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CARBON FIBRE REINFORCED PLASTIC (CFRP) PRESTRESSED MASONRY

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ABSTRACT

A 3 metre high diaphragm wall has been constructed and post-tensioned with four Carbon Fibre Reinforced Plastic (CFRP) prestressing strands and a new anchorage system. Losses of the prestressing forces in these tendons have been monitored. On a theoretical basis, cooling the wall as in a Canadian winter will cause a reduction in prestress force, as opposed to steel tendons, which increase their force with decreasing wall temperature. This has been verified by experiment. The wall was also tested in flexure to cracking. The flexural strength and deformation have been compared to those predicted by analysis.

INTRODUCTION

Post-tensioned masonry diaphragm walls have superior strength and serviceability over plain masonry diaphragm walls. A major drawback associated with post-tensioning is corrosion of the steel tendons, particularly unbonded tendons. In order to eliminate this problem we have replaced steel tendons with Carbon Fibre Reinforced Plastic (CFRP) tendons. CFRP tendons have several advantages over steel tendons due to their higher durability, corrosion resistance, lighter weight, and higher tensile strength (Hercules Aerospace, 1995, Rostasy et. al., 1993, and Santoh et. al., 1993). A suitable anchorage system, which allows the full strength of the tendons to be developed with minimal losses at transfer, has been designed (Sayed-Ahmed and Shrive, 1998a) and thus permits the use of CFRP tendons for post-tensioning.

Temperature effects can have a significant impact on prestressed structures. The effects of temperature change are especially important in climates such as in Canada where a wall, and subsequently the CFRP tendons, may be subject to temperatures ranging from -40°C to +40°C. Temperature variations result in corresponding variations in the level of prestressing which are dependent upon the differences in the coefficients of thermal expansion of CFRP and masonry. A test was conducted to determine the prestress losses in CFRP tendons due to temperature changes. The temperature of a post-tensioned diaphragm wall was lowered to -65°C on one face while the other face remained open to the laboratory (+20°C). The wall was also tested in flexure, as a propped cantilever, with a horizontal line load applied at midheight. Results were compared to those predicted analytically, considering both flexural and shear deformation.

MASONRY DIAPHRAGM WALL POST-TENSIONED WITH CFRP TENDONS

A 3 metre high diaphragm wall was built with clay bricks and Type S mortar ($f_{m\ 28d}$ = 11.2 MPa). The wall was constructed on a reinforced concrete base beam and a reinforced concrete capping beam was mortared to the top of the wall. The dimensions and reinforcing details of the base and capping beams are shown in Figure 1. The wall is 3 metres high, excluding the base and capping beams, and has the cross-section shown in Figure 1.

In order to produce uniform compressive stress over the cross-section of the wall, the wall was prestressed using four 8 mm CFRP (Leadline^{TM}) tendons placed at the centroid of each C-section of the diaphragm wall (see Figure 1). The lower anchorages were seated on the tendon and then fed through holes in the capping beam, through the wall cavities, and finally through holes in the base beam. The anchorages were locked in place using notched steel plates slid through the channels in the base beam. The top anchorage was seated on the tendon using the seating frame shown in Figure 2. The seating load was 65 kN per tendon. After seating, the frame was removed and a 25 mm steel plate was placed under the locking nut. Post-tensioning was achieved using the second frame shown in Figure 2. The load applied was measured by the upper load cell while the lower load cell remained unloaded. When the desired load was reached, the locking nut was locked against the anchorage. At this point the anchorage was released from the anchor adaptor: the upper load cell now read zero while the lower load cell indicated the actual prestressing force transferred to the wall. The difference between this reading and the previous reading in the upper load cell was the prestress loss at transfer and was found to be nearly zero. As can be seen in Figure 3 the prestressing was achieved in two steps. The tendons were post-tensioned sequentially, first to about 30 kN each and then to the initial prestressing load of about 65 kN on each tendon. This prestressing load produced a uniform stress of approximately 0.7 MPa on the wall cross-section. While this prestress level is not high, even low prestress levels are effective in improving the structural performance and serviceability in a wall, as demonstrated by Shaw et. al. (1986).

THERMAL EFFECTS IN CFRP PRESTRESSED MASONRY DIAPHRAGM WALLS

Masonry and CFRP tend to expand and contract different amounts when subjected to a variation in temperature. Compatibility requires that the length changes be the same, resulting in a change in the prestress level. These effects are important in the design of prestressed masonry. In Canada, a wall can be constructed and prestressed in the summer when temperatures are as high as +40°C and then exposed to temperatures of -40°C in the winter. It is also possible that in the winter the interior side of a diaphragm wall could be maintained at room temperature (≈+20°C) while the exterior is subjected to -40°C.

Change In Prestress Level Due To Thermal Effect

Temperature variations have the opposite effect on the prestressing force in CFRP tendons as they do in steel tendons. The losses in prestressing force are dependent on the difference in the coefficients of thermal expansion. The CTE of steel is higher than that of masonry and thus for a decrease in temperature there is an increase in prestress force in steel tendons. There is some disagreement over the value of the CTE for CFRP. Some sources report a negative value (e.g. Daniel and Ishai, 1994) while others report a positive value (Mitsubishi Chemical Corporation, 1996). The CTE of the CFRP tendons used to prestress the wall was determined experimentally (Sayed-Ahmed and Shrive, 1998b). The CTE for the 8 mm Leadline™ used was found to be $(-0.86 \pm 0.09) \times 10^{-6}/°C$. This is lower than the CTE for masonry so we expect a loss in prestress when the temperature of the wall decreases. Prestress loss due to thermal effects is predicted by Equation 1 for temperature varying linearly across the wall and concentric tendons (Sayed-Ahmed and Shrive, 1998b):

$$\Delta P = \frac{\Delta T + 0.5\delta T}{\dfrac{1}{E_p A_p} + \dfrac{1}{E_m A_m}}(\alpha_p - \alpha_m) \qquad \text{(Equation 1)}$$

Where, the subscripts m and p refer to masonry and prestress tendons respectively; α_m and α_p are the CTEs for the masonry and the tendons; A_m is the cross sectional area of the masonry wall stressed by n tendons and A_p is the area of these n tendons; E_m and E_p are the Young's moduli of the masonry and the tendon materials. The linearly varying temperature change can be considered as a uniform change of ΔT and a linearly varying change from zero to a maximum of δT.

THERMAL TEST ON CFRP PRESTRESSED MASONRY DIAPHRAGM WALL

Test Set Up

A thermal test was conducted on the 3 metre high diaphragm wall. An insulated cavity was built on one side of the wall and dry ice was placed within the cavity as shown in Figure 4. An aluminium sheet was placed next to the masonry surface to protect the masonry. A gap of 50 mm was left for the dry ice. Two layers of insulation (R20 each) formed the other side of this gap. The insulation was enclosed by plywood sheets. Great care was taken to ensure uniformity of temperature by insulating and enclosing the side cavities of the wall. The insulated gap was filled with dry ice to begin the cooling cycle. The dry ice was continuously topped up as it settled and evaporated. 24 hours after beginning the cooling cycle, the plywood, insulation, ice and aluminium were removed and the warming cycle began. Wall temperatures were measured by thermocouples placed on the ice side, in the cavity (on the web and in the air), and on the room temperature side (Figure 5). The temperatures monitored during the cooling and warming cycles are shown in Figure 6. Although the top of the wall did not reach steady state due to the repetitive settling/evaporation and refilling of the dry ice, steady state conditions were reached at the midheight and bottom of the wall.

Thermal Test Results

The webs provide connectivity between the two flanges resulting in a temperature gradient across the wall. The gradients across the bottom and midheight of the wall are shown in Figure 7. With the reading locations used, the temperature gradients can be taken as approximately linear across the wall at both the steady state condition and at approximately halfway through the cooling cycle. The linearity of the temperature gradients across the wall also does not seem to be affected by using either the thermocouple in the cavity that is on the web or in the air. Previous experimental results (Shrive et al. 1996) also indicate that temperature gradients across masonry diaphragm walls can be taken to be approximately linear. These experiments also failed to show any significant difference between gradients taken across the cavity in the air or in the web. The gradients obtained by Shrive et al. (1996) are shown in Figure 8.

The prestressing forces were continuously monitored throughout the cooling and warming cycles and are plotted in Figure 9. The effects of the thermal changes are evident in this figure. A prestress loss of approximately 4.8 kN (1.2 kN/tendon) was experienced by the CFRP tendons. Using 4 #8 Leadline^{TM}, with 2250 MPa nominal tensile strength (A_p= 184mm^2, E_p =142 GPa, A_m=0.396m^2, E_m =8 GPa, and α_p =-0.9x10^{-6}/°C), the prestress loss predicted by Equation 1 is 1.8 kN/tendon for the temperature change experienced by the masonry diaphragm wall (α_m=6x10^{-6}/°C). If we take α_m=8x10^{-6}/°C instead, Equation 1 predicts the prestress loss to be 2.3 kN/tendon. The actual values of E_m and α_m have not been determined for the wall constructed. The value of E_m (assumed to be 8 GPa) has little effect on the outcome of Equation 1 while the value assumed for α_m has a significant effect on the calculated prestress loss. Lack of knowledge of an accurate value for α_m accounts in part for the differences between the calculated and experimental results. The discrepancy is also partly due to the top section of the wall not being cooled the same as the rest of the wall. Although the change in prestress level is small, it is significant enough to be considered in the design process.

FLEXURAL BEHAVIOUR OF A CFRP PRESTRESSED DIAPHRAGM WALL

Two flexural tests were performed on the masonry diaphragm wall described previously. The test set up is shown in Figure 10. The base beam was fixed to the load floor and the capping beam was tied to a column via a threaded rod. This rod doubled as a load cell in order to determine the reaction force at the top of the wall. The load was applied at midheight across the width of the wall. Several Linear Strain Converters were placed at various locations on one face of the wall to measure deformations during loading. In the first test the wall was loaded to 120 kN, prior to cracking, and unloaded. In the second test, the wall was loaded until it cracked (138 kN). Both shear and flexural cracks appeared. The load was released, with the flexural crack closing but the shear crack remaining visible at some locations. Subsequently, the wall was loaded to 100 kN and then cyclically loaded three times between 50 kN and 100 kN. No further cracking was observed.

In initial analysis of the flexural test data, the wall was assumed to behave as a propped cantilever. Observation of the top reaction load in relation to the applied load indicates that the wall behaves neither exactly as a propped cantilever nor exactly as a simple span. The reaction was approximately 40% of the applied load whereas for a propped cantilever it should be 31% and for a simple span it would be 50% of the applied load.

Flexural Stiffness of CFRP Prestressed Masonry Diaphragm Wall

The modulus of rupture of masonry depends significantly on the time of day the masonry was placed and also varies from one day to the next. Based on test results (Shrive and Tilleman, 1992), the flexural bond strength for a certain type and strength of masonry was found to vary by 100% on a given day of construction. From the results of the second flexural test on the wall, the tensile stress in the masonry at failure was between 0.23 MPa (propped cantilever analysis) and 0.8 MPa (simple span analysis). These are the stresses as calculated at midheight where the flexural crack occurred. Propped cantilever analysis gives the flexural stress at the base of the wall to be 0.39 MPa although no failure occurred at the base. For the propped cantilever analysis, the variation in stress between that calculated at the base and that where the crack developed, is 70%. The fact that a crack did not develop at the base could be due in part to the time taken in constructing the wall and also to the partial fixed/free behaviour of the base.

The flexural stiffness of the wall was determined theoretically assuming an E_m of 8000 MPa and a G_m of 3500 MPa. Accounting for both flexural and shear deformation the theoretical stiffness of the diaphragm wall is 146 kN/mm. Results from the second test indicate a flexural stiffness of 142 kN/mm before cracking occurred. If only flexural deformation is considered, the stiffness of the wall is 588 kN/mm. Obviously, the shear deformation is significant and needs to be considered in the appropriate aspects of serviceability design of diaphragm walls. Actual midheight deflections, for 70 kN of applied load, were 0.59 mm in the first test and 0.45 mm in the second test. The calculated deflection for an applied load of 70 kN is 0.48 mm. The flexural deformation for this load is 0.12 mm while the shear deformation is 0.36 mm.

SUMMARY

A thermal test was conducted on a CFRP prestressed masonry diaphragm wall. The prestress losses were monitored and compared to those predicted by simple equations. The temperature gradients across a masonry diaphragm wall were found to be approximately linear making the use of these equations reasonable for a first estimation. Due to the differences in the coefficients of thermal expansion, the changes in prestress level in a wall prestressed with CFRP tendons are opposite to changes experienced by walls prestressed with steel tendons. In masonry, a temperature drop below that at which the CFRP were prestressed, results in a prestress loss. In flexure, the shear deformation of wide diaphragm walls is as significant as the flexural deformation.

ACKNOWLEDGMENT

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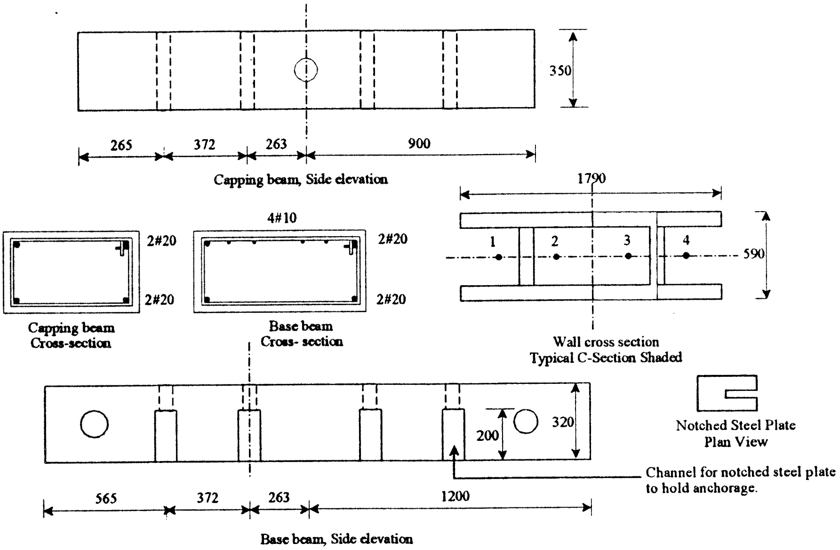


Figure 1. Capping, base beams and cross section of the wall.

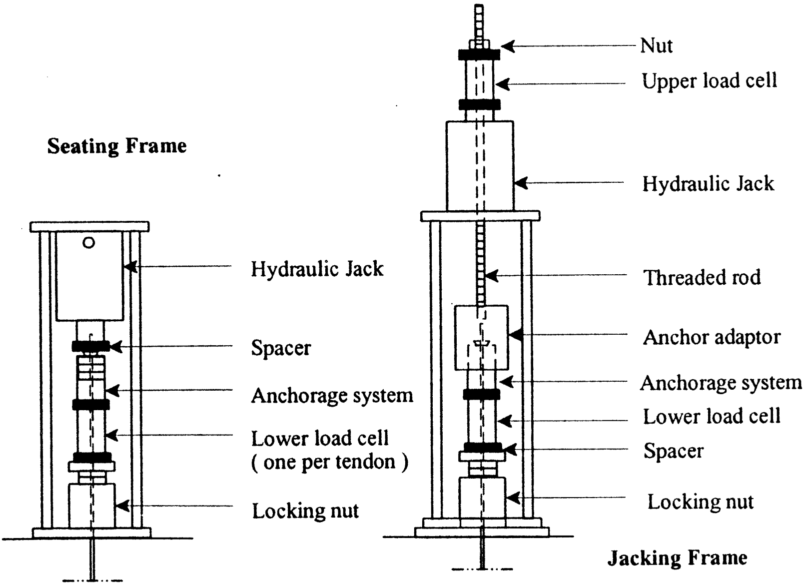


Figure 2. Schematic of seating and the jacking frames used to post-tension the wall.

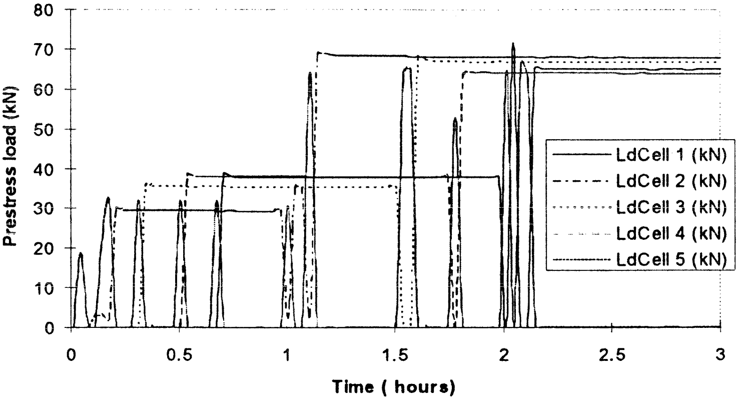


Figure 3. Prestressing sequence. Load cells 1-4 are lower cells for the four tendons and load cell 5 is the upper load cell.

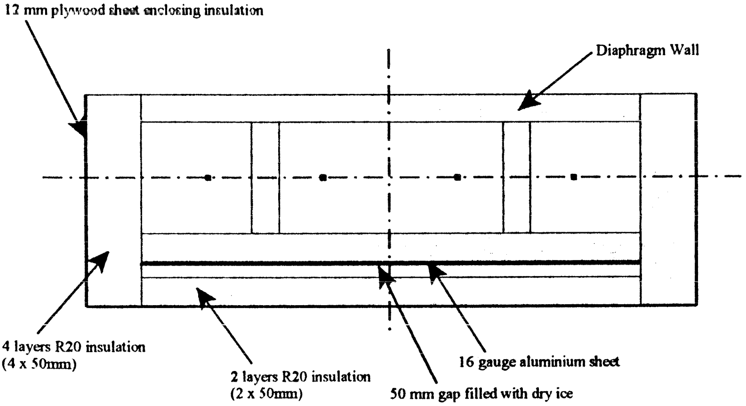


Figure 4. Thermal Test Set Up

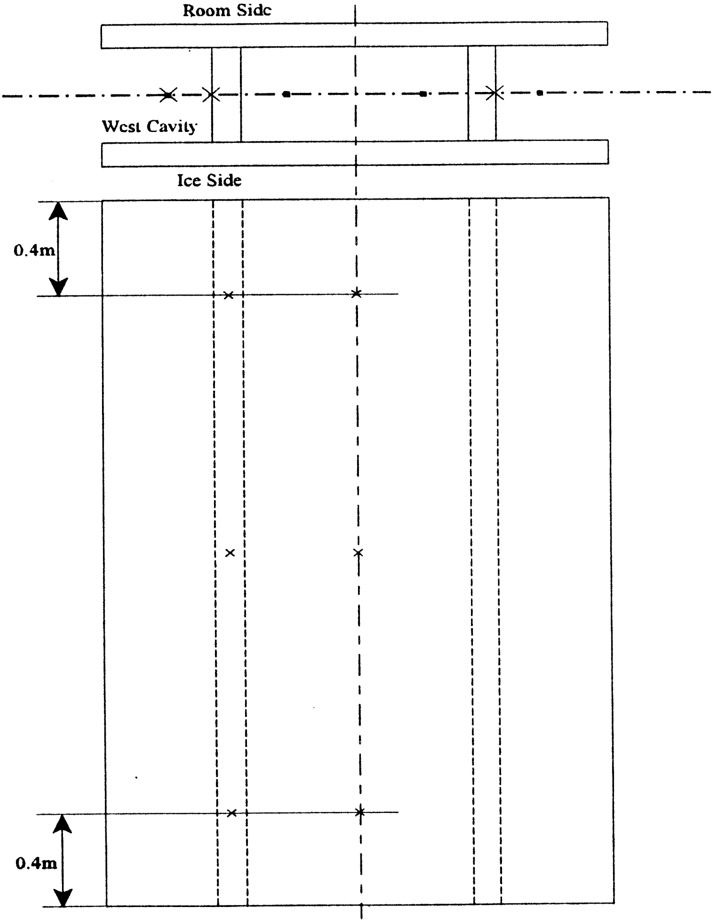


Figure 5. Placement of Thermocouples

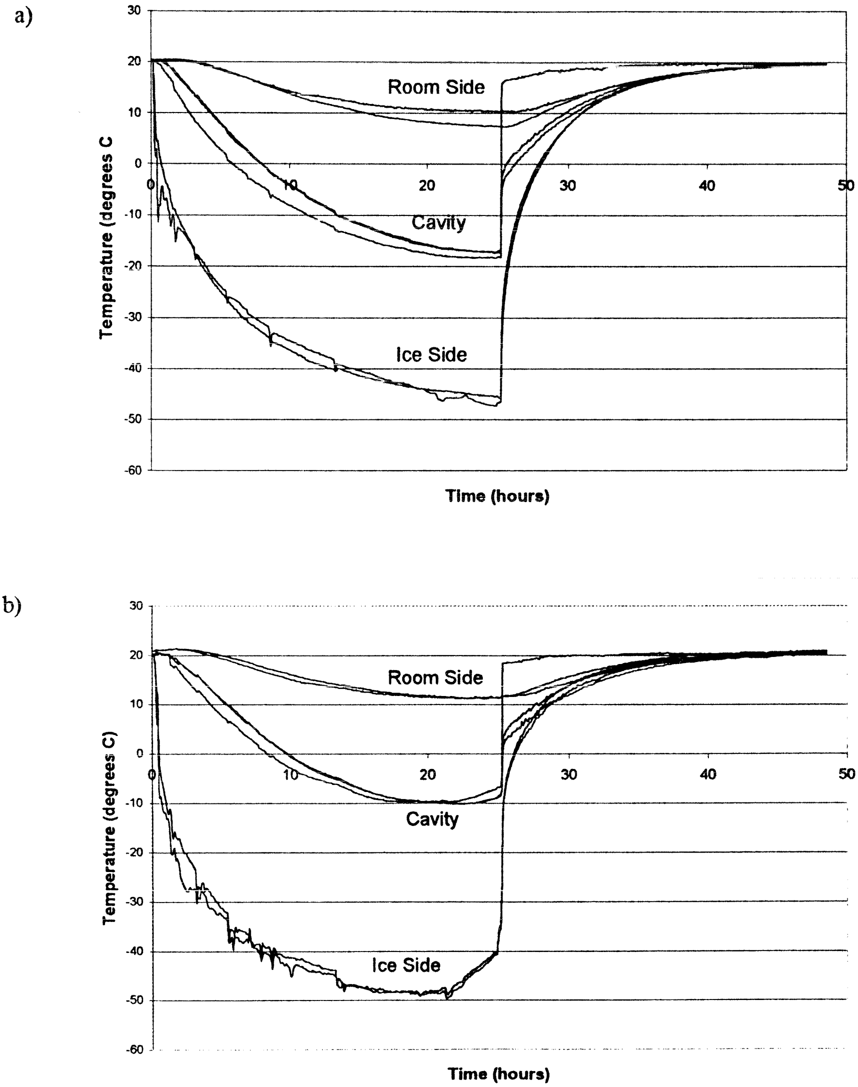
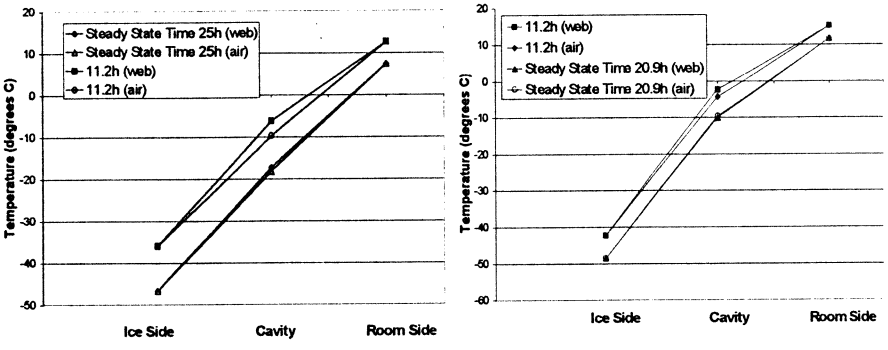
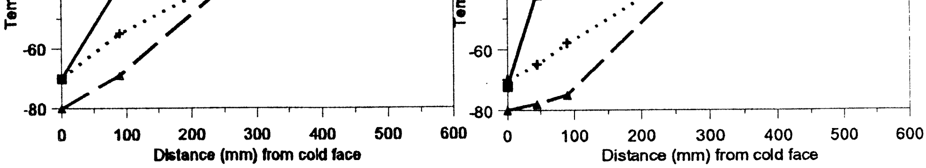


Figure 6. Cooling and Warming Cycles
(a) bottom of wall, (b) midheight of wall



(a) (b)

Figure 7. Temperature Gradients Across Diaphragm Wall
a) at the bottom, b) at midheight



(a) (b)

Figure 8. Temperature Gradients in a Diaphragm Wall
a) Across the Cavity, b) Across the Web (Shrive et al. 1996)

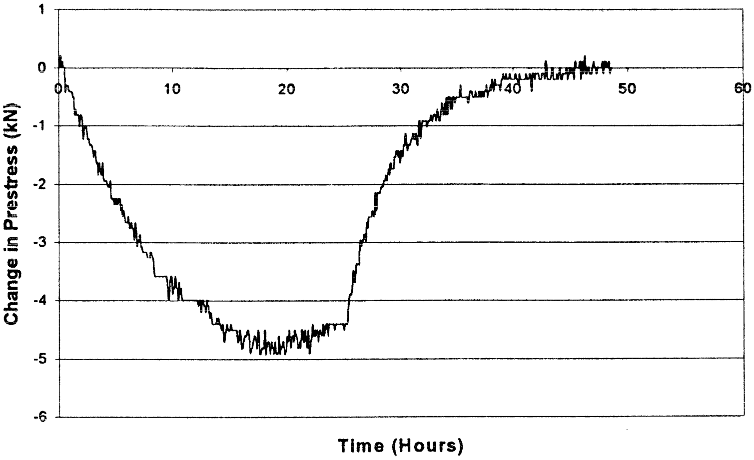


Figure 9. Prestressing Force

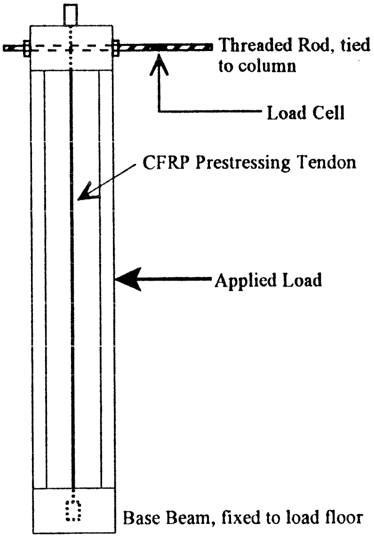


Figure 10. Flexure Test Set Up