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NCMA TEK

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CONTROL JOINTS FOR CONCRETE MASONRY WALLS - ALTERNATIVE ENGINEERED METHOD

TEK 10-3
Movement Control (2003)

Keywords: bond beams, construction details, control joints, crack control, engineered method, joint reinforcement, reinforcing bars, reinforced concrete masonry, shrinkage, wall movement

INTRODUCTION

This TEK supplements TEK 10-2B *Control Joints for Concrete Masonry Walls - Empirical Method* (ref. 3). The reader is encouraged to refer to that TEK for other pertinent information such as construction details and location of control joints.

Controlling shrinkage cracking historically has been addressed by limiting the moisture content of units at the time of placement in the wall and indirectly incorporated the effects of variations in temperature and cement carbonation as well as drying shrinkage. In 2000 however, due to problems associated with maintaining moisture controlled units (Type I) in that state until placement in the wall, moisture controlled units (and Type designations) were removed from ASTM C 90 (ref. 4).

In view of this, the concrete masonry industry has developed an engineered approach to controlling cracking which examines each of these three parameters separately, and then incorporates them into a single Crack Control Coefficient. In general, this engineered approach is more complicated and requires more detailed knowledge of the masonry characteristics than the empirical approach of TEK 10-2B (ref. 3), which is based on historical solutions that have proven successful over many years of experience for a broad geographic distribution. The empirical method is the most commonly used method and is applicable to most conventional building types. The engineered method is generally used only when unusual conditions are encountered such as dark colored units in climates with large temperature swings.

ENGINEERED CRACK CONTROL CRITERIA

The engineered criteria was developed to produce a more rational approach to crack control in concrete masonry - particularly in areas of high seismicity where relatively large amounts of continuous reinforcing steel are used. Also addressed is additional reinforcement around openings to provide strengthening and allow placement of the control joints at locations other than at the openings. The effectiveness of this method depends on reliable criteria being correctly incorporated into the project design, the materials meeting the requirements of the project specifications, and the masonry being constructed in accordance with the project drawings.

The engineered criteria is based on a Crack Control Coefficient to accommodate internal volume changes. Once the internal movement due to volume change has been estimated, the designer can control crack width to a maximum value by 1) limiting the distance between control joints when used in combination with a minimum amount of horizontal reinforcement or 2) incorporating a predetermined, higher amount of horizontal reinforcement (when needed for structural purposes) to limit crack width without the use of control joints.

Crack Control Coefficient

The Crack Control Coefficient (CCC) is an indicator of anticipated wall movement. Concrete masonry unit shortening per unit length is estimated by including the possible combined effects of movement due to drying shrinkage, carbonation shrinkage and contraction due to temperature reduction. The Crack Control Coefficient value itself is determined by summing the coefficients of these three properties for a specific concrete masonry unit. It is a function of unit mix design and production/curing methods.

The total linear drying shrinkage is determined in accordance with *Standard Test Method for Linear Drying Shrinkage of Concrete Masonry Units and Related Units*, ASTM C 426 (ref. 5). ASTM C 90 (ref. 4) limits total linear drying shrinkage of concrete masonry units to 0.00065 in./in. (mm/mm). Note that this is based a saturated condition (immersed in water for 48 hours). In the field, units will probably be no higher than 70% of saturation. Therefore, the highest realistic drying shrinkage potential realized in the field will be around 0.00045 in./in. (mm/mm) or 0.54 in. in 100 ft (13.7 mm in 30.48 m). It for this reason that the *Building Code Requirements for Masonry Structures* (ref. 1) stipulates the use of only 50% of the total linear drying shrinkage determined in accordance with ASTM C 426 (ref. 5) for design.

Table 1—Criteria for Controlling Cracking in Reinforced Concrete Masonry Walls[a]

		Crack Control Coefficient in./in. (mm/mm) 0.0010	0.0015
Maximum wall panel dimensions[2]	length, ft (m)	25 (7.62)	20 (6.10)
	length/height ratio	2 ½	2
Minimum horizontal reinforcement ratio A_s/A_n		0.0007	0.0007

[a]Notes:
1. A_s = cross-sectional area of steel, in.^{2}/ft (mm^2/m).
   A_n = net cross-sectional area of masonry, in.^{2}/ft (mm^2/m).
2. Maximum wall panel dimension criteria need not apply for walls with a minimum horizontal reinforcement area, A_s of 0.002 times the net cross sectional area of the masonry, A_n – see Table 3.
3. See Table 2 for maximum spacings of reinforcement to meet 0.0007 minimum horizontal reinforcement ratio A_s/A_n.
4. The minimum horizontal reinforcement ratio criteria need not apply for walls with a length not exceeding one half the maximum length values shown above.
5. CCC's less than 0.0010 may be available in some areas and spacing could be adjusted accordingly for this as well.
6. This criteria is based on an analytical study over a wide geographical area with wide temperature and and material property variations. Control joint spacing may be adjusted up or down based on local experience.
7. As shrinkage is related to moisture content, consider using the higher crack control coefficient for masonry units that are wet from lack of protection while stored on the jobsite.

Carbonation shrinkage is an irreversible reaction between cementitious materials and carbon dioxide in the atmosphere. It occurs over a long period of time and there currently is no standard test method to determine it. Therefore, it is recommended that 0.00025 in./in. (mm/mm) be used for the carbonation shrinkage coefficient – 0.3 in. in 100 ft (7.6 mm in 30.48 m).

Thermal coefficients for concrete masonry units typically range from 0.0000025 to 0.0000055 in./in./°F (0.0000045 to 0.0000099 mm/mm/°C) (refs. 4 and 5). For design purposes, the value of 0.000004 in./in./°F (0.0000081 mm/mm/°C) be used as outlined in the *Building Code Requirements for Masonry Structures* (ref. 1). Based on a temperature change of 70 °F (38.9 °C), this would translate to a thermal contraction value of 0.00028 in./in. (mm/mm) or 0.34 in. in 100 ft (8.5 mm in 30.48 m).

The CCC is the sum of the potential length change due to each of these three parameters and for typical concrete masonry units varies from 0.00063 to 0.00108 in./in. (mm/mm). This range corresponds to a 100 ft (30.48 m) long wall shortening 0.76 to 1.30 in. (19.3 to 33.0 mm).

Control Joints with Horizontal Reinforcement

The most common (and usually most cost effective) method of controlling cracks in concrete masonry is to use control joints in conjunction with a minimum amount of horizontal reinforcement between the joints. Reinforcement is often required for wind or seismic resistance and it is prudent to utilize it for assisting in crack control as well. The amount of horizontal reinforcement needed is based on limiting cracks to a width of 0.02 in. (0.51 mm) since water repellent coatings can effectively resist water penetration for cracks of this size. Based on this premise and the CCC criteria discussed earlier, control joint spacing criteria are presented in Table 1 utilizing a minimum horizontal reinforcement ratio of $A_s/A_n \geq 0.0007$. A_n is the net area of the vertical cross-section of the wall. For hollow unit masonry and partially grouted masonry it essentially is the total thickness of the face shells times the height of the wall plus the additional area provided by any grouted bond beams. Table 2 presents the maximum spacing of the various sizes of typical horizontal reinforcement to meet the 0.0007 criteria. The wall panel length to height ratio and the maximum length of wall panel criteria in combination with horizontal reinforcement in Table 1 are based on historical field experience and analytical studies.

Horizontal Reinforcement Only

In some regions of the country, significant amounts of horizontal reinforcement are required for structural purposes, i.e. Seismic Performance Categories D and E. Studies have shown that horizontal reinforcement of sufficient quantity can effectively limit crack width in concrete masonry walls. For standard reinforcing bar sizes, horizontal reinforcement spacings up to 48 in. (1219 mm) o.c. have been shown to effectively control cracking without the use of control joints. It has also been shown that horizontal reinforcement provides internal restraint, which results in transfer of tension from the masonry to the reinforcement, resulting in more frequent but much smaller cracks. As the level of horizontal reinforcement increases, cracking becomes more uniformly distributed and crack width decreases.

When a crack is formed, tension in the masonry is released. This masonry tension is transferred to the reinforcement at the time of crack formation. Therefore, reinforcement should be sized such that the resulting tensile force in the reinforcement does not exceed the yield strength of the steel. This keeps the steel within the elastic range and minimizes the crack width to a point where control joints are not necessary in the design.

To ensure the steel is within the elastic range, the width of a crack at the horizontal reinforcement location would be limited to the yield strain of the steel multiplied by the length of reinforcing bar being strained:

crack width = $?_s\, l$

where: $?_s$ = yield strain of steel, f_y/E_s
= 60,000 psi / 29,000,000 psi (413 MPa/ 199,810 MPa)
= 0.002 in./in. (mm/mm)
l = length of reinforcing bar being strained

For example, a bar being strained at its yield strain over a length of 10 in. (254 mm) would limit crack width to 0.02 in. (0.51 mm).

In order to meet this criteria of limiting the steel to the elastic range, the tension in the masonry ($T_m = F_t A_n$) just prior to crack formation must be less than the yield strength of the steel ($T_s = f_y A_s$):

$F_t A_n \leq f_y A_s$, or

$A_s \geq F_t A_n / f_y$,

where:

F_t = average tensile strength of masonry. A vertical

Wall thickness, in. (mm)	No. 5 (M 16)	No. 4 (M 13)	No. 3 (M 10)	4 x3/16 in. (MW 18)	4 x 8 gage (MW 13)	4 x 9 gage (MW 11)	2 x3/16 in. (MW 18)	2 x 8 gage (MW 13)	2 x 9 gage (MW 11)
Table 2—Maximum Spacing of Horizontal Reinforcement to Meet the Criteria As > 0.0007An[1]									
	Maximum spacing of horizontal reinforcement, in. (mm) Reinforcement size								
Ungrouted or partially grouted walls									
6 (152)	144 (3658)	128 (3251)	64 (1626)	72 (1829)	56 (1422)	48 (1219)	40 (1016)	24 (610)	24 (610)
8 (203)	144 (3658)	96 (2438)	40 (1016)	64 (1626)	48 (1219)	40 (1016)	32 (813)	24 (610)	16 (406)
10 (254)	136 (3458)	80 (2032)	32 (1219)	56 (1422)	40 (1016)	32 (813)	16 (406)	16 (406)	16 (406)
12 (305)	120 (3048)	72 (1829)	24 (610)	48 (1219)	40 (1016)	32 (813)	16 (406)	16 (406)	16 (406)
Fully grouted walls									
6 (152)	72 (1829)	48 (1219)	24 (610)	24 (610)	16 (406)	16 (406)	8 (203)	8 (203)	8 (203)
8 (203)	56 (1422)	32 (813)	16 (406)	16 (406)	8 (203)	8 (203)	8 (203)	8 (203)	—
10 (254)	40 (1016)	24 (610)	16 (406)	16 (406)	8 (203)	8 (203)	8 (203)	—	—
12 (305)	32 (813)	24 (610)	8 (203)	8 (203)	8 (203)	8 (203)	—	—	—

1. A_n includes cross-sectional area of grout in bond beams

crack would pass through a head joint and then a block in alternate fashion. The tensile strength of typical masonry units is 200 psi (1.38 MPa) and the tensile strength of a typical head joint is 25 psi (0.172 MPa). Average tensile strength is, therefore, 225 psi / 2 or 112 psi (0.772 MPa).

f_y = yield strength of steel reinforcement
= 60,000 psi (413 MPa)

Substituting these values, the criteria becomes:

$A_s \geq 0.0019\, A_n$

When this condition is met, there is sufficient horizontal steel to limit masonry cracking to widths of 0.02 in. and control joints may be eliminated as stated in footnote 2 of Table 1. Table 3 indicates the amount of reinforcement that will meet this criteria for various concrete masonry walls.

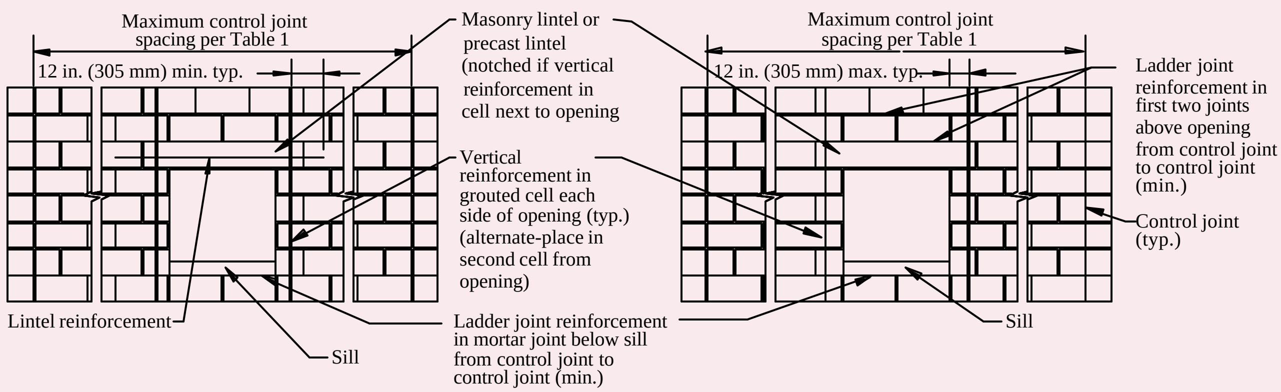
Wall thickness, in. (mm)	No. 6 (M19)	No. 5 (M16)	No.4 (M13)
Table 3—Maximum Spacing of Horizontal Reinforcement to Meet the Criteria As > 0.002An[1]			
	Maximum spacing of horizontal reinforcement, in. (mm) Reinforcement size		
Ungrouted or partially grouted walls			
6 (152)	48 (1219)	48 (1219)	32 (813)
8 (203)	48 (1219)	40 (1016)	24 (610)
10 (254)	48 (1219)	32 (813)	16 (406)
12 (305)	48 (1219)	24 (610)	8 (203)
Fully grouted walls			
6 (152)	32 (813)	24 (610)	16 (406)
8 (203)	24 (610)	16 (406)	8 (203)
10 (254)	16 (406)	16 (406)	8 (203)
12 (305)	16 (406)	8 (203)	8 (203)

1. A_n includes cross-sectional area of grout in bond beams

Control Joints in Vertically Reinforced Walls

In plain masonry walls, control joints are typically placed at an opening as it is a weak point subject to cracking due to the reduced masonry cross section. This requires the control joint above the opening to be aligned with the end of the lintel, cross under the lintel via a slip plane, and then proceed through the opening (ref. 3). In walls containing vertical reinforcement, however, the cell adjacent to the opening usually is grouted and reinforced. Using the same type of detail would require the control joint to cross the vertical reinforcement thereby preventing movement and defeating the purpose of the control joint. However, if the opening is completely surrounded by reinforcement as detailed in Figures 1 and 2, the area through the opening is strengthened and control joints can be placed outside the opening. For best performance the vertical reinforcement should be placed in the cell immediately adjacent to the opening. However, due to congestion in the cell at this location, vertical reinforcement is often placed in the second cell from the opening. On large openings, it is recommended to grout the cell next to the opening as well as the cell containing the reinforcement to provide additional resistance for attaching the door or window frame. These details may also be used in unreinforced walls and walls utilizing steel lintels since the area through the opening is strengthened by the additional reinforcement.

When utilizing these details and the wall segments on either side of openings are designed to resist the lateral loads applied directly to them plus those transferred from the opening enclosure, shear transfer devices such as preformed gaskets (see TEK 10-2B, ref. 3) are not necessary. However, some designers still incorporate them to limit the relative movement between the two panels on either side of a control joint thereby reducing the stress on the joint sealant and providing longer life.



By Extending Lintel Reinforcement (Preferred)

By Adding Joint Reinforcement (Alternate)

Figure 1—Reinforcement Around Openings Option

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