[7]

| | | | |
|---|---|---|---|
| Radial displacement at outside caused by | external pressure | $\Delta r_{(r_1=b)} = -\frac{bp_o}{E_c}\left(\frac{a^2+b^2}{b^2-a^2} - \nu_c\right)$ | |
| | internal pressure | $\Delta r_{(r=b)} = \frac{2a^2bp_i}{E_c(b^2-a^2)}$ | |
| | | Radial | Circumferential |
| Stresses at outside caused by | external pressure | $\sigma_{r(r=b)} = -p_o$ | $\sigma_{t(r=b)} = -p_o\frac{b^2+a^2}{b^2-a^2}$ |
| | internal pressure | $\sigma_{r(r=b)} = 0$ | $\sigma_{t(r=b)} = \frac{2p_i a^2}{b^2-a^2}$ |
| Stresses at inside caused by | external pressure | $\sigma_{r(r=a)} = 0$ | $\sigma_{t(r=a)} = -\frac{2p_o b^2}{b^2-a^2}$ |
| | internal pressure | $\sigma_{r(r=a)} = -p_i$ | $\sigma_{t(r=a)} = p_i\frac{a^2+b^2}{b^2-a^2}$ |

Table 2.1-2. Displacements and Stresses in Cylindrical Concrete Shell

$a = 480$ in. $E_c = 5(10)^{-6}$ psi

$b = 612$ in. $\nu_c = 0.15$

| | | | |
|---|---|---|---|
| Radial displacement at outside caused by | external pressure | $\Delta r_{(r=b)} = -495.33(10)^{-6} p_o$ | |
| | internal pressure | $\Delta r_{(r=b)} = 391.29(10)^{-6} p_i$ | |
| | | Radial | Circumferential |
| Stresses at outside caused by | external pressure | $\sigma_r = -p_o$ | $\sigma_t = -4.1968\ p_o$ |
| | internal pressure | $\sigma_r = 0$ | $\sigma_t = 3.1968\ p_i$ |
| Stresses at inside caused by | external pressure | $\sigma_r = 0$ | $\sigma_t = -5.1968\ p_o$ |
| | internal pressure | $\sigma_r = -p_i$ | $\sigma_t = 4.1968\ p_i$ |

$$\frac{\sigma_{t_{p_o}}}{\sigma_{t_{p_i}}} = \frac{-5.20\ p_o}{4.20\ p_i} = -1.238\ \frac{p_o}{p_i}$$

Obviously, when the ratio of the equivalent pressure exerted by the steel to the pressure exerted by the internal gas is equal to the reciprocal of the wall ratio, we may assume for estimating purposes that the concrete is essentially unstrained circumferentially at the outside and that the circumferential stress is negligible at the inside. In this instance, where the reciprocal of the wall ratio is 0.783, 0.807 psi applied externally will neutralize the critical tangential stress caused by one psi gas pressure (assuming elastic behavior).

The spring rate of the concrete at the interface is shown graphically in Fig. 2.1-1 where interface pressure is plotted against radial recession of the interface, line AB.

It is assumed that the creep and shrinkage behavior of the concrete mix has been studied and that 1800 psi at the time of prestressing has been selected as an appropriate maximum circumferential compressive stress at the inside of the shell. The expression for the foregoing stress from Table 2.1-2 is

$$\sigma_{t(r=a)} = -5.1968\ p_i$$

Solving for interface pressure at time of prestressing

$$p_o = \frac{-1800}{-5.1968} = 346.4 \text{ psi}$$

In order to emphasize the desirability of high ultimate-strength steel, the performance of 100 kpsi minimum ultimate strength carbon-steel bars will be compared with that of ASTM A421-58T cold-drawn high-carbon wire of 0.192 inch nominal diameter and 250 kpsi minimum ultimate tensile strength. At room temperature, Young's modulus is close to 29 mpsi for carbon steel. Following the ACI Building Code, the steel will be at a tension of 0.70 ultimate after prestressing and the area of steel required is therefore

$$A_s = \frac{p_o b}{\sigma_{ts}} = \frac{346.4(612)}{0.7\begin{Bmatrix}100{,}000\\250{,}000\end{Bmatrix}} = \begin{Bmatrix}3.029\\1.211\end{Bmatrix}$$

that is, 3.029 $in^2$ of bar per meridional inch of shell or 1.211 $in^2$ of wire per meridional inch of shell. The respective radial growths are

$$\Delta b = \frac{b\sigma_{ts}}{E_s} = \frac{p_o b^2}{E_s A_s} = \frac{(612)(10)^{-6}\ p_o}{29\begin{Bmatrix}3.029\\1.211\end{Bmatrix}} = \begin{Bmatrix}4264\\10665\end{Bmatrix}(10)^{-6}\ p_o$$

Unclassified
ORNL-DWG 64-3148

Condition:

Initial Prestress

After Shrinkage, Creep and
Relaxation Stabilization

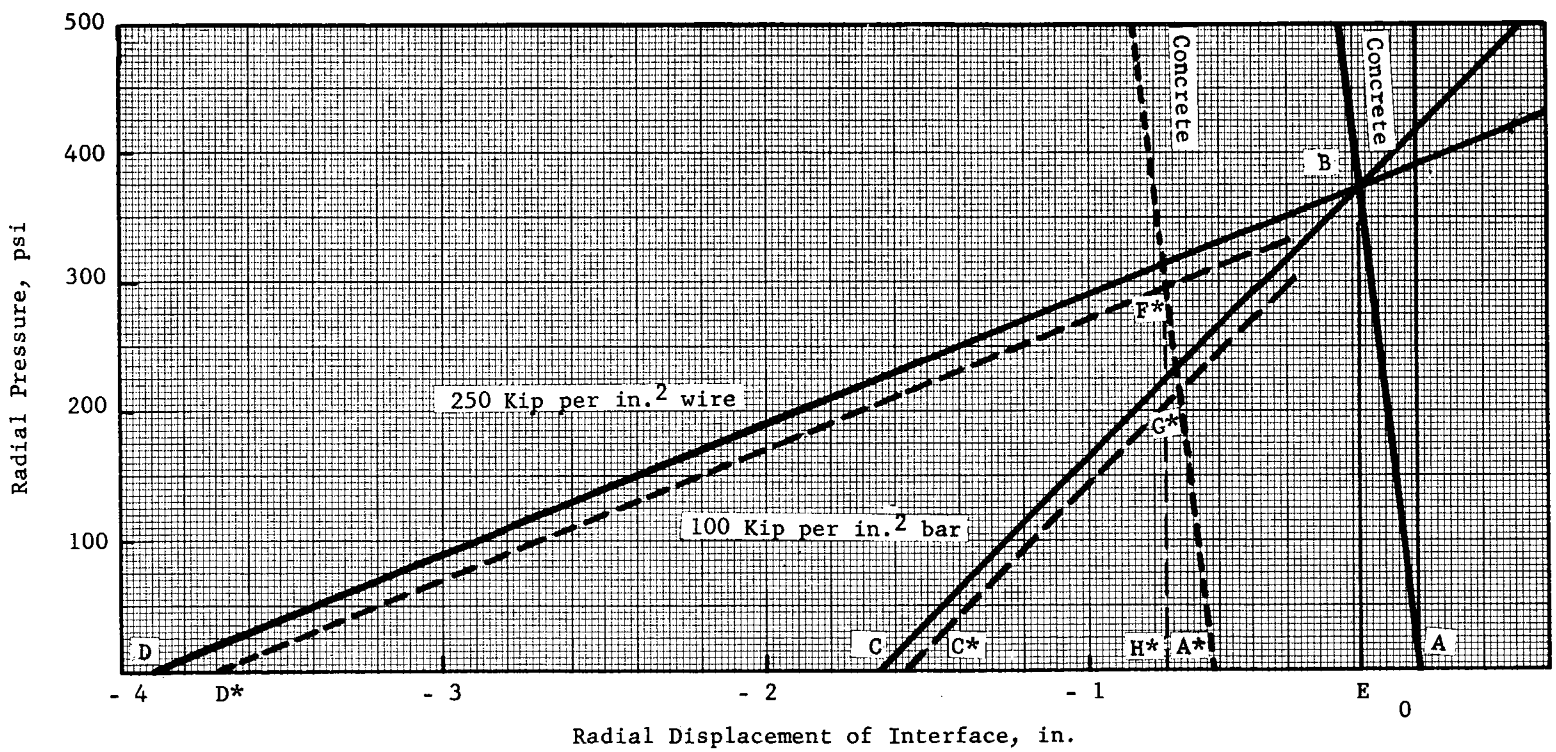


Fig. 2.1-1. Prestress Pressure Developed as a Function of Displacement.

The radial growths at completion of prestress corresponding to $p_o = 346.4$ psi are:

for the bar tendons

$$\Delta b = 4264(10)^{-6}(346.4) = 1.477 \text{ in.},$$

for the wire

$$\Delta b = 10{,}665(10)^{-6}(346.4) = 3.694 \text{ in.},$$

and for the concrete

$$= -495.3\,(10)^{6}\,p_o = -0.172 \text{ in.}$$

These conditions are shown in Fig. 2.1-1 as lines CB, DB and AB, respectively, the steel being in equilibrium with the shell at the intersection B, corresponding to initial prestress pressure of 346.4 psi. The free face of the concrete is at A; the prestressing reduces the 612 in. radius by 0.172 in. to E. The wire has been stretched 3.694 in. on the radius from the "free" condition at D to E; the total elastic interference is DA = 3.866 in. If recourse were had to the 100 kpsi ultimate bar, its "free" radius would have been increased from C to E, so that its total interference with the concrete CA, would be but 1.649 in. The areas of triangles EAB and DEB or CEB, to the scale of the plot, represent the potential energy (in units of inch pounds per square inch of interface) stored elastically in the concrete and the wire or bar respectively. It should be noted that only 0.172/3.866 = 4.45% of the energy enters the concrete if the high-strength wire is used, but this fraction becomes 10.4% with the bar steel, because the latter, by virtue of 2.5 times the area, is stiffer by that multiple. It is important to note that the strain of the concrete is very small; adequate elastic follow-up for service life must be provided elsewhere.

A rather good approximation for the deformation of the concrete under external pressure may be had by using the mean tangential stress over a cross section

$$\sigma_{tc(\text{mean})} = -\frac{p_o b}{t}, \qquad t = a \quad b - a ,$$

to calculate an equivalent displacement of the mean radius

$$\Delta b_{\left(r = \frac{a+b}{2}\right)} = -\frac{p_o b}{tE_c}\left(\frac{a+b}{2}\right) = 506(10)^{-6}\,p_o ,$$

that is, about 2% higher than the Lame' value for the wall ratio under consideration here, namely $-495.3\ (10)^{-6}\ p_o$ from the table. From this it would be expected that the deformations of the concrete and the prestressing steel would be nearly in the ratio of their mean radii and inversely as the product of their areas and moduli:

$$\frac{\Delta b_c}{\Delta b_s} \approx \frac{r_{c(\text{mean})}}{r_{s(\text{mean})}} \frac{E_s A_s}{E_c A_c} .$$

This approximation gives ratios of deformations of concrete to that of steel

$$\frac{\Delta b_c}{\Delta b_s} \approx \frac{(546)(29)\begin{Bmatrix}1.211\\3.029\end{Bmatrix}}{(612)(5)(132)} = \begin{cases}0.047 \text{ wire}\\0.118 \text{ bar}\end{cases}$$

compared to

$$\frac{0.171}{3.692} = 0.046 \text{ for wire}$$

and

$$\frac{0.171}{1.477} = 0.116 \text{ for bar}$$

from the more nearly correct Lame' equation. Obviously, the effect of the lower modulus of the concrete is wholly overshadowed by its greater area.

## 2.2 Loss of Prestress

As has been mentioned previously, under the working conditions of a reactor vessel, both shrinkage and creep of the concrete and relaxation of the steel would probably tend to be higher than found in other applications. Therefore they will be estimated on the high side (Table 2.2-1).

| | |
|---|---|
| Strain caused by creep of concrete per psi of stress at initial prestressing | = - 0.4(10) $\mu$ in. hr/inch |
| Shrinkage strain of concrete | = - 300(10) $\mu$ in. hr/inch |
| Stress relaxation in steel, including slip at anchors | = 5% of prestress tension |

While it might be argued that the smaller elastic interference inherent in the bar prestressing would tend to cause more rapid unloading of the members during the service life and thereby reduce the total relative plastic deformation, the effect of thermal cycling must not be disregarded. The members of the lower ultimate strength prestressing system, because of its stiffness, will suffer proportionally higher transient loadings during temperature changes that will tend to offset loss of isothermal prestress. The larger percentages of yield strength during cyclic stress experienced by the bars in their live-load increment cycling will certainly help to reduce their total-life relaxation relative to that of the wire.

For the example under consideration the total plastic strains at end of

Table 2.2-1. Estimation of Contributions to Loss of Prestress

| | | ACI Building Code 1963 Ch. 26, Ref. 27 | Criteria for Prestressed Concrete Bridges Bu. Pub. Roads 1954 Ref. 88 (Lin) Appendix D | Ref. 89 (Libby) Appendix B 1961 | Ref. 90 (Evans & Bennett) | Ref. 91 (Creasy) |
|---|---|---|---|---|---|---|
| Elastic Deflection of Concrete | | | | | | |
| Pretensioned | | | | | | |
| Post-tensioned | | | | | | |
| Shrinkage of Concrete | | | | | | |
| Pretensioned | Strain | | 0.0002 | 0.0003 | 0.0003 | 0.0002 |
| Post-tensioned | Strain | | 0.0001 | 0.0002 | 0.0002 | |
| Creep of Concrete | | | | | | |
| Pretensioned | | | 16 $f_{cs}$ | 100 to 300% of elastic strain in concrete | 0.14 $f_{si}$ | |
| Post-tensioned | | | 11 $f_{cs}$ | | 0.10 $f_{si}$ | 8.4 $f_{cs}$ |
| Relaxation of Steel | | | | | | |
| Pretensioned | | | 0.04 $f_{si}$ | 0.02-0.08 $f_{si}$ | 0.04 $f_{si}$ | - |
| Post-tensioned | | | 0.04 $f_{si}$ | 0.02-0.08 $f_{si}$ | 0.02 $f_{si}$ | 0.07-0.10 $f_{si}$ |
| Friction | | $T_o=T_x\epsilon^{(KL+\mu d)}$ | | $T_o=T_x\epsilon^{(KL+\mu d)}$ | $T_o=T_x\epsilon^{(KL+\mu d)}$ | $T_o=T_x\epsilon^{(KL+\mu d)}$ |
| Slip and Elastic Compliance of Anchorage | | | By test | | | |

useful life become:

Concrete

| | |
|---|---|
| Creep strain | $-0.4(10)^{-6}(1800) = -720(10)^{-6}$ |
| Shrinkage strain | $-300(10)^{-6}$ |
| Total | $-1020(10)^{-6}$ |

Steel

Creep strain $\dfrac{0.05 \begin{Bmatrix} 70,000 \\ 175,000 \end{Bmatrix} (10)^{-6}}{29} = \begin{matrix} 121(10)^{-6} \text{ bar} \\ 302(10)^{-6} \text{ wire} \end{matrix}$

The corresponding changes in the free radii of the members become

$$\Delta b_{c(\text{plastic})} = 612(-1020)(10)^{-6} = -0.624$$

$$\Delta b_{s(\text{plastic})} = 612 \begin{Bmatrix} 121 \\ 302 \end{Bmatrix} (10)^{-6} = \begin{cases} 0.074 \text{ bar} \\ 0.185 \text{ wire} \end{cases}$$

Thus returning to Fig. 2.1-1, the point A will be displaced 0.624 in. to the left to point A*, point D, 0.185 in. to the right to D* at -3.866 + 0.185 = 3.681 in. from original surface of concrete, and point C, 0.074 in. to the right to C* at -1.649 + 0.074 = -1.575 in. from the original outside surface of the concrete. Plastic deformation will have reduced the elastic interference existing immediately after completion of prestressing from 3.866 to 3.057 inches in the case of 250 kip wire and from 1.649 to 0.951 if 100 kip bar had been used.

The foregoing loss of prestress will lower the radial pressure at the interface of the steel and concrete. Equating the total of the elastic displacements resulting from this pressure $p_o{*}$ to the elastic interference for each case gives

$$495(10)^{-6}\, p_o{*} + 10665(10)^{-6}\, p_o{*} = 3.057 \text{ (wire)},$$

$$495(10)^{-6}\, p_o{*} + 4264(10)^{-6}\, p_o{*} = 0.951 \text{ (bar)},$$

so that the asymptotic prestressing pressures approached early in the service life would be, for wire

$$p_o{*} = 274.0 \text{ psi},$$

and for bar tendons

$$p_o{*} = 199.8 \text{ psi}$$

These would correspond to elastic deformations in the concrete of

$$-495(10)^{-6} \begin{Bmatrix} 274.0 \\ 199.8 \end{Bmatrix} = \begin{matrix} -0.136 \text{ wire}, \\ -0.099 \text{ bar}, \end{matrix}$$

hence the interface displacement is - 0.624 - 0.136 = - 0.760 in case of wire and - 0.723 with the bars, the abscissae of the intersections F* and G* of the dotted rate-of-pressure lines in the figure. The ordinates of F* and G* are, of course, the $p_o$* values. Obviously the stiff, low-strain bar system has suffered the greater loss of prestress under the assumptions stated previously. The remanent prestress fractions are

$$R = \frac{274}{346} = 0.77, \text{ wire}$$

$$= \frac{200}{346} = 0.58, \text{ bar}$$

2.3 Effect of Working Loads

The preliminaries to assigning design pressure ratings to the two constructions are now in hand. The loading to be considered here is simply the design gage pressure of the coolant. Here the vessel wall is assumed to be isothermal; if it were not, thermal stress problems would also demand attention in setting pressure ratings.

A significant criterion in any prestressed concrete structure is the loading that will cause the compressive stress in the concrete to pass through zero into tension, so that a small increase in loading will initiate cracking. In the vessel, under increasing internal pressure, stress reversal will first appear in the hoop stress at the inner surface. As was true of the level of prestress pressure, the elastic response of both the steel and the concrete are involved in the relation between internal pressure and tangential components of resultant stress distribution.

It can be shown from the relations in Table 2.1-1 that

$$\sigma_{t(r=a)} = 0$$

provided that

$$\frac{p_i}{p_o} = \frac{2b^2}{a^2+b^2}$$

Corresponding to this pressure ratio, the strain at the interface will be small, but generally not zero:

$$\Delta r_{(r=b,\sigma_{ta}=o)} = \frac{-bp_o}{E_c}\left[\frac{a^2+b^2}{b^2-a^2} - \nu_c - \frac{p_i}{p_o} \cdot \frac{2a^2}{b^2-a^2}\right]$$

$$= -\frac{bp_o}{E_c}\left(\frac{b^2-a^2}{a^2+b^2} - \nu_c\right)$$

The fraction in the parenthesis approaches zero for a thin wall and unity as the bore radius a becomes very small; the parenthetical value changes sign when the fraction equals Poisson's ratio. For the example vessel

$$\Delta b = -\frac{612}{5}(.238 - .15)(10)^{-6} p_o$$

$$= -10.8(10)^{-6} p_o$$

that is, about 2% of the strain caused by prestress pressure acting alone remains at the pressure ratio corresponding to zero hoop stress at the inner surface of the concrete.

As has been shown previously, the pressure ratio for zero tangential stress at the inner surface in the example concrete shell is

$$\frac{p_i}{p_o} = \frac{2b^2}{a^2+b^2} = 1.238 \rightarrow \sigma_{tpo}(r=a) = -\sigma_{tp_i}(r=a)$$

Obviously, under elastic conditions, radial displacement of the interface to the right by internal pressure loading will induce increasing prestress pressure along the extensions of the straight lines D* F* (for wire) or C* G* (for bar); interest now lies in the stabilized regime of creep, shrinkage, and relaxation that will characterize the major part of the service life of the vessel. In fact, the steel may be looked upon as a strain transducer, at least for isothermal conditions, subjected to no other influence than the compulsion to assume equal extension with the outer surface of the concrete. As radial strain increases linearly with increasing coolant pressure, lines F* A* and congruent G* A* are a graphic representation of the progress of transferring the support of interface pressure from the strained concrete to the coolant as the compressive strain in the concrete lessens and passes its free unstrained radius at A*.

Starting from the known interface pressure and known radial displacement at F* or G*, when the coolant pressure is zero, the statically indeterminate change in interface pressure and consequent change in displacement of the interface corresponding to any coolant pressure are readily computed from two equations:

(a) change in radial displacement of concrete equals change in radial displacement of steel

$$-\frac{b\Delta p_o}{E_c}\left(\frac{a^2+b^2}{b^2-a^2} - \nu_c\right) + \frac{2p_i a^2 b}{E_c\ (b^2-a^2)} = \frac{\Delta p_o b^2}{E_s A_s}$$

(b) change in radial displacement of steel is a unique function of change in interface pressure

$$\Delta b = \frac{\Delta p_o b^2}{E_s A_s}$$

Considering the case of the wire system in the example shell, condition (a) yields

$$- 495.33(10)^{-6} \Delta p_o + 391.29(10)^{-6} p_i = 10,665(10)^{-6} \Delta p_o$$

For example, the "zero stress in concrete" condition requires that

$$\frac{p_i}{p_o} = \frac{p_i}{\Delta p_o + 274.0} = 1.238$$

or $p_i = 1.238 \Delta p_o + 339.2$

substituting in the strain equation gives

$$\Delta p_o = 12.5 \text{ psi}$$

$$p_o = 274.0 + 12.5 = 286.5$$

$$p_i = 355.2$$

From condition (b) the increase in radial displacement of the steel corresponding to $p_i = 355.2$ is

$$\Delta b = 10,665(10)^{-6} \Delta p_o = 0.133 \text{ in.}$$

so that the radial displacement of the interface becomes

$$- 0.760 + 0.133 = - 0.627$$

only 3 mils short of point A*.

A similar calculation for the bar system gives the values for the zero-stress condition shown in Table 2.3-1, and also interface pressures corresponding to increments of 100 psi in the coolant pressure, $p_i$, for both systems. These results are shown in larger-scale replots of region of interest in Fig. 2.3-1 for wire system and Fig. 2.3-2 for bar system. Vertical lines KLJ for $p_i = 354.7$ (wire) and MNP for $p_i = 275.3$ (bar) correspond to "zero tangential stress at inside surface of concrete" for the two systems. Radial pressure at the interface may be read from line F* K and G*M. Lines F* L & G* N divide the pressure intercepts such as K J and M P into the fractions of the internal pressure loads carried by the increase in strain of the steel, e.g.,

$$\frac{K\,L}{K\,J} \text{ and } \frac{M\,N}{M\,P},$$

and the fractions carried by loss of strain in the concrete, e.g.,

$$\frac{L\,J}{K\,J} \text{ and } \frac{N\,P}{M\,P}$$

It may be noted that the smaller radial strain needed to place the concrete in danger of cracking in the lower-ultimate-stress system does not signify that

Table 2.3-1. Response of Stabilized Prestressed Shells to Coolant Pressure

| System | Point in Plot | Radial Displacement in. | Radial Stretch of Steel in. | Interface Pressure psi $p_o$ | Coolant Pressure psi $p_i$ |
|---|---|---|---|---|---|
| Wire 250 Kip | D* | -3.681 | 0 | 0 | |
| | | -0.9 | 2.781 | 260.8 | |
| | F* | -0.760 | 2.921 | 274.0 | 0 |
| | | -0.7225 | 2.959 | 277.6 | 100 |
| | | -0.685 | 2.996 | 281.0 | 200 |
| | | -0.6475 | 3.033 | 284.5 | 300 |
| | Zero Stress | -0.627 | 3.054 | 286.5 | 354.7 |
| | A* | -0.624 | 3.057 | 286.7 | |
| | | -0.610 | 3.071 | 288.1 | 400 |
| Bar 100 Kip | C* | -1.575 | 0 | 0 | |
| | | -0.9 | 0.675 | 158.3 | |
| | G* | -01723 | 0.852 | 199.8 | 0 |
| | | -0.688 | 0.887 | 208.0 | 100 |
| | | -0.653 | 0.922 | 216.2 | 200 |
| | Zero Stress | -0.6266 | 0.9485 | 222.4 | 275.3 |
| | A* | -0.624 | 0.951 | 223.0 | |
| | | -0.618 | 0.957 | 224.4 | 300 |

Unclassified
ORNL-DWG 64-3149

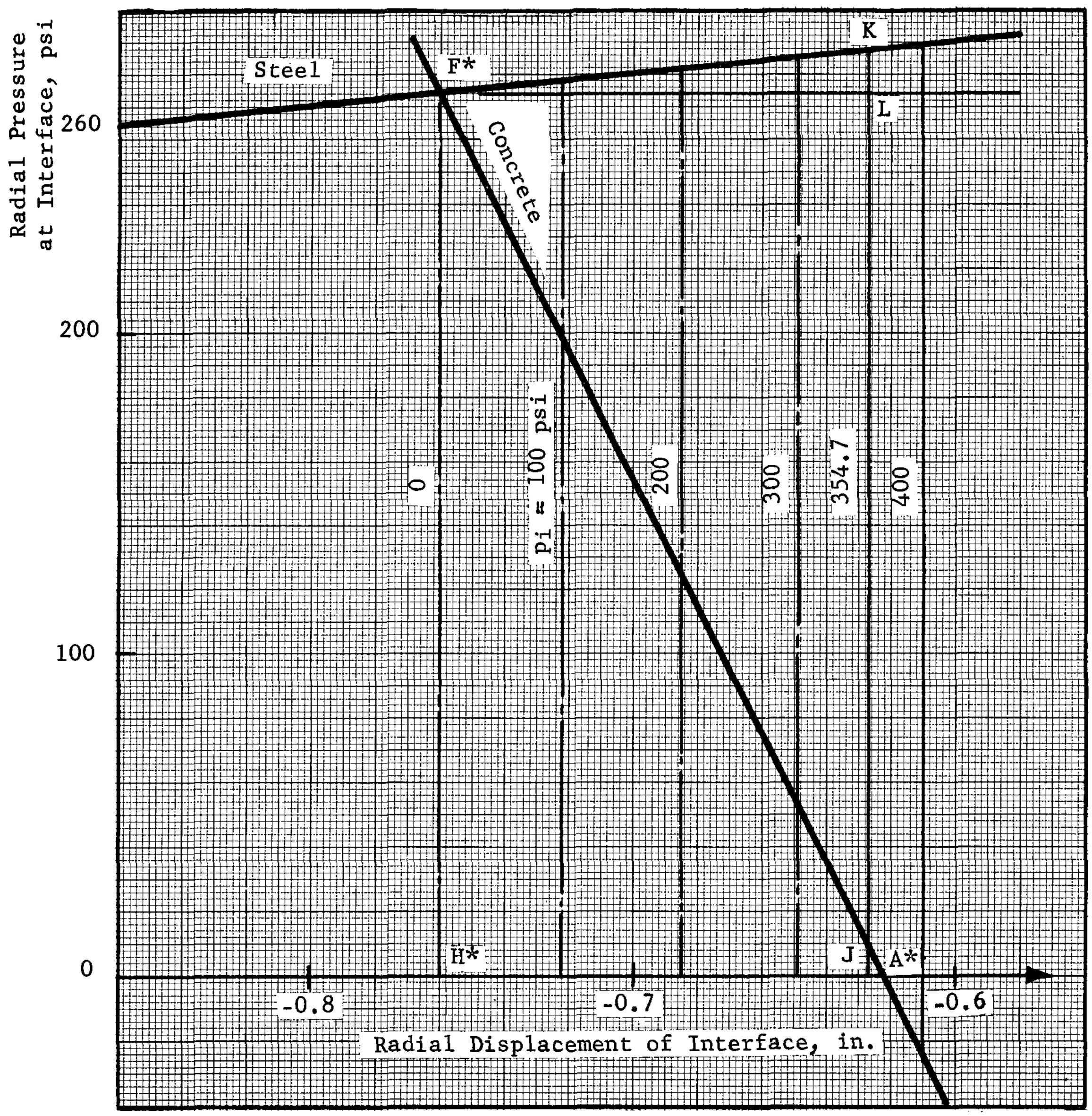


Fig. 2.3-1. Relation of Radial Displacement and Interface Pressure to Pressure of Coolant After Stabilization of Creep and Shrinkage. (250 Kip/in.$^2$ Wire System.)

Unclassified
ORNL-DWG 64-3150

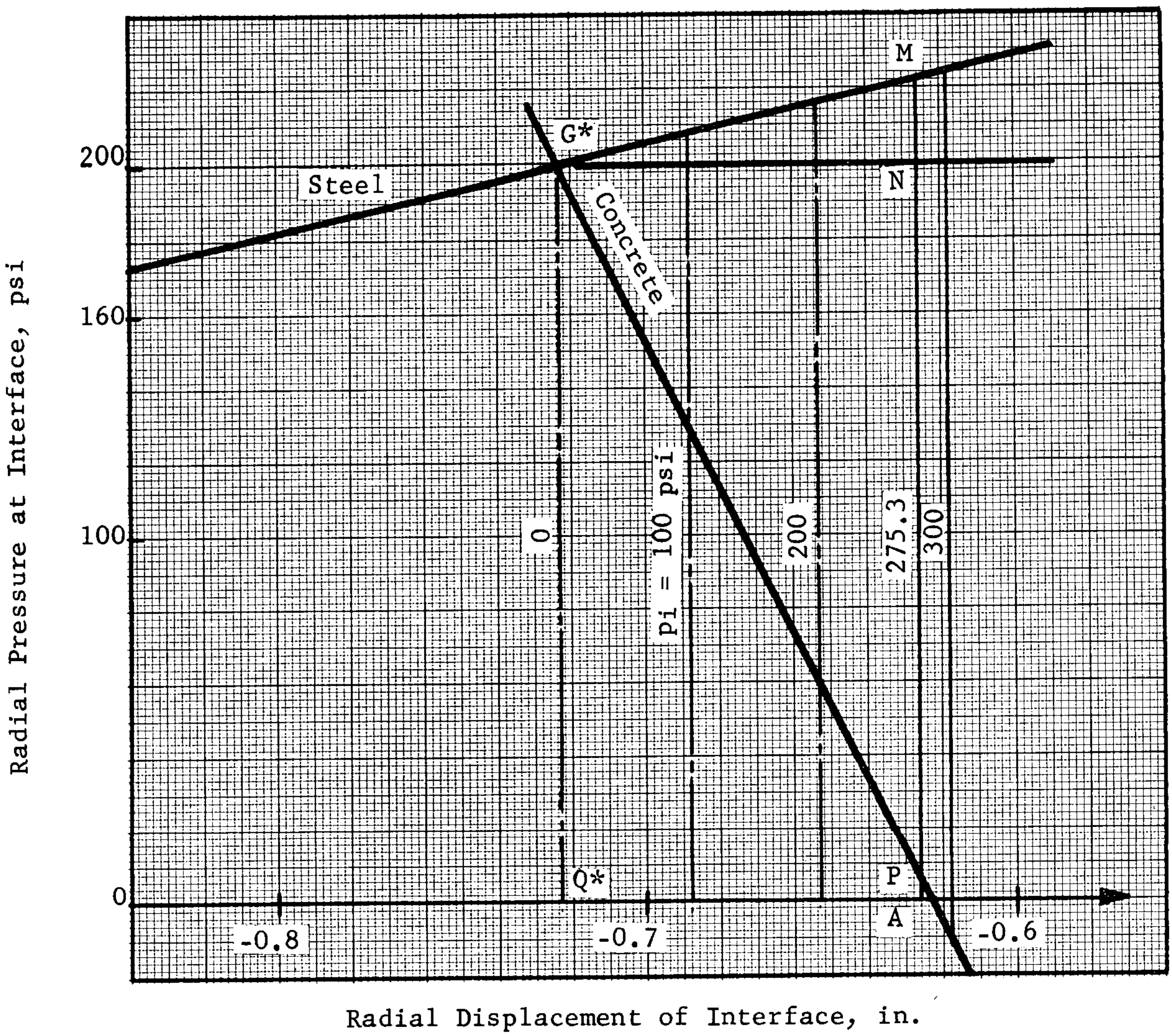


Fig. 2.3-2. Relation of Radial Displacement and Interface Pressure to Pressure of Coolant After Stabilization of Creep and Shrinkage. (100 Kip/in.$^2$ Bar System.)

the steel is not approaching trouble also. For the concrete stress reversal, the radial stretches of 0.133 in. for wire and 0.0965 in. for the bar caused by the coolant pressure will induce stress increments

$$\Delta \sigma_{ts} = E_s \Delta \epsilon_s = E_s \frac{\Delta b}{b} = 29(10)^6 \frac{\begin{pmatrix} 0.133 \\ 0.0965 \end{pmatrix}}{612} = \begin{matrix} 6300 \text{ psi} & \text{wire} \\ 4570 \text{ psi} & \text{bar} \end{matrix}$$

The live load increments as a fraction of ultimate tensile strength would be 6300/250,000 or 2.5% for the wire and 4570/100,000 or 4.6% for the bar. Five percent ultimate tensile strength live load increment has been recommended as a limit in tank practice[58] to avoid fatigue troubles in anchors and grouting bonds.

The selection of a design pressure would be strongly influenced by the level to cause stress reversal in concrete with due regard to the need of avoiding cracking because of likelihood of damage to liner, if used, and reinforcements around penetrations and head junctions. The moderate-strength bar steel system discussed here requires purchase and installation of 2.5 times the weight of the high strength wire. After stabilization of shrinkage and creep it shows a 22.5% (estimated) lower coolant pressure for stress reversal, and nearly twice the live-load increment of stress expressed as a fraction of its ultimate tensile strength of steel.

## 3. Environmental Considerations

From the foregoing discussion the conclusion follows that the prestressing must be capable of a large amount of strain to avoid inherent prestress losses due to the inelastic behavior of concrete. Also it is obvious that the inelastic properties of concrete must be understood and controlled to a degree which assures that the properties of the prestressing can be maintained.

It is impossible to evaluate the properties of the working materials in a prestressed concrete reactor vessel without knowing the environmental conditions. These will vary depending upon the reactor application and they cannot be defined in an absolute fashion. Their nature can be discussed in qualitative terms to establish where design limits must be imposed. The suitability of prestressed concrete reactor vessels to various reactor concepts may thereby be inferred from the reactor operating conditions. To help the reader in appraising the usefulness of prestressed concrete in various reactor systems, the influence of environment on the structural properties of PSC are discussed here.

The important conditions to be considered are:

1. Temperature effects on strength.
2. Working pressure and related design stress.
3. Thermal stress effects.
4. Creep behavior of concrete and steel as a function of temperature and stress level.
5. Shrinkage behavior as a function of humidity, temperature, physical size and time.
6. Nuclear irradiation.

### 3.1 Temperature Effects

Many observers have noted the sensitivity of concrete structures to nominally high operating temperatures as well as temperature gradients. The water content of concrete and its low thermal conductivity account for most of its structural peculiarities. The conventional practice in concrete design is to maintain concrete temperatures well below the boiling point of water so that there will be no gross migration of free water from the structure. Loss of water accounts for most shrinkage effects and probably influences creep behavior. Temperature gradients introduce strains in the concrete which may assert themselves to a degree that overrides the primary stress conditions in a concrete structure. Prestressed concrete designs must allow for the further complication that the steel itself may change its physical properties with elevated temperature. Each of these considerations must be properly balanced to develop a workable design.

3.1.1 Thermal Stress

The low tensile strength of concrete establishes the principal limitation on acceptable thermal stresses. The difficulty is compounded by the poor thermal conductivity of concrete which permits even a small heat source to develop a substantial temperature gradient. Prestressed concrete structures in reactor applications may behave somewhat differently from conventional structures since temperature at the inner surface of the concrete causes a compressive load at that point, but the net effect is to require additional load on the prestressing to prevent cracking at the exterior of the structure.

Techniques for analyzing the thermal stresses in concrete have been developed by Bonsall,[8,9] Hannah,[10] and Samelson and Tor,[11] and are based on the elastic behavior of concrete in accordance with Hooke's Law. As shown by Bonsall's sample calculation, a small gradient could create very high stresses. The obvious conclusion is that such stresses must be avoided or the acceptance of thermal stress cracking incorporated in the design. This latter condition has not been accepted in most designs executed up to now, thus setting a limit on the maximum operating temperature of the concrete and the permissible temperature gradient. It has been argued by Waters and Barrett[25] that this is an unnecessarily severe limitation since cracking may be controlled by conventional mild steel reinforcing as in conventional reinforced concrete.

Rapid temperature transients can introduce an untenable condition if sufficiently severe. The rate of thermal diffusivity for concrete is quite low and exposure to rapid changes in surface temperature will create thermal stresses of substantial magnitude at the surface which are completely independent of the overall structural behavior of the body and may cause spalling at the concrete surface. No experimental work has been identified which would be useful in fixing limits on this type of transient, but it is presumed that conditions which would cause severe dehydration of the surface must be avoided even for short intervals of time.

3.1.2 Strength-Temperature Relationship of Concrete

Apart from thermal stress considerations, there is the question of the effect of temperature on the concrete strength. In reactor systems the design may be greatly simplified if some portion of the concrete can operate at temperatures approaching the coolant temperature. Experimental work[12] has shown that for short time exposures Portland cement retains 33 to 85% of its strength up to 800°F. At higher temperatures there is a rapid deterioration in strength as the structural integrity of the concrete is destroyed by decomposition of the hydrated calcium silicate crystals. Higher temperature limits can be realized with the calcium aluminate concretes used as refractories in high temperature furnace applications.

Tests by Livovich[24] have indicated that strength deterioration in concrete at elevated temperatures may be attributed to alternate hydration and dehydration of the cement crystals, causing gross dimensional changes in the crystal structure. Although most concrete specialists are inclined to the view that temperatures above the boiling point of water will materially reduce the strength of concrete over a long period of time, there does not appear to be any body of experimental evidence to substantiate this view. It may be that the concern develops from the implied alternation of the hydration and dehydration processes. If so, the environment in a prestressed concrete reactor vessel may be controlled in such a way as to minimize this effect.

Tests by Malhotra[12] have provided a relationship between temperature and strength which should prove useful to the concrete reactor vessel designer. These show that for short exposures to temperatures up to 600°F concrete retains 80 to 95% of its compressive strength when tested hot. Further deterioration took place during cooling, so that cold specimens exhibited residual strengths as low as 55% of the normal (unheated) strength.

#### 3.1.3 Strength-Temperature Relationship of Prestressing Steel

Since conventional practice in the design of prestressed concrete is to use hard drawn wire at very high stress values (about 170,000 psi initial prestress and net loads of 130,000 psi), it is important to consider the possibility of creep in the wire due to temperature, stress, or a combination of these two effects. Creep in prestressing wire has been investigated by Day, Jenkinson, and Smith.[13] They have established that hard drawn wire will creep significantly at temperatures above 300°F when a stress equal to 60% of ultimate strength is applied. Thus the maximum temperature of high strength prestressing wire is fixed by this limitation and even though concrete may be operable at temperatures up to 600°F, prestressing wire must be kept out of such high temperature zones. Although no investigation has been reported in the literature, it may be that alloy steel wires and bars could be developed to circumvent this limitation if there were sufficient incentive.

#### 3.1.4 Nuclear Radiation Effects

In the use of prestressed concrete for a nuclear reactor vessel combined with a biological shield, the effect of radiation on the physical properties of both the prestressing steel and the concrete must be considered. The introduction of thermal stresses due to heat deposition has already been discussed in Section 3.1.1. The limits of heat deposition are established by thermal stress limits and no other difficulty is foreseen; however, the properties of the vessel materials could be affected by radiation.

Among the conditions to be avoided is loss of moisture in the concrete, if this is the principal source of hydrogeneous neutron shielding. Although each reactor concept presents its own requirements, there is little likelihood that the concrete will suffer more than 60% moisture loss below 800°F in terms of neutron shielding since a large fraction of the hydrogen is tied up in the hydrate crystals.

Fast neutron damage to pressure vessel steels has aroused concern[18] because of loss of ductility. The behavior of prestressing and reinforcing steels under irradiation has not been investigated experimentally and no information is reported in the literature about their behavior. Pending the development of experimental information to the contrary, it would be prudent to shield prestressing steel from significant fast neutron dosage, thus requiring that the steel be placed in the outer portion of the concrete or that internal shielding be provided.

The structural properties of concrete as affected by radiation are similarly lacking in experimental information, but there is no concern for anything other than a change in the bonding characteristics of the cement and it does not appear likely that radiation effects would be deleterious in this respect at reasonable dosage levels.

#### 3.1.5 Stress-Temperature Effects on Inelastic Behavior

The previously discussed environmental conditions for prestressed concrete reactor vessels cannot be divorced from the working conditions which make prestressed concrete useful for this application. Some measure of temperature rise must be accepted and a high level of compressive stress in the concrete is assumed. Even under the most favorable temperature conditions, the concrete will shrink when it sets, but a limit can be established for the shrinkage if moisture migration can be prevented by maintenance of a high humidity environment for the concrete and a uniform temperature. Since this cannot be expected in a reactor vessel, the design must include an allowance for continuous shrinkage. Two types of shrinkage are involved: an "autogeneous" form which may be independent of moisture level and a form commonly known as "drying shrinkage" associated with the migration of moisture. The latter is responsible for the bulk of concrete dimensional change due to shrinkage. Its effect can be very large and a great deal of investigatory work has been done to establish its significance in the concrete structure.

A detailed discussion of this work is presented in Section 6.2, and it will suffice for now to observe that the net effect is seen as a loss of prestressing which must be allowed for in concrete design.

In addition to shrinkage, the concrete undergoes a change in structural character in an applied compressive stress field which is analogous to metal creep and in concrete structural design is treated in a similar manner. It is difficult to divorce shrinkage behavior from creep and their ultimate effect is the same, a loss of prestress.

Both creep and shrinkage are temperature senstive, accelerating at higher temperatures and decelerating at reduced temperatures. Creep rates for concrete, like creep rates for metals, are affected by the stress level. The net result is that the creep characteristic of concrete, like shrinkage, enters strongly into the evaluation of its structural behavior and a large amount of experimental work has also been directed in this area. The experimental work is also discussed in Section 6.2. When shrinkage and creep effects are combined they represent changes in physical dimension comparable to elastic dimensional changes caused by structural loads and they may, therefore, govern the ultimate capability of a concrete structure.

#### 3.1.6 Environmental Chemical Attack

Generally speaking, the environments associated with a reactor vessel are not likely to affect the physical characteristics of a reactor vessel in a chemical sense. By their nature, such installations are located in non corrosive ambients and strong acids or bases do not have to be considered. Probably the most important factor is the possibility of normal rusting of the prestressing steel by humid air. Conventional practice in prestressing has been to grout the steel to limit rust action, but this may not be consistent with the inspection requirement for the prestressing. Galvanized cable is used in many structures as further protection and from observations of cables in use more than 90 years[14] (for example the Brooklyn Bridge), it has been seen that galvanizing can be very effective. The effectiveness of galvanizing requires that the steel not be submerged in liquid water for extensive periods, and water condensation in the vicinity of the prestressing must be avoided. The operating temperatures are generally conducive to assuring this condition in a reactor vessel, but further assurance may be attained by the application of a corrosion protective grease as was done in the Savannah River containment vessel.[15]

## 4. Functional Requirements of a Concrete Reactor Vessel

The simple requirement that a concrete reactor vessel be capable of holding a high temperature fluid at high pressure understates the demands on a reactor vessel. It is primarily a power plant structure with the containing function an incidental though essential feature. As a power plant structure, the implied needs are openings for maintenance, nozzles for supply and return of the power transmission fluid, nozzles for reactor control rods, openings for instrumentation, foundations for support of equipment, anchors for piping subjected to flexure, seals against leakage of coolant, and biological protection of operating personnel from nuclear radiation. To these are added the requirements which make the vessel function properly, such as its cooling system and insulation for protection against overheating and thermal stresses, its performance monitoring system, and its closure attachments. Although these demands seem to be large and imposing, they can be defined by a discussion of a few basic features.

### 4.1 Vessel Liner

The use of concrete as a pressure container does not require that it perform as a leak tight seal. In many fluid systems small leakage rates are an accepted penalty of little economic significance and without any danger to the external environment. In reactor systems the vessels must be leak tight, first to assure containment of dangerous radioactive constituents in the coolant, and second, to prevent loss of the coolant which may be of substantial value. The simplest approach to a liner is that used in the Savannah River containment vessel in which a thermal setting plastic coating is sprayed on the inside of the concrete as a seal. The experience at Savannah River has shown that such a seal has limited effectiveness and metal liners are used in gas-cooled reactor vessels for the French G-2, G-3 and EDF-3 vessels as well as the British Oldbury vessel. The metal liners are not pressure containers but leakage seals. They are supported by the vessel structure and pressure loads are transmitted to the concrete when necessary by yielding of the liner material. Waters and Barrett[25] have noted that such stresses may exceed the yield strength in a steel liner. Clearly, when yielding is permitted there must be some limiting device to prevent elongation to failure, and the fit between the liner and the concrete is established by this limit. Because of the large size of vessels usually considered for reactor systems, local attachments must be provided to prevent accumulation of dimensional variations in a single area of gross weakness. The thickness of the liner can be established by the permissible elongation permitted in the liner as it conforms to the shape of the concrete vessel.

### 4.2 Insulation and Cooling

The need for temperature control of the pressure vessel requires a heat sink for nuclear heat deposition and a barrier to limit the flow to the heat sink. If the barrier were sufficiently good so that the small amount of heat passing through the concrete did not create high thermal stress, and if the heat could be dumped to the atmosphere, then only insulation would be needed. Thus it is important to establish the capability of the insulation before defining the vessel cooling system. In low temperature systems such as the Marcoule G-3 reactor where the reactor coolant bathing the vessel wall was 25°C and where the graphite reactor reflector essentially eliminated the bulk of the neutron heating, there was no need for insulation or a large amount of supplemental cooling. In the more demanding applications at EDF-3 and Oldbury, a metal reflective insulation is used on the inside of the liner to reduce heat flow from the reactor coolant to the vessel wall and this is supplemented by a water cooling system attached to the liner. In addition a laminated shield consisting of graphite and iron is used to intercept nuclear energy from gamma radiation and neutron leakage which would normally be captured at the vessel wall.

These insulating devices assure that concrete does not rise in temperature significantly and the water cooling system collects heat passing through the liner before it can impose a thermal load on the concrete structure. The water system further assures that the liner and the concrete remain at comparable temperatures so that differential thermal expansion does not affect the liner structure.

### 4.3 Large Openings

Maintenance access and coolant ducts indicate the need for openings in the range of 2 to 30 feet in diameter depending upon the reactor systems. Such large openings cause edge effects at the vessel wall because of the introduction of bending stresses. Furthermore, the liner effectiveness can only be insured if its sealing function can extend through the opening. The accepted practice in the few vessels currently under design is to embed a heavy steel nozzle in the concrete, surround it by conventional steel reinforcing to absorb the concentrated loads in the concrete, and weld it to the liner. There does not appear to be an alternative to this approach.

### 4.4 Small Openings

Generally, small openings, widely separated, will not have any material effect on the concrete vessel structure. The sealing requirement is still imposed as in larger openings, but the supplemental reinforcing may be avoided and the nozzle attachment allowed to move with the metal liner. Local stresses in the liner may need to be avoided by local stiffening and anchoring in the nozzle vicinity.

Where nozzles, though small, must be closely spaced, the overall effect can be to soften the structure. Difficulties can be avoided by assuring a minimum ligament size between nozzles, but thus far nothing has been reported about the minimum permissible spacing for nozzle clusters. The needs concerning this design feature may impose severe limitations on reactor vessel design, especially in control rod locations, and may indicate areas where a group of small nozzles must be handled as a large opening.

### 4.5 Foundations

Both the size and weight of a concrete reactor vessel indicate a need for a substantial supporting device. Dead loads from the vessel weight are not of themselves particularly severe except during the construction stage, since they are not terribly large compared to the pressure loads even though they may be additive. The more important consideration is the effect of large changes in temperature that cause the vessel to expand and contract. The foundation must not interfere with this movement or very large stress concentrations will develop at the juncture between the vessel and its foundation. The current practice in European installations is to separate the vessel from its foundation by a membrane such as neoprene rubber which will allow freedom for the vessel to move independently of its foundation. Only frictional effects then interfere with this movement.

### 4.6 Closures

In previous discussions of openings, the need for methods of closing the opening during reactor operation is immediately apparent. The structural form of the closure may be like that used in a conventional vessel: a steel flange, a canned concrete block, a welded dome, or similar device. The controlling requirement is the need to transmit the load applied to the closure into the concrete structure. Shear locks, wedges, imbedded bolts and similar devices can be employed. The difficulty of solution to the requirement is determined by the size of the opening to be managed and nothing is reported in the literature which gives a clue to the limitations which must be imposed by this design feature.

### 4.7 Heads

The ideal pressure vessel is a sphere by virtue of the symmetry of its pressure and thermal loading and its means of resisting these loadings without discontinuities. Slightly less ideal are spheroids, ovoids, and toroids. All these shapes require that the prestressing cables follow the vessel curvature. Neither are such shapes necessarily able to accommodate the required internal components without waste of space. The first objection may be lessened or overcome by modification of the outer wall face toward various polyhedrons. By maintaining axisymmetry but using a straight line generator, the right circular cylinder, cone, and hyperboloid can be developed. These may have a more convenient interior shape as a container as well as allowing straight axial prestressing tendons. The hyperboloid permits straight tendons of both right- and left-hand obliquity with respect to the axis of revolution, thus accepting a complete prestressing network of straight elements that is efficient for a moderate range of obliquities near the waist of the hyperboloidal surface. The outer surfaces of the hollow cylinder or cone may also be modified toward a prism or pyramid of a polygonal cross section, thereby straightening the circumferential tendons.

The truncated conical, cylindrical, and hyperboloidal surfaces all have open ends that must be closed by partial spheres, oblate spheroids, cones or flat heads, listed in the order of increasing discontinuity-bending moments at the junction. Relief may be obtained by interposing a toroidal knuckle.

Several authors advocate inverting a dished head, so that the coolant pressure is carried on its convex surface and its meridional membrane stresses become compressive. However, the further claim that the prestressing then required for the head is reduced to a system of circumferential tendons at the rim, to resist the meridional thrust, needs examination. It must be noted that springing the dome in this manner will induce considerable meridional bending in all but nearly flat heads. The rotational compatibility of the rims of the dome and of the shell under prestress alone and in combination with thermal stresses and internal pressure must also be investigated before accepting the inverted head without provision of meridional prestress.

## 5. Design Basis

If concrete pressure vessels are to be accepted for nuclear vessel applications, some method must be established to show their structural adequacy. A suitable basis may be derived by investigating construction precedents where design analysis can be correlated with statistical evidence of reliability. If no suitable precedent exists, then a logical structural model must be defined, followed by analytical and experimental verification before such vessels can be considered acceptable on safety grounds.

### 5.1 Design Precedent

Codes of minimum requirements for power boilers and unfired pressure vessels have evolved[16] as reasonable bases for design, fabrication, and inspection in order to satisfy legal and insurability criteria in an orderly way. For steel nuclear-reactor vessels the new problems arising were handled by special rulings of the ASME Power Boiler Code Committee; these have been incorporated into the latest edition of the code. More detailed and specific design rules are found in the Tentative Structural Design Basis for Reactor Pressure Vessels and Directly Associated Components, PB-151987, UNS Bureau of Ships, December 1958[17] and its recent classified revision. No such basis exists for Nuclear PCPV's.

In Britain, concrete-shielding and PCPV designers have been guided to some extent by the following codes of practice and standards:

British Code of Practice CP115, "The Structural Use of Prestressed Concrete in Buildings."

CP2007, "The Design and Construction of Reinforced and Prestressed Concrete Structures for the Storage of Water and Other Aqueous Liquids."

British Standard BS 12, "Portland Cement."

BS 146, "Portland Blast Furnace Cement."

BS 269, "Steel for Prestressed Concrete."

BS 18 for methods of testing high-tensile wire.

BS 785 for mild steel.

BS 1144 for cold-worked steel.

BS 882 for grading fine aggregate.

In the United States, the corresponding guides are the Building Code of the American Concrete Institute (ACI), tentative reservoir and concrete pressure piping codes of the waterworks associations, and the Standard Specifications of the American Society for Testing and Materials (ASTM) as named in the foregoing codes appropriate for PCPV materials.

Particularly pertinent is the 1963 revision of the "Building Code Requirements for Reinforced Concrete."[27] This code provides minimum requirements for the design and construction of plain, reinforced, or prestressed concrete, or composite structural elements of any structure erected under the requirements of the general building code of which this code forms a part. For special structures, such as arches, tanks, reservoirs, grain elevators, shells, domes, blast-resistant structures, and chimneys, the provisions of this code govern where they are applicable.

Chapter 26 or the ACI Code is devoted to prestressed concrete. Although structures for containing pressurized fluids are specifically exempted from its requirements, it is believed that a PCPV for a reactor must satisfy them in spirit as a minimal assurance of safety. The stress limitations from a number of sources is given in Table 5.1.

The ASME Boiler Code, Section VIII, Unfired Pressure Vessels,[16] gives the following basis for establishing stress values for steel: At temperatures below the creep range allowable stresses are established at the lowest value of stress obtained from using 25 per cent of the specified minimum tensile stress at room temperature, or 25 per cent of the minimum expected tensile strength at operating temperature, or 62.5 per cent of the minimum expected yield strength for 0.2 per cent offset, at temperature. No credit is allowed for any improvement in room-temperature tensile strength by special heat treatment. The ASME Code sets the following requirements:

(a) At high temperatures the allowable stresses are based on 100 per cent of the stress to produce a creep rate of $100\ (10)^{-6}$ per 1000 hours, or 100 per cent of the stress to produce rupture at the end of 100,000 hours, based on conservative average of test values. Generally the creep strength is well below the rupture strength.

(b) In the transition range of temperatures, the stress allowances are limited to values obtained from a smooth curve joining the low and high temperature ranges, but not greater than 62.5 per cent of the minimum expected yield strength at temperature.

(c) The membrane stresses may not exceed the product of allowable stress and weld efficiency, after suitable corrections for corrosion and stress distribution in thick shells. Rules are given to limit bending stresses in flat heads and blind flanges to values shown to be safe by extensive experience. Openings are reinforced by replacing the material required by membrane stress close to the penetration. Rulings are included that limit certain discontinuity

Table 5.1. Permissible Stresses as Fractions of Material Test Values

| | Basic Test Value [Ref. 27] | ACI Building Code 1963, Chap. 26 [Ref. 27] | Criteria for Prestressed Concrete Bridges Bu.Pub.Roads 1954 [Ref. 88, Appendix D (Lin)] | Joint Committee ACI-ASCE Appendix B [Ref. 89 (Libby-1961)] | Draft Code of Practice, ISE London, Converted to Cyl. Test Spec. [Ref. 93 (Billig-1951)] | Evans & Bennett [Ref. 90] |
|---|---|---|---|---|---|---|
| Temporary Stresses | | | | | | |
| in Concrete | | | | | | |
| Compressive, Pretensioned | $f'_{ci}$ | 0.60 | 0.60 | 0.60 | 0.40 | 0.32 - 0.40 |
| Post-tensioned | $f'_{ci}$ | 0.60 | 0.55 | 0.55 | | 0.32 - 0.40 |
| Bearing at Anchorage (at transfer) | $f'_{ci}$ | $f_{cp}=0.6f'_{ci}(A'_b/A_b)^{1/3}$ but less than $f'_{ci}$ | | $f_{cp}=0.6f'_{ci}(A'_b/A_b)^{1/3}$ | | |
| Tensile | $f'_{ci}$ | $3(f'_{ci})^{1/2}$ | 0.05 | Zero to $6\ (f'_{ci})^{1/2}$ | | 0.5 Modulus of Rupture at Transfer |
| in Prestressing Steel | | | | | | |
| Tensile During Jacking | $f'_s$ | 0.80 | 0.80 | 0.80 | 0.70 | 0.70 |
| After Anchoring | $f'_s$ | 0.70 | | 0.70 | 0.65 | |
| Stresses under Design Load | | | | | | |
| in Concrete | | | | | | |
| Compressive | $f'_c$ | 0.45 | 0.40 | 0.40 Bridge Members<br>0.45 Building Members | 0.32 | 0.32 - 0.33 |
| Bearing at Anchorage | | - | - | - | - | - |
| Tensile | $f'_c$ | Zero to $6(f'_c)^{1/2}$<br>Note A | zero<br>Note B | Zero to $6\ (f'_{ci})^{1/2}$ | | $0.04\ f'_c$ |
| in Prestressing Steel (Effective Prestress) | $f'_s$<br>$f_{sy}$ | 0.60 } whichever is<br>0.80 } smaller | 0.60 } whichever<br>0.80 } is smaller | 0.60 } whichever is<br>0.80 } smaller | $0.60\ f'_s$, or .75 of 1% Proof Stress whichever is less | |

Table 5.1. Permissible Stresses as Fractions of Material Test Values (Continued)

| | Basic Test Value [Ref. 27] | Magnel 1954 [Ref. 92] | European Codes [Ref. 88 (Lin), p 384] Belgium | England | France | Germany | Switzerland | Recommended (Ref. 34) |
|---|---|---|---|---|---|---|---|---|
| Temporary Stresses | | | | | | | | |
| in Concrete | | | | | | | | |
| Compressive, Pretensioned | $f'_{ci}$ | 0.58 | 0.45 | 0.50 | 0.50 | 0.45 | 0.40 | 0.60 |
| Post-tensioned | $f'_{ci}$ | 0.58 | | | | | | |
| Bearing at Anchorage | $f'_{ci}$ | Never exceed prin. tensile stress of 0.2 $f'_c$ (reinforced) | 0.73 | 0.62 | 0.93 | 0.61 | - | $f_{cp}=0.6f_{ci}\sqrt[3]{A'_b/A_D}$ |
| Tensile, Unreinforced | $f'_{ci}$ | | 0 to 0.045 | 0.075 | 0.04 | 0 to 0.11 | - | 0 |
| Reinforced | $f'_{ci}$ | | 0.09 | - | - | 0.14 | - | Design as reinforced concrete |
| in Prestressing Steel | | | | | | | | |
| Tensile During Jacking | $f'_s$ | 0.66 | 0.66 | - | - | 0.60 | 0.70 | 0.80 |
| After Anchoring | $f'_s$ | 0.60 | 0.60 | - | 0.67 | 0.55 | - | 0.70 |
| Stresses Under Design Load | | | | | | | | |
| in Concrete | | | | | | | | |
| Compressive | $f'_c$ | 0.33 | 0.33 | 0.41 | 0.35 | 0.34 | 0.40 | 0.45 |
| Bearing at Anchorage | | | | - | | 0.31 | | |
| Tensile, Unreinforced | $f'_c$ | | 0 to 0.045 | 0 to 0.075 | 0 to 0.025 | 0 | - | |
| Reinforced | $f'_c$ | Not greater than 0.10 (principal tensile caused by shearing force) | 0.09 | - | - | 0.11 | - | Design as reinforced concrete |
| in Prestressing Steel | $f'_s$ | 0.60 | - | - | 0.57 | - | - | 0.60 |
| (Effective Prestress) | $f_{sy}$ | 0.80 | - | - | 0.57 | - | - | 0.80 |

stresses at head junctions and flanged connections.

The USN Bureau of Ships Tentative Structural Design Basis for Reactor Pressure Vessels PB151987 OTS 1958[17] deals with secondary stresses, thermal stresses, and fatigue damage in steel vessels in a comprehensive manner. It is expected that similar material is being included in Section III of the ASME Code to be issued in 1963.

Billig[28] supports the use of British Codes BS 1500-158 and CP115:159 as relevant to concrete pressure vessel design. In the United Kingdom there is as yet no firm acceptance of any design code for vessels currently under construction and such codes are used only on an ad hoc basis.[29]

The appraisal of design adequacy requires that some basis for acceptance be established. Thus far application of available codes is poorly defined. The ensuing discussion indicates some approaches under consideration for the establishment of a suitable design basis.

## 5.2 Structural Model

### 5.2.1 Loading Conditions

Pressure. A number of differences exist between the behavior at high loadings of concrete as incorporated in a PCPV and that of the steels commonly accepted for application to pressure vessels. In particular, when establishing rules for pressure testing, attention is called to the relatively low strength of young concrete as compared to the strength it would exhibit when mature, the comparatively low elongation of concrete in tension as compared to steel that severely limits its ability to redistribute excessive local (secondary) stresses without permanent damage, and the irreversible micro-cracking (whose onset occurs at low levels of stress, progressing to an alarming extent at a stress level approaching the short-time ultimate stress) that besets concrete. Such behavior argues against combining transient and normal pressure stresses and then arbitrarily superimposing a 50% hydrostatic over pressure test in the name of safety, as for steel vessels. Nevertheless, some form of over pressure test is needed in order to monitor the ultimate behavior of the vessel.

There is a diversity of opinion[29] regarding the selection of a suitable pressure test for prestressed concrete vessels ranging from a value equal to 1.15 times the design pressure to one which equals twice the normal working pressure. While it is premature to pass judgement on the validity of the arguments for these extremes, the character of the concrete structure would tend to discourage a high over pressure test level. It would lead to operating the vessel at high concrete compressive loads under normal operating conditions, thus affecting its structural conditions adversely (the premise here is that under working conditions concrete has the advantage of being nearly a passive structure) or alternatively permitting the concrete to crack during the overpressure tests.

Thermal Loading. The introduction of temperature gradients into concrete is a practice held within severe limits for conventional concrete structures. Thermal stresses, even when biased in a way which favors the compressive loading of the concrete, reduce the margin available to accommodate the normal pressure loads on the structure. In furnace design, refractory concrete is often subjected to very high thermal gradients, but even in these cases the compressive strength of the concrete limits the acceptable level and high thermal stresses are often accommodated by permitting tensile cracking. In prestressed concrete where tensile cracking is to be avoided, the normal practice is to insulate the concrete from high temperature levels and to sweep the surface of the concrete with a relatively low temperature coolant. The thermal loading question then resolves into one of the maximum temperature gradient permissible across the concrete without sacrificing the strength capability assigned to the prestressing.

In all designs for reactor vessels which are currently being developed, this has fixed a maximum concrete temperature which is under 160°F[5] and assumes that the temperature gradient is the difference between that level and the external ambient temperature. Higher temperatures could be accommodated provided they did not interfere with the prestressing structure. In order to do so, designs may have to depart from standard elastic evaluation methods for concrete and a basis for this departure remains to be established.

5.2.2 Mode of Failure

Unlike a steel vessel, the stresses in the concrete are normally at their maximum compressive level when the system has no pressure load, while the prestressing steel is preloaded initially to its maximum level during construction. The applied pressure load opposes the prestressing force applied by the steel and relieves the compressive strain on the concrete. At design pressure load the concrete is thus near its lowest stress level. The prestressing steel, on the other hand, retains the tensile loading applied at installation regardless of whether the stress is applied by the pressure load or the concrete compression. A transient effect is introduced by temperature gradients across the concrete which may increase the compressive stress at the hot concrete surface and introduce a tensile component at the cold surface. The addition of thermal stress is necessarily felt as an increase in tensile strain on the prestressing steel and must be considered in the prestressing.

For the purpose of evaluating the ultimate behavior of the vessel when operating conditions exceed design limits, it is necessary to understand the response of the vessel when unexplained causes change the loading in the concrete from compression to tension. Since the tensile strength of concrete is very small, only a minor tensile stress will cause tensile cracking (this stress level does not exceed 10% of the concrete compressive strength), although this may represent a substantial increase in the pressure load. At the time of concrete tensile cracking, the steel stress increase will not have exceeded 10% of its yield strength so that there is no gross structural effect on the steel at this time. Consequently, interest is centered on the behavior of the concrete. Extrapolating from model tests, Gill and Hanna[2] and Billig[6] have argued that a failure preceded by rupture of the vessel liner will cause the concrete cracks to open allowing the pressure to leak off until the concrete prestressing can override the pressure to reclose and recompress the concrete so that explosive failure is nearly impossible. Mr. Campbell-Allen[29] has postulated that failure of the liner followed by overpressure might allow the coolant pressure to permeate the concrete so that the working pressure acts on the surface area of the cracks or perhaps on a larger effective circumferential surface of the concrete, creating an excessive pressure load which is not immediately self corrective and may be considered explosive though not necessarily in the sense of vessel disintegration.

The use of bonded reinforcing may prove to be effective as a crack arrester[28] since the bond around the reinforcing can absorb the wave of high strain prerequisite to a running crack if the bond and adjoining reinforcing is not already loaded to the point of failure. Conventional bonded reinforcing or even grouted prestressing cable could serve this purpose in limiting the propagation of a brittle fracture through the concrete. The protective feature of bonded reinforcing is predicated on the application of the Griffith crack propagation theory[31] which indicates that a crack will not propagate unless the strain release energy is greater than the energy binding the free surface of the body together. A demonstration of a more exact model of the crack propagation phenomenon has been attempted for materials exhibiting some plastic deformation by Irwin[32] and Orowan.[33] An experimental determination of the critical strain energy release rate by Kaplan[30] on a variety of concrete specimens has indicated that the propagation of cracks does, in fact, occur at stress levels below those predicted by simple elastic theory.

The character of failure in the prestressing steel appears to be such that an abrupt and catastrophic change in the structural condition of the vessel from this cause is improbable. Each cable and even each strand of wire is independent of all others. Propagation of failures must therefore occur progressively. The prestressing is itself ductile so that stretching rather than rupture will be the form of the failure. Furthermore, the wire quality should be extremely uniform by the nature of its manufacturing process so that local flaws of a serious nature are unlikely. Hence, the most probable form of failure to be considered in the prestressing cable is one involving loss of prestressing, the results of which are identical to those identified for tensile failure of the concrete.

The prestressing steel anchors are a source of potential difficulty. The loads are concentrated locally in these areas and bending as well as pure compression loads must be accommodated. Concern arises for shear failures in these areas and crack propagation is therefore of major concern. It is in the anchorage areas that great dependence must be placed on bonded reinforcement to resist bending and prevent shear failure in the concrete.

The introduction of elevated temperatures can not lead to any serious effect on abrupt failure characteristics. High temperature, though causing ultimate deterioration of the concrete strength cannot instantaneously affect a concrete structure. The very low thermal diffusivity of concrete permits the concrete to behave as an insulating device so that high temperatures would require long time periods to penetrate into the structure. Thus short time transient effects which plague the steel vessel designer can be essentially ignored in concrete design and long time effects are perfectly suited to safe monitoring of the vessel's operational condition.

A philosophy of design which logically treats the failure behavior of concrete vessels has been proposed by Waters and Barrett.[25] It is predicated on the assumption that the reactor operators are competent and able to correct operating deficiencies given ample time and warning of failure and that protective devices will provide prompt relief in the event of overpressure. Its purpose is to provide a design which is immune to the catastrophic conditions resulting from a rapid depressurization incident. These premises are directly comparable to those accepted for steel pressure vessels. The philosophy proposes not only to assure the vessel integrity, but the reliability of the vessel lining. From a safety viewpoint, the integrity of the liner is of secondary interest and may be ignored for the purpose of this discussion. The essentials of the approach are that:

1. Cracking due to secondary stresses is permissible if controlled by bonded reinforcing in such a way that cracks do not penetrate to the inner concrete surface.

2. Shear failures must be prevented by reinforcement in sufficient quantity to assure that excessive loading can only introduce tensile failure in the structure under load.

The application of these design criteria permits the acceptance of the premise that only a progressive failure is possible and the vessel is therefore adaptable to monitoring and control within safe operational limits. The presumption is:

1. Errors in the evaluation of long-term stress distribution can be observed and corrected prior to catastrophic failure.

2. That loss of prestress by concrete deterioration, excessive thermal strain or other unexplained conditions can be observed and corrected prior to failure.

The acceptance of these criteria appears to be highly consistent with steel vessel practice. Even the shear failure protection is reasonably analogous to premises accepted for bolted flange vessel closures.

The implications of these requirements are that the designer must know the load distribution in the vessel (possibly by pretesting and/or model study) and that he can provide sufficient margin between catastrophic failure loads and conditions where failure symptoms can be detected to permit adequate time for corrective action.

Using these criteria as a basis for design, the limiting load factor for a pressure vessel may be defined as the ratio of the pressure at which the plastic deformation becomes equal to the elastic deformation at design pressure to the design pressure.

$$\text{Limiting load factor} = \frac{P_L}{P_D}$$

The tension cracking corresponding to a loading of $P_L$ would be readily detected. If the ultimate load factor is defined as the ratio of bursting pressure $P_B$ to design pressure $P_D$,

$$\text{Ultimate load factor} = \frac{P_B}{P_D}$$

then a minimum difference of

$$\left[ \frac{P_B}{P_D} - \frac{P_L}{P_D} \right] \geq 0.5$$

is suggested as an adequate margin for safety between the loading to cause regions of concrete tensile cracking and the loading to deform the vessel without limit (burst). This could lead to an ultimate load factor of two for a suitably-monitored vessel with adequate control of loadings. Effective structural design demands a thorough and complete knowledge of the properties and behavior of the materials to be used under the proposed service loadings and environmental conditions as a prerequisite to making the foregoing argument compatible with the nature of prestressed concrete container structures. If the logic is accepted, it can become a basis for design rules.

### 5.2.3 Safety Margin

Since it is a concession to imperfect knowledge of operating excursions and to the shortcomings of analytical methods, safety margin is a rather hazy concept. Where a long experience with a large number of similar structural components exists, statistical correlations are possible and a practical feel for adequacy of design is built up. Nevertheless, it is disturbing to realize that failure of concrete under combined stress, particularly diagonal tensile cracks in shear, cannot be correlated exactly with any presently conceived theory of failure. In any new application under new loading conditions, the afety margin must be re-examined.

In structural concrete design, classical elastic analysis is often supplemented by ultimate load analysis as a safeguard against blind reliance on elastic calculations of inherently inelastic structures. Consequently, two independent margins of safety are frequently of interest, one concept based on the excess of some limiting stress capacity over a calculated load elastic stress intensity, the other on the excess of a loading that would produce plastic failure over an operating load.

True ultimate-load design demands knowledge of the time-dependent behavior of materials and non-linear analyses. Model tests to search the validity of such procedures are generally more complicated than the tests of material specimens and of elastic models that suffice under the regime of elastic analysis. The practices recommended in the codes discussed in Section 5.1, are tailored to the results of experimental work aimed at accounting for the elasto-plastic properties of concrete.

## 6. Materials

In general terms concrete may be defined as a conglomeration of coarse aggregate such as crushed stone and fine aggregate such as sand, held together by a cement created from the hydration of calcium or aluminum silicates. The mixed proportions of these various constituents, their size distribution, and the water content at the time of placement, all contribute to the physical properties of the solid structure. Iron reinforced concretes, or ferro-concretes as they are usually described, contain a network of steel wire, steel bars, or for prestressed concrete, steel cable, providing tensile strength in the heterogeneous body to improve the imbalance which exists between tensile and compressive strength in concretes. Conventional reinforcing is usually bonded to the concrete by the cement layer at the surface of the steel reinforcing. This is often the case in prestressed concrete, but alternatively, external anchors can provide a means of mechanically combining the steel reinforcing with the concrete which is independent of cement bonding. This last mentioned combination is of particular interest for prestressed concrete pressure vessels. The varieties of concrete and the types of steel used in reinforced concrete structures cover such a wide range that a comprehensive review of the available information would be unmanageable. However, it is worthwhile to selectively review some of the materials which are specifically pertinent to reactor vessel installations so that the character of these structures can be better understood.

At the outset it is important to recognize that for prestressed concrete, high compressive strength concrete in combination with high tensile strength steel is essential to the successful use of prestressed structures. The reasons for this have already been discussed in Section 2. For those applications already in use or under construction, a nominal concrete crushing strength of 5000 psi is used in combination with 250,000 psi ultimate strength prestressing cable to attain the necessary structural property for reactor pressure vessels. This discussion is directed toward materials which may exhibit these properties and is diverted only where specialized considerations make alternate material of interest in supplementing the high strength properties of the concrete.

## 6.1 Concrete Constituents

Considering first the concrete mixes of use in structural applications, these may easily be treated in cement and aggregate categories even though they must be considered in combination when describing properties of a concrete mixture. A comprehensive discussion of cement and concrete is covered in "The Chemistry of Cement and Concrete"[35] by F. N. Lea and C. H. Desch.

### 6.1.1 Cements

Only Portland cements and high alumina cements have major interest for concrete vessels. The first mentioned is primarily a mixture of finely ground lime (calcium oxide) and silica ($SiO_2$) with lesser quantities of alumina ($Al_2O_3$), iron oxide ($Fe_2O_3$), and magnesia ($MgO$), depending upon the material source, with the nominal proportions of 3 parts lime to one part silica. The high alumina cements are generally mixtures of equal parts lime and alumina with lesser quantities of iron oxide, silica and magnesia. Portland cements can be characterized as high strength materials requiring about 28 days to approach their ultimate strength capability. They are generally used in applications where adequate time is available for curing, high early strength is not critical, workability is important, and very high temperatures are not anticipated. The high alumina cements are of great interest where high early strength is required and/or high temperature resistance is of interest. They are frequently used in high temperature furnace applications or where rapid development of early strength is imperative. High alumina cements can only be used when careful control is maintained over the workability of the mix prior to setting.

Cement paste or mortars have often been investigated separately from concrete mixtures and have been shown capable of compressive strengths equal to those found in concretes. When used in concrete mixtures, the strength of the concrete is directly dependent upon the fraction of cement in the mix. A Portland cement concrete designed for 5000 psi compressive strength will contain about 20% cement by weight of dry mixture, the remainder being roughly equal portions of fine and coarse aggregate. The importance of the water/cement ratio in strength consideration has universal acceptance, even though the rheological behavior of the moisture in concrete is poorly understood. The effect of water/cement ratio on strength is discussed by Gilkey[36] and provides a practical review of its significance. For 5000 psi compressive strength concrete, useful in prestressed concrete systems, a water/cement ratio range of 0.4 to 0.5 is a useful working value and a fairly stiff mixture in terms of workability is implied thereby. Hughes[87] has proposed a rational basis for concrete mix design developed from practices used in the Berkeley nuclear power station, principally for high quality structural and shielding concrete . It provides excellent practical guidance to the selection of concrete mixes.

For high alumina cement refractory concretes for furnaces, the ratio of cement to aggregate is roughly the same (about 20% cement) and the water/cement ratio is once again in the range of 0.4 to 0.5 on a weight basis. Mixing practices for refractory concrete are given in the Refractory Concrete Manual[37] and the Aluminite Cement Mortar Manual[38] of the Universal Atlas Cement Division of the U. S. Steel Corporation.

The principal points of difference between the Portland cement concretes and the high alumina cement concretes are that the former because of their relatively slow setting property are more easily worked during placement, while the latter are better suited to very high temperature environments (500°F and above). It is expected that the high alumina cements would be used only where it was imperative to expose the concrete vessel to high temperature environments.

In addition to these two previously mentioned cements, the use of pozzalanic cements deserves some brief comment. These are generally high silica fines mixed with Portland cement and are often made by using furnace fly ash as the high silica constituent. They have the property of inhibiting the alkali aggregate reaction. They reduce the expansive behavior ot concrete associated with differential thermal expansion ot interest in the thermal stress evaluation of concrete systems.

6.1.2 Aggregates

The fine aggregate used in concrete mixes is almost exclusively clean river sand because of its improved workability. Recommended seive sizes are given in the ACI Standards. The coarse aggregate may be limestone, basalt, granite or comparable material whose shear and compressive strength are known to be inherently higher than cement mortars. Generally for prestressed concrete applications, shales or silty material are avoided. For best characteristics, size distribution within the aggregate should cover a fairly narrow range. A large quantity of fines will tend to reduce the strength properties of the finished concrete structure. A large quantity of coarse aggregate will reduce the workability of the concrete. A range of aggregate sizes whose maximum is approximately 1 1/2 in. and in which 70% is not smaller than 1/2 in. is a typical size distribution for high grade reinforced concrete. A discussion of aggregate selection is given in the "Design and Placing of High Quality Concrete" by D. A. Stewart.[39] A practical discussion of mixing practice is also covered.

Aggregates for refractory concretes are a specialized field. Perhaps the only one of interest in prestressed concrete applications would be crushed firebrick. Typical aggregate distribution is given in the Refractory Concrete Manual[37] with a range of 12-15% fines recommended for optimum workability and maximum strength.

#### 6.1.3 Expansive Agents

The addition of calcium sulfoaluminate to serve as an expansive agent will permit the development of compressive stresses in ferro-concretes without mechanical loading of the prestressing steel. Experimental investigations[86] of these chemically prestressed concretes show that there is some loss of concrete compressive strength through the addition of the expansive agent. Nevertheless the technique has value[40] in some structural applications and Lin[79] has verified that this type of concrete is amenable to structural analysis.

Concretes of this type exhibit high water content which could have value in neutron shielding but are likely to be sensitive to elevated temperature. The place of chemical prestressing in PCPV installations is not easily identified. It can be of use only where concrete is restrained until it has attained strength. Possibly it could prove effective as supplemental prestressing in the outer perimeter of a vessel to compensate for tensile stresses resulting from temperature gradients.

### 6.2 Concrete Material Properties

#### 6.2.1 Strength

Portland cement concrete is a dependable and competitive structural material with a broad range of applications; nevertheless, the phenomena that enable it to withstand loading under various environmental exposures over a long time with acceptable deflections are by no means well understood. The complex structure of concrete does not fit into the simple mechanisms or theories of strength that have been proposed in order to make the designer's problem more straightforward. Concrete is not one material, but a sort of random-rubble masonry, a heterogeneous mass of graded coarse and fine aggregates cemented together by hardened cement paste. Usually the aggregate particles are stronger and considerably stiffer than the paste, but unlike many two-phase materials, the bond between the cementitious paste matrix and the faces of the grains of aggregate tends to be weaker than the hardened paste itself.

The paste is subject to large volume changes that result from movements of the gel and capillary water therein. Often the shrinkage stresses that result from these internally generated strains build up faster than they can be relieved by creep and generate microcracks at the bond interfaces in concrete that has never been loaded. Curing is but one of the many job operations that must be carefully carried out in order to attain the performance that the designer of the structure has predicted. Further, water migration in the paste is influenced by humidity, ventilation and temperature of the environment and by externally-applied loading throughout the working life of the structure. The resulting intensification of the strain imcompatibilities of the paste with the aggregate particles and of the concrete with reinforcement steel and prestressing tendons may materially affect the strength of the structure.

Not only the strength and stiffness of the aggregate, but also its volume ratio, maximum size and grading, shape, specific surface, surface roughness, and chemical nature play interrelated roles of varying consequence in the structural resistance of concrete. Reinforcing steel offers a sort of filamentary super-stiff aggregate that properly directed, spaced, and proportioned gives the designer a means to offset the weakness of concrete in tension, and hence improve its resistance to bending and shear loads. A high statistical level of confidence in the integrity of reinforced concrete structures results from the conservative assumptions used in conventional design practice, even though the true mechanisms by which the resistance or strength of the composite structure is generated under complex loadings are not completely understood.

Although tensile reinforcing steel in economical amounts will be strained sufficiently to allow concrete cracking under load, it assures a closely-spaced pattern of narrow cracks that is often not objectionable. On the other hand, the addition of yet another structural component to the complex, namely prestressing tendons, affords the practical attainment of a concrete structure essentially free of loading cracks. For conceptual clarity prestress may be regarded simply as an advantageous field of compressive stress applied to the concrete mass by the tendons before subjecting the structure to service loading. The intent is to bias regions of potential tensile stress toward lower intensities or even completely into compression under maximum working load. In practice, it is found that economical prestressing requires higher quality concrete and more care in design and execution of the job than required for comparable reinforced concrete construction. Even so, new questions arise as to safe yet economical design limits that cannot yet be fully answered. The prolonged heating under stress of a prestressed reactor vessel poses the question of how to predict deterioration of materials and changes in dimensions with consequent loss of prestress as a function of the temperature and loading history.

6.2.1.1 Compressive Strength

In a practical sense, the designer is enabled to cope with the internal structural complexity of concrete mainly through the well-established techniques of the cylinder strength test. By this means the mix design using the available aggregates may be checked. Cylinders made from the job mix as placement proceeds are cured and stored in a manner to simulate reasonably the environment of the work itself. Specimens to indicate compressive strength of the structure at any chosen time after placing are thus acquired. Provided that the diameter is large compared to the largest piece of aggregate to minimize (1) the influence of the wall of the cylinder mold on the consolidation of nearby aggregate, and (2) the length is about two diameters to avoid the frictional restraint of the machine platens in the middle region, the test cylinder approaches statistically an element of volume in the work itself. The ultimate stress from the cylinder test, $f'_{cs}$ (usually at 28 days) has become the principal criterio of strength in domestic codes. The cube test remains in general use in Europe as the counterpart of the cylinder test. Because the lateral frictional restraints at the ends act jointly to strengthen an area of the middle cross section, a given concrete will show a cube strength that is in the neighborhood of 1.25 times the cylinder strength.

Although the loading of the test cylinder is unidirectional and compressive, no such limitations apply to the resultant internal stresses in the concrete. The relatively-rigid aggregate particles of various sizes and shapes are urged together by the deflections of the paste and slip of the bond. Because of the interlocking of aggregate particles tapering in opposite directions along the load axis, shearing stresses are induced and hoop tensions increasing toward the surface are built up in the cylinder. The stress distribution in a loaded test cylinder is extremely complex. This internal structural complexity of concrete may be responsible for the present lack of satisfaction among concrete designers with classical elasticity and plasticity approaches that have been rather successful in the field of design using more homogeneous materials.

6.2.1.2 Tensile Strength

Even though prestressed concrete is designed on the assumption of purely compressive loading, the tensile strength of the concrete is still important. Tensile strength is required to withstand shearing loads. In a thick-walled reactor vessel the tensile strength of the concrete in hoop tension is not negligible. Moreover, if tensile cracking is to be avoided, knowledge of the tensile strength is needed. Because of the problems posed by test grips, moment-free universal joints, and the random and rapidly-varying location relative to the axis of the test machine of the strain centroid, direct tests of tensile strength have been generally avoided by recourse to modulus of rupture (in flexure).

In a recent series of tests[41] the beam test was compared with the Hondros split-cylinder test and used to investigate the effects of previous storage under compression, the rate of loading, and temperature at time of test (between room temperature and -4°F), on the tensile strength of a standard concrete mix of 1/2 inch maximum aggregate. The equation $f_t' = A + 54 \log_{10} R$

A = constant for the concrete under test    R = loading rate in psi/min

was found to be valid for R between 1 and 10,000 psi per min. Tensile strength increases over room temperature values at zero °F. The split-cylinder test was easier to carry out and gave more consistent results. For the mix used the split-cylinder tests compared with the flexure tests as follows:

| Type of Test | Split-Cylinder | Modulus of Rupture |
|---|---|---|
| Average tensile stress at failure, psi | 455 | 690 |
| Standard deviation, psi | 30 | 58 |
| Standard deviation, percent | 6.6 | 8.4 |
| Fraction of $f_c'$ | 0.085 | 0.13 |

The ratio of tensile strength as determined by the split-cylinder test to the modulus of rupture was 0.66.

Winter[42] points out that tensile strength appears likely to be a separate property of any given concrete that cannot be uniquely defined by some specific relation to the cylinder strength; yet, there is a dependence between compressive and tesnile resistance. Intuitively the lateral bursting failure frequently observed in the cylinder test, coupled with a large increase in compressive strength conferred by a small amount of circumferential wire reinforcement on a test cylinder suggests that the compression test may be strongly influenced by the tensile resistance of the concrete. Values of $f_t'$ varying from 0.03 $f_c'$ to 0.10 $f_c'$ are reported, with rupture modulus as high as 0.15 $f_c'$.

Ultimate splitting stress test results are compared with ultimate flexural stress indications in Fig. 6.2.1-1(a) and with direct tensile stress tests in Fig. 6.2.1-1(b), both from reference 50. The ultimate strength averaged 28 percent lower by splitting than by flexure and 36 percent lower by direct tension. All stresses were calculated by elastic theory.

6.2.1.3 Stress vs Strain

By recourse to extremely stiff testing machines[43] with provision for maintaining equal strain at four stations 90 degrees apart while moving the specimen laterally with respect to the machine axis[46] it has been possible to

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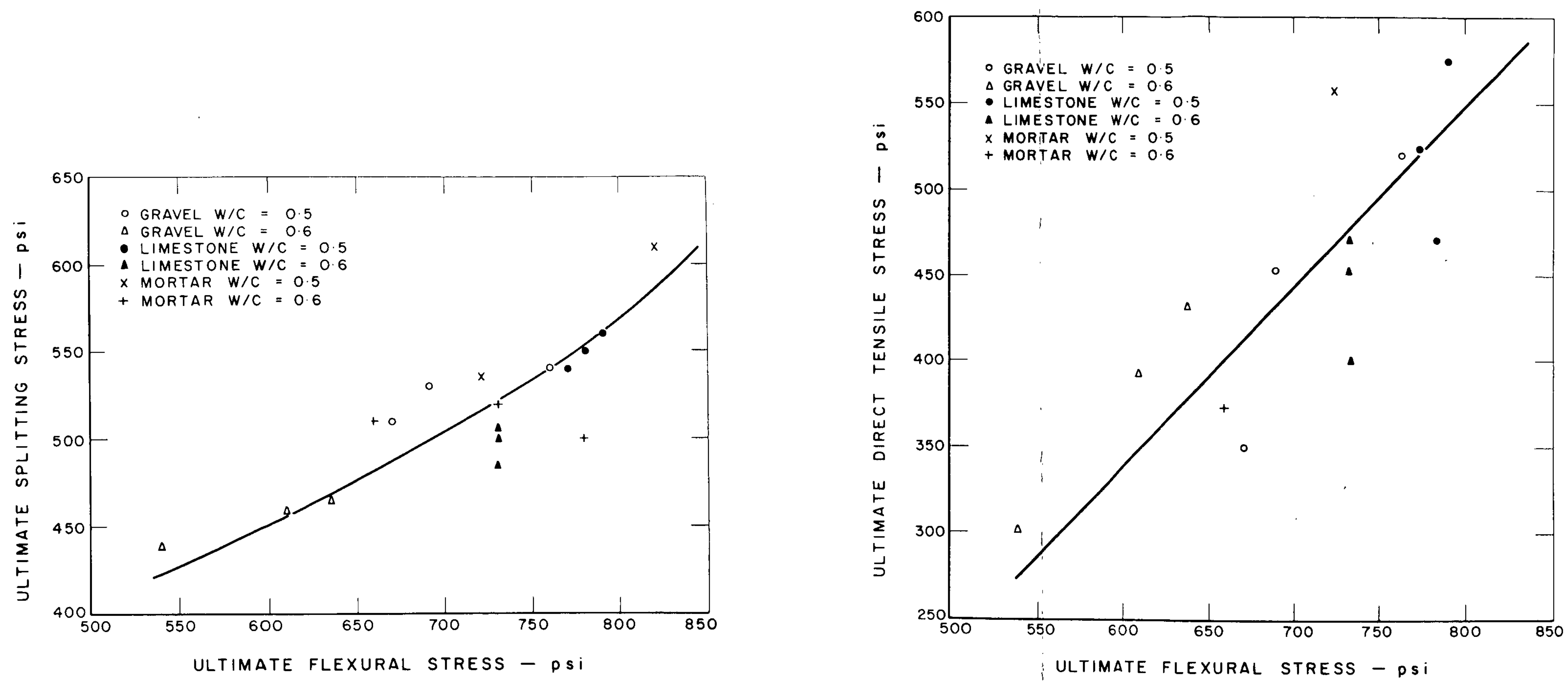


(a) Comparison of tensile stresses at ultimate failure in the flexure and splitting tests.

(b) Comparison of tensile stresses at ultimate failure in splitting and direct tension tests.

Fig. 6.2.1-1. Ultimate Strength Tests for Concrete (Ref. 50).

record the descending branch of the stress-strain curve for compressive specimens, Fig. 6.2.1-2. It is well worth noting that the ductility of concrete decreases as the ultimate strength increases somewhat analogous to carbon steel quenched and drawn at progressively lower temperatures; a question is thus raised as to the true value of extremely high $f_c'$ in structural design. Hsu et al.[26] and Kaplan[30] have proven that microcracking progresses rapidly near the ultimate strength; presumably the descending branch results as the ligaments able to carry load became narrower, fewer, and perhaps more devious.

The stress vs strain relation for concrete is the key to the stress distribution at a section that experiences a space gradient of strain, as in flexure. The otherwise statically-indeterminate stress distribution (excepting in regions of gross cracking or shear-flow disturbances near reinforcing steel or larger pieces of aggregate or other departures from the plane of the unloaded section) closely approaches the stress vs strain curve as established by a carefully conducted series of compressive tests on cylindrical or prismatic samples of comparable history.[45]

To achieve rational procedures for ultimate strength design, the stress-strain relationships under sustained load for concrete of considerable age, and at various loading rates must be established. Rusch[46] reviews a number of the research programs toward this end. It is necessary that strain, not load, be applied at a constant rate for useful results in research. Using automatically programmed machines to apply a uniform strain to the midsection of the specimen at a uniform rate, Fig. 6.2.1-3 was obtained for concretes of 3000 psi average strength and loaded 56 days after casting. The deformations are not purely elastoplastic; the slower the rate of loading, the greater the creep and shrinkage. While other variables such as type and content of cement, grading and stiffness of aggregates, temperature and moisture will influence results, strength and time will predominate to produce similar relationships for other concretes.

The loading of actual structures takes place in a less favorable way since the load may be applied rather quickly and then held relatively constant. Fig. 6.2.1-4 shows the influence of the degree of sustained loading ($f_c/f_c'$ at 56 days) on the deformation and time elapsed up to failure in the Munich test equipment.[46] The failure in a conventional short-time test occurred in abcut 20 min. at an ultimate strain of about 0.0025 in./in. For sustained loads of degree less than unity there are two outcomes. If the degree of loading exceeds the ultimate strength for sustained loading, strain eventually increases rapidly and fracture follows; for loading below the sustained strength, deformation approaches a limiting value of stabilized strain.

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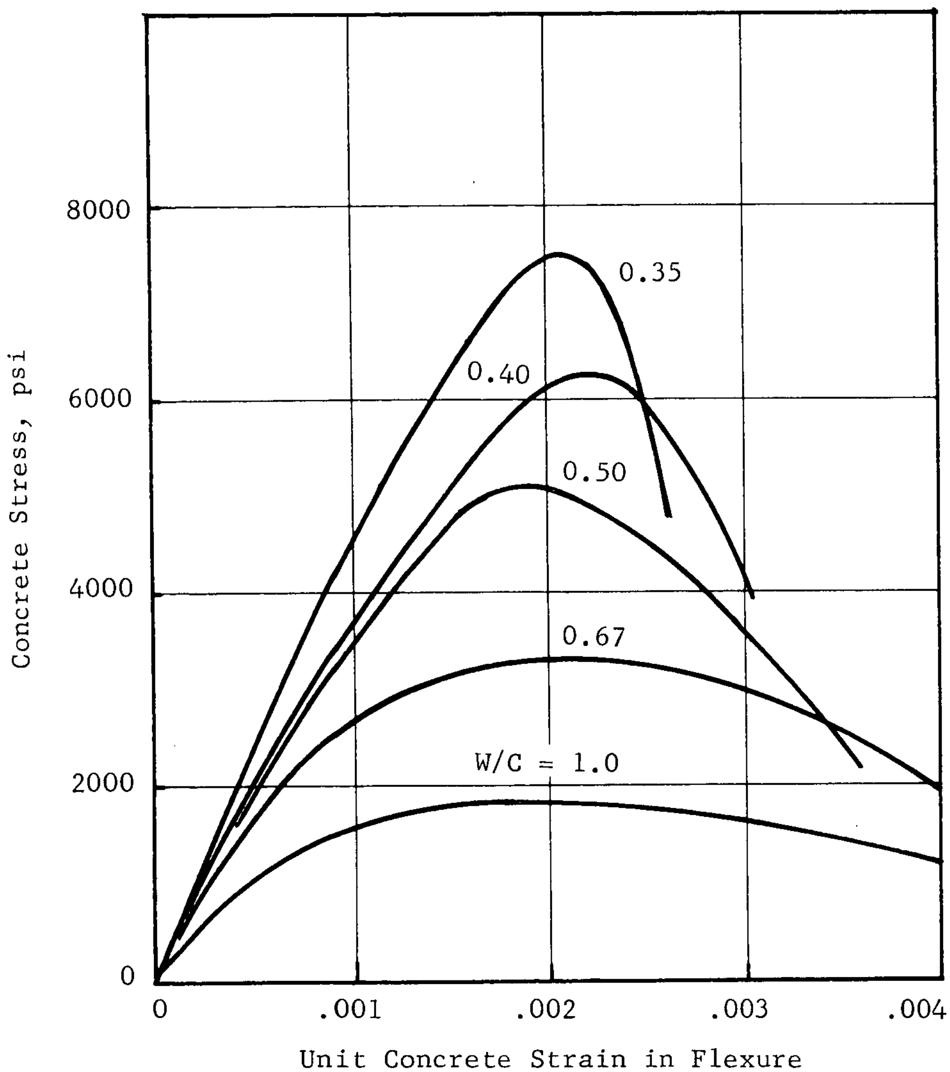


Fig. 6.2.1-2. Stress-Strain Curves for Concrete - Age 28 Days. (Ref. 43).

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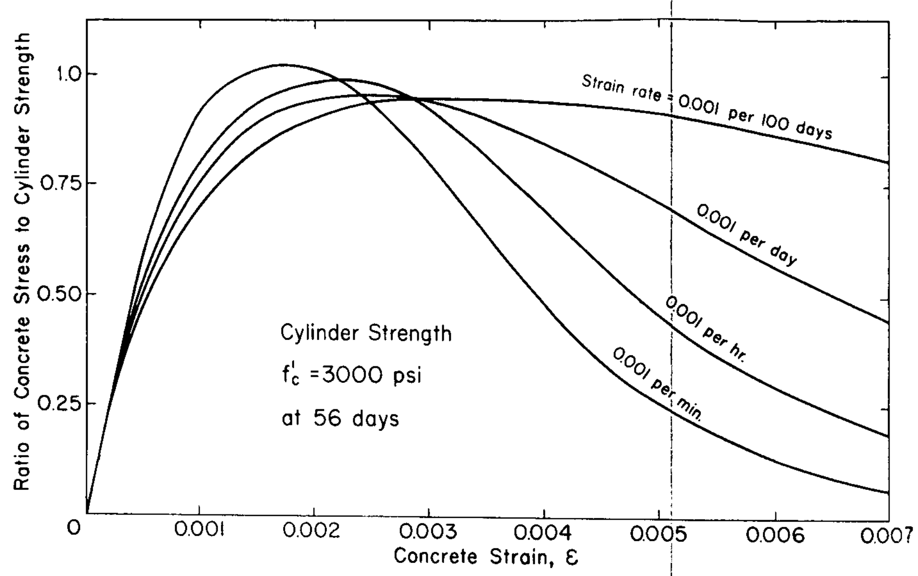


Fig. 6.2.1-3. Stress-Strain Curves for Various Strain Rates of Concentric Loading (Ref. 55).

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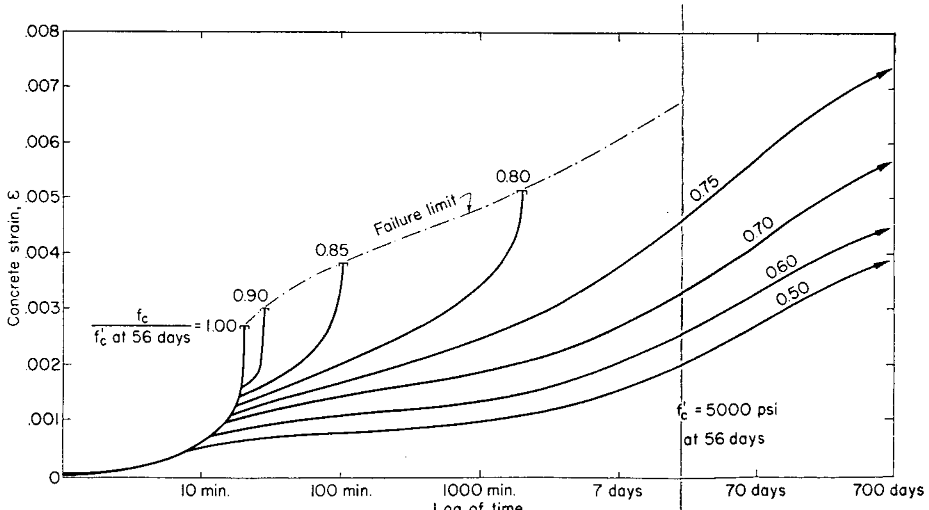


Fig. 6.2.1-4. Strains Under Sustained Load Applied at Concrete Age 56 Days (Concentric Loading of Prisms at 56 Days). (Ref. 55)

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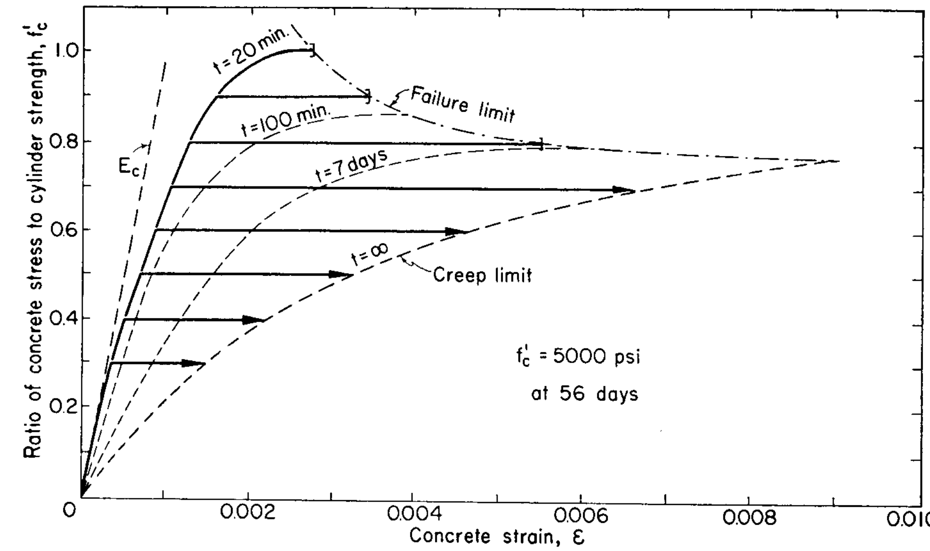


Fig. 6.2.1-5. Influence of Load Intensity and Duration on Concrete Strain (Ref. 55).

These data are replotted in Fig. 6.2.1-5 as loading intensity against strain with time as a parameter. The fracture and stabilized creep envelopes indicated, bracket all possible relations of stress and strain for this concrete and environment.

The Munich tests have shown that sustained load strength of a concentrically loaded concrete specimen amounts to at least 75 percent and averages 80 percent of the short-time test on a twin specimen cast at the same time and tested soon after the collapse.

Kaplan points out[30] that microcracking overrides creep as a source of inelastic behavior so that stress-rupture tests above the loads causing intense microcracking, near the sustained load limit, cannot be correlated with creep tests at lower stresses where plastic flow of the paste without cracking is the dominant mechanism of deformation. There is a rough parallel between microcracking of concrete under high sustained load and the voids that form at crystal joints during third-stage creep of metals, at least in the marked increase in rate of strain associated with this phase.

Kaplan cites the results of many workers to show that deviation from linearity in a short-time test marks the onset of cracking. He presents results for a total of 272 specimens for four types of tests in 17 mixes of concrete and mortar at cracking and ultimate stress. Loading was at 1060 psi per minute in compression and 150 psi per minute in direct tension. Flexure and splitting with stress was reported at onset of cracking and at 95 or 100 percent ultimate load. Ductility at 95 percent ultimate load is nearly independent of the type of coarse aggregate and of water ratio for the mixes used, but falls by about one third as the fraction of coarse aggregate rises from zero to 50 percent, as seen in Fig. 6.2.1-6. Ultimate strength, however, was dependent on type of aggregate and water ratio, Fig. 6.2.1-7, and much more sensitive to volume of coarse gravel aggregate than it was in the case of limestone. Microcracking occurred considerable below the ultimate failure loads. Tensile stresses and strains at onset of cracking decrease as volume of coarse aggregate in the mix is increased. This may be caused by internal stress and strain concentrations caused by the restraint of the coarse aggregate inclusions.

6.2.2 Reinforced Concrete Structural Behavior

Examination of Fig. 6.2.1-5 indicates a domain between the design stress limit of 0.45 $f_c'$ and the critical stress (near 0.78 $f_c'$ in this instance) having a combined elastic and creep strain under sustained load greater than the short-time ultimate strain. If concrete in this domain could be proven by research to be free of immoderate cracking and otherwise undamaged (as the Munich tests already indicate), it would serve to reassure designers that the new high-strength steels could indeed be used efficiently in columns and

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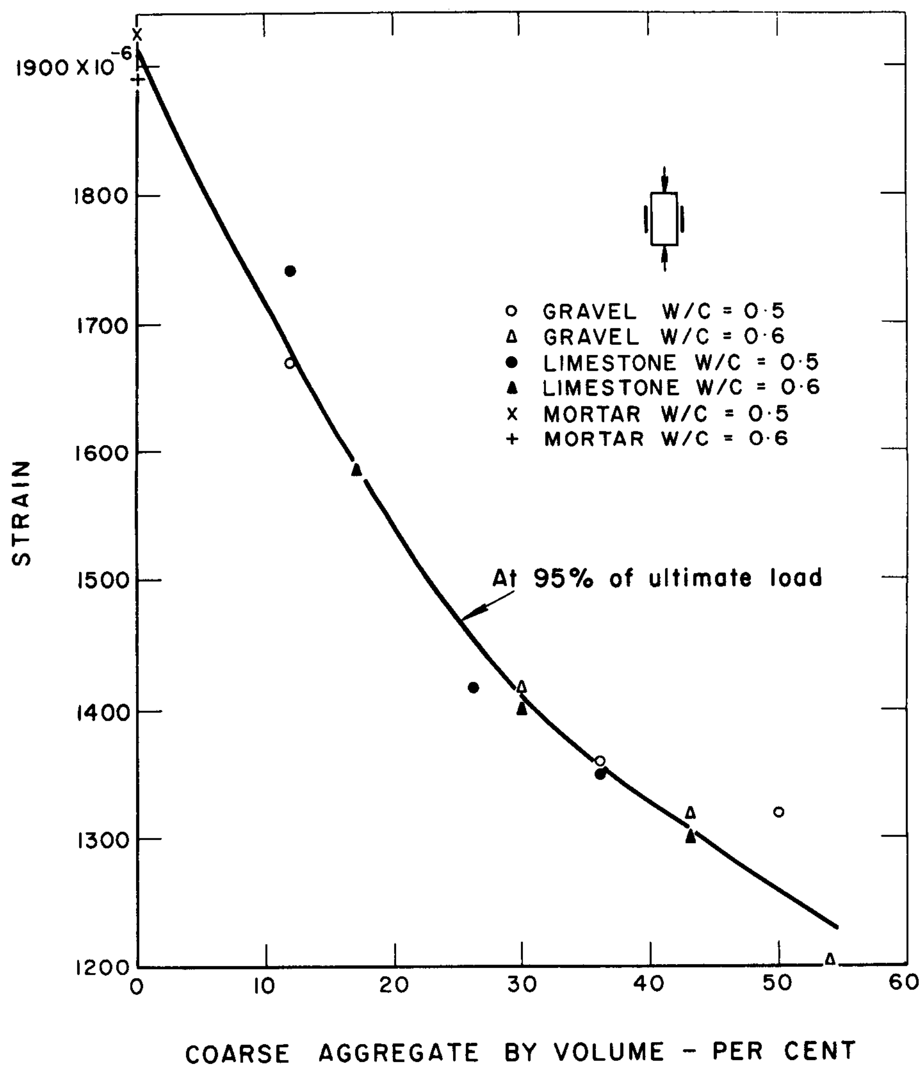


Fig. 6.2.1-6. Relation Between Compressive Strain at 95 Percent of Ultimate Load in Compression Test and Percentage of Coarse Aggregate by Volume. (Ref. 30)

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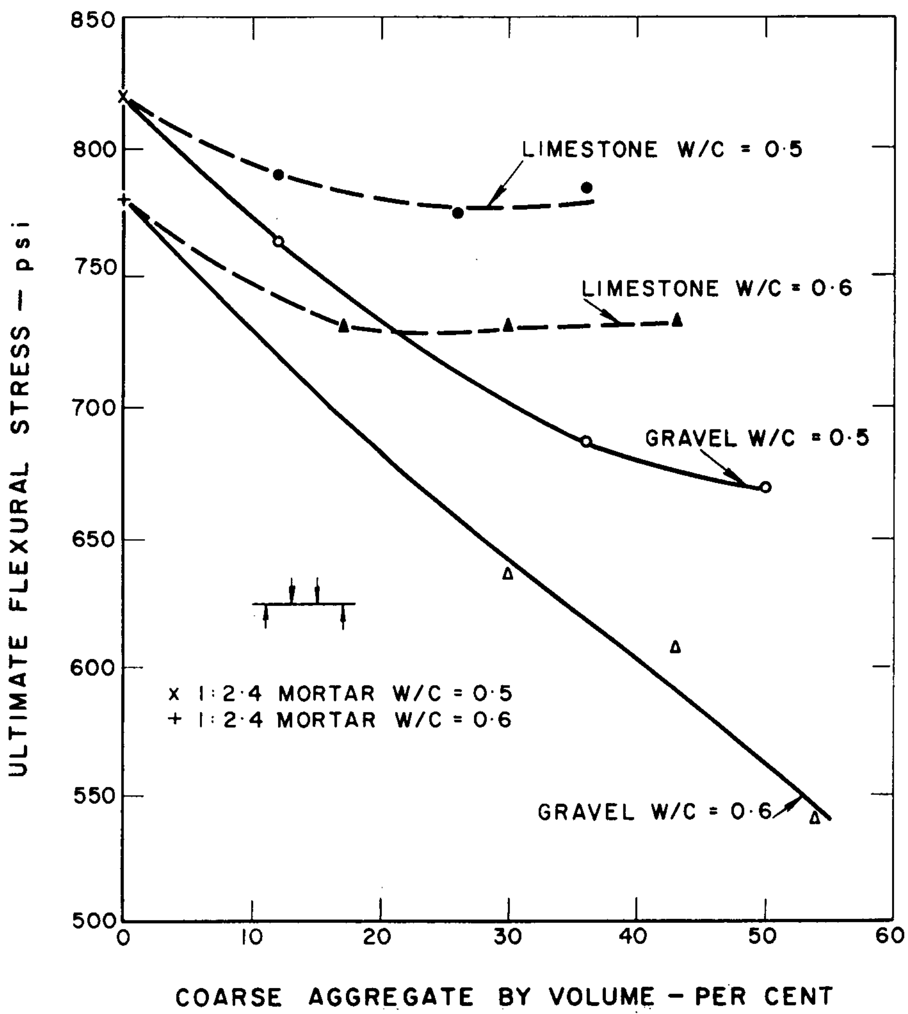


Fig. 6.2.1-7. Relation Between Flexural Stress at Ultimate Failure and Percentage of Coarse Aggregate by Volume. (Ref. 30)

compressive-side flexural concrete. Under gradually applied overloads to strains of 0.0025 or 0.003 (stress in steel 75,000 to 90,000 psi), the concrete would remain sound and continue to carry at least its design load.

References[43,46,47,48] review earlier analytical approaches for predicting the plastic stress distribution at the ultimate load of real structures as an introduction to tests for establishing structural properties and checking the validity of theoretical work. There appears very real hope of considerable improvement in the understanding of the lifetime behavior of reinforced and prestressed structures, provided that the gaps in the knowledge of the time-dependent and temperature-dependent properties of concrete are diligently investigated.

6.2.3 Inelastic Behavior

6.2.3.1 Shrinkage

According to Ross[49] three kinds of movement (other than the large plastic strains recorded near the ultimate stress) are recognized in concrete: (1) drying shrinkage, independent of stress, (2) elastic strain on loading, and (3) creep, defined as the difference between time-dependent movements of stressed and unstressed elements otherwise identical. Although it is customary to speak of "drying" shrinkage, cured concrete that is sealed by a vaporproof membrane from its environment will exhibit an autogeneous linear shrinkage strain of something like $40(10)^{-6}$ after one month increasing to $100(10)^{-6}$ after five years.[50] Autogeneous shrinkage would be experienced by a thick monolithic concrete reactor vessel wall. It is customary to include autogeneous hydration shrinkage with the larger movement caused by actual escape of water in the term "drying shrinkage". Throughout this review "volume changes" will be specified in linear change per unit length, a pure number.

Shrinkage of hardened concrete is of the same order as elastic and creep strains, so that it is a major problem in designing prestressed concrete During the setting process it must also be taken into consideration. The choosing of aggregate and the planning of formwork and placing schedules, particularly if a steel liner is to be used, affect the end results.

Hydration of cement results in a reduction in volume of the system of cement plus water.[51] This contraction is rapid and largely completed while the concrete is still plastic. Plastic shrinkage is aggravated by loss of water by vibration, evaporation, bleeding, or suction. Cracking may result if obstructions interfere with uniform shrinkage. Such restraints may be created by reinforcements or prestressing ducts, large aggregate particles, unfavorable form geometries or simply resistance to flow offered by an older substrate of concrete. Cement-rich mixes and early setting promote greater early shrinkage and set the stage for more extensive green cracking.

Powers[52] has presented an informative elementary discussion of the physical phenomena that appear to be involved in drying shrinkage, that is, strains caused by ambient conditions of humidity as they influence water movement within cured concrete. Concrete is composed of aggregate (rock particles) separated but bonded by cement paste. Cement paste is made up of any anhydrous particles of cement clinker remaining, the mass of cement gel formed by hydration that includes at least 28 percent porosity, and the capillary cavities within the boundaries of the gel. Gross voids such as air bubbles are not considered part of the paste.

In concretes composed of ordinarily-used aggregates, volume changes are confined mainly to the cement paste. The electron microscope shows that the colloidal gel is a mass of ribbons and crumpled foils. Gas-absorption techniques indicate that these foils are not less than two nor more than three molecular layers in thickness.[53] The interstitial spaces are called gel pores. A cubic inch of gel would average about $(10)^{18}$ particles of gel and something like 35 percent of gel pores.

Each unit volume of the original clinker makes about two volumes of gel, replacing the cement and part of the original water space. The lower the w/c ratio, the less capillary space in proportion to gel in the mature paste. Powers speculates that the average width of a gel pore is 3 or 4 diameters of a water molecule so that van der Wahl forces between gel particles are weakened by adsorbed water on the gel surfaces. He attributes drying shrinkage to withdrawal of adsorbed water, the primary factor being a drawing together of gel particles under van der Wahl forces, and a secondary effect the interfacial shrinkage or crumpling under the increase of the particles' own surface tension. Valence forces would come into play only during actual rupture of the gel in compression and this accounts for its weakness in tension.[53]

In gel so dense that it is without capillaries, the pore water at saturation is 26 percent of the volume. If all this water is removed, the paste shrinks about 3 percent in volume or 1 percent linear shrinkage. Removal of molecular layers of water is able to bring this about because there are $(10)^{18}$ foil-shaped gel particles per cubic inch. Water in the capillary cavities being beyond the range of molecular attraction has no effect on volume. The effect has been investigated[52] in mixtures of pulverized silica and cement forming mortar specimens. The w/c ratio increased with silica content, giving rise to larger proportions of capillary cavities. The neat cement specimen shrank linearly indicating few if any capillaries. With increasing silica content the capillaries gave up water more readily to lower the rate of shrinkage with respect to drying, but at the end of the drying period the foregoing shrinkage rates were all nearly equal, because almost exclusively adsorbed water constituted the net loss from each specimen. The important point in shrinkage is not water loss, but how the water inventory is being redistributed between the active surfaces of the gel ribbons and the inert storage in capillaries, voids, and within the intergranular pores of the aggregate.

Brunauer[53] recently gave a more detailed account of work on the physiochemical interplay of lime and silicate in the presence of aluminum and magnesium in determining the specific surface area and the cementitiousness of gel. The amount of area and its quality are the main factors in the strength, creep, and shrinkage properties of concrete. He credits aluminum with counteracting the tendency of the gel foils to roll into fibers as they do in compounds of pure tricalcium and $\beta$-dicalcium silicate.

Restraint of shrinkage by aggregates

It is common experience that neat cement specimens are much more prone to drying cracks than mortar specimens. The freedom of many aggregates from significant drying shrinkage offers a powerful means to control shrinkage. Pickett[54] was able to get fair correlation between a theoretical formulation of aggregate restraint and test results. He represented an aggregate particle by a small sphere within an infinite body and by equating the elastic reduction in shrinkage of the body to the elastic compression of the sphere he eliminated the interface pressure. Denoting the volume of aggregate per unit volume of concrete by g, he expressed the rate of change of shrinkage with change of g and integrated to obtain

$$S = S_o(1-g)^{\alpha}, \text{ or } \log \frac{S_o}{S} = \alpha \log \frac{1}{1-g}$$

for plotting purposes

where $\alpha = \dfrac{3(1-\nu)}{1+\nu+2\,(1-2\nu_g)\,E/E_g}$

and $S_o$ = shrinkage if no aggregate were present

$\nu$ = Poisson's ratio

Pickett tested the validity of the relation by shrinkage tests on mortar prisms 1 x 7/8 x 11 1/4 in., prepared and cured in accordance with Table 6.2.3-1. In Fig. 6.2.3-1 the plotted results for high early strength cement are found to be well represented by the theoretical straight line for $\alpha = 1.7$ in the case of w/c = 0.35, but w/c - 0.50 shows only fair correlation. Pickett suggests that reasonable elastic constants for $\alpha = 1.7$ are $\nu = 0.2$, $\nu_g = 0.25$, and $E/E_g = 0.21$. L'Hermite[55] gives a range of $1.2 \lessapprox \alpha \lessapprox 1.7$ from experimental work on shrinkage.

In spite of the rational shortcomings of the formulation in neglecting specimen boundaries and the influence of the large areas of paste-aggregate

Table 6.2.3-1. General Outline of Conditions in Study. (Ref. 54)

| Cements used | Aggregates used | Percent aggregate by absolute volume† | $W/C$ by weight |
|---|---|---|---|
| High-early-strength | Silica flour | 0 | 0.5 |
| | | 5 | |
| | Standard Ottawa sand | 15 | 0.35 |
| Laboratory mix of four brands of Type I | | 30 | |
| | Graded Elgin sand | 50 | |
| | | 65 | |

*Specimens were sealed in steel molds ⅞ x 1 x 11¼ in., that were stood on end for 2 hr during setting of mortar. Each mold was turned end-for-end every 5 min. Curing was for 7 days under water at 78 F. Drying was for 224 days or longer at 50 percent relative humidity, 76 F and the wetting period was 84 days under water at 74 F. Two specimens were made of each combination, a total of 72.

†As will be noted later, there were slight deviations from these values.

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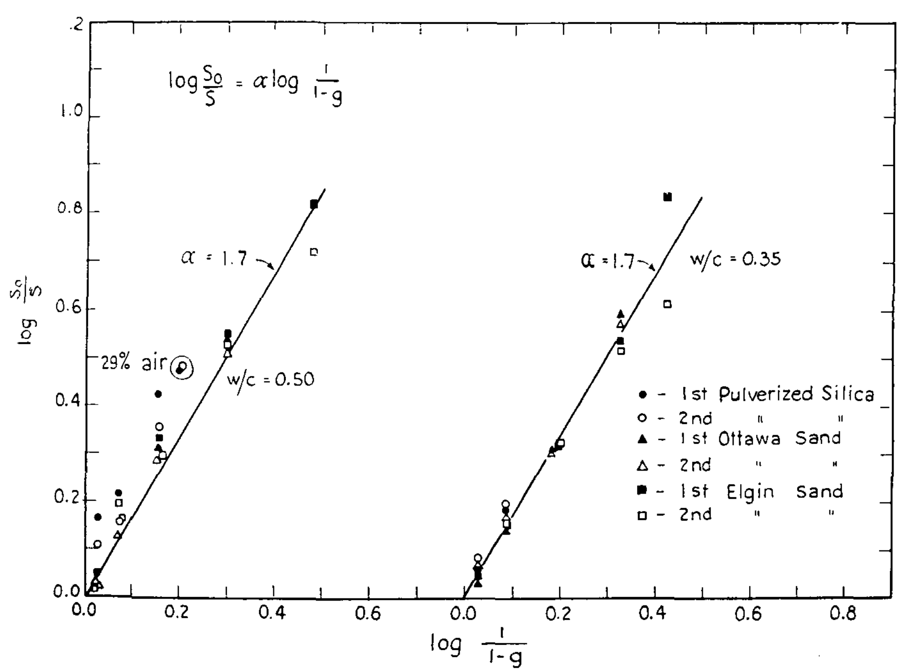


Fig. 6.2.3-1. Effect of Aggregate on Shrinkage. (Ref. 54)

interfaces on E and $\nu$ of the paste, Pickett's correlation is useful in explaining the effect of aggregate distribution on shrinkage.

In 1928 tests were started at the University of California to determine the effect of the mineralogical character of aggregates on creep and shrinkage.[56] One test series used six different types of aggregate with the same cement with a constant aggregate-cement ratio (5.67 by weight) and a constant water-cement ratio (0.59 by weight). All specimens received identical curing treatment: fog at 70°F for 28 days. Storage was in air at 70°F and 50% RH. The shrinkage of these 4 in. diameter, 15 inch long cylinders per unit length is plotted against a log time scale for 23 years in the lower nest of Fig. 6.2.3-2. The specimen identified as "gravel" contained local sand and gravel, principally graywacke and other sandstones. The instantaneous deformation for creep and shrinkage specimens over 23 years were as follows:

| Aggregate | Instantaneous Deformation at 800 psi Compression | 23 Year Shrinkage |
|---|---|---|
| | $(10)^{-6}$ | $(10)^{-6}$ |
| Sandstone | 281 | 1260 |
| Gravel | 275 | 1140 |
| Basalt | 224 | 850 |
| Limestone | 222 | 650 |
| Quartz | 212 | 550 |
| Granite | 205 | 850 |

Apparently the stiffness of the aggregate has an effect on shrinkage; the superior performance of limestone is probably a partially-chemical phenomenon. It is worth noting that even after 23 years, these small specimens were shrinking slowly, although in every case more than half of the 23 year shrinkage had taken place at age 90 days from end of cure.

The ACI Manual of Concrete Inspection (3rd Ed., p. 33)[95] gives data showing that for a given water-cement ratio, the rock content may be as much as 80% or as little as 60%, depending on the maximum size of the graded aggregate. According to Pickett's equation, 1/4 in. maximum rock size would give 3 times the shrinkage of 6 in. maximum rock size corresponding to the foregoing 0.6 and 0.8 rock contents.[52] The converse, the reduced restraint of larger aggregate if aggregate-to-cement ratio is held constant, is seen in Fig. 6.2.3-3 from reference 56. The coarser aggregate also formed a specimen that exhibited 9% less rigidity.

Clay in the aggregate pervades the paste as flocs that have a higher shrinkage than cement paste, leaving in effect voids that greatly increase concrete shrinkage. Effective washing of sand and gravel is part of shrinkage control.[52]

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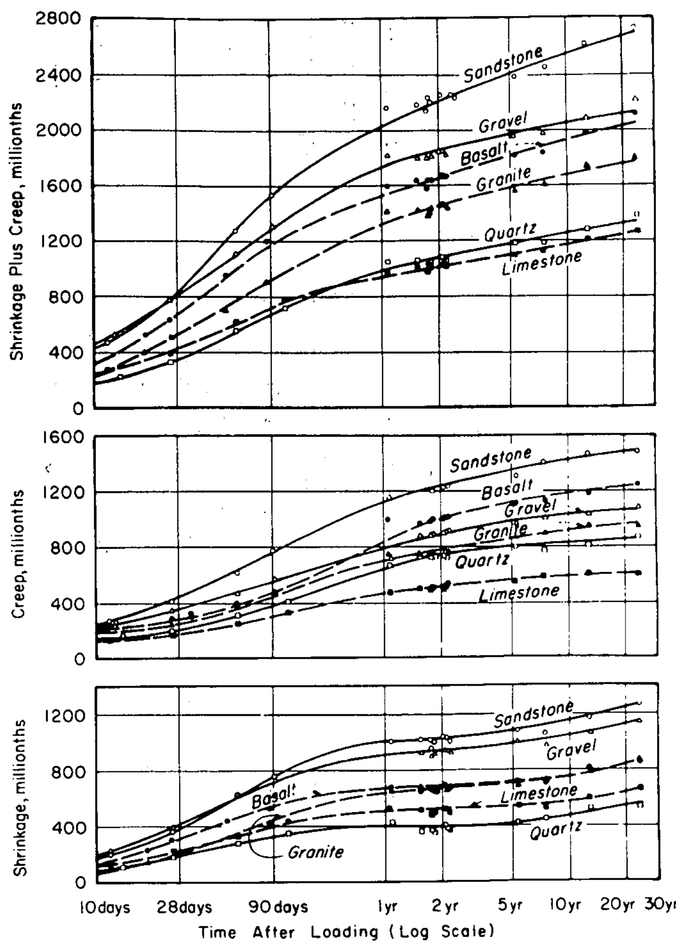


Fig. 6.2.3-2. Effect of Mineralogical Character of Aggregates Upon Creep and Shrinkage. (Ref. 56)

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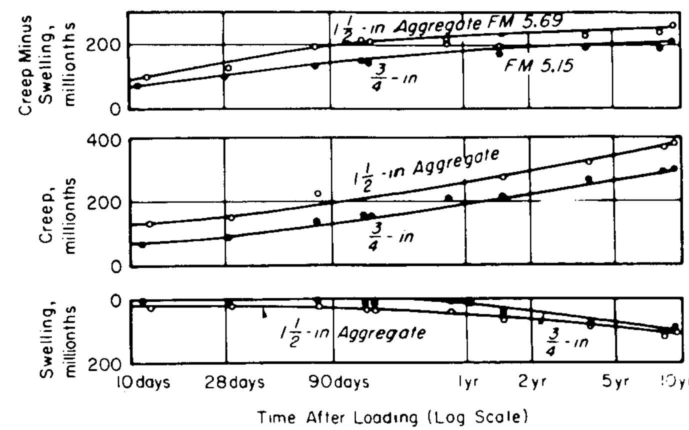


Fig. 6.2.3-3. Effect of Size of Coarse Aggregate Upon Creep and Expansion While in Moist Storage. (Ref. 56)

## Cement Characteristics versus Shrinkage

The primary cement characteristics that influence shrinkage are (1) fineness of grinding and (2) the relation of the gypsum content to the chemical composition.

Coarse clinker particles retained on the 200 mesh sieve probably never become hydrated, but act as aggregate. The Series 17 test results from reference[56] displayed in Fig. 6.2.3-4 illustrate the effect of fineness on the shrinkage of Types I and IV compositions over a period of 19 years under the conditions stated below the plots. The authors state that where drying shrinkage is not an item of importance, as in mass structures, concretes containing coarse-ground, low-heat cements have enjoyed freedom from cracking.

## Carbonation

The effect of carbonation caused by environmental $CO_2$ on shrinkage and surface cracking has been recognized only recently, so that drying-shrinkage tests in general include carbonation. Although a distinctly different action, the final carbonation shrinkage is strongly influenced by environmental humidity, as shown in Fig. 6.2.3-5. Apparently low humidity does not provide enough carbonate radicals, but on the other hand, too complete saturation with water hinders migration of carbon dioxide into the concrete. As a result carbonation peaks at about 55% relative humidity. Carbonation shrinkage cracking has been most troublesome in exposures inside buildings. Verbeck[57] notes the possibility of controlling shrinkage, increasing strength, and reducing permeability of precast products by deliberate precarbonation under controlled humidity. Powers[58] presents a hypothesis of a mechanism of carbonation shrinkage that is compatible with recent experimental findings.

## Moist Curing

Since prolonged moist curing promotes more complete hydration of cement particles, it is to be expected that subsequent drying shrinkage of the paste would increase.

Powers[52] points out that the effect of protracted curing on the shrinkage of concrete is complicated by the restraint of the aggregate. Curing not only increases the capacity of the paste for shrinkage by eliminating clinker particles, but it also increases modulus of elasticity and reduces the rate of creep at a given stress. The result is that prolonged curing makes paste more prone to crack when severely restrained. If cracking relieves stresses around aggregate particles, the overall shrinkage might thereby be diminished. Various laboratory tests on specimens of mortar and concrete indicate that prolonging the period of moist curing before beginning drying reduces the amount of shrinkage, other data show the opposite. Internal cracking may be causing the

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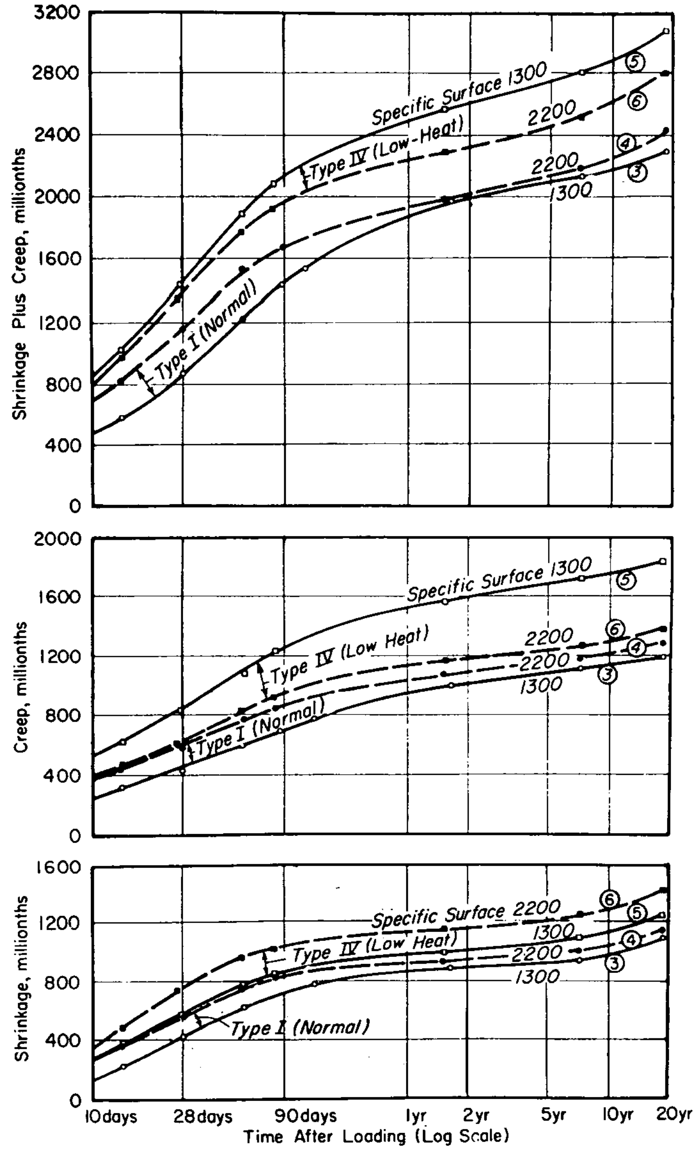


Fig. 6.2.3-4. Effect of Composition and Fineness of Cement Upon Creep and Shrinkage. (Ref. 56)

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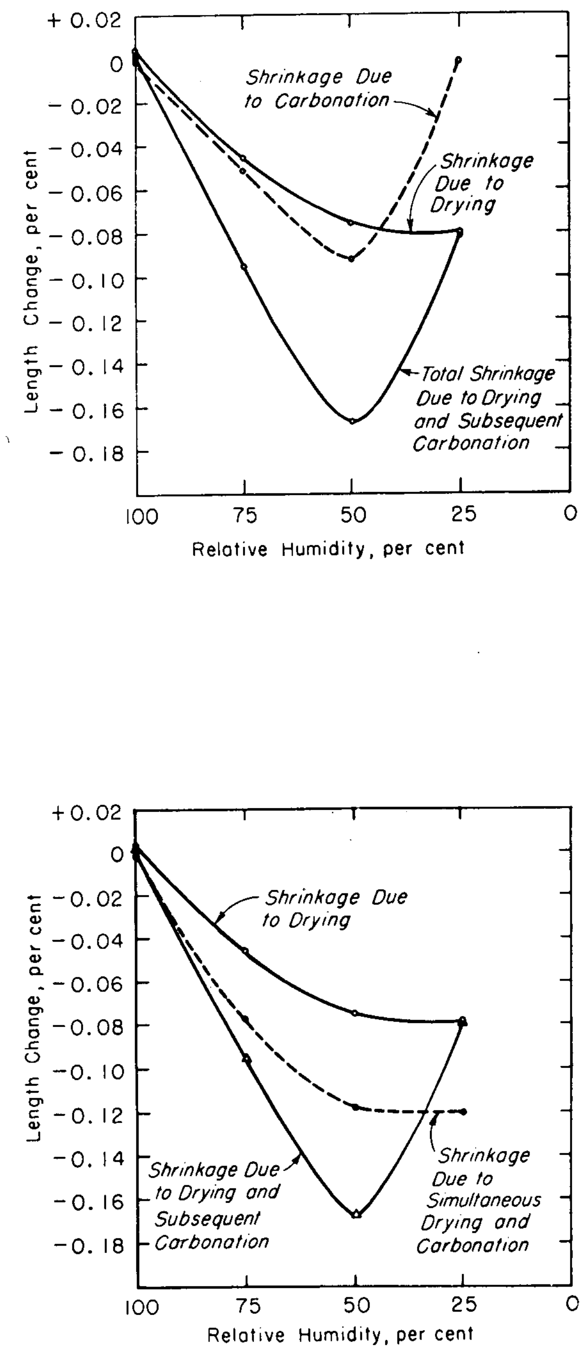


Fig. 6.2.3-5. Drying Shrinkage and Carbonation Shrinkage as Functions of Humidity; Interaction of Drying and Carbonation, Together and Tandem, as Functions of Humidity, After Verbeck. (Ref. 57)

variation. In general, length of moist curing is a relatively unimportant factor in the control of shrinkage.

Figure 6.2.3-6 from reference 56 shows that the effect on shrinkage is small if moist cure is prolonged for 90 days instead of 28 days. The particular conditions of the test were: 4 in. diam cylinders, 1 1/2 in. maximum aggregate, Type I cement, a/c ratio 5.05 by weight, w/c ratio 0.69 by weight, cured moist at 70°F, and stored in air at 70°F and 70% relative humidity. The longer cure lowered (or delayed) the shrinkage only slightly. The total shrinkage was rather small because of the 70% RH storage. There is an appreciable stiffening at 10 days of loading in creep by the 90-day cure; the creep curves for equal loading trend apart for the first year, but are much closer percentagewise at 21 years. The authors reported a very marked decrease of creep for the 90-day specimens during the period immediately after application of load.

Keeton[59] provides a comparison of 175-day shrinkage for curing ages of 8 and 24 days at 100% RH for 3 in. diam specimens. Figure 6.2.3-7 indicates the measured shrinkage vs age with length of cure and relative humidity of storage environment as parameters.

Environmental Humidity

Between a cement gel particle and the surface environment of the concrete lies a complex diffusion path of gel pores, capillary cavities, voids, and aggregate pores. Between concretes great variation in the space gradient of water vapor pressure needed to produce a given flux density of water movement would be expected. Naturally temperature and space variation of temperature would have a profound influence on the available driving gradient. The past history of humidity exposure and temperature with time over a span of years influences the distribution of inert and adsorbed water in massive concrete and in turn affects shrinkage.

The effect of relative humidities of storage of 50, 70, and 100% on shrinkage of 4 in. cylinders of concrete are shown in the lower plot of Fig. 6.2.3-8 over periods of 19 or 22 years.[56] The specimens are described on the figure. The shrinkage of the Naval Civil Engineering Laboratory specimens is plotted in Fig. 6.2.3-9 over 175 days from end of 8-day cure for storage at relative humidities of 20 and 50% at 73.4°F for 3, 4, and 6 in. diam cylinders (L = 3D).

Thickness of Section

Figure 6.2.3-9 shows that for the conditions tested the shrinkages were directly proportional to the cylinder diameters at about 20 days with 50 RH, but at 175 days the shrinkage of the 3 in. diam cylinder was only something like 25% more than that of the 6 in. diam cylinder.

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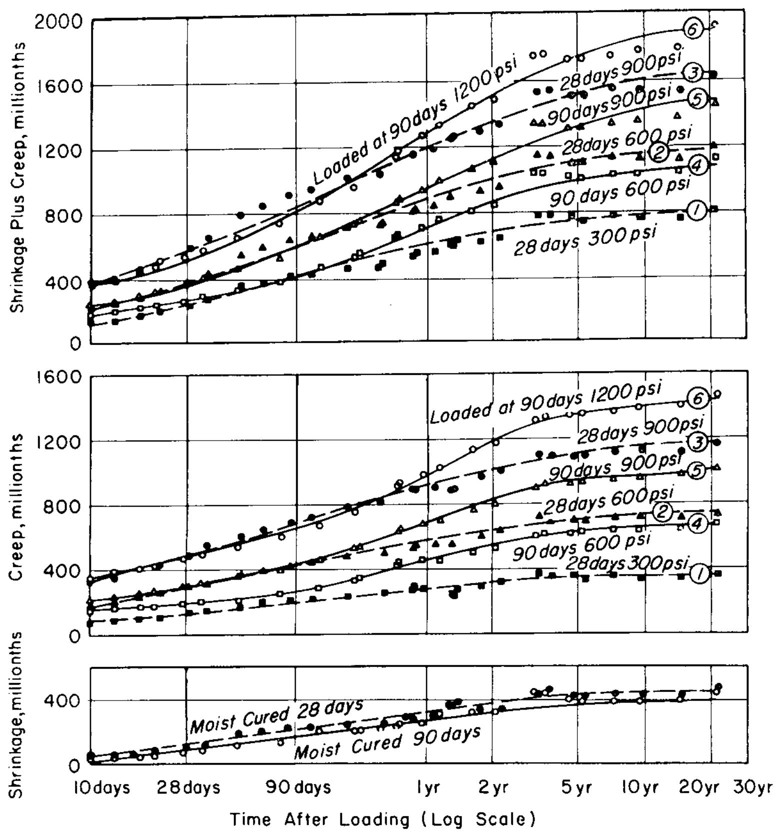


Fig. 6.2.3-6. Effect of Age and Intensity of Loading Upon Creep and Shrinkage. (Ref. 56)

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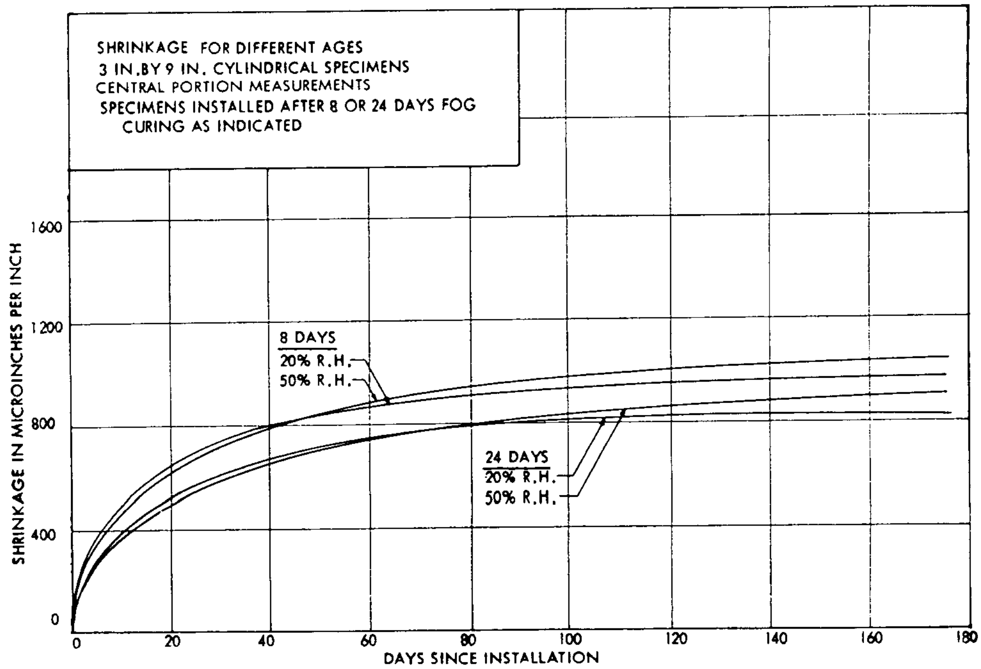


**Figure 29. Shrinkage of 3- by 9-in. specimens, central portion measurements.**

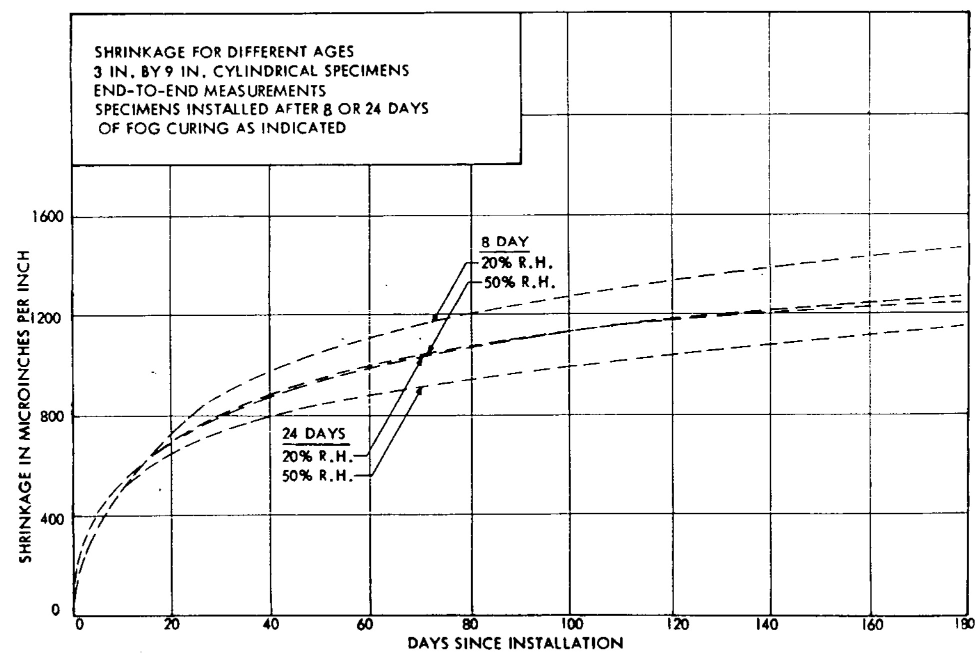


Fig. 6.2.3-7. Shrinkage of 3- by 9-in. Specimens, End-to-End Measurements and Central Portion Measurements.

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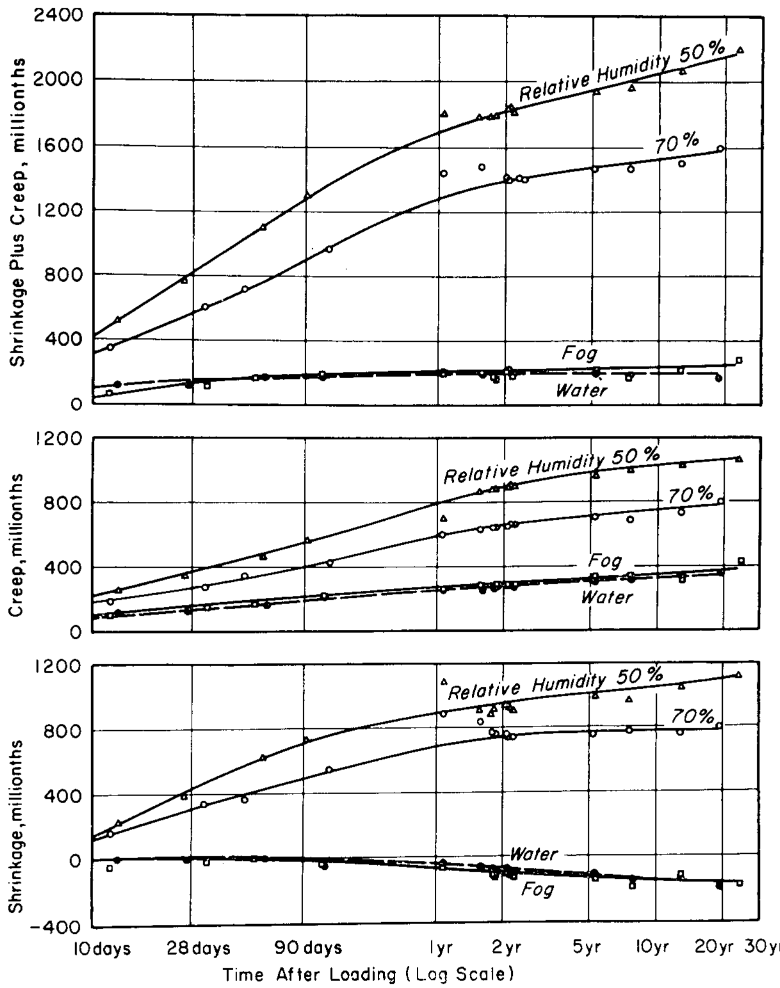


Fig. 6.2.3-8. Effect of Moisture Condition of Storage Upon Creep and Shrinkage. (Ref. 56)

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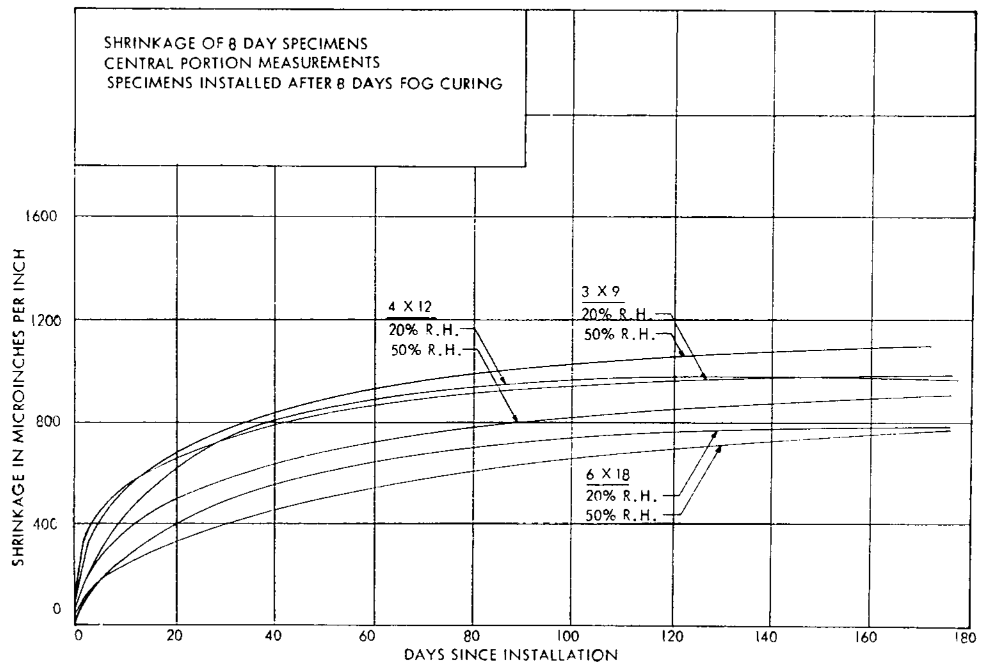


**Figure 27. Shrinkage of 8-day specimens, central portion measurements.**

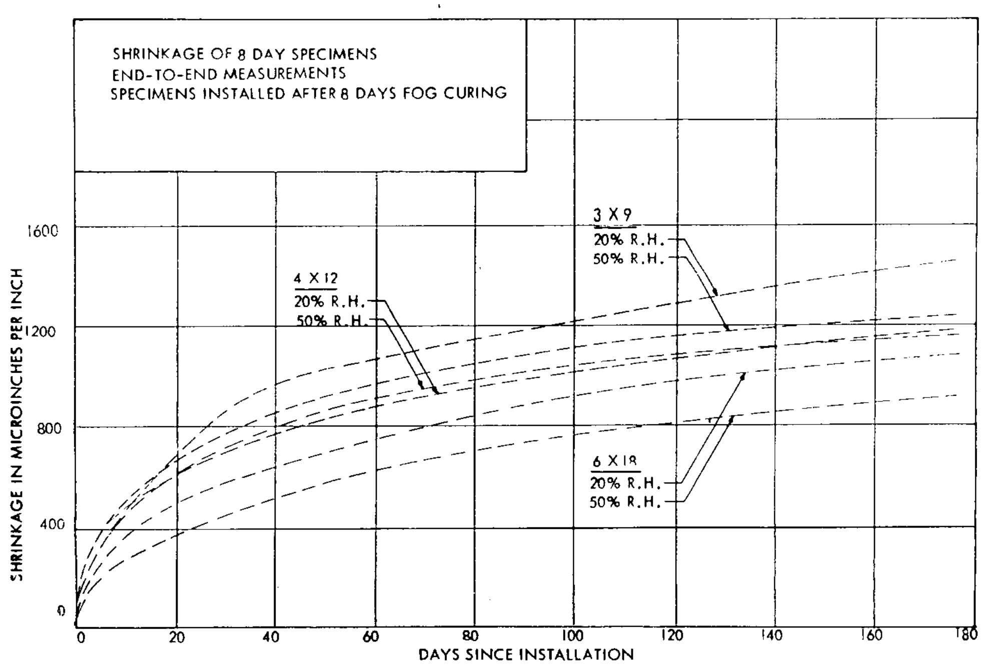


Fig. 6.2.3-9. Shrinkage of 8-Day Specimens, Central Portion Measurements and End-to-End Measurements. (Ref. 59)

## Ambient Temperature

Heating experiments on young concrete are complicated by the restraint on shrinkage introduced by the more rapid hydration at elevated temperatures. In reference 44 tests of shrinkage at elevated temperatures are reported. Cylinders 4.5 in. diam and 12 in. long were cast from a typical 1:2:4 concrete mix, w/c ratio 0.45 by weight. The cylinders were demolded at age one day and placed under water for three more days, then stored at 17°C in 90 RH air until 10 days old. Figure 6.2.3-10 shows shrinkage against time for 62 days at 20, 50, 80 and 100°C. The relative humidity was not reported.

## High Pressure Steam Curing

Subjecting cement paste to saturated steam of 350-450°F results in a non-colloidal paste structure composed of relatively few particles per unit volume.[52] Specimens so treated show little shrinkage or swelling with change in moisture content. Strength is severely reduced. Menzel[60] showed that blending pulverized silica with cement gives a steam-cured product having normal strength only half the shrinkage of normally-cured concrete without silica flour. An autoclave of 135 to 425 psig working pressure is of course required.

## Control of Shrinkage

Rule-of-thumb factors for estimating the importance of various measures for controlling shrinkage are shown in Table 6.2.3-2 taken from reference 57, showing the ratio of departure from minimum shrinkage for a given structural component caused by each poor choice (from shrinkage standpoint) and their cumulative effect.

Insofar as thickness prolongs drying so that the extension stress induced in the dryer surface layers has more time to relax in the earlier life of a structure, the integrated linear change per unit length caused by "shrinkage" may be materially reduced by creep in thicker sections, even at great age, after deformation attributable to drying shrinkage becomes small. The experimental evidence indicates that long-time shrinkage is less as section thickens;[56] possibly relaxation may be helping other factors to produce the apparent size effect.

### 6.2.3.2 Creep

Although creep is helpful to the designer in relaxing the stresses set up by the stress raisers, particularly those caused by differential shrinkage, it may lead to the difficulties of progressive loss of prestress; cracking at structural discontinuities or junction of members with poorly maoched creep deformations; plastic deformation of reinforcing steel; rupture of bond; and buckling or rupture of liner.

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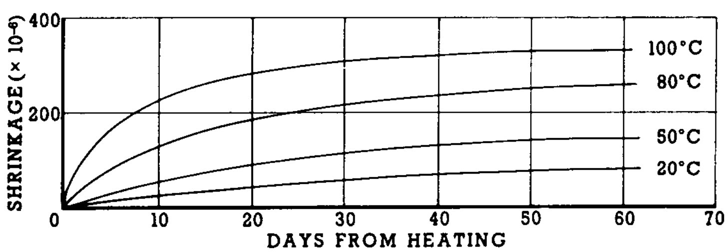


Fig. 6.2.3-10. Graph Showing Variation of Shrinkage with Time for Unsealed Specimens Maintained at Various Constant Temperatures. (Ref. 44)

Table 6.2.3-2. Factors in Shrinkage of Concrete (Reference 52)

| | Individual Factor | Cumulative Effect |
|---|---|---|
| Characteristics of Cement: | | |
| From optimum gypsum to gypsum deficiency | 1.5 | 1.5 |
| From 15% residue 200-mesh to 0% residue | 1.25 | 1.9 |
| Compressibility of Coarse Aggregate: | | |
| From quartz to Elgin gravel | 1.25 | 2.4 |
| From Elgin gravel to expanded shale | (1.4) | |
| From steel punchings to Elgin gravel | (1.4) | |
| From steel punchings to expanded shale | (2.0) | |
| Rock Content of Concrete: | | |
| From 6 in. max. size agg. to 1/4 in. max. size* | (3.2) | |
| From 1 1/2 in. max. size agg. to 3/4 in. max. size** | 1.3 | 3.1 |
| Stiff consistency to wet consistency*** | 1.20 | 3.7 |
| Cleanness of Aggregate: | | |
| No clay in aggregate to much bad clay | 2.0 | 7.4 |

* Computed from g = 0.8 and 0.6 respectively
** Computed from g = 0.74 and 0.70 respectively
*** Computed from g = 0.74 and 0.71 respectively

Terms like shrinkage, swelling, and creep do not correspond to intrinsic properties of a material. They have been adopted to designate the behavior of laboratory specimens under environmental conditions. The capillary theory appears to account most nearly for the complex response of gel, mortar, and concrete to loading and environment. The concept of creep as the difference between the time-dependent movements of stressed and unstressed elements otherwise identical is clear cut, but in examining the state of stress on the boundary of an element in a structure it is manifestly impossible to identify quantitatively such components of stress as capillary pressure, vapor tension for each opening and the randomly directed reactions of aggregate particles on each other, whether directly or through varying thicknesses of gel bedding.

## Principal Factors in Creep

Much effort has been devoted to studying the influence on creep deformation of the different variables in the mix, curing process, environment, and loading. This work was largely confined to rather small laboratory specimens and models in which it is difficult to avoid errors caused by drying shrinkage and to maintain the conditions that would exist in massive concrete. The effect on creep exerted by the cement, the aggregate, certain admixtures, water ratio, curing practice, size of section, working environment, duration, intensity, and type of loading have been given considerable attention.

Neville[61] has drawn the following conclusions regarding the effects of cement on creep: the composition and fineness of the cement have a small influence on creep; low-heat (Type IV) cement creeps more than normally calcined cement and is sometimes used where shrinkage cracking or earth movements are troublesome; aluminous cement creeps less than Portland cement; fineness increases creep.

Fluck and Washa[62] have summarized the creep behavior of plain and reinforced concretes. Aggregates of high modulus of elasticity are able to restrain creep. Hard, dense aggregates with low absorption, good bonding properties graded for low voids give low creep. Quartz and dense limestone give low creep, granite and basalt are acceptable, while sandstone and shale allow greater creep. Contaminants borne by aggregates such as clay increase creep. Physical and chemical compatibility of aggregate and gel promote structural soundness and thus enhance creep resistance.

Admixtures must be regarded with suspicion until their effect on creep is established. Pozzolans tend to increase creep deformation while a number of air-entraining agents have shown little influence.

Creep is nearly proportional to the amount of paste in concrete regardless of the water-to-cement ratio under the stipulation that the sustained stress is a given fraction of the ultimate strength at the time of application of the load. With a fixed sustained stress, creep decreases with water/aggregate ratio, illustrating another advantage of power vibration over hand rodding. The former makes lower water content and/or higher aggregate content practicable.

Duration of cure may be regarded as equivalent to age at time of loading unless the intervening exposure is definitely deleterious. Creep decreases with longer cure, but if the sustained stress is fixed as a fraction of ultimate strength at time of loading, age has less importance. L'Hermite[55] shows that the ratio:

$$\frac{\text{creep of specimen loaded at } \underline{a} \text{ days age}}{\text{creep of specimen loaded at 7 days age}}$$

decreases linearly with logarithm of a/7.

Experiments with oven drying at 105°C to establish the effect of evaporable water content at time of loading on creep indicate that creep would disappear with complete loss of evaporable water. Autoclaving in steam at 180°C results in low creep that stops in 100 days.

Humidity in storage, that is during the period the load is sustained, affects creep strongly. A normal 50% RH (RH = relative humidity) seems to result in creep 2 to 3 times that in moist storage. At 20% RH the integrated creep closely parallels that at 50% RH. Alternate wetting and drying increase creep; compressive load appears to inhibit the swelling in moist environment that would be shown by an unstressed specimen.

Creep increases rapidly with temperature of storage, at least during the first ten days of loading. Creep of sealed specimens at 80 days was 4 times as great at 100°C as at 20°C.

The rate of creep decreases with duration of loading in a roughly exponential manner, but creep does not stop in thirty years under constant conditions. Although the concre e is maturing and its elastic modulus is increasing, creep persists at a progressively slower rate. In Davis' tests[56] if creep in one year is unity, after 10 years it is 1.26, after 20 years 1.33, and after 30 years 1.36.

Unsteady loading accelerates creep analogous to results from steel at elevated temperature.[55] Applying a load 500 times a minute for 1 day (720,000 repetitions) caused the same creep as an equal steady load induced in 28 days; 13.9 days ($10^7$ repetitions) was equivalent to 600 days (43 times the elapsed time) of steady loading.

Little work has been done on creep in tension and torsion, on shearing, creep at bond, and under other biaxial and triaxial combined stress conditions. Tensile creep is more rapid in early stages than compressive creep and tensile strain leads quickly to rupture. However, a pure-torsion creep test indicated exactly the same axial shrinkage as an unstressed control specimen. Shrinkage should not cause torsion and torsion caused no shrinkage, but this implies that the creep deformations attributable to diagonal tension and diagonal compression are of equal magnitude and oppositely directed.

Although there is considerable variation in results, Poisson's ratio in creep is apparently on the order of the elastic ratio, 15 percent or less, indicating that creep is accompanied by considerable volume change.

A question of great concern to the designer of a prestressed concrete reactor vessel is whether the great mass of concrete required, or more specifically, the length of the mean diffusion path by which water may escape offers any hope for reduction in deformation levels for creep as it certainly does in the case of drying shrinkage. Measurements on dams have indicated that it has a great effect, generally explained by the slowing of evaporation. Concrete stored in water creeps less than a sealed specimen, and a sealed specimen creeps less than an unsealed one. Two mechanisms appear to be operative: internal shrinkage stresses, and hydration. Surface shrinkage puts the core under a compression load but in a relatively moist medium, so that its creep is less pronounced than the surface creep. Although delayed, considerable core creep will be caused in thin specimens by the progressive and sustained shrinkage of layers closer to the surface. In thick bodies the moist, bulky core is so dominant as to suppress any great effect from the stretched skin that absorbs almost the entire strain incompatibility by tensile creep, thin layer after thin layer, as drying progresses. As for hydration, a bulky vessel wall will be treated to a protracted saturated cure inside the relatively thin layer that can be affected by the drying conditions at the surface. Hydration will be far advanced when desiccation affects an appreciable fraction of the mass. Thus mass concrete creeps less because it is better hydrated, just as a specimen loaded in dry air will creep less after a long moist cure.

### 6.2.4 Concrete Fatigue

It is customary in reinforced concrete design to treat live loads separately from dead loads in structural analysis and to consider the frequency of load application. The role of fatigue in concrete pressure vessel design is determined by these cyclic live load characteristics. A reactor may be subjected during its lifetime to millions of small temperature variations by changes from ambient to operating temperature during reactor shutdowns for maintenance, and infrequent changes in pressure during shutdowns. The reactor vessel is not likely to be sensitive to the minor temperature changes in the reactor. By design practice the vessel wall is always cooled by the reactor inlet coolant which is maintained at constant temperature regardless of reactor power conditions. In current European design the vessel's temperature is controlled by a separate cooling system which divorces the vessel from the reactor power transients. Thus cyclic temperature stresses of concern in concrete reactor vessels may be limited to conditions during startup and shutdown of the reactor and even then they may be tempered by the large heat capacity of concrete and the low thermal diffusivity leading to a very sluggish response to transients. Consequently, the behavior of concrete under temperature transients may not for many designs introduce serious fatigue problems. However, where it is desired to use the concrete at high temperatures, this can be a matter of concern and must be examined.

Pressure loading introduces similar fatigue questions, but these too will be infrequent, occurring only during shutdowns for maintenance or accident conditions. An exception is made in the case of reactors using the Brayton cycle where pressures are varied significantly with reactor power output.

The area of greatest concern may be in the vicinity of penetrations where power levels may change heat transfer conditions significantly, but it is more likely that the transients of greatest interest from a fatigue standpoint are those associated with starting and stopping power operations and depressurization conditions.

Nevertheless, it is important to know how the structure behaves under live load changes. Work carried out on structural components used in bridges, highways, and railroad crossties gives an indication of the cyclic load behavior of concrete structures. A general review of research on the fatigue of concrete appeared in 1958[63] and one confined to prestressed structures in 1957.[64]

6.2.4.1 Fatigue of Plain Concrete

Up to ten million cycles, plain concrete has a compressive fatigue strength (for zero to maximum compressive stress) of 50-55% of static ultimate crushing strength. Since the flexural fatigue strength is usually nearly 55% of the ultimate static flexural strength, but shows a variation of between 33 to 64% of ultimate, and failure is invariably tensile for spans a reasonable multiple of depth, it can only be concluded that this is also close to the tensile fatigue strength (up to $10^7$ cycles). Moreover, as would be expected from the vastly greater compressive strength, no difference is detected between one-way and fully-reversed bending strength of plain concrete beams. The interaction of steady and alternating components of stress are presented in Fig. 6.2.4-1, a modified Goodman diagram for plain concrete beams from reference 65. No evidence of an endurance limit for concrete has been found.[65]

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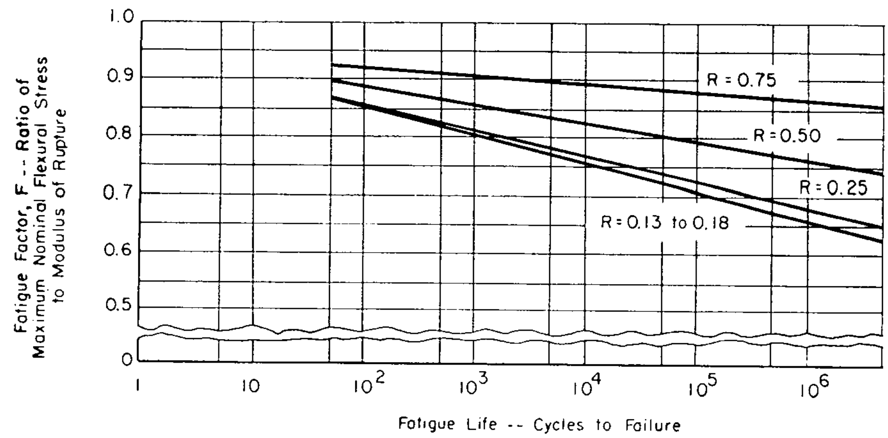
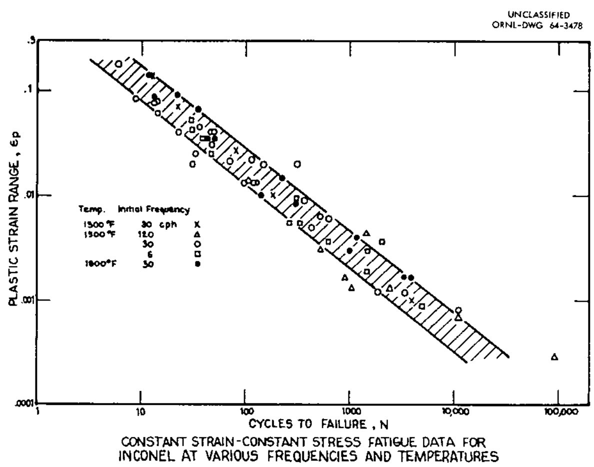



(a) Effect of the range of stress on the behavior of plain concrete under fatigue loading.




(b) Modified Goodman diagram showing the effect of the range of stress on the fatigue limit of plain concrete under repeated loading.

Fig. 6.2.4-1. Influence of Mean Stress on Fatigue Strength of Plain Concrete in Bending. (Ref. 65)

Under repetitive loading, depending on the intensity of loading and the ratio of maximum to minimum cyclic compressive stress, the modulus of elasticity changes in various ways. Secant moduli gradually deteriorate under heavy loading. Permanent strain (creep) is quickly apparent, increasing with the steady-stress component. The stress-strain diagram acquires an upward concavity (and even an S-shape just before failure) with considerable width in the hysteresis loop. On the other hand, moderate cyclic loading may increase the linear range and the fatigue strength in somewhat the same manner as work-hardening affects austenitic steels; sometimes a definite stiffening has been observed.

Well-cured, high-quality concrete is believed to exhibit a high and consistent fatigue strength ratio (F), as a general rule. Rate of testing has no effect above 1 cps. At 10 cycles per minute creep begins to play a role in lessening fatigue strength. Fatigue tests on bond have been erratic.

The effect of temperature on fatigue strength does not appear to have been investigated. Presumably, as in metals, it would have a marked influence on the damage caused by the steady component of stress in slow cycling. Fatigue and creep plastic damage are difficult to separate in this gray area of time dependence.

6.2.4.2 Fatigue of Reinforced Concrete Structural Components

Most failures of reinforced beam specimens were caused by steel failure that apparently arose from stress concentration and abrasion at cracks. If the longitudinal reinforcement is critical, the specimens seem to have a true endurance limit at $10^6$ cycles of 60 - 70% of static ultimate strength.

Comprehension of the complex interaction of shear, diagonal tension, and bond adhesion and discernment of their roles in the order of events leading up to fatigue failure of reinforced beams has been difficult. Chang and Kesler[66] state working ground rules briefly: beams that are flexurally weak statically, will generally fail in flexure (steel failure) in fatigue, provided the maximum shear in fatigue is not more than 60% of the static diagonal cracking strength; flexural failure is possible after diagonal cracking provided the cracks do not concentrate the compressive zone fast enough to permit shear-compression failure. This requires that the fatigu noment be less than 70% of static ultimate flexural strength; if the foregoing shear and moment ratios are much exceeded shear-compression failures generally result.

Verna and Stelson[67] extend these conclusions over the $10^3$ to $10^7$ cycles domain and show good evidence that the critical fatigue moment and shear ratios are functions of the number of cycles. Figure 6.2.4-2 reproduced therefrom, divides the whole F-N plane into regions:

I. No failure in any beam in any mode is likely.

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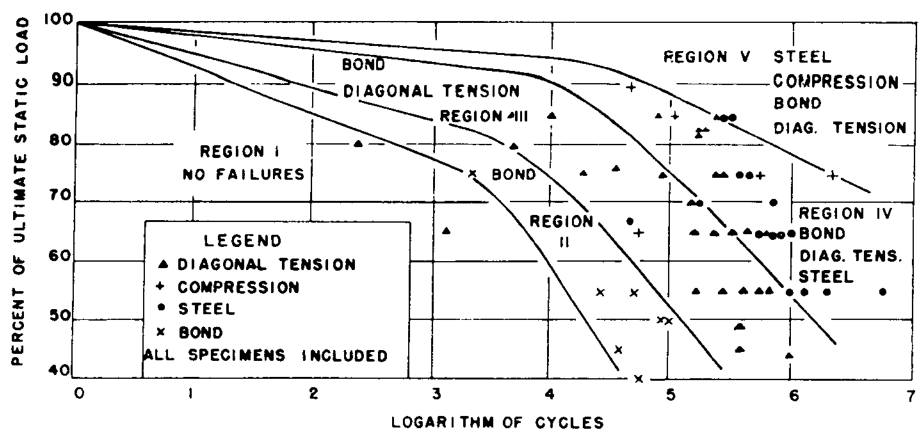


Fig. 6.2.4-2. Regions of Failure by Mode. (Ref. 67)

II. Bond failure is the only mode likely.

III. Diagonal tension may occur as well as bond.

IV. Steel failure may occur if bond or diagonal tension failures do not forestall it.

V. Absolute-destruction region where compression failure will end the test of any specimen escaping the other modes.

It is curious that the experimental points could have been fitted with straight tangents to the right hand limb of the authors' zone boundaries. These tangents would intersect the 100% of ultimate line between $10^3$ and $10^4$ cycles as the scatter-band boundaries for ductile round or rectangular steel beams do. More tests are needed below $10^3$ cycles to discover if some plastic redistribution of stress is effective in suppressing fatigue failures at low numbers of flexure cycles.

Of course this classification applies only to rectangular-cross-section beams without shear reinforcement, the variables being concrete strength, steel area, steel perimeter, and beam depth. Compression failures by fatigue in the concrete in beams deliberately designed to be critical in the compression zone have been reported as low as 60% of static strength. Also, in narrow-web specimens fatigue shear may bring failure at 40% of monotonic ultimate load.

6.2.4.3 Fatigue of Prestressed Concrete Specimens

Beams under repeated load show failures entirely different in nature from static failures. Fatigue failure of tendons at their separator bearings can lead to sudden major cracking of concrete in the plane of maximum moment.[94] The current design rules give adequate protection to the concrete in fatigue; cause of failure appears to be confined to the prestressing steel in all reported work. Bonded steel fails at cracks. Bonds give trouble only in short, deep spans. Specimens that do not fail by fatigue show full static strength in subsequent tests. The percentage of static strength developed in fatigue is generally higher for prestressed than for reinforced beams. Of course plastic-hinge theory for limit-design is not applicable to repeated loading.

Work is needed on the fatigue strength of the wire at anchors, bonds, cracks, separators, anti-friction shoes and, in fact, any contact that causes cyclical bending or lateral compression to be superimposed on the working tension of the steel. Obviously, short spans of prestress and nicked or fretted tendons are potential sources of trouble.

6.2.5 Cement-Aggregate and Cement-Steel Bond Strength

Efficient concrete design from the structural viewpoint requires the aggregate, cement paste, and the bond between them to be of almost equal strength. Such efficiency is utopian, primarily because of the bond strength which is usually the weakest of the three structural components. Bond strength has been measured for various aggregate-cement paste combinations.[68] The surfaces of the rock were prepared similarly to eliminate surface irregularities as a variable. The bond strength between Portland cement and different aggregates varied considerably. No great difference appeared among different samples of the same rock type. Reactive rock could not be associated with an increase in bond strength development. In other respects, the bond followed the same behavior noticed in cement paste tests; strength increases with decrease in water content and with age. In all tests, the strength of the cement paste was greater than the bond. Elastic strength tests of mortar with a 3 to 1 sand-cement ratio and water-cement ratios of 0.4, 0.5, and 0.6 showed that elastic failure takes place in the cement paste rather than in the bond between paste and aggregate.[69] Mathey and Watstein[70] have experimentally investigated the bond strength in concrete, using high yield strength steel reinforcing. They conclude as follows: A linear relationship exists between critical bond stress and the ratio of bar diameter to embedment length for the 1/2 in. bar but not for the 1 in. bar. The ratio of bond stress to tensile stress is equal to the ratio of diameter to 4 times the embedded length is suggested as the basis for design. The bond strength decreases with embedment length for a given bar. It also decreases with an increase in diameter for a given length-diameter ratio. Since diametral size influences bonding strength, the conclusion advanced should apply only to the two sizes tested.

#### 6.2.6 Shielding Properties

General shielding requirements and how these requirements can be incorporated into concrete for nuclear vessels has considerable bearing on acceptability standards. Neutron radiation is the most difficult to shield.[85] The main design problem is the arrangement of a correct balance to effect the most efficient and economical shield against all radiation. Hydrogen is the most effective element in slowing down (thermalizing) neutrons over the entire energy spectrum. Boron is effective in capturing thermal neutrons. It releases alpha particles which are easily shielded. The use of boron in concrete is debatable because of its high cost and its poor qualities of delaying setting time and weakening concrete. Boron is often uneconomical for concrete shielding in reactors surrounded by heavy steel tanks and thermal shields, but when needed the addition of calcium chloride restores the concrete strength and eliminates the setting delay to lend some practicality to its use. Calcium chloride is avoided in prestressed concrete.

Dense material is needed to shield against gamma rays. Since concrete is an economical and effective material for shielding stationary reactors, a high density concrete is often preferred to the low density type because its use leads to thinner walls.[45] Aggregates of hydrous iron ore increase the 150 lb per cu ft of ordinary concrete to 220 lb per cu ft. The increased water content provided by this aggregate increases the hydrogen available for attenuating neutrons thereby improving its biological shielding properties. Barite aggregate meets the density requirements but its other physical properties render it less suitable for shields.[83] On the other hand, magnetite meets most of the requirements. Magnetite concrete is made successfully by conventional methods if the cement paste is dense and tough enough to float the heavy aggregate particles.[84]

6.2.7 Heat Transfer Properties

Concrete materials are best described as heat insulators rather than heat conductors. Lea and Desch[35] indicate that concrete thermal conductivity can vary from 0.8 to 2.0 Btu/hr/ft$^2$/°F/ft. Generally the higher thermal conductivities are associated with more dense concretes or those having a high moisture content. Very low density refractory concretes (around 50 lb/ft$^3$) can have thermal conductivities which are in the range of 0.12 Btu/hr/ft$^2$/°F/ft extending up to 0.5 Btu/hr/ft$^2$/°F/ft for higher density refractories (115 lb/ft$^3$) containing crushed firebrick. Because of these low thermal conductivities, very little heat can be transferred through concrete structures without large temperature gradients.

The heat capacity of concrete is in the range of 0.2 to 0.25 Btu/lb/°F[35] so that the thermal diffusivity of concrete is quite low. It is because of this property that short time transients are not generally of significance in the structural behavior of the concrete except at the heated surface where spalling can occur.

6.2.8 Thermal Expansion

Although for general purposes no significant difference exists between the coefficient of expansion for steel and concrete, there is in fact an important variation between these materials. The coefficient of expansion for steel is usually taken to be 6.5 x $10^{-6}$ inches/inch/°F, while concrete may vary from 3 to 8 x $10^{-6}$ depending upon the aggregate type and content.[35] The practice normally followed is to select a concrete mix which minimizes the difference in expansion coefficient, but when high thermal gradients are involved, there may be some advantage in selecting a concrete mix which biases the concrete thermal stress in a favorable direction.

Thermal stresses in concrete are usually evaluated by elastic analysis. They are held within reasonable bounds by imposing severe temperature restrictions. For high temperature systems where tensile cracking must be tolerated, as in refractory installation, Crowley[71] has suggested that concrete structures designed in such a way that a residual compressive stress remains in the high temperature surface even though cracking must be tolerated in the cold surface. This assures that the integrity of the concrete will be maintained. In such cases the thermal expansion is used to offset drying shrinkage strains which are invariably introduced in refractory concretes. The methods by which this approach can be adapted to prestressed concrete have not been defined, but potentially a similar technique could be followed, thus minimizing the prestressing demands on the steel reinforcing.

### 6.3 Reinforcing Steel

In normal reinforced concrete applications, steel reinforcing is used as a crack control device and the distribution of steel in the concrete is such as to limit the size of the cracks to a level at which the amount of concrete in compression can balance the loading of the steel in tension. The concrete is then effective only if the fraction permitted to crack is relatively small. Steel, on the other hand, can be used effectively in this manner only if it is permitted to strain substantially. The balance between these two material properties is such that practive has shown mild steel having a working stress of 16,000 to 18,000 psi in tension and an elastic modulus of 28 x $10^6$ matches well with 3000 to 5000 psi concrete having an elastic modulus between 4 and 5 x $10^6$. Little need be said about the character of reinforcing steel used in controlled cracking applications. The ACI Standards adequately describe the requirements of these materials. A wide variety of steel shapes are available to suit the various types of structures in general use.

Where prestressing is used there is no longer the necessity for concern about the cracking of concrete and the maximum attainable strength in prestressing steel can be utilized. The attention of the prestressed concrete vessel designer is therefore directed toward high strength steels for prestressed applications.

The first applications of prestressed concrete used high strength steel bar, and this material is still used frequently for precast shapes, but the most effective form of steel reinforcing has been found to be stranded wire cable. The chemical composition of such cable has been standardized in ASTM specification A421-59T.[72] It is a high carbon steel wire (about 0.8% carbon) suitable to hard drawing. The manufacturing techniques for wire cable are discussed by Bannister.[73] He identifies the following properties as valuable in stranded wire: 1) Reasonable flexibility, 2) capability of behaving as a homogeneous unit under tension, 3) ability to withstand heavy transfer forces without distortion, 4) ability to bond well with concrete. These requirements lead to the use of a 7-wire stranded cable in which a straight central core wire is wrapped with six helical wires as the most useful for prestressed wire cable. Stranded wire 0.25 inches in diameter is close to the maximum tolerable in 7-wire construction. Where larger wire groups are desired, mixtures of 2 or 3 sizes of wire can be used to retain flexibility with increasing size.

6.3.1 Steel Structural Properties

Prestressed concrete requires that steel prestressing loads be very large. These are normally attained by using ductile, high tensile strength steel. As discussed in Section 2, a large amount of permissible elastic strain must be provided in the prestressing to allow for losses due to inelastic deformation of the concrete. The importance of ductility is an obvious requirement derived from the need to provide a steel which cannot fail abruptly without prior indication of distress.

The steel of greatest interest for prestressed concrete vessels is a high carbon steel wire which is hard drawn to attain the required strength and dimensionally stabilized by stretching and stress relieving the wire. As shown in Table 6.3.2-1, the strength of the wire decreases with increasing diameter, leading to practical limits of about 9/32 in. diameter in prestressing applications. Alloy steel rods are used for prestressing in many applications where the advantages of large diameter offset the reduced strength capability, but sizes exceeding 2 in. diameter are not used even for alloy steel prestressing rods. By grouping small wire strands it is possible to realize the high strength capability of the wire and also utilize large steel cross sections where required. Cables up to 2 in. diameter are available commercially for prestressing applications. Much larger cables have been used in suspension bridge installations, but the difficulties in attaining uniform tension and suitable fatigue characteristics in the cable anchors must be resolved before size extensions can be attained in prestressing cables. Nevertheless, larger size cables should be feasible if a demand for them is indicated.

Steel is a malleable alloy of iron containing carbon, manganese, and other alloying constituents. Steels whose distinctive properties mainly result from their carbon contents are classed as carbon steels. Alloy steels owe their distinctive properties to the effects of other elements separately or in conjunction with carbon.

High tensile strength is ordinarily derived from higher carbon content. Additional alloying constituents modify or enchance such engineering properties as toughness, depth of section that may be heat-treated effectively, strength at elevated temperatures or corrosion resistance. The cooling rates of heat-treatable steels from above their transformation temperatures to atmospheric temperatures profoundly affect their mechanical properties. Slow cooling or annealing of medium and high carbon steels renders them relatively soft, rapid cooling or quenching hardens them. Intermediate properties are obtained by cooling in the atmosphere (normalizing) or by reheating quenched material to 400 to 1200°F and then cooling in air. The higher tempering temperatures give a progressively weaker but more ductile product. "Patenting" achieves a similar result by rapid cooling to a tempering temperature followed by cooling in air. When applied to high-tensile steel wire the term "stress-relieving" refers to holding for a considerable period at a low temperature. As the term implies, stress-relieving allows the differential axial stresses set up by plastic shearing during the wire drawing process to creep to lower intensities. As-drawn, the remanent compressive stress would act as an internal reaction to reduce the apparent tensile elastic range of the wire.

By metal stretching to about 2/3 of ultimate strength the wire elastic range can be biased toward the tensile direction of the stress axis. Since a tendon will never be subject to axial compression, this tensile shift is a clear gain. During this process some strain hardening results, reducing the elongation capability. The elongation limit can be enhanced by a mild heat treatment in molten lead at 700°F. This combined stretching and heat treating is called "stabilization" and is usually specified for high quality prestressing cable. The effect of stress-relief and stabilization on the stress-strain characteristics of hard drawn wire is shown in Fig. 6.3.2-5.

High-tensile carbon steel wire owes its strength to cold working; it is drawn through dies that reduce its diameter slightly during each pass, with intermediate annealing treatments. Cold-drawn prestressing wire suffers a degradation in strength with increasing diameter (see Table 6.3.1-1) that limits its practical use to 7 mm (9/32 inch) diameter. A similar limitation makes heat-treated alloy rods larger than two inches diameter unattractive.

### 6.3.2 Strength of Steel Evaluation

The tension test is the customary way of investigating the mechanical properties of steel. A standard specimen of suitable length/diameter ratio is centrally and progressively loaded while an extensometer system senses the elongation in a specified gage length. The testing machine traces a curve of load vs elongation of which typical shapes are shown in Fig. 6.3.2-1, that gives the derived stress-vs-strain curves for several classes of steel products. Such curves contain much of the information on the structural behavior of steel that is needed by the designer. The slope of the straight portion of the curve is the elastic modulus. The "proportional limit" is defined by a barely-detectable plastic set. The "yield point" is found only in low carbon steel and is characterized by a sudden loss of load with increasing strain. For other steels, the "yield stress" is defined as that

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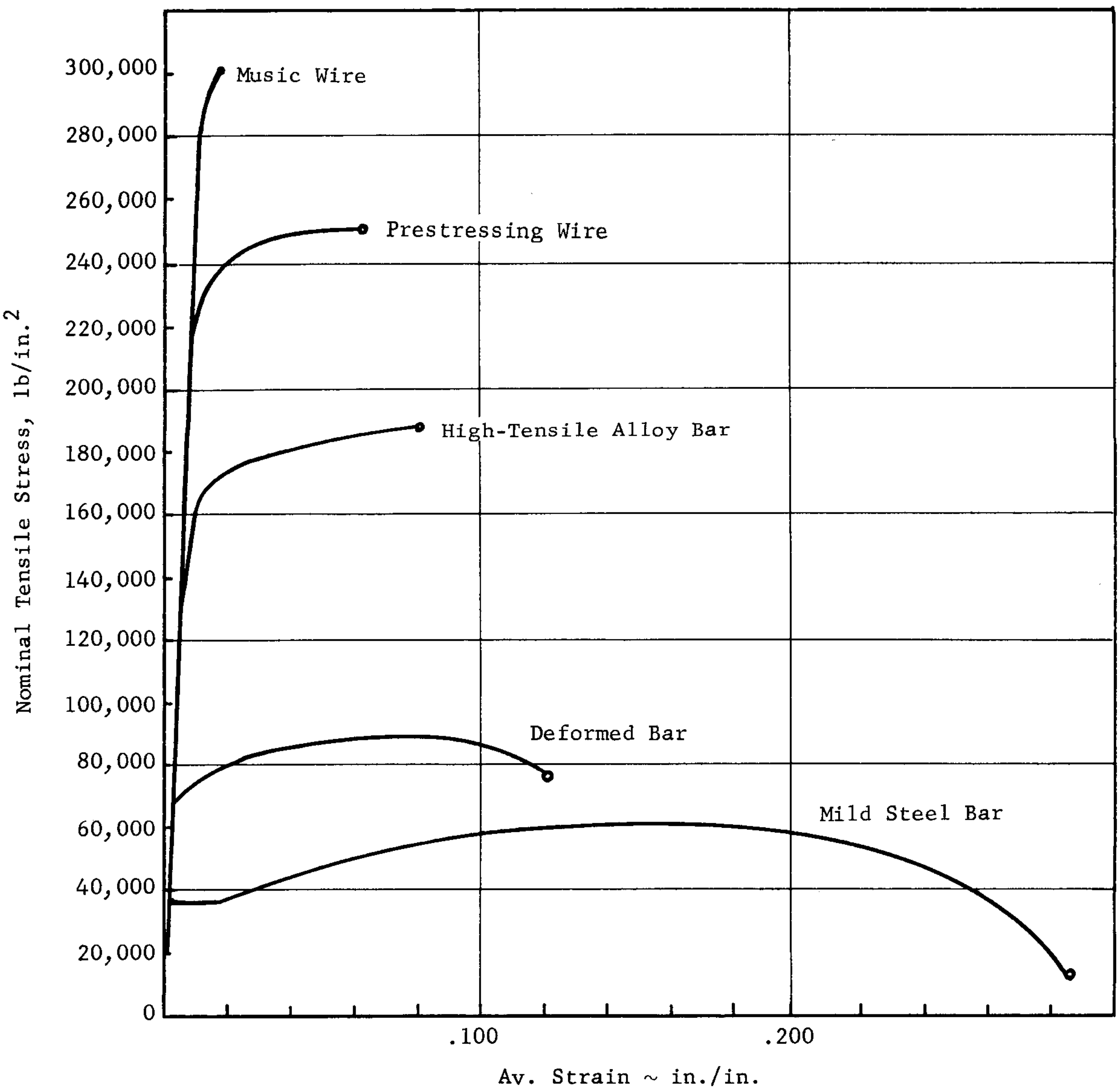


Fig. 6.3.2.1. Tensile Tests of Various Steels.

corresponding to a considerable plastic offset such as an average strain of 0.002 inches per inch in 2 inches. The "ultimate strength" is the maximum value of the fictitious stress curve based on the original cross section. The drooping branch beyond the maximum is caused by pronounced necking. The "true stress" based on the cross section of the neck invariably rises for a steel specimen until actual fracture starts. The total elongation at fracture and the reduction in area suffered by the neck are two indications of "ductility". The typical progressive reduction in ductility that goes with increasing yield strength is evident in Fig. 6.3.2-1.

The ability of steel metallurgists and fabricators to reproduce closely the strength and type of stress-strain relations specified by the designer is a major factor in the universal acceptance of steel throughout the whole range of structural applications. A comprehensive discussion of the stress-strain behavior of steel is provided by Kennedy.[74]

The elastic characteristics in typical tensile tests of solid uncoated stress-relieved wires for prestressed concrete of No. 6 wire gage and 5 mm and 7 mm diameter are shown in Fig. 6.3.2-2. The minimum stress corresponding to a total elastic and plastic average strain in the gage length of 0.7% is sometimes used in specifying prestressing wire. The "yield stress" at 0.2% plastic offset is indicated on the curves. In Britain the term "proof stress" generally at 0.1% plastic offset is used in place of "yield stress." Table 6.3.2-1 shows weights and minimum ultimate tensile strengths guaranteed by one manufacturer for some commonly-used sizes of solid stress-relieved prestressing wire. The superior specific strength of the smaller wire is apparent.

Fig. 6.3.2-3 gives a similar record of the elastic behavior of a tensile test on 3/8 inch 7-wire strand. Engineering data for commercial strands are presented in Tables 6.3.2-2 to -5. The small losses in strength caused by torsional, bending, and lateral compressive stresses induced in a helically-laid strand or rope, are largely compensated by the gains from using small, inherently stronger, wires, and by the stabilizing treatments used. Recently developed techniques of galvanizing or tinning exact only a minor toll of ultimate strength that is offset by greatly increased resistance to corrosion.

Fig. 6.3.2-4 shows the type of elastic behavior found in alloy-steel prestressing bars. Table 6.3.2-6 shows one manufacturer's recommended initial and working prestressing forces and guaranteed ultimate loads for bars of 1/2 to 1 1/8 in. diam. Although only about 3/5 as strong per unit cross section, such bars compete successfully with wire and strand for a range of applications.

Carbon steel loses both its ductile yield strength and its brittle cleavage strength as its temperature is lowered, but at the "nil-ductility-

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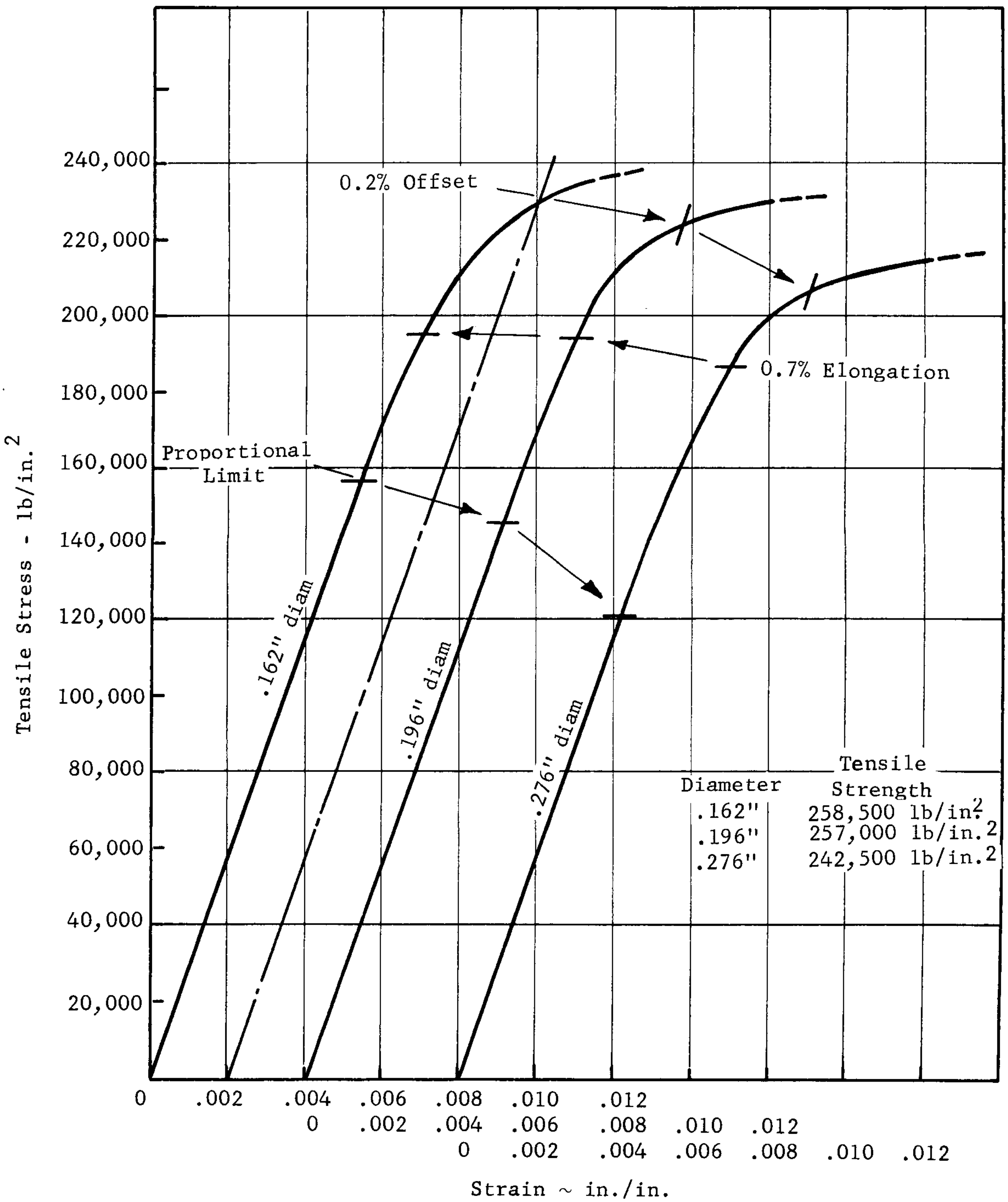


Fig. 6.3.2-2. Typical Tensile Tests in Elastic Region on Uncoated Stress-Relieved Wire for Prestressed Concrete (Roebling).

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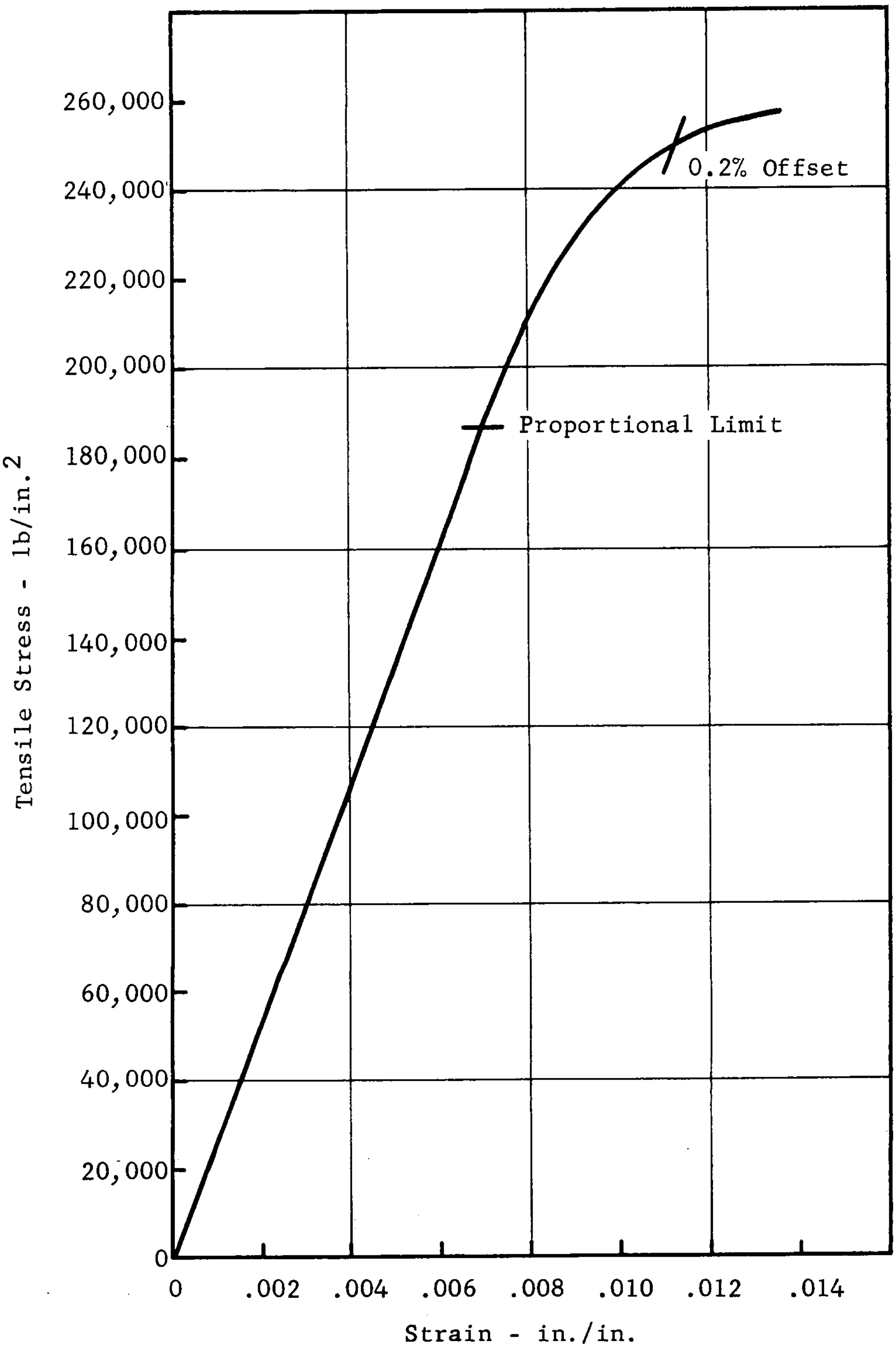


Fig. 6.3.2-3. Typical Tensile Test in Elastic Interval on 3/8 in. Diam 7-Wire Strand for Prestressed Concrete (Roebling).

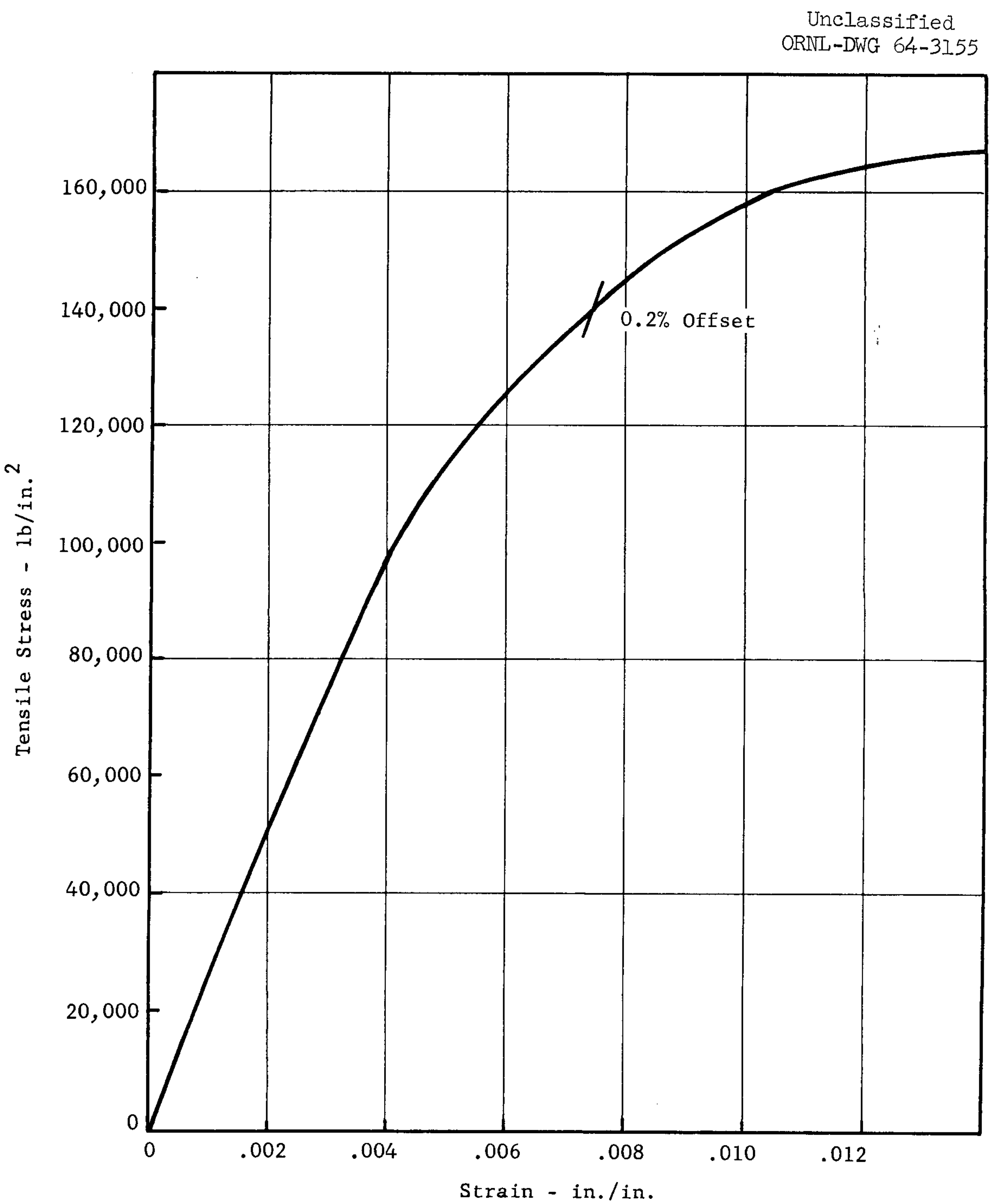


Fig. 6.3.2-4. Typical Tensile Test in Elastic Interval for High-Alloy Bars for Prestressed Concrete (Rods, Inc.).

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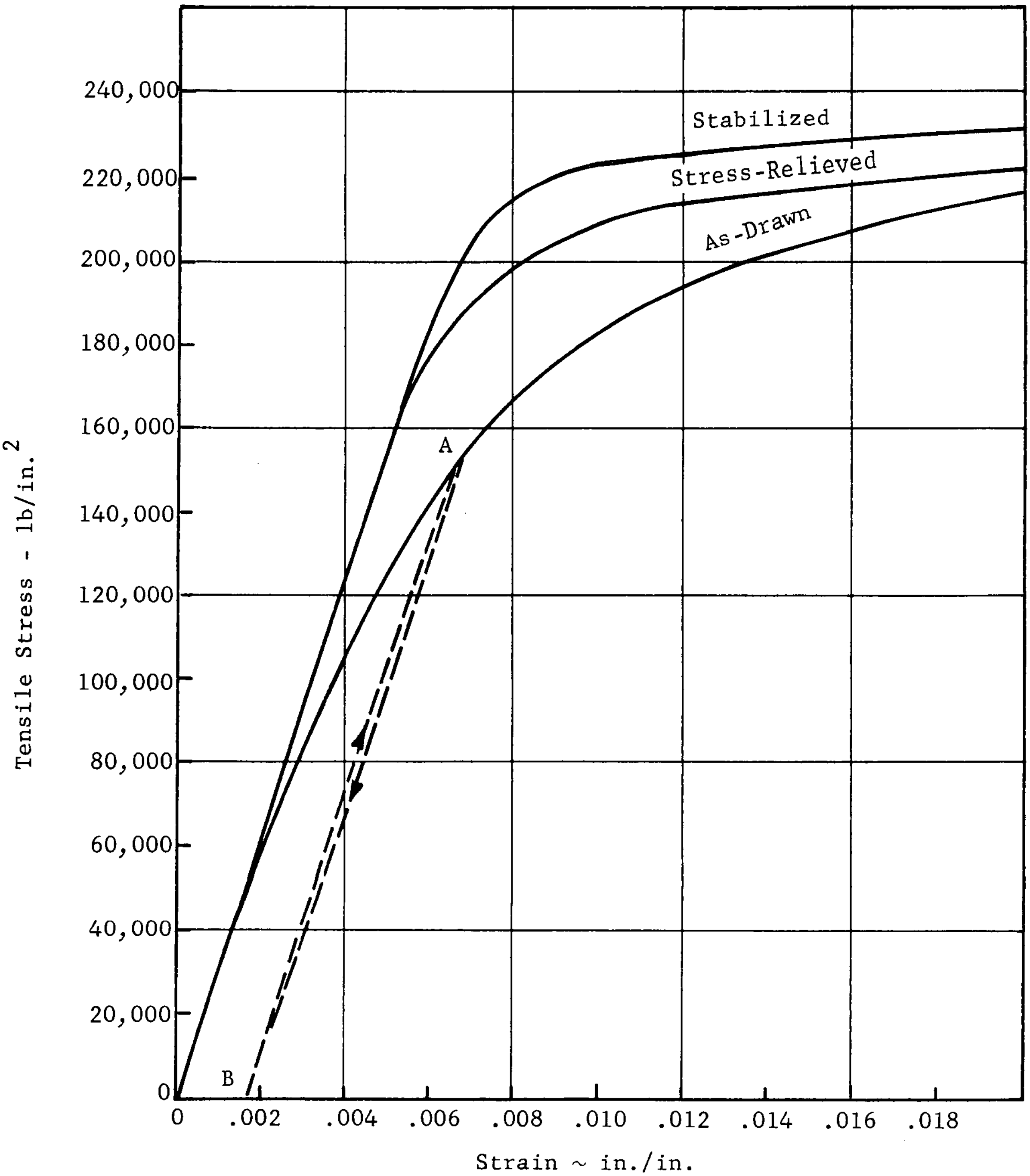


Fig. 6.3.2-5. Effect of Post-Drawing Treatments on 0.200 Diam Wire.

Table 6.3.2-1. Solid Stress-Relieved Wire for Post-Tensioning (Ref. 75)

| Diameter in. | Area $in^2$ | Weight lb/1000 ft | Min. Ultimate Strength $lb/in^2$ |
|---|---|---|---|
| 0.276 (7 mm) | 0.05983 | 203.2 | 236,000 |
| 0.250 | 0.04909 | 166.7 | 240,000 |
| 0.196 (5 mm) | 0.03017 | 102.5 | 250,000 |
| 0.192 (#6 gage S.W.) | 0.02895 | 98.32 | 251,000 |

Table 6.3.2-2. Roebling 7-Wire Galvanized Strand for Prestressed Concrete (Ref. 75)

| Nominal Diameter inches | Weight Pounds per 1000 ft | Approx. Gross Metallic Area $in^2$ Including Galvanizing | Minimum Breaking Strength lb |
|---|---|---|---|
| 1/4 | 122 | 0.0356 | 7,400 |
| 5/16 | 198 | 0.0578 | 12,100 |
| 3/8 | 274 | 0.0799 | 16,500 |
| 7/16 | 373 | 0.1089 | 22,500 |
| 1/2 | 494 | 0.1438 | 31,000 |

Table 6.3.2-3. Roebling 7-Wire Uncoated Strand for Pre-Tensioning (Ref. 75)

| Nominal Diameter inches | Weight Pounds per 1000 ft | Approximate Area $in^2$ | Ultimate Strength lb |
|---|---|---|---|
| 3/16 | 73 | 0.0214 | 5,500 |
| 1/4 | 122 | 0.0356 | 9,000 |
| 5/16 | 198 | 0.0578 | 14,500 |
| 3/8 | 274 | 0.0799 | 20,000 |
| 7/16 | 373 | 0.1089 | 27,000 |
| 1/2 | 494 | 0.1438 | 36,000 |

Average modulus of elasticity $27(10)^6$ $lb/in^2$

Table 6.3.2-4. Standard Galvanized Bridge Strand Physical Properties and Design Data (Ref. 75)

| Nominal Strand Diameter* | Number of Wires | Approximate Weight lb/ft* | Approximate Gross Metallic Area Incl. Galv., sq. in* | Min. Breaking Strength in Tons of 2,000 lb |
|---|---|---|---|---|
| 1/2 | 7 or 19 | 0.52 | .149 | 15.0 |
| 3/4 | 19 | 1.18 | .336 | 34.0 |
| 1 | 19 | 2.00 | .577 | 61.0 |
| 1 1/2 | 51 | 4.70 | 1.36 | 138.0 |
| 2 | 93 | 8.48 | 2.43 | 245.0 |
| 2 1/2 | 139 | 13.25 | 3.81 | 376.0 |
| 3 | 183 | 18.90 | 5.400 | 538.0 |

* Approximate value - more accurate figures will be furnished on request.

Minimum modulus of elasticity of strands when prestretched - 24,000,000 psi.

Design data of larger strands up to 4 in. diameter will be furnished on request.

Sizes and constructions available from stock will be furnished on request.

Table 6.3.2-5. USS, American Steel and Wire Division
Strands for Prestressed Concrete

| Nominal Strand Diameter in. | Number of Strands and Wires | Weight Pounds per 1000 ft | Area of Metal $in^2$ | Breaking Strength, lb | |
|---|---|---|---|---|---|
| | | | | Galvanized lb | Uncoated lb |
| 1/4 | 1 x 7 | 121 | .0352 | 7,350 | 8,960 |
| 5/16 | 1 x 7 | 205 | .0595 | 12,400 | 14,900 |
| 3/8 | 1 x 7 | 273 | .0792 | 16,500 | 19,600 |
| 1/2 | 1 x 7 | 516 | .150 | 31,000 | 35,500 |
| 5/8 | 1 x 7 | 812 | .236 | 49,000 | 53,400 |
| 3/4 | 1 x 19 | 1160 | .336 | 70,000 | 79,900 |
| 7/8 | 1 x 19 | 1610 | .468 | 97,000 | 108,000 |
| 1 | 1 x 19 | 2060 | .597 | 123,000 | 134,000 |

| Mechanical Properties of Wire | Galvanized | Bright |
|---|---|---|
| Min. U.T.S. ($lb/in^2$) | 220,000 | 238,000-268,000* |
| Min. elongation at U.T.S. | 4.0% in 10 in. | - |
| Approx. Y.S. at 0.7% elongation ($lb/in^2$) | 160,000 | 67% U.T.S. |

* Depending on wire size

Table 6.3.2-6. Properties of Stressteel Tensioning Units
Stressteel or Lee-McCall System
Truscon Division, Republic Steel Corporation, Youngstown, Ohio

| Bar Diameter in. | Area $in^2$ | Weight lb/ft | Initial Prestressing Force lb | Working Force (after losses of 15%) lb | U.T.S. $lb/in^2$ |
|---|---|---|---|---|---|
| 1/2 | 0.196 | 0.668 | 19,600 | 16,680 | 28,400 |
| 5/8 | 0.306 | 1.04 | 30,600 | 26,050 | 44,400 |
| 3/4 | 0.442 | 1.50 | 44,200 | 37,600 | 64,100 |
| 7/8 | 0.601 | 2.04 | 60,100 | 51,100 | 87,200 |
| 1 | 0.785 | 2.67 | 78,500 | 66,800 | 113,900 |
| 1 1/8 | 0.994 | 3.38 | 99,400 | 84,500 | 144,000 |

Basis:

| | |
|---|---|
| Guaranteed minimum ultimate strength | 145,000 $lb/in^2$ |
| Initial prestress | 100,000 $lb/in^2$ |
| Working prestress | 85,000 $lb/in^2$ |

transition" (NDT) temperature the brittle cleavage strength equals the ductile yield strength and ductility disappears. Actually many other factors besides temperature are involved, such as grain structure, imperfections, notches, and local variations in carbon content, so that the temperature, usually defined as the maximum of NDT, is a range of temperatures at which a running crack and brittle fracture may occur. Sustained radiation can increase the NDT of some steels significantly. Because of the high stress conditions in prestressing wire, local conditions under transient might in some cases exceed elastic limits. Since abrupt failure violates the prescribed failure mode, embrittlement must be avoided in prestressing cable. Hence, radiation damage to prestressing cable must be avoided.

### 6.3.3 Inelastic Behavior Considerations

The ordinary tensile test at room temperature when performed with reasonable dispatch, records only a part of the plastic deformation that would correspond to sustained high-stresses. All steels continue to elongate under a tensile load, although it is difficult to detect the increase of strain after a short time unless the stress is very high or the temperature is elevated. Fig. 6.3.3-1 presents creep behavior diagrammatically for the purpose of defining terms. The total strain is shown as a function of time for a specimen under a constant tensile load. In a sense, all strain is time-dependent, but elastic stress and strain are propagated into a body at its acoustic velocity, while plastic strain propagates slowly and usually must be measured over long time periods. Thus the ordinates of points A and B are indefinite, depending on the speed of loading and the sensitivity and resolution of the instrumentation.

From B to C the strain grows at a lessening time rate. Microscopic examination reveals continuing formation and growth of well-distributed slip-band systems, more numerous in a grain so oriented that a weak plane of its crystal lattice lies close to a direction of maximum shear, that is, 45 degrees from the direction of the tensile loading.

As time goes on, a nearly-linear interval from C to D is reached, at or near the minimum creep rate, where rates of diffusion processes having softening influences are in equilibrium with those of the strain-hardening processes. In creep tests at stresses corresponding to long times to rupture, this linear interval occupies a large fraction of the test time. At slightly-lower stresses, rupture will not take place in finite time. At temperatures near atmospheric, creep of carbon steel becomes negligible after a short period at loads in the neighborhood of the yield strength.

In the third stage of creep the strain rate rises toward rupture, apparently because of a disintegration of the joints between the more distorted and

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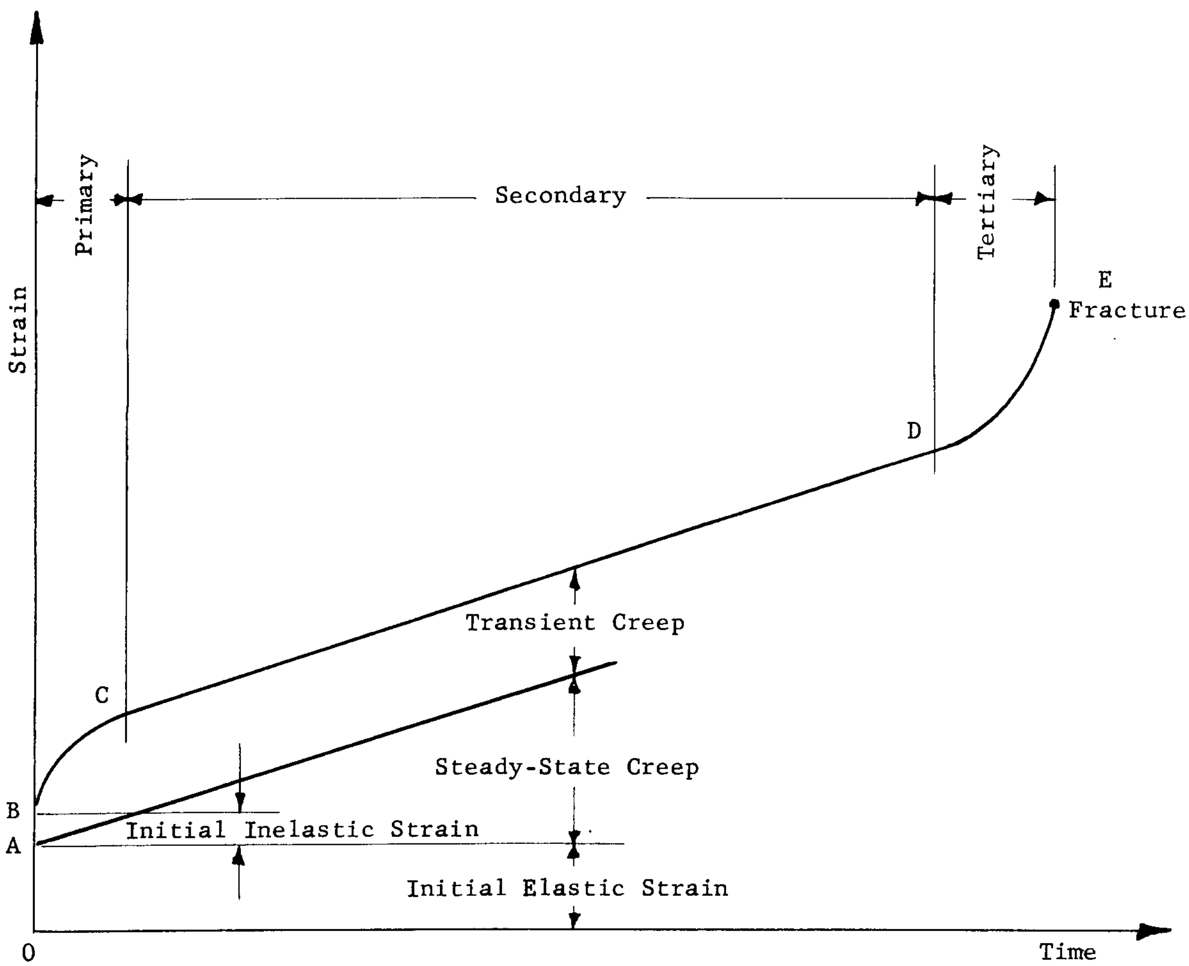


Fig. 6.3.3-1. Diagram of Strain vs Time for Classifying Inelastic Behavior of Steel Under Sustained Load.

mismatched grains. Creep rate at a given load increases, and yield strength and elastic modulus are lowered as the temperature is raised.

In prestressing steel, the creep rate is of interest mainly as it is reflected by the consequent relaxation of the prestress. As pointed out in Section 2, the elastic strain in the steel should balance at fifteen or more times that of the concrete. Therefore tests of relaxation of stress at constant strain are useful to show the inelastic behavior of tendons under steady temperature conditions, inasmuch as coolant pressure changes will have little influence on the strain in a tendon.

The first 8 hours of relaxation tests[73] of 0.200 inch diameter solid wire are shown in Fig. 6.3.3-2 for three levels of strain, in the as-drawn and in the stress-relieved condition. At the highest initial strain, stress-relieved wire exhibited more plastic flow than as-drawn wire after 15 or 20 minutes, but at a constant strain corresponding to 134,400 psi the stress-relieved wire showed only 60 percent of the 8 hour relaxation of the as-drawn specimen. Fig. 6.3.3-3 shows the loss of stress at constant total strain during the first 25 days after loading to 179,200 psi for a 19-wire strand in four different conditions. At this rather high loading the as-stranded material again showed less relaxation than in the stress-relieved condition. The specimen that had been stabilized at a high tension showed by far the best resistance to relaxation. Standard sizes (1/4 to 1/2 inch) of prestressing-quality seven-wire strands can be depended on not to exceed 1 1/2 percent total relaxation from 179,200 psi (80 long tons per square inch) at constant total strain and room temperature.

6.3.4 Fatigue

Fatigue fracture of steel, like creep rupture, is a plastic phenomenon. However, under cyclic loading, the damage is generally limited to a relatively few wide slip bands compared to the widely distributed networks under steady state strain. These become incipient cracks and potential sites for the final fracture in a small fraction of the fatigue life measured in cycles. Under repeated loading the algebraic net plastic strain is nearly imperceptible, but the summation of the absolute values of plastic strain over a large number of cycles has been found to be large for ductile steels.

As a class, steels exhibit an endurance limit, that is, an amplitude of fully reversed stress below which failure will not occur in a finite number of cycles. Unless corrosive effects are present, steel that has not failed at $10^7$ constant-amplitude cycles will generally not fail. For smaller numbers of cycles (low-cycle fatigue), higher amplitudes can be withstood. In Fig. 6.3.4-1 the curve A is the average of a large number of tests on many types of steels in the form of a smooth bar subjected to fully-reversed axial stress.

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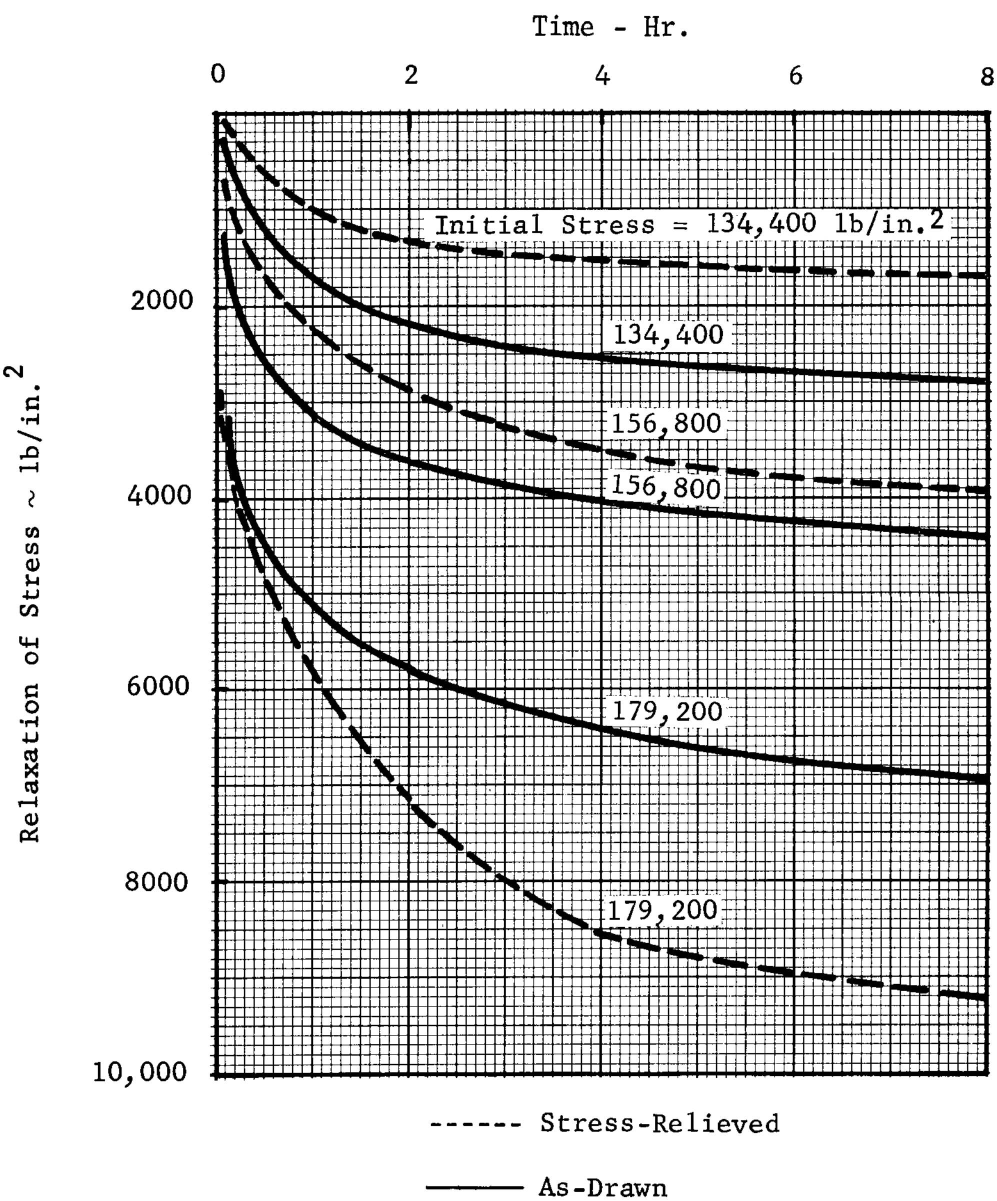


Fig. 6.3.3-2. Effect of Initial Stress on Relaxation of Stress for 0.200 diam Solid Wire (Ref. 73).

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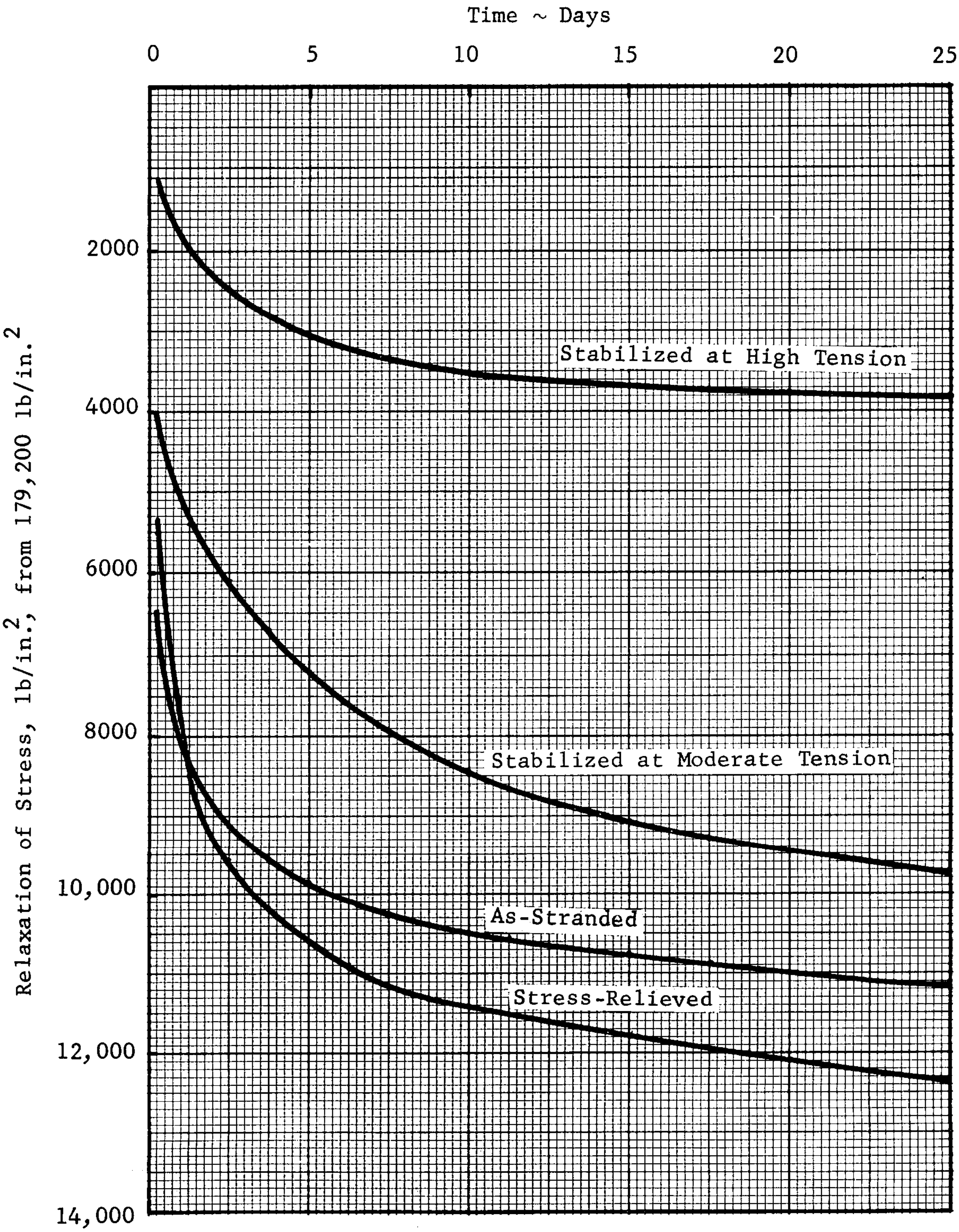


Fig. 6.3.3-3. Relaxation of Stress vs Time for 19-Wire Strand, as Affected by Treatment (Ref. 73).

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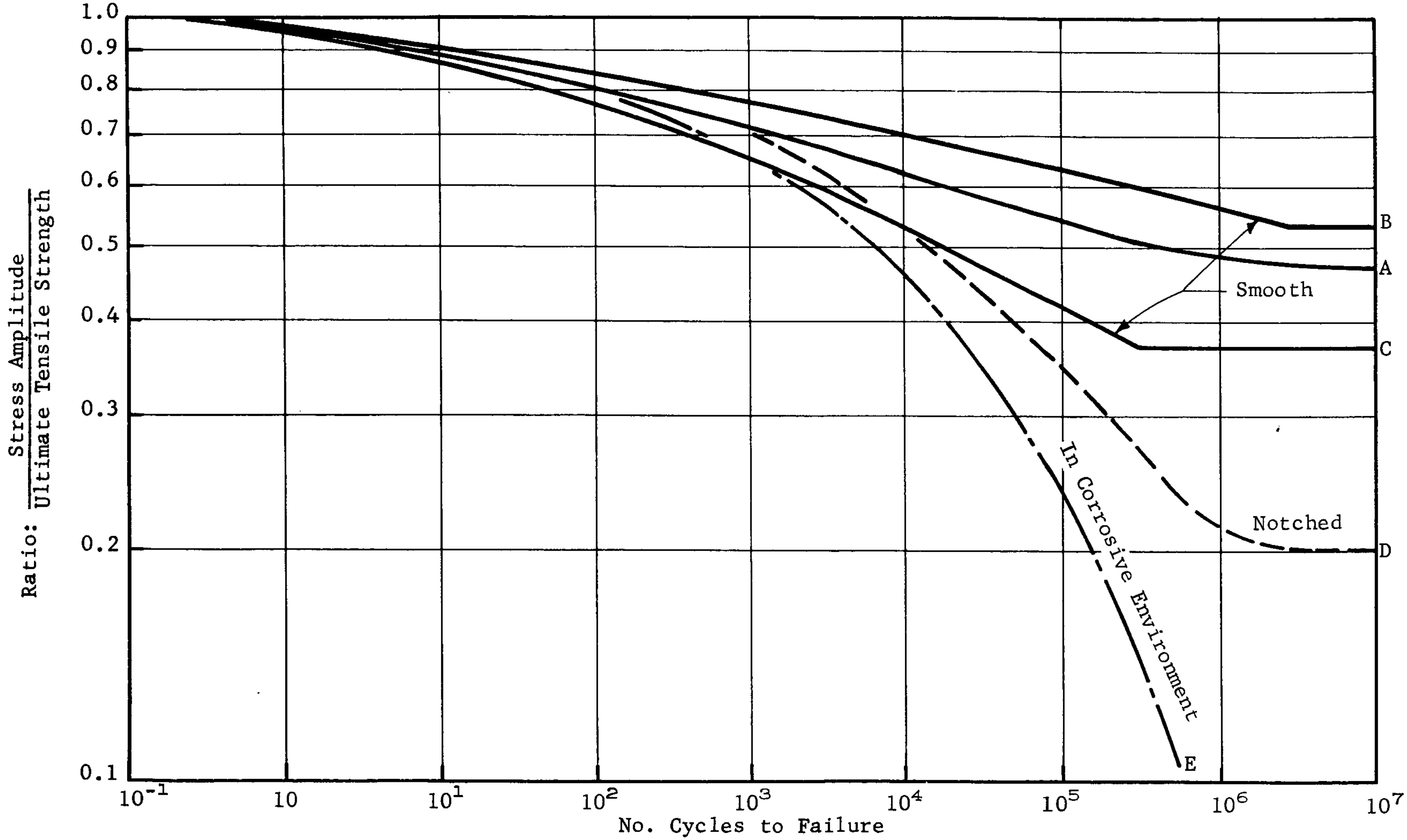


Fig. 6.3.4-1. Stress Amplitude Ratio vs No. Cycles of Completely-Reversed Axial Stress.

The dimensionless plot shows the ratio of the stress amplitude to the ultimate tensile strength vs number of cycles to fracture for a push-pull test, extending from unity at a quarter cycle (the tensile test) to approximately 0.47 at $10^7$ cycles. To get the curve shown, the low-cycle points would have been strained at rates close to that at which the tensile test was made; the high-cycle right-hand part of the curve is probably insensitive to strain rate except in the in the unsteady-creep regime.

The scatter band between broken curves B and C will include substantially all such tests on steel. In notched specimens, the restraint afforded by the material beside the notch (a biaxial restraint of slip) seems to suppress the influence of the stress concentration at the notch until $10^3$ or $10^4$ cycles are reached. At larger numbers of cycles the fatigue strength suffers increasing reduction from the notch as in curve D, where the "fatigue stress-concentration factor" at $10^7$ cycles would be .46/.20 = 2.3 for this particular geometry of notch. The tensile half of the cycle seems to open the structure (particularly at incipient cracks and grain joints) to corrosion, so that if moisture or corrosive ions are present, the endurance limit disappears and the high cycle part of the test points might lie along curve E, or even lower for slow cycling in seawater, for example. Long holding time per cycle under creep conditions will produce a similar effect on the test results, but it should be recognized that stress corrosion and unsteady creep border upon separate phenomena from those at work in fatigue fracture.

To design prestressing tendons and their anchor systems, it is necessary to consider the difficult problem of the interaction of steady and cyclic loading in steel. Suppose that pressurization and heating cause stress in a tendon to rise from $S_{min}$ at A to $S_{max}$ at B in Fig. 6.3.4-2, and after a long period of base-load operation the shut down operations return the tendon to the stress level $S_{min}$ at C. In effect this represents a steady or mean stress $S_M$ upon which is superposed an *alternating stress of amplitude* $S_A$ where

$$S_M = \frac{S_{max} + S_{min}}{2}$$

and

$$S_A = \frac{S_{max} - S_{min}}{2}$$

The diagrammatic representation of safe limits for the interaction of steady and cyclic stresses in machine parts is evolving somewhat along the lines of Fig. 6.6.4.3. *It is rather realistic in a range of applications* sandwiched between the upper bound of the nil ductility transition region where brittle fracture is possible and the lower boundary of the region of

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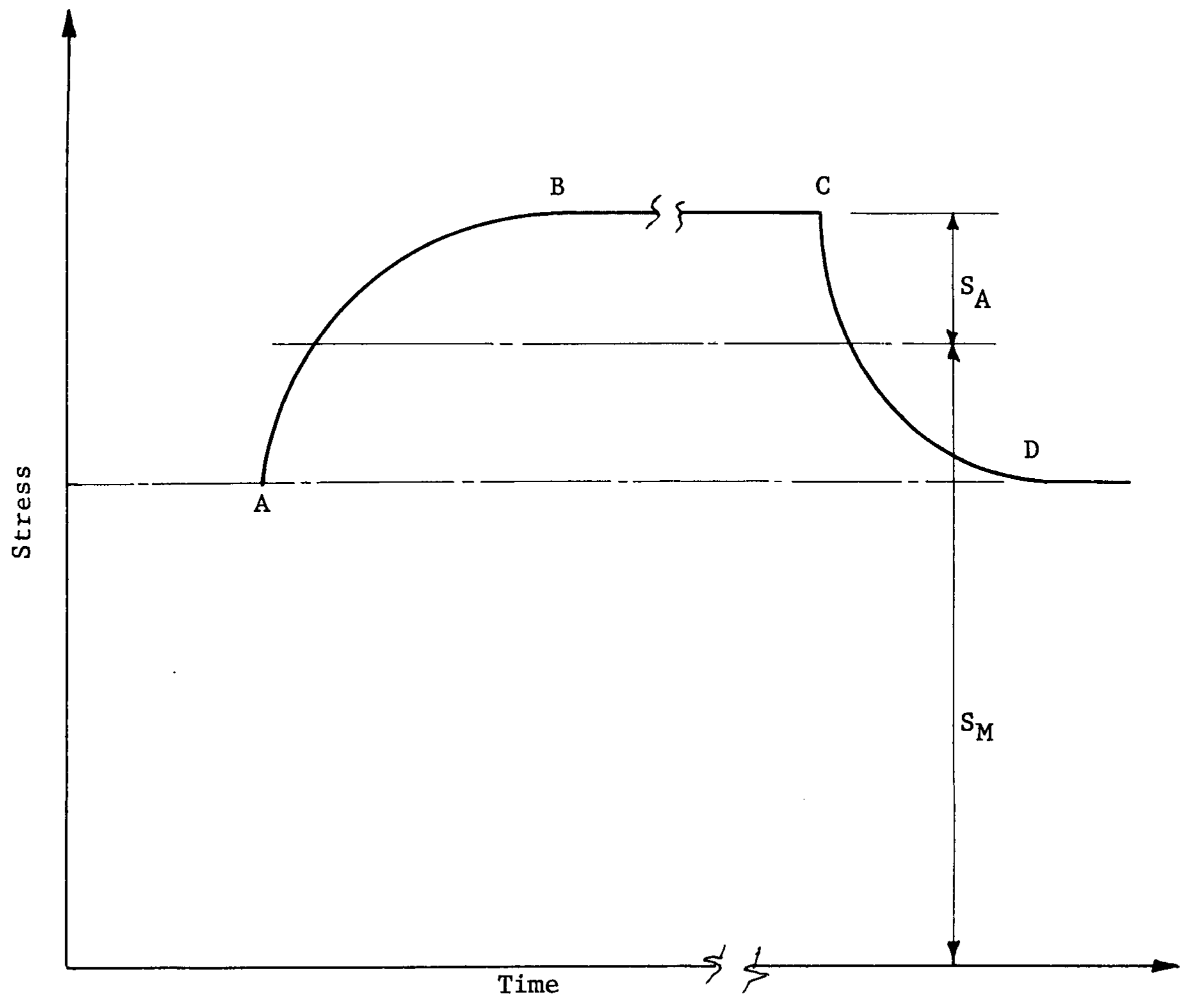


Fig. 6.3.4-2. Cyclical Nature of Stress Variation.

high temperature and/or high stress where creep damage, phase changes, or loss of dimensions may play dominant roles. The failure line from experimental investigations into completely-reversed-stress fatigue is a skewed parabola characterized by a small negative slope at its intercept with the alternating stress axis. This intercept $S_f$ is then the fatigue strength of a smooth specimen of the material in question undergoing completely-reversed axial stress for the number of cycles for which the particular diagram is constructed in the environment of interest. $S_f$ and $S_u$ (the minimum ultimate tensile stress) have received more attention than the segment between them; the chord joining these intercepts is a conservative approximation of the failure curve.

$S_a$, the allowable fully-reversed alternating stress amplitude for a structural component free of stress raisers, is chosen as a fraction of $S_f$ that recognizes the variations possible in its fabrication and the margin of ignorance of the application. If a chord joins $S_a$ and $S_u$, the safe triangular region included between it and the axes will have a right hand tip where dimensional integrity is unsatisfactory, for example, a tendon might relax so that design prestress would be impaired. If $S_b$ is defined as the limit of the minimum stress at which satisfactory elastic behavior is assured, then a line at a negative slope of unity through $S_b$ will exclude this inadequately elastic tip.

For multiaxial stressing such as exists at an anchor of a tendon, the maximum-shear theory should predict the strength of steel adequately. Solderberg[76] has suggested an approach for evaluating fatigue strength of steel. The plots of each of the three algebraic differences of principal stresses vs time must be searched for maximum range of the several classes of cycles that characterize the service. Corresponding to each class a number of cycles, amplitude of alternating stress and mean stress intensity will be established. From fatigue tests on the actual component the equivalent fatigue stress-concentration factors k may be obtained by which the alternating stress intensities must be multiplied. Then the points corresponding to each of the three mean stress intensities and their corresponding $k\ S_{alt}$ amplitudes must fall within the safe region of Fig. 6.3.4-3 to be acceptable.

It is unlikely that the exact stress distribution in a strand at its anchor can be predicted analytically or measured. Hence the fatigue performance of a combination of such components over the proposed ranges of amplitudes and mean stresses and other operating conditions under which they will be used must be probed by direct fatigue experiments if economic but safe application is to be achieved.

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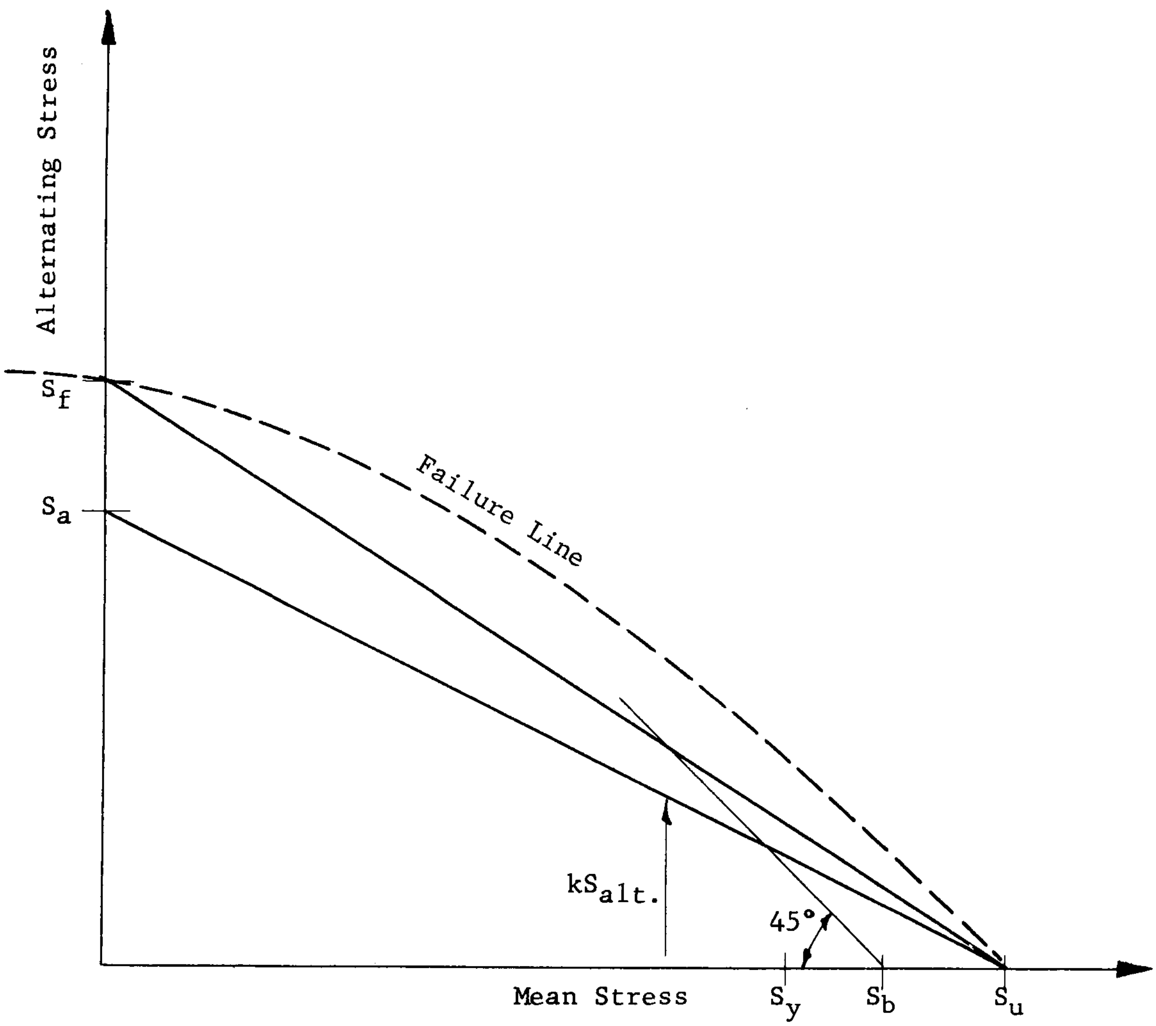


Fig. 6.3.4-3. Limits of Interaction of Alternating and Mean Stress Intensities in Fatigue.

# 7. Construction Practices

## 7.1 Concrete Placement

The builder of a PCPV for a nuclear reactor is challenged to apply many skills of planning, organization, and execution in order to meet the specifications of the designer and to produce a large, complex, safe, useful structure in the most economic manner. A basic requirement is realistic coordination of the design provisions with the problems of construction. For example, the size of control rod openings and cost of the system to accommodate misalignment might be reduced by tolerances on the concrete that would be unduly burdensome to the builder. Excessively high minimum-strength requirements for concrete might ease the designer's problem in a certain region of the structure, but such requirements exact a notoriously stiff penalty for slight deviations from the optimum combination of mix design and placement techniques that are needed to meet them.

Accuracy Requirements. The largest factor in meeting dimensional tolerances is the geometrical precision and rigidity of the formwork. The interface with the curing concrete must not deflect excessively under wet hydraulic pressure or subsequent reactions from the shrinking concrete. These considerations put restrictions on the rate and sequence of placing concrete, the curing practices, and the program of removing forms.

Although green concrete creeps readily under reactions from the forms, these reactions must be available at small deflections. It would be appropriate to foster a cooperative effort between designer, inspector, and builder in order to foresee and record the effects of heat of hydration, differential shrinkage at structural discontinuities and placement joints, and the general influence of early shrinkage on the departure of final dimensions from those of the fo mwork, or from nominal design dimensions.

Formwork. The ACI committee on formwork has summarized[77] comprehensively formwork problems, present practice and its need for improvement, the responsibility of the engineer-architect, and suggested specifications for design, construction and materials of formwork. Forms for special structures, including mass concrete, are discussed together with particular techniques such as prepacked grout, slip forms, permanent forms, forms for prestressed concrete construction, and composite construction. Although it is general, the report presents the difficulties with formwork and the approaches to overcoming them clearly.

In addition to hydraulic pressure normal to the interface and shrinkage reactions, vessel formwork may be subjected to placement impact, vibration, earth pressures, traffic, and wind loads. It must support securely the reinforcement steel, ducts for prestress tendons, liners for penetrations and closure openings, and inserts for support of internals. Leakage would cause adjacent unsoundness of concrete.

The liner and its stiffeners and shoring offers a framework that may permit assembling many of the foregoing insertions in advance of placement on the site. In a similar way, the prestressing pilasters might take advantage of shop instead of field conditions and be precast with anchors and reinforcing steel precisely located and with their own prestressing systems tensioned. On arrival and installation at the site, they would become permanent form members. The cost and substantial nature of formwork for massive concrete favors the procedure of incorporating it into the design as permanent forms, excepting temporary shoring, falsework and guying components. If the paneling of the outer form is steel, it would have some value as a protective lagging for a PCPV.

Crowding at a vessel site, as on ship ways, favors prefabrication of form panels with their stiffening in conveniently-sized modular units. Improved conditions in the shop over "in the air" at the site suggest that the main framing of the formwork ought also to be considered for prefabrication in packages not unduly troublesome to transport.

Because the hydraulic reactions on the inner and outer forms are of the same order, and the potential of the outer form to cooperate with the inner in resisting shrinkage, the total formwork structure might be reduced by a closer pitching of radial ties. Since such ties, if left in place in the concrete, would inhibit buckling of the liner, and could serve as shear steel, producing a desirable triaxiality of reinforcement, it is worth attempting to devise means of insuring them from becoming, through subsequent bond failure, the sites of leakage paths for radiation or atmospheric moisture. Such means might take the form of bifurcations at narrow whiffle-tree blocks. Against moisture, small plate collars welded to the steel and imbedded in the concrete would preserve the bond as well as act as waterstops.

Reuse of forms, while attractive as a saving in material, probably would drag out the placement schedule. To avoid elaboration of falsework, recourse may be had to the earlier placed portion of the vessel and to increased cross-tying for support of working forms. Slipforms represent the ultimate form reuse since a constant cross-section structure such as a bridge pier, tank, or tunnel is, in effect, placed as a continuous member. When slipforms are used, an elaborate jacking and guiding apparatus is required that will pay for itself only if kept in continuous use.

Placing. Concrete must be placed and vibrated or rodded with dispatch after introducing water to the mix in order to develop its potential strength. Handling and placing methods must avoid segregation of aggregate or bleeding; either results in excessive void formation and weakness. Mix design must be in accord with the handling and placing technique. A number of concurrent processes and inspections must be programmed and coordinated for successful results.

In the conventional method batches from the mixer are placed in hoppers and fed by gravity to buggies that place it in the forms from just above the concrete in place, or if the forms are deep, through elephant trunks. The concrete is not allowed to fall freely more than a few feet to avoid segregation of coarse aggregates. For a thick-walled PCPV the work will be open enough so that direct vibration of the concrete (rather than of the form) would be practical. Vibration permits low voids with low w/c ratio, but must be halted before bleeding is induced. The prestressing ducts must not be deformed or punctured by the rodding or vibration means.

Special mixing and pumping equipment is available for placing ordinary concrete (as well as grout) through a pipeline, an extension of the conventional placement method.

Difficulties with segregation and with voids in complicated or inaccessible shapes have caused development of the prepacked method where the form is filled with coarser aggregate (wetted) and a mortar grout, often containing agents to lower its viscosity and increase its wetting power, is injected or intruded into the form, generally from the lowest point, with a positive-displacement pump. Maximum possible strength in the massive sections is compromised in order to upgrade the concrete in the problem regions.

Puddling has also been used in placing large or dense aggregates such as steel punchings in shielding. The sequence is a reversed intrusion; the aggregate is vibrated into a previously placed mortar bed.

Grouting joints of composite construction or prestressing tendons in their ducts requires care in providing clear passages, a thin neat cement, or at most, low proportions of finest aggregates, and adequate pressure. Surfaces prone to suction must be tempered by presaturation. Adequate clear vents at the extremities of the flow paths must be provided and checked for complete filling before cutting off the grout pressure.

Pneumatically-placed mortar has provided a dense, impervious cover for helically wound continuous wire tendons. The dry cement-sand mix is blown through a nozzle wherein the water is introduced by pressure and the turbulent stream of rather dry discrete particles allowed to impinge on the work with appreciable velocity head.

Although it would be theoretically possible to place the concrete for a large PCPV monolithically, such a feat in human foresight and cooperation, with requirements for extensive stockpiles and standby facilities, is not essential. A good joint can be made in green concrete by chipping away the top 2 or 3 inches of the previous lift, cleaning, and grouting with neat cement before proceeding with the placement of the next lift. A mature concrete joint needs care in cleaning the exposed aggregate and in curing the new lift to avoid damage to the new bond by the shearing forces set up in drying shrinkage, though fortunately, the younger concrete will creep readily in this critical period.

Admixtures are available that will delay or inhibit setting and bonding. Added to the final course of a lift of concrete, they will facilitate cleaning the aggregate to obtain a good bonding and keying with the subsequent course. Both intrusion and puddling also offer means of getting partially-exposed clean aggregate at the top of a lift.

The possibility of simplification and better control of process variables lends attraction to the procedures of composite construction with factory precast members, assembled with grouted joints and finally prestressed. For example, the barrel of a vessel might be formed of precast staves, or of ring-shaped laminae that could themselves be segmented. Such approaches to prestressed masonry might become feasible if non-shrinking, dense, and adherent grouts were developed. However, the problems of sealing prestressing tendon openings across such joints are rather formidable for a PCPV. Composites of individually prestressed units are now widely used for buildings and bridges. Development work on expansive cements for self stressing is being pushed.

Curing. A wet or sealed environment is needed for concrete during its curing period. This is a relatively easy requirement to meet in the case of massive concrete. Heat of hydration is more difficult to control in high-strength mass concrete placed at high rates. Even with lean mixes in dams the internal heat generation produces a 50°F temperature difference that persists for months. Differential shrinkage and thermal stresses may induce excessive cracking while curing. Remedial provisions have included low heat cements and internal water-cooling pipes. It has been suggested[78] that limiting this cooling to the surface and to the early (high creep capability) portion of cure would engender a final residual compressive stress at the surface (a biaxially-prestressed skin) that should tend to inhibit tensile-cracking tendencies. It does not appear desirable to increase the compression at the inner surface by this method because of the likelihood of crushing the concrete and thus damaging the liner support at structural discontinuities during prestressing after curing is completed. Externally, complete elimination of cracking is less compelling, but the foregoing technique may be useful in regions of high local external stresses.

## 7.2 Prestressing Steel Installation

### 7.2.1 Cable Size and Handling

Representative linear prestressing systems are classified in Table 7.2.1-1 by Lin.[79] The principal division is between post-tensioning if the concrete itself supplies the opposing reaction for stretching tendons, and pretensioning if other means are used. Almost all pretensioning is done in prestressing benches to which the form must be brought. Such benches require a large investment in foundation and framing, but are appropriate and versatile for mass production of items readily transported. Occasionally the mould supports the pretensioning force, for example, in centrifugally-cast prestressed components. The Shorer and the Chalos systems carry the pre-tensioning force of a group of tendons on a centrally located steel tube that is removed after transfer of load. During the casting operation, both pre-tensioned wire and strand are generally held by serrated wedges. Strand-vise is a patented quick-opening clamp. The Dorland clip with wire inside is crimped sinusoidally by a die to obtain short transmission lengths with larger wires.

Actual stressing of the tendons is generally done with hydraulic jacks that suit the size and number of wires to be pulled and the geometry of the anchors used in the particular system. Magnel's jack stresses the wires in pairs. The Freyssinet jack, using two coaxial pistons, pulls as many as 18 wires at once. The outer piston achieves the desired wire tension and the inner piston drives the conical wedge home to force the wires against the internally serrated outer cone.

The Leonhardt and Billner systems dispense with patent anchors by casting the concrete in two parts separated by a slot in which jacks are placed for prestressing. In the Billner system the jacks are removed after partially grouting the slot. Simple loops at the ends of the tendons transfer the pre-stressing load. Leonhardt water jacks are filled with grout and remain a permanent part of the structure.

Both electrical heating and chemical expansion are potentially usable for prestressing and both have been used in prestressed concrete installations. The former takes advantage of the electrical resistance of the wire to raise its temperature by passing a current through it. While the wire is heated the grout is applied and allowed to set. Cooling the wire causes it to contract, pulling the concrete into compression through the steel-cement bond.

Table 7.2.1-1. Linear Prestressing Systems
(Ref. 79)

| Type | Classification | Description | | Name of System | Country |
|---|---|---|---|---|---|
| Pre-tensioning | Methods of tressing | Against Buttresses or Stressing Beds | | Hoyer | Germany |
| | | Against Central Steel Tube | | Shorer | U.S. |
| | | | | Chalos | France |
| | Methods of Anchoring | During Pre-stressing | Wires | Various Wedges | |
| | | | Strands | Strandvise | U.S. |
| | | For Trans-fer of Prestress | Bond, for small wires | For most Systems | |
| | | | Corru-gated clips, for big wires | Dorland | U.S. |
| Post-tensioning | Methods of Stressing | Steel Against Concrete | | For Most Systems | |
| | | Concrete Against Concrete | | Leonhardt | Germany |
| | | | | Billner | U.S. |
| | | Expanding Cement | | Lossier | France |
| | | Electrical Prestressing | | Billner | U.S. |
| | | Bending Steel Beams | | Preflex | Belgium |
| | Methods of Anchoring | Wires, by Frictional Grips | | Freyssinet | France |
| | | | | Magnel | Belgium |
| | | | | Morandi | Italy |
| | | | | Holzmann | Germany |
| | | | | Preload | U.S. |
| | | Wires, by Bearing | | B.B.R.V. | Switzerland |
| | | | | Strescon or Prescon | U.S. |
| | | | | Texas P.I. | U.S. |
| | | Wires, by Loops and Combination of Methods | | Billner | U.S. |
| | | | | Monierbau | Germany |
| | | | | Huttewerk Rheinhausen | Germany |
| | | | | Leoba | Germany |
| | | Strands, by Bearing | | Roebling | U.S. |
| | | | | Wayss & Freytag | Germany |
| | | Bars, by Bearing | | Lee-McCall | England |
| | | | | Stressteel | U.S. |
| | | | | Finsterwalder | Germany |
| | | | | Dywidag | Germany |
| | | | | Karig | Germany |
| | | | | Polensky & Zollner | Germany |
| | | | | Wets | Belgium |
| | | | | Bakker | Holland |

This method requires fairly high temperatures and is unsuited to hard drawn wire because of annealing effects. The use of calcium sulfoaluminate expansive cements cause concrete to expand during the curing process, and if the concrete bond with the steel sets up prior to the expansion then the effect is similar to that caused by heating the steel, since differential expansion leads to compressive loads in the concrete. These techniques eliminate frictional effects as they are not dependent upon steel slippage to transmit the tension load to the steel, but they have, in common with all other grouted prestressing systems, the disadvantage that no measurement of the prestressing effectiveness can be attained in place and retensioning or replacement of the cable is wholly impractical.

The most diverse field for invention has been associated with methods for anchoring the tendon in the post-tensioning system. The frictional grips are universally dependent on wedges arranged to be further tightened by the pull of the tendon after transfer. The Freyssinet system holds a circular array of wires with a single wedge. The Holzmann system uses a single wedge to hold a stack of layered wires sandwiched between filler plates giving a close-pitched rectangular lattice of wires seemingly with good space efficiency in the friction anchorage zone. All these systems require direct friction force between the prestressing steel and the wedged surfaces, and are therefore of questionable value for stranded cable.

B.B.R.V., Strescon, and Texas P.I. all use spheroidal upset buttons in the wire which bear in suitably-contoured counterbored holes on a bearing plate through which the wires are threaded. Chocks back of the bearing plate maintain the desired strain.

In the loop classification, it has already been mentioned that the Billner and Leonhardt systems simply use a return bend in the concrete to anchor each pair of wires. The Leoba system provides a threaded stud carrying a tee head with symmetrically-disposed saddle grooves for an equal number of wire loops on either end of the head. Monierbau casts the spread wires in zinc or alloy within a steel cone having a threaded end rod much in the manner of the Roebling strand anchor shown at the top in Fig. 7.2.1-1. All the threaded-end-rod systems provide space for the jack socket to be threaded on beyond the nut. Roebling varies this by using a threaded coaxial hole for the jack as in the second and last sockets of Fig. 7.2.1-1, shortening these units.

Lee-McCall is typical of the bar systems that attempt to develop nearly the full strength of the bar by a long forward taper in the threads of the bars, couplings, and terminal nuts, the engagement being on the order of two bar diameters. Thus it is necessary, after jacking and before relieving jack, to run the nut home on the taper and then insert appropriate blocking between

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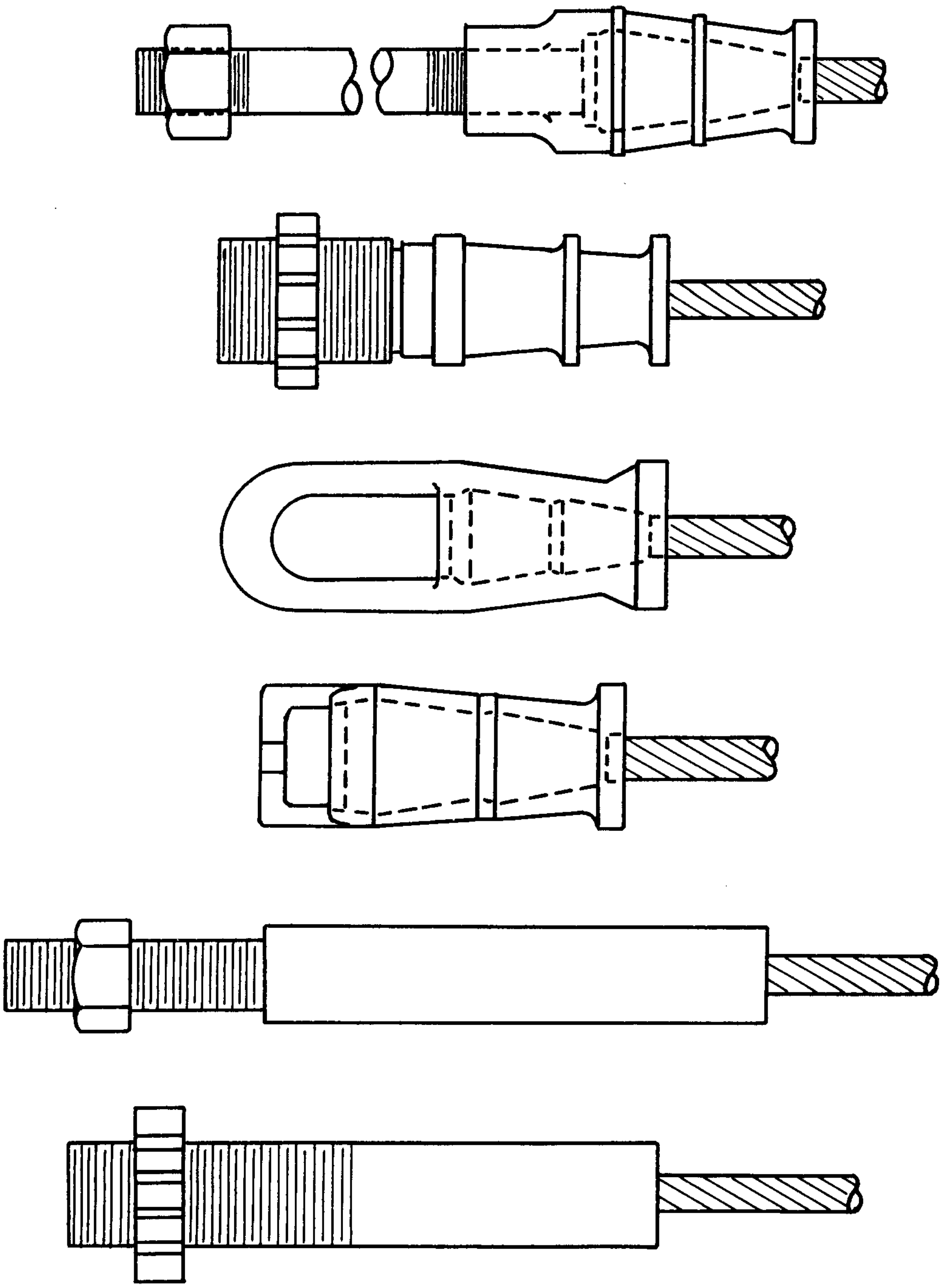

Fig. 7.2.1-1. Special Sockets (Roebling).

the washer face and bearing plate.

The underlying principle in any wire-gripping device intended to develop a high fraction of the potential strength of the wire must be a gradual application of lateral pressure and deformation proceeding away from the strained length so that the lateral pressure does not become great until the integrated shear flow into the grip has materially reduced the tension in the wire. In this way the algebraic difference of principle stresses is raised as little as practicable above the wire tensile stress. Although this stress-raising phenomenon has little effect in a ductile material in a tensile test where the load is increased monotonically, it is critical for cyclic loading. Since complete grouting of tendons is not attractive in a nuclear reactor vessel, the choice of method of holding the wires must be partly guided by the behavior of friction grips, button heads, and low-melting point alloy embedded sockets under cyclic loading.

A very practical consideration in choosing a system of prestress is its space efficiency. In the run of the tendons it is desirable that they be concentrated into a minimum number of groups to consolidate the space for placing and vibrating the concrete, for installing reinforcing steel, and to provide general accessibility for construction and inspection operations. In the anchorage region, poor volume-efficiency of the anchorage devices creates a highly subdivided concrete phase that will require a quantity of steel to transfer the prestressing load to a substantial concrete zone capable of sustaining it, or alternatively an extended stepped-pilaster construction with anchors in staged groups exterior to the structure and thus wasteful of prestressing steel and supporting structure.

Space efficiency seems to favor the use of steel strand in a socket grip for the main tendons of a large high-pressure reactor vessel.

Any of the friction grip, button-head, or socket systems, if left ungrouted, may be retightened without disturbing the wire-anchor interface in a manner that would materially aggravate the cold-work damage to the wire. The adjusting threaded member or chocking can easily be proportioned to exceed the strength of the tendon that loads its.

The Preload wire winding system is adapted to cylindrical water tanks having a vertical axis; it might be considered for circumferential prestressing of reactor vessels. A single wire is wound on the outer surface of the tank in a spaced helical layer. Prestressing tension is maintained by drawing the wire cold through a die mounted with the wire reel on a carriage that propels itself by a drive chain wrapped around the tank. The wire is subsequently protected by a layer of pneumatically-placed grout; additional layers may be superimposed as required. The system eliminates the cost and friction losses in anchors, but retightening would appear to require a winding with new wire.

Table 7.2.1-2 lists sizes and space-efficiency data concerning the largest tendons that can be accommodated per anchor of commercially-available systems; the first four use solid wires, the last three systems employ groups of strands.

A PCPV demands the post-tensioning treatment, because of the magnitude of the total prestressing load on an item that is not being considered for mass production. In this method a duct for each tendon is cast into the concrete along with local reinforcement and load-spreading steel for the anchor at each end. The tendon may consist either of one or more smooth wires, or of one or more multi-wire "strands" as the mill-twisted product is known.

Although there are many schemes for forming the passage for an internal post-tensioning tendon, the most economical satisfactory method of effecting freedom of the tendon for tightening appears to be the flexible metallic tube of galvanized mild steel. It is positioned and firmly supported with the tendon in place before placing concrete. The tendon ought to be galvanized and greased at the mill for corrosion protection during shipping and installation. If the placement plan requires deep lifts of concrete, rigid tubing may be required for duct lining in order to prevent inward leakage of concrete into the duct. Such leakage would interfere with post-tensioning operations and would prevent completion of grouting where grouting is planned.

### 7.2.2 Antifriction Measures for Prestressing Cables

The use of curved prestressing tendons introduces a frictional component to the applied prestressing forces. When the friction factor is high, as would be expected with dry surfaces, the effective tensile load may be materially reduced by these friction losses. One of the merits indicated for the Preload "wire winding" technique is that the wire is not required to slip over the prestressed surface during the prestressing because the wire is locally stressed by a slight amount of wire drawing during the wrapping operation. For other types of cable the effective prestressing can be increased if friction losses in transmitting the prestressing force throughout the cable are minimized by antifriction techniques. In the G-2 and G-3 designs, lubricated sliding shoes were attached to the cable and allowed to slide along the surface to reduce this frictional component. The sliding shoes were lubricated with molybdenum disulphide grease and the coefficient of friction is reported[80] to be in the range of 0.02 to 0.025. The problem in G-2 and G-3 is considerably more severe than in most other vessel designs because the cable encircles the vessel. At Oldbury where a helical wrap is employed a lubricant is applied directly to the cable midsection to reduce friction losses. The Du Pont designed Savannah River containment shell, in which cables covered a 90° arc between anchoring pilasters, utilized rust inhibitive grease to reduce friction.

Table 7.2.1-2. Prestressing Systems
(Ref. 25)

| System | No. of Wires | Wire Dia. | Area(A1) sq. ins. | Working Load (56% Ult) L. tons | Duct Area (A2) sq. ins. | Anchor Space (Anchor crs)[2] (A3) sq. ins. | $\frac{L}{A2}$ T/sq. in. | $\frac{L}{A3}$ T/sq.in. | Proportion of Load Pulled by Single Jack | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| Parallel multi-wire with threaded anchor bolt | 66 | 7 mm | 3.95 | 221 | 7.069 | 198 | 31.3 | 1.115 | Whole | - |
| Parallel multi-wire with Magnel-Blaton anchorage | 96 | 7 mm | 5.75 | 322 | 35 | 362 | 9.2 | 0.915 | 1/48 | Separators in ducts |
| Parallel multi-wire with nail head at anchor head | 55 | 7 mm | 3.29 | 184 | 15.9 | 256 | 11.58 | 0.72 | Whole | - |
| Parallel multi-wire wedged into tapered holes in bearing plate | 38 | 7 mm | 2.375 | 133 | 5.94 | 81 | 22.4 | 1.642 | 1/38 | - |
| | No. of Strands | Strand Dia. | | | | | | | | |
| Parallel multi-strand with Freyssinet anchorage | 12 | 0.6" | 2.64 | 153 | 7.069 | 169 | 21.7 | 0.906 | Whole | - |
| Parallel multi-strand wedged into tapered hole in bearing plate | 7 | 0.7" | 2.282 | 140 | 5.94 | 81 | 23.6 | 1.73 | 1/7 | - |
| Parallel multi-strand with Magnel-Blaton anchorage | 27 | 0.5" | 3.89 | 250 | 20 | 228 | 12.5 | 1.098 | 1/27 | Separators in duct |

A friction loss of 10% of the restressing load was assumed in the Du Pont design analysis. These practices indicate that, while friction must be considered in the design of prestressing systems, techniques are available to hold the losses to reasonable levels.

### 7.2.3 Grouting

The practice of grouting the tendon in place by pumping the duct full after post-tensioning has been widely adopted to insure against any further tendency toward slippage at the anchor and to furnish protection for the tendon from corrosion. Further, the claim is advanced that necking or fracture of a wire would then cause loss of prestress only in the region included within the transmission length on either side of the failure. On the other hand, to obtain effective bonding in reasonable transmission length, the members of the tendon must be spaced so openly that the poor volume efficiency of the tendon will be a serious burden during construction in a restricted ligament or critical region requiring heavy reinforcement. All possibility of retightening a particular tendon is lost by full-length grout. It is believed that such full-length grouting would be out of place at present in a nuclear power application, because the long-time effects of temperature on shrinkage and creep, and thus on loss of prestress are not presently known. Possibly the assurance of slow progressive failure of the vessel might be lessened by grouting. If stresses during an unforeseen operating incident caused cracking to propagate to a tendon normal to the crack surfaces, this tendon would have less ability to absorb the strain input from the opening crack without rupture if grouted than if free to elongate between its anchors. It must be conceded, however, that experimental evidence indicates that great potential for controlling tension cracking is possessed by filaments of bonded steel. A factor in this is the potential of the bond during overpressure to raise the local stress in the steel to the ultimate before a crack gets very wide, provided the short-time transmission length is short. On the other hand, with ungrouted steel, the steel tension would not rise materially above the prestress value before the tensile strain in the concrete becomes so high as to cause general opening and propagation of the tension cracks. This would give warning of distress, but at a cost of considerable permanent damage.

## 7.3 Liner Installation

A steel sealing membrane to assure negligible leakage of the coolant and to avoid contamination of the coolant by dust from the concrete is an established feature of prestressed-concrete nuclear-power reactor vessels currently in operation or under construction. Such a membrane would also prevent pressurizing of the concrete pores, a condition threatening the structural integrity of the concrete in the event of sudden depressurization of coolant. It appears likely that the foregoing aims, as well as good thermal continuity and well-distributed support for the liner can be most readily realized by placing the liner against the inner surface of the concrete of the vessel wall.

Although a very thin liner of sheet steel that would be gas tight could be rolled and fabricated, it is desirable that it be thick enough to be stable with a reasonable span between the members connecting it to the concrete. For convenience and dispatch, it appears desirable to be able to erect the liner with its supports, the wall-cooling piping harness, and the internal falsework necessary to enable the assembly to serve as the inner form for the concrete placement, at a station well to one side of the foundation site. Thus when the foundation is complete, the liner unit would be scheduled also to be ready to set over it.

The design, construction, inspection and testing of the liner might easily fit the practices approved for pressure vessels for nuclear applications. Qualification of welding procedures, radiography, and leak testing would adhere to the ASME Power Boiler Code and its nuclear interpretations. Particular attention would be devoted to accommodating incompatible strains of the liner and the prestressed concrete members of the vessel in a safely elastic manner, yet with adequate support of the liner to avoid excessive bending stresses either from lateral coolant pressure or from instability of the liner spans.

## 7.4 Cooling

A concrete reactor container may be kept relatively cool by insulation and a coolant such as water. The container at Oldbury, United Kingdom will have a dimpled stainless steel foil to protect the liner and to limit the temperature in the wall to 65°C.[81] In addition, it will have a water cooling system consisting of 3/4 in. parallel pipes embedded in the concrete. They will be arranged vertically on a 12-inch pitch and will terminate in a series of headers where the flow will be controlled. Coiled pipes near the access ports will offer additional cooling in these regions.

The cooling system of the vessel will serve to remove both the heat entering the liner from the primary coolant and a large part of the heat arising from radiation of the concrete. Hence good thermal conduction is needed from the liner to the cooling pipes as well as from the concrete of the vessel wall to the pipes. If the structure that supports the liner is designed to hold the cooling conduits during placement of the concrete, it would be easy to provide adequate thermal conduction between them.

### 7.5 Insulation Methods

The development of useful insulating techniques for application to prestressed concrete reactor vessels does not introduce requirements which are unique because of the presence of prestressed concrete. In gas-cooled reactor systems where high temperature coolants are contained in carbon steel reactor vessels, a metal foil insulating material is often used to protect the reactor vessel from high temperature environments above the nominally selected working temperatures between 650° and 750°F. If the coolant can operate at a low temperature near one end of the reactor, then arrangements can be worked out to eliminate the need for insulating the vessel surface from the high temperature outlet environment. Nevertheless, an insulating technique is used even in such concepts to prevent transfer of heat between the high temperature reactor coolant outlet and the vessel cooling medium. In most water cooled reactor systems, the maximum coolant temperature is sufficiently low so that no high temperature protection for the carbon steel vessel need be provided; however, thermal transients in the vessel's metal wall are an area of concern. In a prestressed concrete reactor vessel, the insulation requirements are somewhat more severe than in steel reactor vessels because of limitations which must be placed on permissible thermal gradients within the vessel. Several forms of insulation can be visualized, but the following three types are considered to be of primary interest:

1. Metal foil insulation made from layers of stainless steel or other bright metals, separated by stagnant gas spaces.
2. Ceramic fibrous insulation, possibly in sealed metal containers.
3. Refractory concrete insulation.

The usefulness of these three devices is dependent upon the reactor concept being pursued.

The first type has been used extensively in gas-cooled reactor systems where it is relatively easy to maintain the stagnant gas layers. This insulation can be fairly expensive when made of high alloy steel foil. Its thermal conductivity is dependent upon the thermal conductivity of the stagnant gas film so that in high pressure coolants it may be very difficult to attain a thermal conductivity much below 0.5 Btu/hr/ft$^2$/°F per ft of thickness. The implied requirement will be a very thick and expensive insulation layer if temperature differences between the concrete and the coolant must be large. Insulation of this type may be more effective if it can be canned and evacuated, but no such insulation is currently available commercially.

The second type of insulation may be used interchangeably with metal foil insulating materials and several reactor concepts have been based on the use of such materials. The lower thermal conductivity of ceramic materials, together with their lower cost, indicates economic incentives for their development. At the same time deterioration of such material in the reactor atmosphere may be more severe than with all metal insulation. Placement of these materials in an evacuated metal container has advantages similar to those suggested for metal foil concepts. Thermal conductivities in the range of 0.1 $Btu/hr/ft^2/°F$ per ft of thickness should be attainable.

The third insulating device, making use of refractory concrete, is suggested as a means of easing the heat loading on the reactor vessel where insulations of the previous types mentioned are not fully effective. Refractory concretes are used in high temperature furnaces as a routine matter and can withstand temperatures in the range of 1200°F for limited periods, while retaining reasonable structural properties. If a concrete reactor vessel is to contain a coolant of high thermal conductivity, e.g., liquid water, then it would be essential that the insulation be placed in such a manner that the coolant itself does not provide a heat transfer path directly to the concrete. Refractories could be used effectively in a zone between the concrete vessel liner and the concrete vessel proper, where their thermal conductivity would be independent of the reactor coolant properties. They could effectively divorce the concrete pressure container from temperature transients and reduce heat losses to the concrete vessel cooling system.

There is little need for development work in the area of metal foil reflective insulations. These have been amply demonstrated[20,21] for gas-cooled reactor systems. However, if there is a need to lower the thermal conductivity of such insulations, then techniques for canning the foil in an evacuated atmosphere could prove extremely useful. For the ceramic fiber insulations a demonstration of canning techniques is similarly needed, but the area of greatest interest is demonstration of deterioration characteristics in reactor operating environments. A limited amount[22,23] of information is reported in the literature about this matter.

The use of refractory concrete as an insulating material has attractive economic advantages, combined with tremendous value as a means of separating the insulation from the reactor coolant environment and deserves maximum attention. Practices[71] used in furnace applications may be used as a point of departure. Since it must be assumed that insulation in this form must be subdivided if it is to withstand substantial thermal stress, methods must be developed for attaching the subdivided segments to the reactor vessel and holding these pieces in place. The arrangement of the insulating segments must be such as to support the reactor vessel liner whose integrity is dependent upon support from the external structure.

### 7.6 Inspection

The inspection practices for prestressed concrete construction have not been well defined. The ACI standards[95] provide some guidance in this area. Abels and Turner[82] list a number of specimen clauses for specifications which suggest requirements for detailed inspection. These include:

1. Reports of the strength of cured concrete specimens before applying post-tensioning.
2. Measurements of tension and length of cable strands for uniformity.
3. Measurement of tensioning force and steel elongation during post-tensioning.
4. Inspection for cracks prior to and after prestressing.

Because these inspection requirements are directed mainly at prestressed concrete beams, some further amplification is in order. In beam construction flexural tests of specimen beams are often required to determine ultimate strength capability. In a concrete vessel a pressure proof test would probably be substituted as is the case with steel pressure vessels. The margin of pressure over design conditions is difficult to define. The test could be expected to be held below the pressure level at which tension is introduced to the concrete. Because of the substantial allowance for long time creep and shrinkage, this test condition might still approach a value equal to 1.25 times the design pressure used in the Savannah River vessel without unnecessarily penalizing the structure. Inspection of materials for flaws is important but standards are not easily established. Savannah River required magnetic particle inspection of anchors; certification of anchor strength; tensile, elongation, ductility, and yield strength of wire; and test to failure of an 8 ft section of cable from each reel: A full time concrete inspector observed the concrete installation. The final proof test of the vessel, as previously mentioned, was a stregnth test at 1.25 times the design pressure. In addition a leak tightness test over a 24 hour period at design pressure was required. These practices are consistent with those used in steel pressure vessels.

Since the Savannah River vessel represents the only nuclear precedent for concrete vessels in the United States, the background of information from which inspection standards can be developed is very limited. Wire strain is the true measure of prestress conditions and direct measurement devices which can measure wire strain during prestressing are probably necessary. Investigators at Sheffield University in England[19] have developed load cells based on photoelastic principles which are adaptable to field use in prestressed concrete installation.

The definition of inspection requirements for liners has no precedent.

Practices used in containment systems where reinforced concrete cells are lined with concrete have been limited to leak tests, but the infrequent and short time loading condi ions in such applications are not comparable to the steady state high pressure loading conditions where metal yielding is inherently a functional requirement of the liner. Elongation limits for liner materials will have to be established, fatigue characteristics will need to be understood, and this in turn correlated with fabrication requirements for the installation.

A final consideration of importance is likely to be measurement of the cooling capability in the concrete cooling system. Some measure of the effectiveness of the cooling system will need to be demonstrated since probably the structural integrity of the structure will be dependent on limiting thermal stress loading. The literature does not indicate any specific approach to this question.

## 8. Operational Control

Regardless of the type of vessel used in a nuclear system, its acceptance is contingent on assurance that the operator understands its physical conditions and can correct any difficulties prior to catastrophic failure. For steel vessels this has taken the form of continuous temperature monitoring of the vessel surface, periodic visual inspection, and the testing of environmental sample specimens. A similar requirement will undoubtedly be imposed on concrete vessels.

### 8.1 Temperature Monitoring

Direct measurement of the temperature of the concrete is readily accomplished in critical locations by imbedding thermowells in the structure. Further control can be exercised by measuring the performance of the vessel cooling system. Fortunately, sudden temperature changes cannot occur because of the large heat capacity and low thermal diffisuvity of the concrete and long periods of time should be available for corrective action.

### 8.2 Prestressing Behavior

Some loss of prestressing is to be expected over a long period of operation and practical design will include allowance for these effects. For the purpose of monitoring, the operator will need some measure of these losses. Direct measure of prestressing steel strain appears to be the most attractive means, although ultrasonic techniques or vibrational measurements might be considered. The photoelastic devices discussed in Section 7.6 appear suited to long term usage for the purpose of operational control.

### 8.3 Liner Behavior

The operating condition of the liner is not of great concern because of safety considerations. Its failure cannot impose a serious structural effect on the vessel that would not be exposed by the prestressing monitors. However, it would be of value for the operator to know areas of weakness in the liner. The initial strength test will readily define areas of concern if the liner is instrumented with strain gages. Some form of stress coating will show up major plastic deformations. Where these are of interest for future operational information, some form of extensometer attached to the liner could be brought through the concrete structure. Thus far, however, no such proposals have been reported in the literature. An immediate indication of liner rupture during failure could be provided by installing pressure gages or coolant leak detectors in the space between the liner and the concrete structure.

## 9. Research and Development

European activities in the construction of large prestressed concrete reactor vessels for nuclear power reactors give a strong indication that vessels can be designed safely for this purpose with presently available technology. Notwithstanding this fact, there remains a large number of gaps in the information needed for the effective use of prestressed concrete in reactor vessels. These include development of design tools, a more thorough understanding of concrete properties and devices for enhancing these properties, experimental techniques for verifying design adequacy, improvements in prestressing steel hardware, refinement of devices for nondestructively monitoring concrete behavior, clear definitions of failure modes in prestressed concrete structures, and means by which concrete can be made compatible with high coolant operating temperatures. A thorough investigation of the research and development needs in prestressed concrete is beyond the scope of this presentation, but some of the more important areas of interest are indicated by this literature review and can be summarized here.

### 9.1 Analytical Techniques

A number of analytical procedures need to be adapted to PCPV design in order to lessen the burden of design calculations needed to verify the safety of an economical structure. In general the precision and range of applicability of such procedures will require some experimental investigation by model tests.

A high-speed digital computer program for stress analysis of axisymmetric thick shells has long been urgently needed for ultra-high-pressure steel vessels subject to differential heating. Such a program would be helpful in designing PCPV reinforcing tensile and shear steel in the crowded critical areas at head junctions, diaphragms, and closures. Plane strain procedures might be adapted through model tests to design of the reinforcement of large penetrations. Work is needed to extend the existing techniques for predicting the behavior of steel vessels at penetration and supports to PCPV's, accounting for the inherent anisotropy of steel and concrete composite structure. Extensive probing of the validity of such analytical procedures by correlative model and full-scale testing is hardly to be avoided.

In general, plastic behavior of structural materials is partly irreversible. The total accumulation of permanent strain usually leads to strength impairment or ultimately to strain fracture. Concrete is no exception. The principle of attempting to accommodate the operating loadings on a PCPV elastically is therefore a desirable practice. However, plastic behavior has a major effect during the concrete curing period and is an important consideration in maintaining prestress and dimensional stability under prolonged mechanical and thermal loading. It can be useful as a warning that overloading is causing microcracking, deterioration of bonding and similar preliminaries associated with progressive failure.

A body of observations concerning the inelastic behavior of small specimens and beams during curing and under load at atmospheric temperature is now available. With these data and others as they become available, the designer must attempt to estimate the corresponding inelastic behavior of his massive prestressed concrete vessel during during, after prestressing, and under the cyclic combination of loadings incident to operation. In addition to the fundamental prediction of loss of prestress in the smooth parts of the shell, attempts to estimate plastic behavior at discontinuities and match plastic strains in the members of a joint must be checked out by monitored model tests. The effect of vessel plastic deformation on the adjacent liner is of particular concern.

### 9.2 Model Studies

Concrete itself is inhomogeneous, imperfectly elastic under short-time loading, and subject to large time-dependent deformations that are but partially attributable to mechanical and thermal loadings. It has limited ability to sustain tension or shear. Its response in tension is different from its response to compression. The flow rule of Tresca (maximum shear theory) is useless for predicting the behavior of concrete under combined stress, because of the profound effect that the normal stress on a plane of shearing has on yielding. While this effect can be easily visualized diagrammatically as Mohr demonstrated, the nonlinearity that it introduces has rendered the behavior of concrete under multiaxial loading intractable for analytical procedures, particularly under creep conditions.

The addition of reinforcement and prestress increases the inhomogeneity and adds anistropy to the loaded medium. Analytical means of design by themselves are thus inadequate and new analytical attacks must be subjected to experimental checks. It is to be expected that work with models of components and of complete prestressed concrete vessels will be more extensive than has been true of steel vessels. Such models ought to be painstakingly planned to to yield a high return of information. The design should be carried out by the best available rational analytical procedures to carry test loadings and withstand test environments and durations that will closely simulate intended service conditions for the prototype.

Control should be precise during such a test. Instrumentation must be ample to record locations and manner of onset of deterioration and failure history throughout the model so that realistic interpretation of results will be facilitated and the adequacy or shortcomings of analysis and other design procedures pinpointed. Naturally the materials, plan of construction, workmanship, and inspection methods used to acquire a model, and the control and recording system of the investigation are also being probed in a model study.

The instrumentation should enable study of onset of tensile cracking, crack arrest, distribution, and control by tensile steel, bond failures, and behavior of reinforcing steel and grouted and ungrouted prestressing tendons as well as the overall strain behavior of the structure.

A given model may test two different head designs and a plurality of of penetration configurations. After elastic and plastic deformation tests have been completed, a model vessel may be overloaded in small increments while the behavior of representative members are carefully monitored to obtain the order and manner of failure for each region. Providing that damage-control measures for temporarily restoring the leaktightness and strength of regions displaying premature distress can be devised, a great deal of insight into the behavior of the prototype vessel might be gained from a single model.

If the model is too small, the bond strength and the lengths of the paths of water migration in the concrete of the prototype will be poorly represented.

It is not likely that stress models of homogeneous materials such as steel or resin will be of much help in the study of the behavior of prestressed concrete pressure vessel prototypes.

In measuring concrete stresses, the strain incompatibilities and the vagaries of the stress trajectories induced by the local variations in stiffness caused by the presence of various sizes of aggregate particles must be taken into account in choosing transducers and gage lengths. Reinforcing steel profoundly affects neighboring stress patterns. Obviously brittle-lacquer patterns and birefrigent coating fringes would be clouded by the randomly-grained rigidity variations between the paste, mortar, and aggregate domains of the concrete surface.

## 9.3 Material Properties

### 9.3.1 Concrete Creep and Shrinkage as a Function of Temperature, Load and Size

Some of the problem areas appearing to need basic research are as follows: influence of temperature level and distribution on moisture migration in concrete, modulation of this influence by imposed stress field, by high ratios of volume to exposed surface; the resultant effect of such influences of bulk and of thermal activation on time-dependent deformations of concrete when unstressed and when subjected to uniaxial and multiaxial stress; the effects on the rate of change and on the terminal values of coefficient of thermal expansion and of thermal conductivity; the response of structural concrete and prestressing tendons to irradiation with stress and temperature as parameters; effect of sustained elevated temperature on the physical and engineering properties on materials of prestressed concrete, for example, the influence of holding temperature on the level of critical sustained stress; best choices of cement varieties and types, nature of aggregate, steel, mix proportions, admixtures, and placing techniques to limit the various types of environmental damage.

#### 9.3.2 Prestressing Steel at Temperature and Under Cyclic Load

Creep tests at controlled load are probably more convenient and precise than relaxation tests under controlled strain of a suitable gage length of prestressing steel. Controlled temperature furnaces capable of sustained and uniform heating over the gage length would be a requisite part of the creep testing facility. Once the functional relation of creep with load intensity and temperature are established, relaxation performance may be integrated numerically.

Although cyclic fatigue tests may be performed expeditiously, rapid testing cannot provide insight into the effects of unsteady creep. The effect of cycled loading periods comparable to actual service history needs examination. Similarly, although metallurgical phase changes are not expected, it must be remembered that creep is the effect of thermally activated phenomena that are strongly influenced by the previous temperature history as well as the load vs time pattern. Sound generalizations of variable temperature behavior cannot be based on constant temperature data alone.

### 9.4 Vessel Openings

#### 9.4.1 Stress Distribution as a Function of Size

Exact thin-shell solution is known for the stress distribution around a radial cylindrical nozzle in a spherical shell. Bijlaard's work gives adequately precise solution for a radial nozzle in a cylindrical vessel remote from other discontinuities if the nozzle is not too large. Experimental work on steel vessels confirms Bijlaard's result that as the ratio of the radius of the opening to the characteristic attenuation length of the vessel wall increases, the discontinuity stresses rise at an incr asing rate. In spite of the introduction of inhomogeneity, anisotropy, great shell thickness, tensile weakness, and composite construction in a concrete vessel, here it can be expected that the difficulty of reinforcing penetrations also will rise at an increasing rate. Intuitively, the tensile membrane cut out by the penetration must be replaced functionally by a tensile hoop around the opening and the shell must be given bending resistance to withstand the lateral loading imposed at the rim of the opening by the hydrostatic end force of the duct. At a given pressure, the latter loading per unit length of periphery must rise with the square of the diameter of the penetration. As mentioned in Section 4, heavy composite steel members are hardly to be avoided.

Model testing must be planned to search the effectiveness of design provisions for carrying the membrane stresses, the bending stresses and the shearing loads at a penetration. Incentive exists for pushing this study into the range of large openings for gas-cooled nuclear power reactor vessels. Though a small opening of itself is not a structural problem, effective reinforcement of the ligaments of a large number of closely grouped control rod openings may require experimental investigation, somewhat analogous to the work that has been done on steel tube sheets.

9.4.2 Closure Methods

Closure design is believed to require a development program including model experiments. Reasonable quickness of access will likely demand a composite steel and concrete structure with rather complex arrangements for spreading the concentrated loading at the fastenings into the shell and cover concrete. The designer's skill in matching the thermal expansions and mechanical strains, both elastic and plastic, to achieve adequate strength in a convenient and economical manner can be sharpened by appropriate model studies.

9.4.3 Interconnection of Steel and Concrete

Any machine or structure depends for success on its designer's ability to foresee internal reactions and to provide for their maintenance throughout the life of the device. Even in its most elementary form prestressed concrete forms a closed loop with its tendons, their anchorages, and the transition zone where each anchor reaction spreads by means of the shearing strain in the anchor plate or in the transmission length of a direct bond anchor when this is used so as to distribute the reaction. There ought to be no weak region in any member to lessen the resistance of the structural loop.

The great disparities in rigidity and strength of concrete and steel have been discussed. Concrete must be reinforced to carry even low intensities of tension or shear. It is evident that in a large reactor vessel the anchorage regions and other major discontinuities will present crowded, complex transition domains where the stress diffusion pattern in, and response of the concrete to, multiaxial loading will need experimental investigation of the validity of analytical attacks.

## 9.5 Liner Behavior

### 9.5.1 Dimensional Requirements and Anchoring Methods

The liner must be supported against the coolant pressure by the vessel so that a good fit is needed without unsupported areas that might fail in fatigue. It must be keyed to the vessel in a manner to prevent rupture by creeping in the thermal-ratchet sense as pressure loading and thermal strain are cycled. To protect its fit, the liner must be prevented from buckling as the vessel is prestressed and suffers plastic deformations. Elastic means of accommodating excessive strains of the vessel at discontinuities ought to be incorporated in the liner. There are enough areas of inexperience in the functions of the liner to justify development testing in conjunction with tests of model vessels.

### 9.5.2 Cooling Methods and Insulation

The liner mechanical support and its thermal connection to the heat removal sink are intimately related and it may prove undesirable to separate these functions. The effectiveness of the heat transmission and removal means can be investigated during thermal testing of a model vessel.

A development program is needed to develop a high temperature insulation that will be effective in helium, or that could withstand vacuum canning in a high pressure coolant.

## 9.6 Material Development

### 9.6.1 High Strength Cables and Anchors

For good design, execution and inspection, a single bridge rope with a working strength of one million pounds with a clean, compact anchorage and jacking system needs to be developed.

### 9.6.2 Refractory Concrete

The development of a refractory concrete having good structural resistance would reduce the problem of cooling the vessel wall.

### 9.6.3 High Strength and Low Shrinkage Concretes

The current practice in the design of concrete structures makes the design pressure of a concrete vessel dependent upon the ultimate compressive strength of the concrete. If the current ACI value of 45% of cylinder crushing strength is taken as the upper limit on design pressure and if 5000 psi crushing strength represents the maximum strength of concrete, then

2250 psi would be the maximum design pressure for a reactor vessel. This is obviously a deterrent to use of these vessels in PWR systems where 2500 psi is a nominal working pressure. Potentially higher strength concretes are attainable by improved curing practices. Advantages may also be taken of the confining nature of prestressing to enhance the concrete loading capability. The capital cost structure of steel vessels in high pressure reactor systems provides considerable latitude for concrete strength improvement at premium cost while still providing designs which would represent singificant cost savings in a power plant installation. Efforts expended in this direction would therefore appear to be warranted.

The limitations on the useful strength of prestressing steel are to some extent determined by the allowances for inelastic deformation of concrete through shrinkage as well as creep. The development of concretes whose time-dependent deformation is lower than presently available will eliminate the steel strength premiums associated with these allowances and will be of overall benefit to the usefulness of concrete vessels.

### 9.7 Failure Characteristics

Monitoring of destructive testing of model vessels would establish failure symptoms and modes of failure in order to provide a realistic basis for safety evaluation. Failures by creation of overpressure, overtemperature, or local weakening of the prestressing would be investigated. Tests would include applications of stored energy for significant intervals comparable to those associated with depressurization accidents in pressurized water and gas-cooled reactor systems.

### 9.8 Monitoring Techniques

Suitable methods for monitoring the behavior of a vessel and its components require investigation. These include the techniques of non-destructive testing, indications of maintenance of tendon tension, and concrete crack detection and measurement.

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