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AN EXPERIMENTAL STUDY OF CREEP AND SHRINKAGE IN POST-TENSIONED CONCRETE MASONRY

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ABSTRACT

Time-dependent prestress losses due to creep and shrinkage lead to a reduction in structural efficiency in post-tensioned concrete masonry walls. This paper investigates the magnitude of creep and shrinkage that can be expected from medium-weight concrete masonry units manufactured in New Zealand. The findings from a creep and shrinkage experiment, lasting approximately two years, are presented and compared with test data from other researchers and values stipulated in international standards. A creep coefficient of 3.0 is found, a value in agreement with the Australian, British and Canadian standards. Shrinkage strains of 1000 microstrain and 600 microstrain for grouted and ungrouted walls respectively, were obtained under laboratory conditions.

KEYWORDS: prestress losses, post-tensioned, concrete masonry, creep, shrinkage.

INTRODUCTION

Large scale structural testing has demonstrated the favourable characteristics that post-tensioned concrete masonry (PCM) walls have over their conventionally reinforced equivalents, such as increased in-plane strength and the absence of residual post-earthquake wall displacements. Damage of PCM walls, when subjected to in-plane lateral loads, typically involves masonry crushing of the wall bottom corners, which can be easily repaired, thereby reinstating the original wall strength and stiffness [1,2,3]. Though the concept of prestressed masonry has been around for decades, the codification of prestressed masonry as a standard construction technique has only begun in more recent years. The British standard BS 5628 [4], the Australian standard AS 3700 [5], the North American code MSJC [6], the Canadian standard S304.1-04 [7], and the New Zealand standard NZS 4230 [8] all contain some guidance for using prestressed masonry in design.

Assessment of time-dependent prestressing losses is of great importance for prestressed concrete masonry structures. The effective prestress level decreases over time due to creep, shrinkage and steel relaxation, hence reducing structural efficiency. Research has indicated that prestress losses in prestressed concrete masonry may range as high as 25% of the initial stress level, depending on the masonry, the initial stress level and the prestressing hardware [9]. Losses are typically attributed to shrinkage, creep, relaxation of tendons, elastic shortening, anchorage ‘draw-in’ (seating losses), friction due to undulations of tendons or thermal effects. The experimental study outlined in this paper investigates only losses arising from creep and shrinkage of concrete masonry.

A considerable number of research projects have been conducted in various countries on creep and shrinkage of concrete masonry, typically lasting about one year. The researchers conducting those studies assessed the magnitude of creep and shrinkage as a function of materials, axial load, time and ambient temperature and humidity [10,11,12].

The precast hollow core concrete masonry blocks ordinarily used in the upper North Island of New Zealand are composed of porous pumice aggregate. With a mass of approximately 1850 kN/m^3, this type of block can be classified as light-weight in the New Zealand context. ASTM C55-03 (13) classifies concrete masonry in three weight groups: [1] light-weight with mass less than 1680 kg/m^3, [2] medium-weight with mass 1680-2000 kg/m^3 and [3] normal-weight with mass higher than 2000 kg/m^3. The North Island concrete masonry block falls in the medium-weight category according to the North America weight specification.

None of the studies previously reported apply directly to the New Zealand lightweight type of concrete masonry and none of the creep and shrinkage experiments directly addressed fully grouted concrete masonry, to the knowledge of the authors. It was therefore considered necessary to conduct a creep and shrinkage study on locally manufactured concrete masonry.

CREEP AND SHRINKAGE THEORY AND EXPRESSIONS

Despite the fact that creep and shrinkage are interactive processes and cannot be completely dissociated, the effects of creep and shrinkage traditionally are dissociated for simplicity as illustrated in Figure 1. This is an extremely simplified description of a complex interaction between time dependent effects.

The initial elastic shortening, denoted '5' in Figure 1, results from application of axial load at time t_0 and is often used as reference for describing the magnitude of creep and shrinkage. The initial elastic shortening reflects the stress level applied to the concrete and the initial elastic modulus of the concrete. Shrinkage of drying concrete, denoted '4', is characterized by volume change in the cement paste due to drying shrinkage and shrinkage due to carbonation.

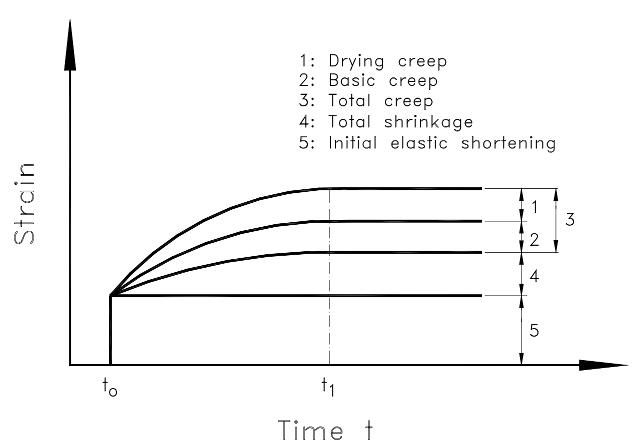

Figure 1 – Change of strain in axially loaded and drying concrete masonry

It is seen that the total creep, denoted '3', is divided into two components: [i] basic creep, which is related to the moisture content in the concrete paste that is available for evaporation (no moisture movement to or from the ambient medium), and [ii] drying creep, which occurs when an axially loaded specimen is not in hygral equilibrium with its surroundings, meaning that moisture is transferred between the specimen and the surrounding air; thus drying while under load increases the creep of concrete.

Figure 1 illustrates that at a certain age, t_1, nearly all creep and shrinkage has occurred. According to Neville [14] creep and shrinkage of concrete take place over long periods. Typically, the shrinkage after 1 year amounts to about 75% of the shrinkage after 20 years. There is no long-term data available to determine the accuracy of these numbers for concrete masonry.

The total creep strain may be expressed by Equation 1. It is assumed that the concrete masonry creep strain ε_{cr} is proportional to the initial elastic deformation ε_{mi} by the factor C_c, called the 'creep coefficient'.

$$\varepsilon_{cr} = C_c \varepsilon_{mi} = C_c \frac{f_{mi}}{E_m} \qquad \text{Equation 1}$$

In this equation, f_{mi} is the initial axial stress on the masonry cross section after anchor lock-off (prestressing + gravity load) and E_m is the initial elastic modulus of the masonry. Assuming that E_m remains nearly constant, Equation 1 can be reduced to:

$$\varepsilon_{cr} = k_c f_{mi} \qquad \text{Equation 2}$$

where k_c is termed the 'specific creep'. Equation 2 is a greatly simplified expression and has considerable appeal to codification and design work because of its simplicity.

EXPERIMENTAL INVESTIGATION

The first creep and shrinkage experiment on prestressed concrete masonry at the University of Auckland was conducted in 1998, and consisted of ten wallettes, of which five were fully grouted, four ungrouted and one partially grouted (2 of 3 flues grouted). Prestress levels ranged between 0.97 MPa and 2.8 MPa and measurements were taken over a period of 373 days [15]. In 2000 a second experiment was conducted to confirm and expand the findings of the 1998 study. Five fully grouted and five ungrouted wallettes were constructed, with prestress levels ranging between 1.00 MPa and 4.00 MPa, and being recorded for 388 days [16]. The use of improved instrumentation and differing environmental conditions, as a result of the location of the wallettes in the Civil Test Hall, led to a discrepancy between the two series and resulted in the initiation of a third series in 2002 to confirm the findings of the 2000 study. The results from the third series are presented herein.

DESIGN OF EXPERIMENTS

The experiments described in this paper were conceived to reflect common concrete masonry construction practice in New Zealand and to reflect realistic axial prestress levels expected in PCM. Each series consisted of 10 wallettes, 1.0 m high, 0.59 m wide and 0.14 m thick with dimensions shown in Figure 2(a) and (b).

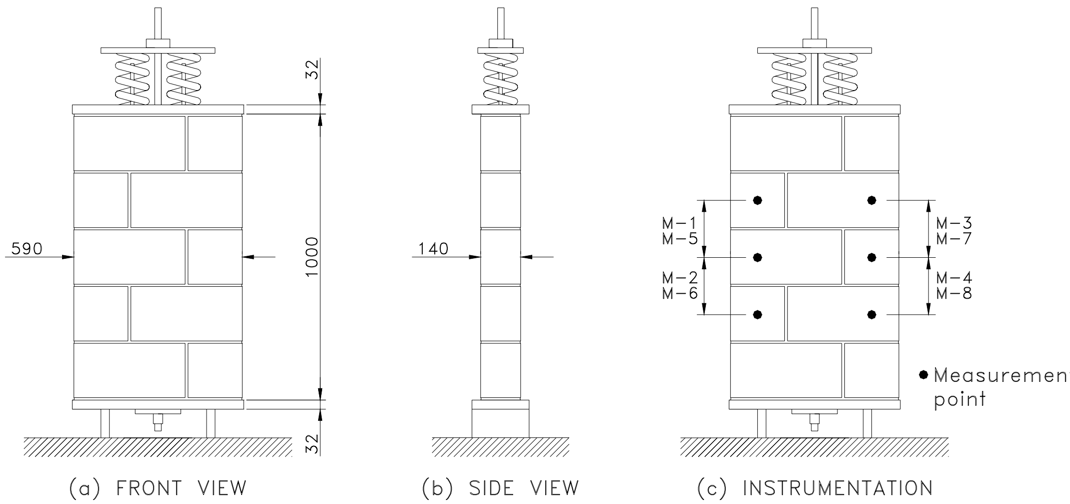

Figure 2 – Wallette dimensions, test setup and instrumentation

All wallettes were constructed of ordinary grey 15 series (140 mm wide) precast hollow block concrete masonry manufactured to the specifications of NZS 3102:1983 [17]. The wallettes were built in running bond. The mortar used was pre-blended Trade Mortar^{TM}, a mortar used for nearly all concrete masonry construction in New Zealand, consisting of cement, sand and a plasticizer with a mix ratio cement:sand of 1:3. Trade Mortar^{TM} conforms to NZS 4210:2001 [18] with a minimum compressive strength of 12.5 MPa. Each series consisted of five fully grouted (FG) and five ungrouted (UG) wallettes. The grout was specified according to NZS 4210:2001 with a minimum compressive strength of 17.5 MPa. A shrinkage compensating agent, SIKA CAVEX^{TM}, was added to the grout shortly before pouring to compensate for immediate shrinkage caused by water removed from the grout by the dry concrete masonry blocks. Shortening of the wallettes was monitored on a regular basis throughout the test duration, while ambient temperature and humidity were measured continuously at half hour intervals.

Wallettes were subjected to prestress on day 0, being 15 days after grouting, and measurements were recorded initially for 319 days. Between day 319 and day 663 data collection was suspended, but the wallettes remained stressed. A number of measurements were taken to assess the shortening after approximately 2 years, with the series concluding 755 days after stressing. The age of the blocks could not be obtained accurately but presumably ranged between 28-52 days at the day of construction. Specifications for the creep study reported here are given in Table 1. Wallettes UG1 and FG1 remained unstressed in order to ascertain the magnitude of strain loss specifically attributable to shrinkage.

The following net areas were used for computation of the initial stress, f_{mi}: For ungrouted wallettes it was assumed that only the block flanges (30 mm wide) and the ends (30 mm wide) were carrying load, as the intermediate webs had alternating locations and therefore could not transfer load. A resulting net area of 40200 mm^2 was found. The cross-sectional area of the fully grouted wallettes was taken as 82600 mm^2 (gross area).

Table 1 – Specifications of creep and shrinkage wallettes

Wall.	Grouting	Prestress f_m day 0 f_{mi}	day 755 f_{mf}	change % of f_{mi}	Masonry properties day 28 $f'_{m,28}$~	day 766 f'_m~	Load ratio day 0 $f_{mi}/f'_{m,28}$
UG1	Ungrouted	0.00	0.00	-	12.0	13.1	0.000
UG2	Ungrouted	1.01	0.95	7%	12.0	13.1	0.084
UG3	Ungrouted	2.00	1.89	6%	12.0	13.1	0.167
UG4	Ungrouted	2.52	2.37	6%	12.0	13.1	0.210
UG5	Ungrouted	3.04	2.89	5%	12.0	13.1	0.253
FG1	Fully grouted	0.00	0.00	-	17.8	18.6	0.000
FG2	Fully grouted	1.02	0.93	9%	17.8	18.6	0.057
FG3	Fully grouted	2.03	1.80	12%	17.8	18.6	0.114
FG4	Fully grouted	2.29	1.95	15%	17.8	18.6	0.129
FG5*	Fully grouted	-	-	-	-	-	-
		MPa	MPa	-	MPa	MPa	-

* failed by vertical splitting during stressing, ~ measured by testing 3 prisms

TEST SETUP AND INSTRUMENTATION

Figure 2(c) shows schematically the test setup and instrumentation configuration used. Heavy coil springs were installed in series with the wallettes between the tendon anchorages in order to keep the prestressing force approximately constant over time. The wallettes were built in the Civil Test Hall and were stored in the basement of the Test Hall after application of axial load.

Axial shortening strains were measured via a digital gauge between measurement points. Each measurement sequence (one measurement over each set of measurement points for all wallettes) that was taken at a particular time was repeated three times to improve the reliability of the data. The Demec instrumentation layout is shown in Figure 2(c). Eight measurements were taken (four on each side of each wallette), in a uniform stress region, directly on the masonry over a gauge length of 200 mm.

A reference gauge length of 200 mm was provided by a reference bar made of INVAR steel with a very low coefficient of thermal expansion. This bar was measured 6 times for each measurement sequence (before and after every two wallettes). It was found using the Demec system that length measurements (and changes) could be replicated with good accuracy for repeated measurement sequences taken at the same day.

EXPERIMENTAL RESULTS

Figure 3 shows the shortening strain (shrinkage + creep) curves for all wallettes, with the unstressed wallettes representing the pure shrinkage strain. Figure 4 shows the creep strain curves. The creep results presented for the grouted and ungrouted wallettes were not corrected for temperature and humidity variations throughout the sampling period. Figure 5 shows the variation of the ambient air temperature and relative humidity (24 hour mean).

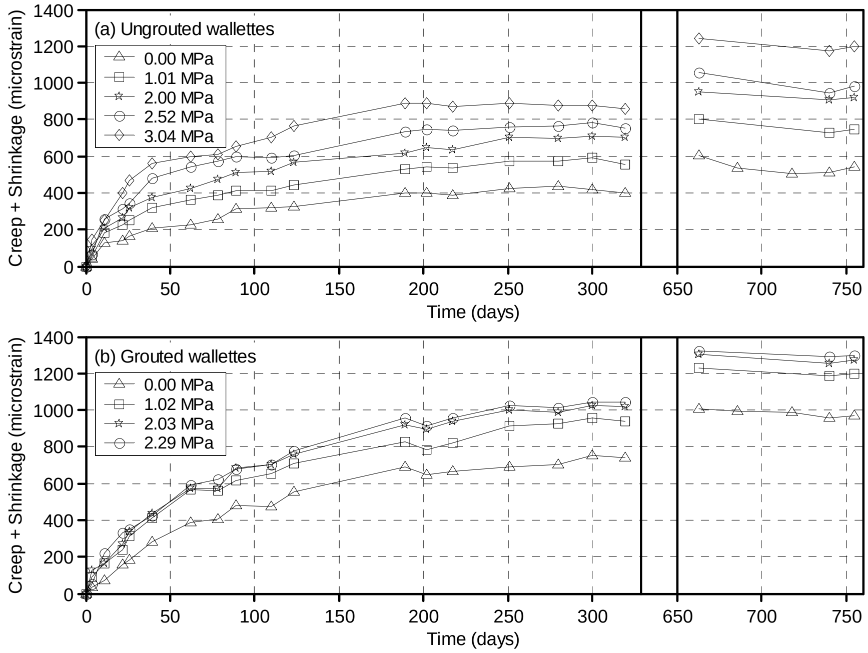

Figure 3 – Creep + shrinkage strain

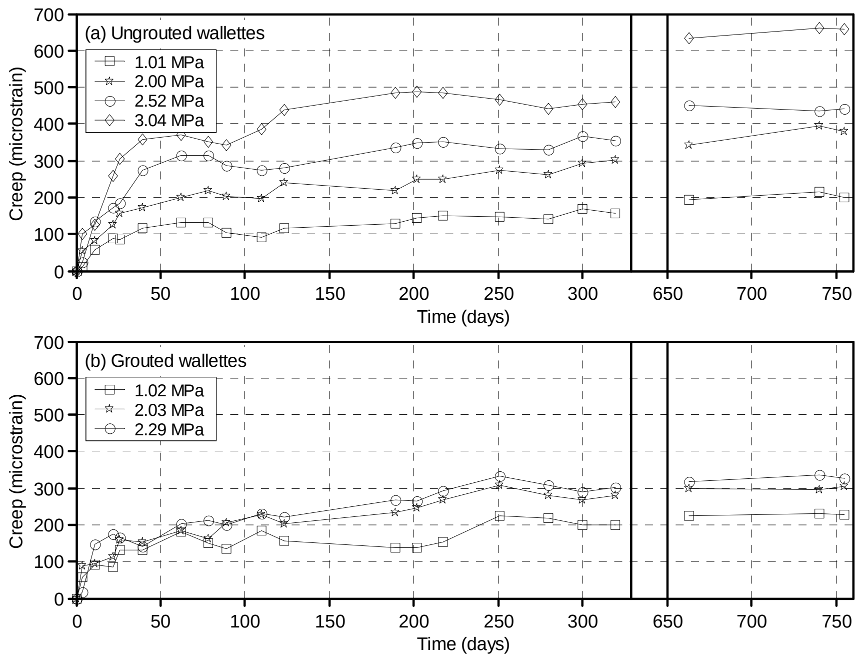

Figure 4 – Creep strain

The shrinkage curves, shown in Figure 3, appear to have begun levelling around 200 days after stressing. Wallette measurements taken between days 663 and 755 show a slight expansion, which is attributed to the increase in both ambient temperature and humidity over this period. At the time of de-stressing (day 755) the ambient temperature and humidity were approximately equal to the values recorded on the day of stressing. Figure 4 indicates that the grouted wallette creep had levelled out after the first 300 days, whereas the ungrouted wallettes continued to creep showing an increase in measured strain between days 319 and 663. The creep curves of all wallettes were approximately level by the end of recording.

A summary of results is presented in Table 2. It is seen from this table that there was good agreement between the measured and estimated initial elastic shortening values. The creep coefficient and the specific creep given in the table were based on the measured initial elastic shortening, ε_{mi}. Wallette FG5 split upon prestressing (day 0) due to the attempted high level of prestress (4 MPa), a level unlikely to be specified in design.

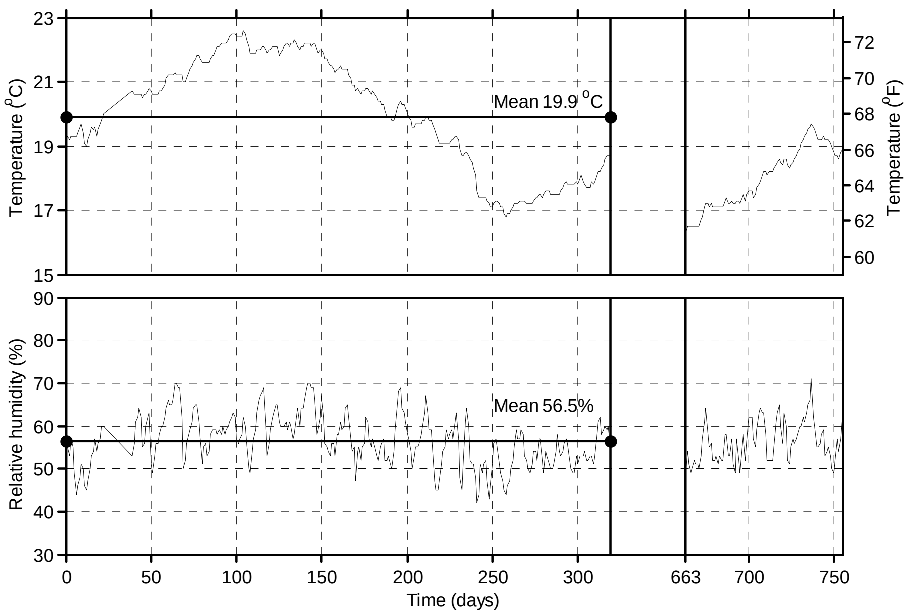

Figure 5 – Ambient air temperature and humidity variations

Table 2 – Summary of results

Wall.	Applied stress f_{mi}	Elastic shortening measured ε_{mi}	Elastic shortening theoretical $\varepsilon_{mi,t}$	Shortening under sustained load* total shortening	Shortening under sustained load* shrinkage ε_{sh}	Shortening under sustained load* creep ε_{cr}	Creep Coeff.[+] C_c	Specific creep[+] k_c
UG1	0.00	0	0	606	606	0	-	-
UG2	1.01	123	106	800	606	216	1.75	213
UG3	2.00	211	208	949	606	395	1.87	197
UG4	2.52	277	263	1058	606	452	1.63	179
UG5	3.04	306	317	1239	606	661	2.16	218
FG1	0.00	0	0	1005	1005	0	-	-
FG2	1.02	83	71	1231	1005	255	3.08	251
FG3	2.03	162	143	1304	1005	309	1.91	152
FG4	2.29	166	161	1322	1005	337	2.03	147
FG5	-	-	-	-	-	-	-	-
	MPa	με	με	με	με	με	-	με/MPa

* maximum values, + based on $\varepsilon_{mi,t}=f_{mi}/E_m$ where $E_m=800f'_m$

DISCUSSION OF EXPERIMENTAL RESULTS

The environment in the Test Hall basement is considered to result in shrinkage strains exceeding those expected for walls in a realistic outdoor environment. This is because an outdoor environment (in New Zealand) is characterised by lower average temperature and higher relative humidity than those of the Civil Test Hall indoor environment. The maximum shrinkage strain measured, ε_{sh} = 606 $\mu\varepsilon$ (microstrain) for ungrouted concrete masonry and ε_{sh} = 1005 $\mu\varepsilon$ for grouted concrete masonry, may therefore be considered conservative. When considering the shrinkage results it should be kept in mind that the measurements only include shrinkage recorded after application of prestress.

The creep and shrinkage strains measured should theoretically be amplified to account for the creep and shrinkage expected to occur beyond the test period. No magnification has been adopted herein due to a lack of knowledge regarding suitable values.

The experimentally derived creep coefficient, C_c, is shown for each wallette in Table 2. The creep coefficients were calculated according to Equation 1, using the maximum-recorded values of creep strain. The results are based on the modulus of elasticity, E_m, defined by Laursen as equalling 800f'_m [16]. Maximum-recorded values for C_c of 2.2 and 3.1, for ungrouted and grouted masonry respectively, were found.

The specific creep is also shown in Table 2 and was calculated from Equation 2, again using the maximum-recorded values of creep strain. The maximum specific creep for ungrouted concrete masonry was k_c = 218 $\mu\varepsilon$/MPa and for grouted concrete masonry was k_c = 251 $\mu\varepsilon$/MPa.

OTHER EXPERIMENTAL WORK

A compilation of recent experimental work on creep of concrete masonry was prepared by Hamilton and Badger [10]. The results were given in terms of k_c and were specific to ungrouted concrete masonry. Values for light-weight and normal-weight units were presented and categorized with respect to the utilized mortar, M, S and N (refer to MSJC [6] for definition of mortar types), with typical mortar compressive strengths N: 5.2 MPa, S: 12.4 MPa and M: 17.2 MPa being adopted. In New Zealand only one type of mortar is commonly used: Trade Mortar™. It's a pre-mixed and pre-bagged product characterised by a specified strength of 12.5 MPa, and would be classified as S-mortar in the US.

Table 3 shows a compilation of research results, including the results found by earlier experiments conducted at the University of Auckland. Only results from experiments lasting more than 300 days are included. Results from the Laursen experiments refer to the study at the University of Auckland in 2000.

CODE VALUES FOR CREEP AND SHRINKAGE

Table 4 summarizes values from creep and shrinkage provisions of AS 3700 [5], BS 5628 [4], Canadian standard S304.1-04 [7] and MSJC [6]. These provisions do not clearly distinguish between grouted and ungrouted masonry, so it is presumed that the values are primarily applicable to ungrouted concrete masonry. It should be noted that the calculation of creep strain due to C_c is dependent on E_m, which is obtained in different ways by the individual codes.

Table 3 – Experimental values of k_c

Researcher	Weight Category	Grouting	Mortar type M	S	N	Duration of loading
Maksoud (12)	normal	no		55.1-74.0		400
Hamilton & Badger (10)	normal	no		92.8-189		300
Laursen (16)	medium	no		198-282		388
Wight & Ingham	medium	no		179-218		755
Laursen (16)	medium	yes		115-219		388
Wight & Ingham	medium	yes		147-251		755
Harvey & Lenczner (11)	light	no	162-186	46.4-51.6	78.7-82.4	300
Maksoud (12)	light	no		74.0-92.8		400
Hamilton & Badger (10)	light	no		34.8-74.0		300
			με/MPa	με/MPa	με/MPa	days

Table 4 – Code defined creep and shrinkage coefficients

Source Document	C_c	k_c	ε_{sh}
AS 3700	2.5	170#	700
BS 5628	3.0	210#	500
S304.1-04~	3.0-4.0	210-280#	100-200
S304.1-04+	4.0-5.0	280-350#	200-300
MSJC*	0.5#	36	100
		με /MPa	με
~normal-weight, +light-weight, *moisture controlled units, #using $E_m = 800f'_m$ and $f'_m = 18$ MPa			

COMPARISON

It is seen from Table 3 that the range of k_c values for ungrouted concrete masonry correlate well with the study conducted by Laursen in 2000, and with the top end of the Hamilton & Badger range. The k_c values for grouted concrete masonry are somewhat larger than those measured for ungrouted masonry and again correlate well with the Laursen study.

Table 4 shows that the AS 3700, BS 5628 and S304.1-04 values of C_c are of similar magnitude, but that the MSJC value is much lower. Comparison of the obtained C_c values to code values reveals that a C_c of the order of 3.0 is consistent. The MSJC value seems unrealistically low.

In terms of estimated shrinkage ε_{sh}, it appears that the BS 5628 and AS 3700 values of 500 με and 700 με, respectively, are similar, but that the MSJC value is much lower, with the Canadian code values lying in between. The value obtained in this study for ungrouted concrete masonry (606 με) is similar to the BS 5628 and AS 3700 values. For grouted concrete masonry a higher shrinkage was measured (1005 με), a value that exceeded the values outlined in all four codes.

CONCLUSION

For grouted and ungrouted concrete masonry it is recommended to use the creep coefficient $C_c = 3.0$, a value in agreement with AS 3700, BS 5628 and S304.1-04. The following shrinkage strains were recommended for design under New Zealand conditions: for grouted walls $\varepsilon_{sh} = 1000$ με and for ungrouted walls $\varepsilon_{sh} = 600$ με.

Creep and shrinkage properties for partially grouted concrete masonry should be based on interpolation between the results for grouted and ungrouted concrete masonry according to the ratio of grouted cells to the total number of cells.

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REFERENCES

1. Laursen, P. T. and Ingham, J. M., Structural Testing of Enhanced Post-Tensioned Concrete Masonry Walls, ACI Structural Journal, V. 101, No. 6, Nov-Dec 2004, pp. 852-862.
2. Laursen, P. T. and Ingham, J. M., Structural Testing of Large-Scale Post-Tensioned Concrete Masonry Walls, ASCE J. of Structural Engineering, V. 130, No. 10, 2004, pp. 1497-1505.
3. Wight, G. D., Ingham, J. M. and Kowalsky, M. J., Shake Table Testing of Post-Tensioned Concrete Masonry Walls, 13th International Brick/Block Masonry Conference, Amsterdam, The Netherlands, July 4-7 2004, pp. 1059-1068.
4. BS 5628: 1995 Part 2:, Code of Practice for use of Masonry. Structural Use of Reinforced and Prestressed Masonry, British Standards Institution, London, 1995.
5. AS 3700-1998, Masonry Structures, Standards Assoc. of Australia, Homebush, NSW, 1998.
6. MSJC, Building Code Requirements for Masonry Structures and Specification for Masonry Structures, Masonry Standards Joint Committee, USA, 2002.
7. S304.1-04, Design of Masonry Structures, Canadian Standards Association, Mississauga, Ontario, Canada, 2004.
8. NZS 4230:2004, Design of Reinforced Concrete Masonry Structures, Standards Association of New Zealand, Wellington, New Zealand, 2004.
9. Schultz, A. E. and Scolforo, M. J., An Overview of Prestressed Masonry, The Masonry Society Journal, V. 10, No. 1, August 1991, pp. 6-20.
10. Hamilton III, H. R. and Badger, C. C. R., Creep Losses in Post-Tensioned Concrete Masonry, The Masonry Society Journal, V. 18, No. 1, July 2000, pp. 19-30.
11. Harvey, R. J. and Lenczner, D., Creep Prestress Losses in Concrete Masonry, Proc. 5th Int. RILEM Symposium on Creep and Shrinkage of Concrete, Barcelona, Spain, 6-9 June 1993.
12. Maksoud, A., Short and Long Term Capacities of Slender Concrete Block Walls, PhD thesis, McMaster University, Hamilton, Ontario, Canada. 1994.
13. ASTM C55-03, Standard Specification for Concrete Brick, ASTM International.
14. Neville, A. M., Properties of Concrete, 4th Ed., Longman Group, Harlow, 1995, 844 pp.
15. Laursen, P. T., Ingham, J. M. and Voon, K. C., Material Testing supporting a study of Prestressed Concrete Masonry, 12th International Brick/Block Masonry Conference, Madrid, Spain, 25-28 June 2000, pp. 921-935.
16. Laursen, P. T., Seismic Analysis and Design of Post-tensioned Concrete Masonry Walls, PhD thesis, University of Auckland, Auckland. 2002. 282p.
17. NZS 3102:1983, Concrete Masonry Units, Standards Association of New Zealand, Wellington, New Zealand, 1983.
18. NZS 4210:2001, Masonry Construction: Materials and Workmanship, Standards Association of New Zealand, Wellington, New Zealand, 2001.