Creep Losses in Post-Tensioned Concrete Masonry

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Prestressed masonry offers a competitive alternative to conventionally reinforced masonry and reinforced concrete in certain applications. While prestressed masonry is a commonly accepted form of construction in Europe, it has seen very little use in the U.S. This is mostly due to the lack of building code provisions for building officials, design guidance for design professionals, and hardware and construction experience for contractors (Schultz and Scolforo (1991)).

Prestressed (more precisely, post-tensioned) masonry is most economical when used in tall single-story industrial/commercial structures. Another potential use of prestressed masonry is sound walls for separating residential area from noisy highways. Unbonded post-tensioned masonry walls could provide savings in time and materials over conventional reinforced masonry construction. Figure 1 shows typical details for a post-tensioned hollow-unit wall.

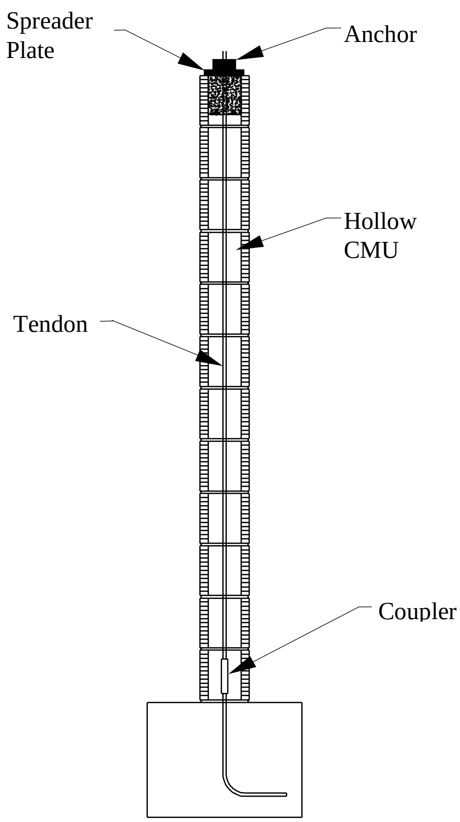

Figure 1—Post-Tensioned Concrete Masonry Wall

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As in prestressed concrete, prestressing losses reduce the effective prestress present in the tendon. In post-tensioned concrete masonry prestress losses are primarily due to creep and shrinkage of the masonry. Creep is an increase in strain over time under a constant stress and occurs in both the units and mortar. Creep losses occur when the prestressing tendon shortens with the masonry. The total creep experienced over the lifetime of the masonry can be several times the magnitude of the initial elastic strain caused by the application of the prestress force. Thus, it is an important consideration for the design of prestressed masonry.

The initial prestress is also reduced by shrinkage of the units and mortar. When concrete masonry is subjected to air with a relative humidity below 100 percent it experiences drying shrinkage, which is an irreversible reduction in volume. Shrinkage losses occur when the prestressing tendon shortens with the masonry. The rate of drying shrinkage decreases with time, so the longer the masonry units are allowed to cure before prestressing, the smaller the prestress loss due to shrinkage.

PRESTRESS LOSS DUE TO CREEP

When designing prestressed masonry, one needs to know the maximum level of shrinkage and creep that will occur in a structure. Knowing these values in terms of total strain allows the designer to calculate the shortening of the tendon over the life of the structure. This shortening can then be expressed in terms of a change in stress by multiplying the modulus of elasticity of the tendon by the total expected creep strain. The change in stress is the prestress loss due to creep and is added to other losses such as shrinkage and relaxation to give the total loss. This total loss is deducted from the initial prestress to give the "effective" prestress.

The goal of this research was to develop recommended values of creep strain that can be used in design to determine the effective prestress. Creep in concrete has traditionally been expressed in terms of the creep coefficient (C_c) defined as follows:

$$C_c = \frac{\varepsilon_c}{\varepsilon_i} \quad \textbf{(1)}$$

Where ε_c = total strain due to creep and ε_i = initial elastic strain. To calculate the creep strain using the creep coefficient, the modulus of elasticity (E_m), and the initial prestress level (f_{mps}) must be estimated prior to construction:

$$\varepsilon_c = C_c \varepsilon_i = C_c \frac{f_{mps}}{E_m} \qquad \textbf{(2)}$$

The *Building Code Requirements for Masonry Structures* (1999) (hereinafter referred to as the MSJC Code) gives the creep of concrete masonry in terms of specific creep (k_c) defined as follows:

$$k_c = \frac{\varepsilon_c}{f_{mps}} \qquad \textbf{(3)}$$

The total expected creep strain can be calculated directly from the specific creep knowing the initial prestress level:

$$\varepsilon_c = k_c \; f_{mps} \qquad \textbf{(4)}$$

The specific creep is the preferred parameter for calculating the expected prestressing losses because only the initial prestress level is needed. Use of the creep coefficient requires that the modulus of elasticity be estimated for design. Further justification for using k_c is given in the section RECOMMENDATIONS FOR MSJC, page 27.

MECHANISMS

Schultz and Scolforo (1991) state that concrete creep research data should not be used directly for estimating creep and shrinkage losses of concrete masonry because:

- mixing and curing processes differ considerably,
- masonry has mortar joints,
- the geometry and configuration of masonry units is quite different from that of precast or cast-in-place concrete.

Therefore, although it is inappropriate to use concrete creep values to calculate prestress losses in concrete masonry, it is useful to review the information available on creep of concrete to better understand the factors affecting creep of concrete masonry. This is appropriate because the creep in concrete is attributed to the internal pressure caused by external loads in adsorbed and interlayer water within the microstructure of the cement paste (Neville, et al. (1983)). This pressure causes migration of the water and resulting volumetric changes. Since concrete masonry is also bound together with portland cement paste, creep is anticipated to be similar in the two systems.

Figure 2 qualitatively illustrates the components of the total associated deformation of a loaded and drying concrete specimen loaded at time t_o. As the figure indicates, total creep strain consists of two components, basic creep and drying creep. Basic creep is affected by the amount of evaporable moisture available in the cement paste. The less moisture present in the cement paste, the smaller the basic creep component. The second component, drying creep, occurs when a loaded specimen is not in hygral equilibrium with the surrounding medium (moisture is being transferred between the concrete and the surrounding medium). If a concrete specimen is loaded after it has reached hygral equilibrium with the surrounding air, then the effect of relative humidity on creep will be less than with a specimen that continues to dry after the application of load.

Note that the creep and shrinkage are represented as additive. Typically, creep of concrete is determined experimentally by using an unstressed control specimen constructed from the same materials and exposed to the same environment as the loaded specimens. The only deformation associated with the control specimen is shrinkage. The strains in the control specimen are subtracted from the strains in the specimens subjected to load, the difference in strain being creep. For applications where creep and shrinkage occur simultaneously this is a valid approach (Neville, et al. (1983)). But in reality, creep and shrinkage are interactive processes that can not be completely isolated and the general effect of shrinkage is to increase the total magnitude of creep. The effect, however, is small and only needs

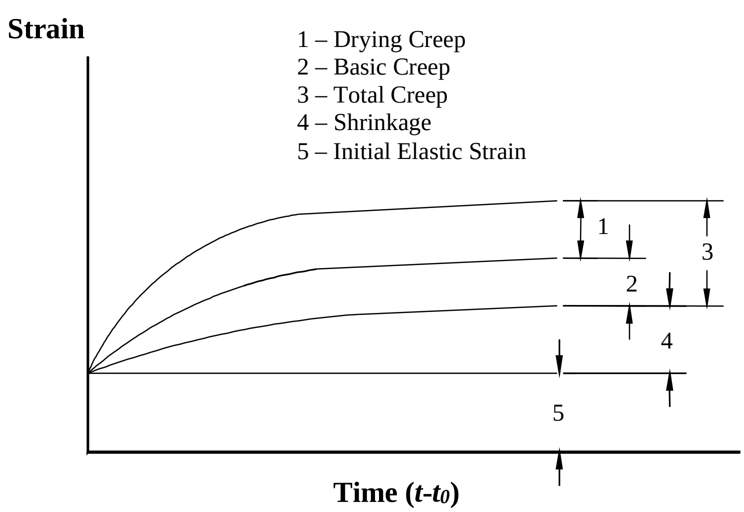

Figure 2—Change in Strain of a Loaded and drying Specimen (Neville et al. (1983))

to be accounted for when there is a need to distinguish between basic and drying creep.

DESIGN OF EXPERIMENT

Prestressing will generally be used on tall walls that have low compressive stresses from gravity load, since relatively low prestress levels will satisfy the design requirements for this application. To determine a reasonable level of prestress to apply to the specimens during the creep test, two typical design applications were selected: a cantilever retaining wall and a commercial/industrial building with tall masonry walls. Note that the prestress levels were selected based on allowing no net flexural tension under design service loads.

The cantilever retaining wall was constructed of 8 in. (203 mm) CMU using a specified compressive strength of masonry , f'_m , of 1,500 psi (10.3 MPa) with a backfill height of 6 ft (1,829 mm). This would be typical of a basement wall. The lateral equivalent fluid pressure was assumed to be 26 $^{lb}/_{ft^3}$ (4,100 N/m^3).

The commercial wall is constructed of 8 in. (203 mm) concrete masonry units (CMU) using an f'_m of 1,500 psi (10.34 MPa) with an unsupported height of 22 ft (6.7 m) and was lightly loaded from the roof dead load. The wind pressure was assumed to be 18 psf and the wall was assumed to be simply supported.

These example calculations indicate that for the retaining wall and tall single-story wall configuration selected, a prestress level of approximately 150 psi (1.03 MPa) on the minimum net section will provide zero tensile stress in the wall cross-section at full lateral load. The stress is based on the minimum net cross-sectional area cut through the face-shell bedded mortar joint. Stress levels of 50 (0.34 MPa) and 250 psi (1.72 MPa) were also chosen because they are reasonable prestress levels for design, and they provide comparison for the effect of the prestress level on the specific creep. The test matrix is shown in Table 1.

TEST SET-UP AND PROCEDURES

Eight masonry walls, 6 ft (1,829 mm) tall x 24 in. (610 mm) long x 8 in. (203 mm) wide (nominal dimensions), were constructed by masons from the local union apprenticeship program under the direction of the program instructor (Figure 3). All walls were built in running bond with face-shell bedding using type S mortar and Type I moisture-controlled units. Four walls were constructed with normal-weight concrete units, and the remaining four were constructed with lightweight concrete units. The normal-weight CMU had a granite aggregate and the lightweight units had an expanded shale aggregate meeting the requirements of ASTM C90. CMU were manufactured in Cheyenne, WY by Powers Brick and Tile Co.

Before the masonry walls were post-tensioned, stiff steel loading frames were positioned on top of the walls to ensure a uniform stress through the entire height of the masonry wall. The walls were constructed with the center masonry cell aligned with a hole in the laboratory's strong floor. The walls were prestressed by inserting high strength steel rods 1¼ in. (31.8 mm) nominal diameter and an ultimate strength of 150 ksi (1,030 MPa) through the middle cell of the walls and stressing against the strong floor and frame. Large rail-car springs were used to maintain the stress in the prestressing bars. Figure 4 shows the details of the specimen construction.

Two control walls, 4 courses (32 in. (813 mm)) high x 24 in. (610 mm) long x 8 in. (208 mm) wide, were constructed at the same time, using the same materials as the prestressed walls. One control wall was constructed with normal weight units and the other with lightweight units.

Table 1. Test Matrix

Specimen	Prestress Level			Aggregate	
	50 psi (0.34 MPa)	150 psi (1.34 MPa)	250 psi (1.72 MPa)	Lightweight Block	Normal Weight Block
Wall A	**X**			**X**	
Wall B		**X**		**X**	
Wall C		**X**		**X**	
Wall D			**X**	**X**	
Wall E	**X**				**X**
Wall F		**X**			**X**
Wall G		**X**			**X**
Wall H			**X**		**X**

Figure 3—Construction of Specimens

Subsequent to construction of the masonry walls, a steel frame was erected to allow proper placement of dial gages to monitor the axial displacement in the walls (Figure 4). We attempted to measure the movement in the units and mortar using a Whitemore gage, but the equipment lacked the resolution necessary to measure the small movements in the mortar and units. This measuring system was abandoned soon after the walls were prestressed and monitoring was initiated.

Dial gages, with resolutions of 0.001 in. (0.025 mm) were placed so as to measure the movement of the walls relative to the strong floor. Readings were taken on both ends of all walls in order to exclude bending displacements caused by prestress eccentricity. In addition, the dial gages were placed at the neutral axis of the out-of-plane bending direction to exclude end displacement due to accidental out-of-plane prestress eccentricity. Dial gages placed at mid-height of the walls provided redundant readings in case a dial gage was disturbed or damaged.

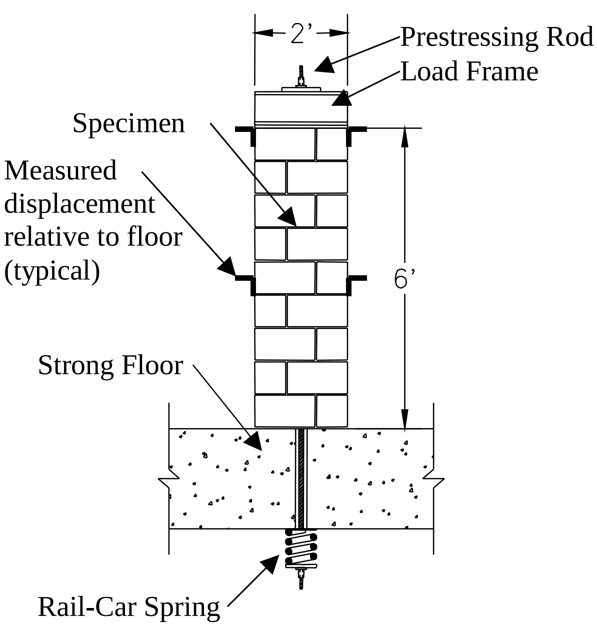

Figure 4—Specimen Test Set-up (1 ft = 305 mm)

Movements were monitored on a daily basis for the first two weeks after prestressing. After two weeks, readings were taken approximately every two to three days. The frequency of readings decreased to one per week after about 150 days following prestressing. Temperature and humidity levels were recorded each time wall movement data was collected.

Over the testing period the temperature ranged between a maximum of 75°F (24°C) and a minimum of 49°F (9°C). The relative humidity in the testing lab fluctuated with the diurnal and seasonal changes in ambient humidity. Both temperature and humidity were monitored and recorded during the test period at each reading of wall movement. In addition, the maximum and minimum temperature and humidity were periodically recorded.

Table 2 gives the testing schedule starting with the date of manufacture of the lightweight and normal weight CMU. All the walls were constructed at the same time. However, walls C and G (both stressed to 150 psi (1.03 MPa)) were stressed 19 days later to investigate the effect of different ages at loading on the creep of masonry. Data collection was terminated on March 6, 1997 giving the masonry walls 295 days (276 days for walls C and G) under load.

Four face-shell-bedded prisms were constructed concurrently with the masonry walls. The compressive test results are shown in Table 3. Strengths were calculated based on the minimum net area at the face-shell-bedded joint and were tested when the data collection was terminated.

Table 2. Schedule of Specimen Construction

Day	Action
0	Normal Weight CMU Manufactured
36	Lightweight CMU Manufactured
45	All Specimens Constructed
63	All specimens but two 150 psi (1.03 MPa) specimens are stressed
82	Remaining two specimens prestressed (delayed prestressing)
358	Test concluded

RESULTS AND DISCUSSION

Figures 5 and 6 show the total movement of the normal weight and lightweight specimens as measured during the test period. The strain in the unloaded wall represents the shrinkage experienced by the masonry following loadingof the other specimens. The total strain for each specimen is made up of the elastic instantaneous strain, the shrinkage strain (as indicated by the plot for the unloaded wall), and the creep strain.

Table 3. Prism Strengths (psi). Tested at the Termination of Data Collection

	Normal Weight	Lightweight
Prism 1	2,250	1,520
Prism 2	1,950	1,630
Average	2,080	1,580

Figures 7 and 8 show the creep strain after the shrinkage strain and initial elastic strain have been subtracted. Note that the initial creep strain for both the lightweight and normal weight specimens is negative. This might indicate that the stressed walls swelled beyond their initial elastic deformation; that swelling could be attributed to the increasing of the relative humidity. However, referring to Figures 5 and 6, and comparing the total strains of the 0 psi (0 MPa) specimen and the 50 psi (0.34 MPa) specimen, we see that over the first 160 days the total strains for both specimens are approximately equal. Total strain is composed of initial elastic strain associated with the prestressing force, shrinkage strain, and creep strain. The control specimen has no elastic or creep strain since it is not stressed. The control specimen and the 50 psi specimen having equivalent total strain, indicates that the control

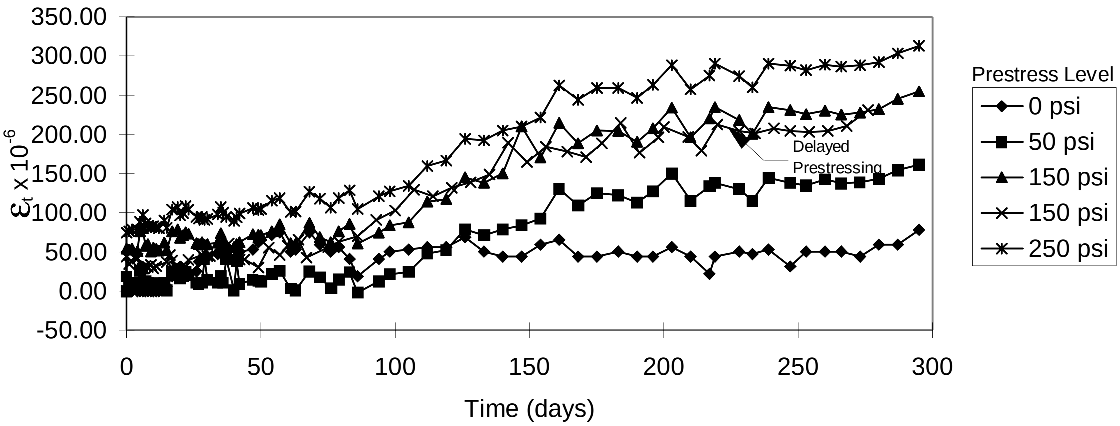

Figure 5—Total Movement for Normal Weight Specimens (1 psi = 0.6895 MPa)

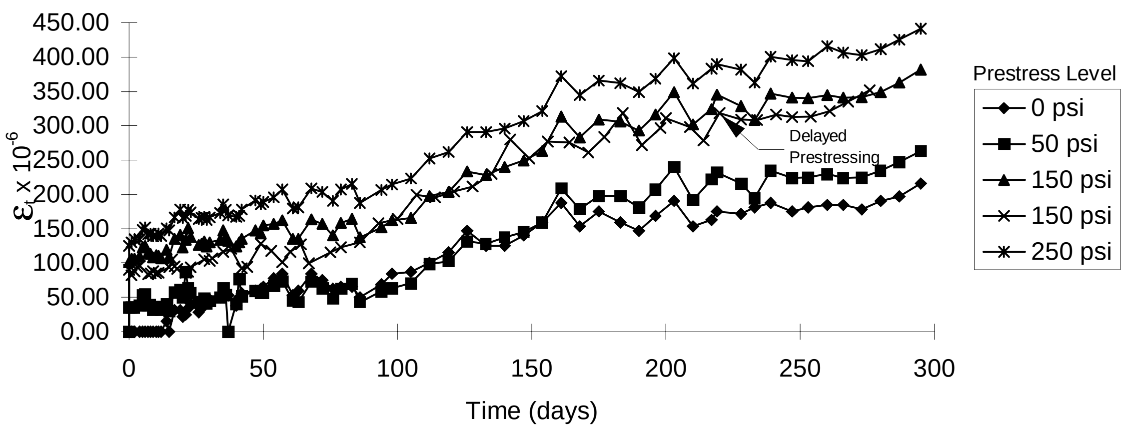

Figure 6—Total Movement for Lightweight Weight Specimens (1 psi = 0.6895 MPa)

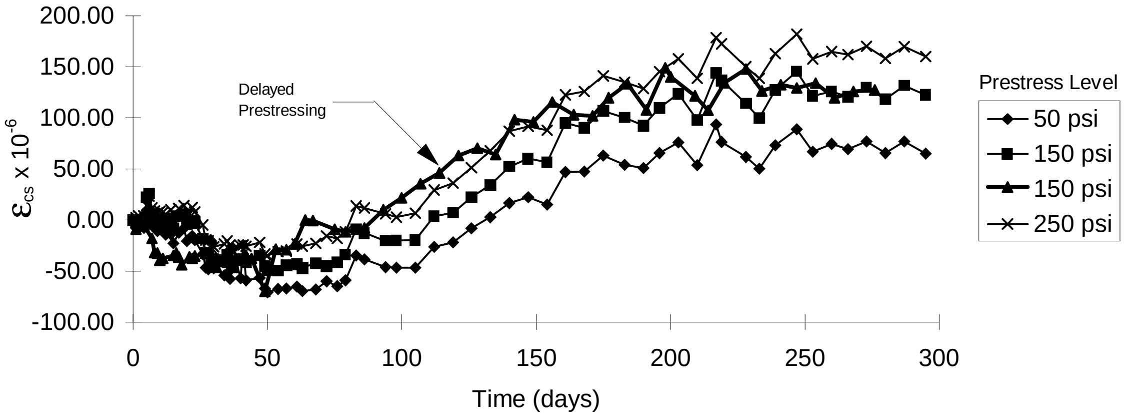

Figure 7—Creep Strain of Normal Weight Specimens (1 psi = 0.6895 MPa)

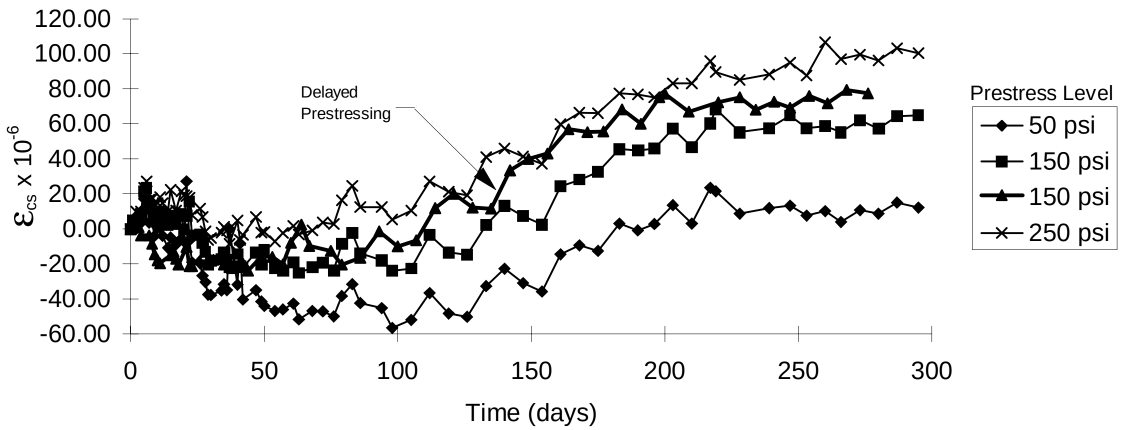

Figure 8—Creep Strain of Lightweight Specimens (1 psi = 0.6895 MPa)

specimen experienced a greater rate (and in turn magnitude) of shrinkage when compared with the stressed specimens.

The reason for the increased shrinkage of the control specimen can likely be attributed to the steel loading frames on top of the stressed specimens. The bottom plate of the steel loading frame completely covered the top of the specimen, effectively enclosing the cells. This restricted the air circulation in the cells, reducing the surface area available for shrinkage of the stressed specimens. The unstressed specimen had no loading plate, allowing shrinkage to occur from inside as well as outside the specimen. It is expected that the ultimate value of shrinkage for the stressed and unstressed specimens should be approximately equal. Although the hollow cells of the stressed walls were covered, the cells were still exposed to outside air, which should facilitate shrinkage. However, this shrinkage may be slower than that of the control because of the restricted air circulation. Specific creep values compare well with values obtained in previous research indicating that, while the creep data does not follow the typical logarithmic decay, the total magnitude of creep at the end of the test period is reasonable.

Figures 9 and 10 show the change in relative humidity (RH) and the rate of creep for the normal weight and lightweight walls, respectively. Period 1 begins with the application of load and continues through approximately day 70. It marks a gradual increase in RH. During this same period, the creep continues with a negative trend. Period 2 begins at approximately day 70 and marks the period where the RH remains constant, corresponding to a change from negative to positive creep. As the RH begins to decline (Period 3) the creep turns positive and continues on this trend for the remainder of the test period. This phenomena is similar for both normal weight and lightweight specimens.

Neville et al. (1983) have shown that the effect of alternating humidity levels on the total magnitude of creep of concrete is not significant. In addition, Ameny (1979) showed that for lightweight concrete masonry the effect of alternating levels of relative humidity on creep was small. However, the RH in these tests was not alternated over the test period, but rather the change was due to the natural seasonal change of RH in the structures lab.

Tables 4 and 5 summarize the total strains measured over the 300-day test period along with the specific creep data for each of the specimens. The specific creep values for normal weight specimens are greater than those for the lightweight specimens at all prestress levels. One would intuitively expect specific creep for lightweight masonry to be higher than the specific creep for normal weight ma-

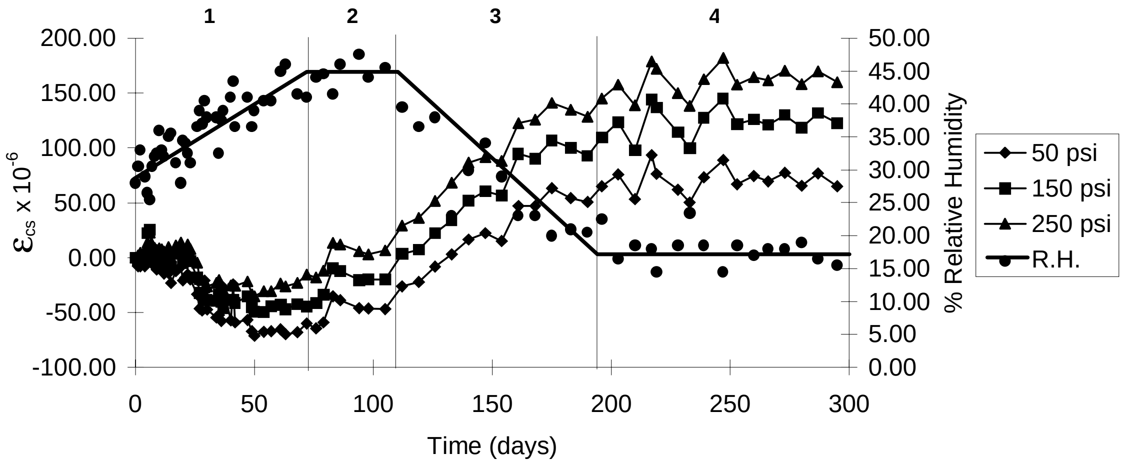

Figure 9—Effect of Relative Humidity on Normal Weight Specimens (1 psi = 0.6895 MPa)

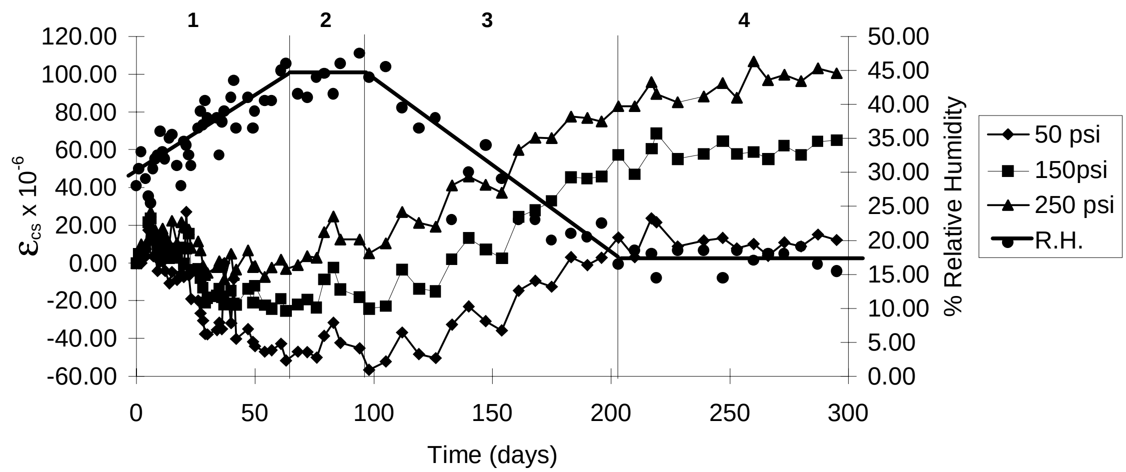

Figure 10—Effect of Relative Humidity on Lightweight Specimens (1 psi = 0.6895 MPa)

Table 4. Summary of Results for Normal Weight Specimens (1 psi = 0.6895 MPa)

Applied Stress (psi)	Strain (x 10^{-6} in./in.)			k_c
	Creep	Shrinkage	Total	(x 10^{-7}/psi)
50	65	78	143	13.0
150	125	78	203	8.2
150 (delayed)	128	75	206	8.5
250	160	78	238	6.4

Table 5. Summary of Results for Lightweight Specimens (1 psi = 0.6895 MPa)

Applied Stress (psi)	Strain (x 10^{-6} in./in.)			k_c
	Creep	Shrinkage	Total	(x 10^{-7}/psi)
50	12	215	227	2.4
150	65	215	286	4.7
150 (delayed)	77	215	292	5.1
250	100	215	315	4.0

sonry. However, the experimental results obtained from this testing indicate otherwise. Maksoud (1994) noted in his own experimental work that for an approximate loading period under one year, normal weight masonry will experience greater specific creep than lightweight masonry, which agrees with the results of this research. If total prestress loss (due to creep and shrinkage) is considered, then the lightweight construction will have 30 to 60 percent higher prestress losses due to creep and shrinkage than that of the normal weight construction.

COMPILATION OF CREEP RESEARCH

Available relevant previous research is summarized in this section for comparison to the results of the experimental work presented and for use in evaluating the current code provisions for creep and shrinkage. Tables 6 and 7 list a number of researchers with the year their results were published, and the value for specific creep calculated from their work. The following briefly describe the research and experimental work presented in the tables.

Tatsa et al. (1973) performed creep tests on normal weight and lightweight concrete blocks in hygral equilibrium, which would significantly reduce the magnitude of the shrinkage creep. We would expect that the total creep in these experiments would be lower than that in the field. Stress levels used were 210 and 140 psi (1.45 MPa and 0.97 MPa) for the normal weight and lightweight test specimens, respectively. Total losses were measured to be 18 percent for the lightweight units and 13 percent for the normal weight units. Tatsa et al. used short panels and full-sized wall panels in the experimental program. All masonry was constructed using type N mortar. The short panels were 32 $^{3}/_{8}$ in. (822 mm) long by 8 in. (203 mm) high by 9 $^{1}/_{2}$ in. (241 mm) wide with individual units of dimensions 16 x 8 x 2.75 in. (406 x 203 x 70 mm). The full size wall panels were 9.5 ft (2.9 m) high by 4.7 ft (1.4 m) width using 8 in. (203 mm) block. The mortar bedding was not given.

Lenczner (1974) investigated creep of concrete blockwork piers. Individual units consisted of a limestone aggregate with a unit strength of 820 psi (5.65 MPa) at the beginning of testing, increasing to 1800 psi (12.4 MPa) one year later when testing was concluded. Compressive testing of the piers gave an f'_{mt} of 615 psi (4.2 MPa) at 28 days, increasing to 750 psi (5.2 MPa) after one year. Test stress levels ranged from 113 to 274 psi (0.78 to 1.89 MPa), while stress/strength ratios ranged from 18.4 to 44.7 percent. Lenczner tested a total of four piers, and was able to monitor the movement of a single bed joint using acoustical strain gages. Individual units were 18 x 9 x 4 in. (457 x 229 x 102 mm).

Ameny (1979) tested lightweight concrete masonry under varying stress/strength ratios, load eccentricities, and environmental conditions. Applied stress levels corresponded to 20 and 40 percent of the tested prism strength, f'_{mt}. Load was applied to the test specimens axially and at varying eccentricities. The temperature in the testing lab ranged from 50°F to 81°F (10°C to 29°C) , and the relative humidity varied from 10 to 60 percent. Test specimens were 5 courses tall and 1 unit wide. The units had nominal dimensions of 8 in. (203 mm) wide x 8 in. (203 mm) tall x 16 in. (406 mm) long. Although all joints were made with type N mortar, Ameny used two different mixes. One mix used masonry cement and sand, the other used Portland cement, lime, and sand. Based on test results Ameny concluded that different mortar constituents do not effect the creep behavior of masonry, but they do effect the modulus of elasticity. Additionally, Ameny determined that creep behavior of the masonry was not affected by load eccentricities.

Maksoud (1994) investigated the creep response of normal weight and lightweight masonry prisms to develop creep functions for finite element modeling. Testing consisted of 4-course masonry prisms using nominal 8 in. (203 mm) hollow CMU stressed to 20 and 40 percent of the tested prism strength. The mean strength for the tested CMU was 2,640 psi (18.2 MPa) for the lightweight and 4,090 psi (28.2 MPa) for the normal weight. Prism strength was 1,944 psi (13.4 MPa) for the lightweight and 3,278 psi (22.6 MPa) for the normal weight masonry. The mortar strength was reported as 1,465 psi (10.1 MPa).

Brooks and Bingel (1994) investigated the effect on creep of varying the applied stress levels, although the reported specific creep value was obtained under constant stress. Tests conducted on the CMU walls (45 in. (114.3 mm) high x 18 in. (457 mm) wide) lasted for 250 days. Loading began

Table 6. Specific Creep Values for Normal Weight Concrete Masonry (x10^{-7} in./in./psi)(1/psi = 145/MPa)

Researcher	Mortar			Duration of Loading (days)
	M	S	N	
Tatsa et al. (1973)	-	-	4.9	210
Maksoud (1994)	-	3.8-5.1	-	400
Badger (1997)	-	6.4-13.0	-	300

Table 7. Specific Creep Values for Lightweight Concrete Masonry (x10^{-7} in./in./psi)(1/psi = 145/MPa)

Researcher	Mortar			Duration of Loading (days)
	M	S	N	
Tatsa et al. (1973)	-	-	9.42	210
Lenczner (1974)	-	-	33.0-56.0	320
Ameny (1979)	-	-	10.3-11.5	110
Harvey and Lenczner (1993)	11.2-12.8	3.56-3.20	5.43-5.68	300
Maksoud (1994)	-	5.1-6.4	-	400
Brooks and Bingel (1994)	-	3.4	-	130
Badger (1997)	-	2.4-5.1	-	300

at 45 days at a stress level of 73 psi (0.50 MPa) and increased in equal 73 psi (0.50 MPa) increments until the stress level tripled at 218 psi (1.5 MPa). The stress level was then decreased back to zero in equal increments so that the loading history was symmetrical.

Harvey and Lenczner (1993) conducted creep tests on double-wythe walls and hollow piers using solid concrete masonry units. The applied stress levels ranged from 60 psi to 470 psi (0.41 to 3.24 MPa) and mortar types M, S, and N were used. It is not known if normal weight or lightweight units were used.

DISCUSSION

It is interesting to note that all of the previous CMU creep research gave specific creep values within the range of 3.4×10^{-7} to 14.7×10^{-7} / psi (0.5 to 2.1×10^{-7}/ MPa) with the exception of Lenczner (1974), who reported 33.0×10^{-7} to 52.0×10^{-7}/ psi (4.8 to 7.5×10^{-7}/ MPa). All of this CMU creep research was performed using masonry walls with the exception of Lenczner's work which used masonry piers. Lenczner applied load to the piers using creep machines that simulated building dead loads more than prestressing loads. The creep machines applied load to the entire cross-section of the pier, whereas a prestressing force is practically a concentrated load. Lenczner's test set-up gave a more even stress distribution near the loading point, which should lead to higher levels of creep.

Lenczner also had unusually high shrinkage (525×10^{-6} in./in. ($13{,}335 \times 10^{-6}$ mm/mm)) in the masonry. Since shrinkage and drying creep are both affected by the drying process, masonry that undergoes significant shrinkage will also be expected to experience increased drying creep.

Another possible factor is Lenczner's use of 4 in. (102 mm) block. If the CMU tested by Lenczner in the masonry piers was solid and fully bedded then the possibility exists that for a given stress level there may be the opportunity for two magnitudes of creep. For example, masonry units with mortar on the face-shells and the webs may experience more creep than units with mortar solely on the face-shells. Stress transferred through the face-shells may disburse into the webs, lowering the magnitude of stress in the face-shells and resulting in lower magnitudes of creep.

The variability of Ameny's elastic strains can be attributed to masonry's nonhomogenous nature. Maksoud noted the same effect since he was able to separate strains in the block from strains in the mortar joint. Creep strains in the block had a linear relationship to the applied stress, however the global creep strains (defined by Maksoud as the creep in the block and joint, inclusive) were nonproportional to the applied stress. Maksoud concluded that the creep effect of the mortar joint may have been the cause. Maksoud reported creep strains in the mortar joints of 3400×10^{-6} and 2872×10^{-6} for applied stress/strength ratios of 0.2 and 0.4, respectively. However, creep tests, also performed by Maksoud, on mortar cylinders with the same mix design as the mortar used in the joints showed significantly lower creep strains. The extremely high creep strains in the mortar joints of the test specimens could be attributed, Maksoud stated, to gaps at the CMU-mortar joint interface, which closed under sustained loading. Maksoud also noted that the high creep strains in the mortar joint could be a result of the lack of accuracy inherent in measuring deformations over a short gage length.

Ameny (1979) reported that creep in the mortar joint is 15 to 30 percent of the overall masonry creep. Because mortar has significantly different material properties relative to CMU it is possible for the creep effect of the mortar joint to force masonry creep to behave nonproportionally to the applied stress.

RECOMMENDATIONS FOR MSJC

The 1999 edition of the MSJC Code includes provisions for the design of prestressed masonry. These provisions do not have specific recommendations for calculating prestress losses, but the Commentory refers to Section 1.8 of the Code for recommendations on calculating shrinkage and creep of concrete masonry. Section 1.8 requires using a specific creep value of 2.5×10^{-7}/psi (3.6×10^{-5}/MPa) for concrete masonry. The Commentary indicates that this value is based on Lenczner and Salahuddin's work reported in 1976. This paper covers almost exclusively clay brick masonry with a reference to one set of tests conducted by Lenczner (1974). There is no reference in Lenczner and Salahuddin (1976) to specific creep of concrete masonry so, the origin of the current MSJC Code value is unclear.

Figures 11 and 12 (normal weight and lightweight, respectively) show all of the specific creep values in current and previous research as a function of the prism strength. The results from Lenczner's 1974 work are not shown on either of the plots (see Table 1) because the results as interpreted from the 1974 paper are nearly an order of magnitude above the remainder of the data, as was discussed previously. We believe that Lenczner's data is not appropriate for use in selecting a value for calculating creep loss and therefore have excluded it from the graphs.

Both the lightweight and normal weight plots show a slightly decreasing trend, as would be expected. It is possible to perform a regression analysis and determine the creep coefficient as a function of the strength of the masonry. Indeed, much of the past research into creep of concrete and clay masonry has focused on the accurate estimation of the actual creep accounting for the site conditions and materials used in construction. In some cases,

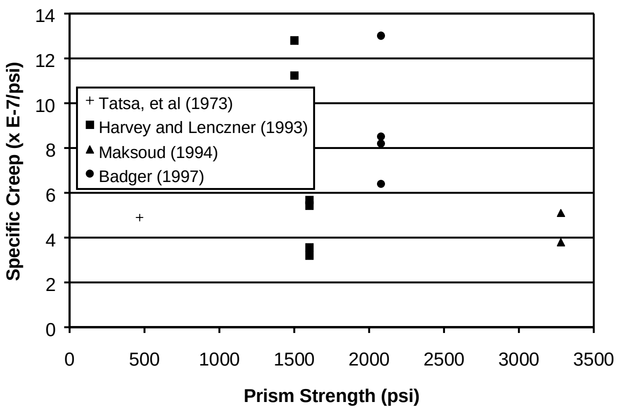

Figure 11—Specific Creep for Normal Weight Units (1 psi = 0.006895 MPa)

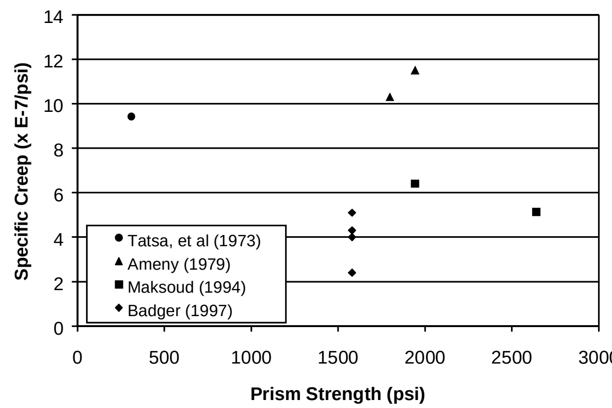

Figure 12—Specific Creep for Lightweight Units (1 psi = 0.006895 MPa)

elaborate equations have been developed to calculate the ultimate value of creep. We feel that this level of perceived accuracy in design is unwarranted for three reasons. First, there has traditionally been very little control over the moisture content of units during the construction phase. This will tend to make the drying creep an unknown for any site condition. Second, the moisture content of the mortar is adjusted by the mason for the current conditions. The moisture content can vary significantly between jobs and even day-to-day on a single project. This will also affect drying shrinkage making accurate prediction very difficult. Third, and most convincing reason is that accurate estimation of creep does not necessarily lead to significant savings of prestressing steel.

Figure 13 shows the increase in tendon material quantities as the specific creep is increased. Each curve represents a different specific creep while the x-axis represents the variation in prestress level. As the initial prestress level (that is, steel strength) increases, the increase in material required is reduced. Assume that the initial prestress level is 150 ksi (1,034 MPa). If a specific creep value of 13 x 10^{-7}/psi (1.9 x 10^{-4}/MPa) is used instead of the value given

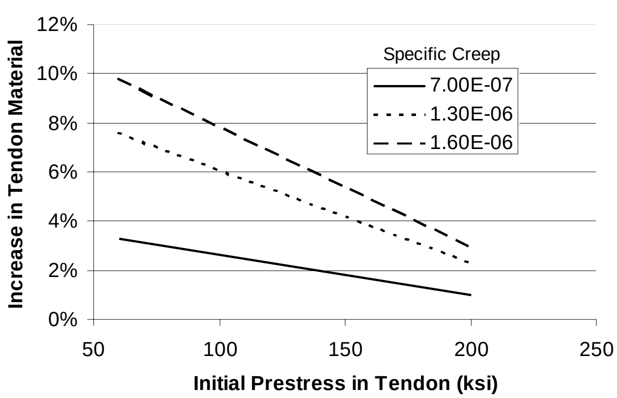

Figure 13—Effect of Specific Creep on Tendon Quantities. Increase in Tendon Material is Based on Comparision to a Design Using a Specific Creep Value 2.5 x 10^{-7}/psi (3.6 x 10^{-5}/MPa)

in the code, the increase in prestressing steel quantity is only slightly greater than 4 percent. Therefore, an increase in creep coefficient of nearly an order of magnitude results in a very small increase in cost. In conclusion, the specific creep should be estimated *conservatively*, but there are no significant cost savings in estimating the creep *accurately*.

It is clear that the current MSJC code value of 2.5 x 10^{-7}/psi (3.6 x 10^{-5}/MPa) significantly underestimates the creep values obtained in the compilation of concrete masonry creep research. Underestimating creep in reinforced masonry (apparently) has not been a significant issue since this specific creep has been in use since the inception of the MSJC Code in 1988. However, underestimating the creep in prestressed masonry has very serious implications. Underestimating creep will lead to long-term prestress losses that are higher than expected, reducing the effective prestress and the factor of safety. It is strongly recommended that the prestressed provisions include recommendations for estimating creep using a single value of specific creep:

$$k_c = 13 \times 10^{-7}/\text{psi} \ \ (1.9 \times 10^{-4}/\text{MPa})$$

This value conservatively encompasses much of the data gathered in the current and past research, and should be used on Type M, S, or N mortar, for normal weight or lightweight block.

PRESTRESS LOSS CALCULATIONS

Figure 14 shows a graph of the prestress losses that would be used in design should the specific creep recommended in this paper be accepted into the MSJC Code prestressed provisions. The graph contains four curves. Each curve represents a different prestress level in the masonry based on the minimum net section properties. Note that the prestress loss increases significantly with the reduction in initial prestress, which is a fraction of the prestressing steel strength. So the higher the prestressing strength, the smaller the loss in prestress and the less pre-

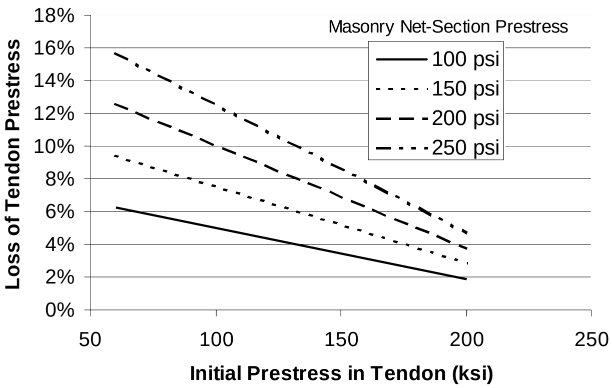

Figure 14—Prestress Losses Using the Proposed Specific Creep Value (1 psi = 0.006895 MPa)

stressing steel is required. This trend holds true for all prestressing losses and confirms that higher strength steels are more efficient for prestressing.

Mackay (1997) indicates that prestressed masonry codes in other countries are much more conservative than the current MSJC Code. He indicates that the British code gives a creep coefficient (C_c) value of three and the draft Australian code gives a value of 2.5. Using $E_m = 1.5 \times 10^6$ psi (10,343 MPa) gives specific creep values of 20 x 10^{-7}/psi (2.8 x 10^{-4}/MPa) and 16.7x10^{-7}/psi (2.4 x 10^{-4}/MPa)for the British and Australian Codes, respectively. This further supports the need to adjust the specific creep values in the prestressed masonry provisions of the code.

CONCLUSIONS

Eight 8 in. (208 mm) hollow concrete masonry walls were prestressed to 50, 150, and 250 psi (0.34, 1.03, and 1.72 MPa) on the minimum net section of face-shell bedded joints. The walls were monitored over 300 days beyond prestressing to determine the maximum creep movement for these relatively low prestress levels. The experimental work was conducted indoors but not in an environmental chamber and was thus subjected to changes in relative humidity and temperature. Unloaded control walls constructed of the same materials were also monitored to remove the shrinkage strain from the total strain experienced by the loaded specimens. These ultimate creep values were used to calculate the specific creep for the particular construction, constituents, and prestress level. The specific creep values compare well with previous creep research. Previous research and that presented in this paper were assembled, summarized, and used to develop a recommended value for specific creep to be included in the prestressed masonry provisions of the MSJC:

$$k_c = 13 \times 10^{-7}\text{/psi } (1.9 \times 10^{-4}\text{/MPa})$$

Based on the experimental results the following additional conclusions or recommendations are made:

1) Seasonal variations in humidity appeared to affect the short-term creep behavior. However, the ultimate creep values at the end of the testing period gave specific creep results that are consistent with previous research.
2) In this research, lightweight concrete masonry will experience greater magnitudes of creep strain than normal weight concrete masonry for a given stress level. Previous research into creep of lightweight concrete supports this finding. In addition, total creep and shrinkage for lightweight concrete masonry is greater than for normal weight.
3) A relatively small age difference at loading (assuming both specimens are at least 28 days old) will not have a significant effect on the rate or magnitude of creep in specimens of similar construction.

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NOTATIONS

C_c = creep coefficient.
E_m = modulus of elasticity.
f_{mps} = initial prestress level.
f'_{mt} = tested prism strength.
f'_m = specified compressive strength of masonry.
k_c = specific creep.
RH = relative humidity.
t = time.
δ_c = total strain due to creep.
ε_i = initial elastic strain.