MSJC PROVISIONS FOR THE DESIGN OF MASONRY DEEP BEAMS

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The theory used for the design of slender beams has a limited applicability to deep beams, resulting in designs that are generally not conservative. Shear warping of the cross section and a combination of diagonal and flexural tension stresses in the body of a deep beam require that deep beam theory be used for design of such members. The Masonry Standards Joint Committee has recently proposed a definition of deep beams as well as design provisions and requirements, which were approved by appropriate committees and by the public and have been incorporated in the 2011 Building Code Requirements and Specification for Masonry Structures. This article summarized the work conducted by the Flexural and Axial Load subcommittee of the Masonry Standards Joint Committee in developing the deep beam requirements.

Keywords: Codes, Regulations, Deep Beams, Design

INTRODUCTION

The theory used for the design of slender beams has a limited applicability to deep beams, resulting in designs that are generally not conservative. Shear warping of the cross section and a combination of diagonal and flexural tension stresses in the body of a deep beam require that deep beam theory be used for design of such members. The Masonry Standards Joint Committee (MSJC) has recently proposed a definition, design provisions and requirements for deep beams, which were approved by appropriate committees and by the public and are incorporated in the 2011 Building Code Requirements and Specification for Masonry Structures (Masonry, 2011).

THEORETICAL BACKGROUND

The fundamental hypothesis of the flexure theory is that stress distribution across the section is proportional to the distance from the neutral axis of bending, i.e., plane sections, through the cross section of a beam, perpendicular to its axis remain plane after the beam is subjected to bending. Beams, however, are hardly subjected to pure bending. Hence cross sections of beams subjected to simultaneous bending and shear do not remain plane, they warp. Warpage of the sections are important for beams with small span-to-depth ratios (Popov, 1990) and smaller the span-to-depth ratio the more pronounced the deviation from the linear stress hypothesis (Park and Paulay, 1975). According to Park and Paulay (1975), the deviation from the fundamental linear stress hypothesis should be considered for beams having span-to-depth ratios less than 2.5. These small span-to-depth ratio beams are commonly found above
warehouse dock doors, in foundations walls, and in shear wall structures that resist lateral forces, and they must be designed using the appropriate theory.

Figure 1 shows the distribution of horizontal flexural stresses at the midspan of a homogeneous simply supported beam having different span-to-depth ratios and being subjected to a uniformly distributed load. Flexural theory predicts, for a beam with a span-to-depth ratio of one, tensile stresses at the bottom fiber equal to $0.75 \frac{w}{b}$, where $w$ is the magnitude of the distributed load and $b$ is the width of the beam. As Figure 1 shows, however, the intensity of the actual tensile stresses at the bottom fiber for such a small span-to-depth ratio beam is $1.6 \frac{w}{b}$, which is more than twice the intensity predicted by the flexural theory. The theoretical development for the stresses on deep beams is given by Dischinger (1932). The report by Dischinger is in German and a summary in English is given by the Portland Cement Association (Design, 1946). Other researchers (Roark and Young, 1985) confirm the development by Dischinger. The design of deep masonry beams, therefore, must consider the deviation from the fundamental linear stress hypothesis; otherwise, designs of these elements will continue to be generally not conservative.

**Figure 1:** Distribution of Flexural Stresses in an Isotropic Homogeneous Simply Supported Beam; from Leonhardt and Walther (1966) and Park and Paulay (1975).
DESIGN CODES PROVISONS
A literature review indicated that masonry design codes from Canada (Design 2004a), the European Union (Eurocode 2005), and New Zealand (Design 2004b) have provisions for the design of deep masonry beams. The United Kingdom had its own code of practice for the use of masonry (Code 2005); however, that code has been withdrawn and replaced by the Eurocode 6 – Design of Masonry Structures (2005). Further review of the literature indicated that deep masonry beam requirements in current masonry design codes are similar to the recommendations by the Comite Europeen du Beton (1970) for the design of deep concrete beams, which are based on the experimental investigation conducted by Leonhardt and Walther (1966). Park and Paulay (1975) provide an excellent summary of the original report.

The only masonry deep beam provision in the US is that given in the ASCE Standard Minimum Design Loads for Buildings and Other Structures (Minimum 2010). The proposed provisions add a definition, minimum flexural tension reinforcement requirement, and minimum distributed horizontal and vertical reinforcement requirement to Masonry Standard Joint committee (MSJC) Standard (Masonry 2011).

NEW PROVISONS FROM MSJC
The new provisions include a definition, a design method, requirements for flexural reinforcement requirements, and requirements for shear and web reinforcement.

Deep Beam Definition: A summary of the literature review findings is presented in Table 1. In most documents, deep beam definition is given in terms of span-to-depth ratio; few documents, however, give the definition in terms of depth-to-span ratio. For direct comparison between the different values, all definitions in Table 1 are given in terms of span-to-depth ratios. For comparison, the British Standard (Code 2005) definitions are also presented. Some of the codes give the definition in terms of clear span and overall depth while others use effective span and effective depth. In addition, some codes differentiate between simple and continuous span.

As previously mentioned, deviation from the linear stress hypothesis is important only for beams with small span-to-depth ratios, and the smaller the span-to-depth ratio the more pronounced the deviation. Although it is theoretically possible to determine the exact span-to-depth ratio at which the linear stress hypothesis is no longer applicable, such high degree of accuracy is not warranted for most practical design problems. Thus, the small variation in values presented in Table 1 is not significant and a slightly larger value can be viewed as representing the conservativeness of a specific code.

The adopted MSJC deep beam definition is a beam that has an effective span-to-depth ratio, $l_{eff}/d_e$, less than 2 for a simple span and less than 3 for a continuous span. A new variable, effective span ($l_{eff}$), has been introduced and is to be taken as the smaller value between (a) the center-to-center distance between supports, $\ell_c$, or (b) 1.15 multiplied by the clear span, $\ell_n$; $d_e$ is the effective depth of the beam. The use of the effective span recognizes that the point of rotation of the beam can be somewhere between the face of the support (lower bound) and the center of the support (upper bound). The upper bound value essentially acknowledges that the point of rotation of the beam cannot be beyond the center of the support.
### Table 1: Deep Beam Definition

<table>
<thead>
<tr>
<th>Standard</th>
<th>Span to Depth Ratio</th>
<th>Span Definition</th>
<th>Depth Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASCE</td>
<td>clear span to overall depth ratio &lt; 2.5 (continuous spans) or &lt; 1.25 (simple spans)</td>
<td>clear span is not defined; assumed distance from face to face of the supports, $\ell_n$</td>
<td>overall depth is not defined; assumed to be overall member height, $h$</td>
</tr>
<tr>
<td>BS</td>
<td>span to depth ratio &lt; 1.5</td>
<td>span is not defined; assumed to be the effective span, which is the smaller of the distance between center lines of supports, $\ell_c$, or the clear distance, $\ell_n$, plus the effective depth, $d$</td>
<td>depth is not defined; appears to mean effective depth according to 8.2.4.1 (f), which is $d$</td>
</tr>
<tr>
<td>CEB</td>
<td>span to depth ratio &lt; 2.5 for continuous spans or &lt; 2 for simple spans</td>
<td>span is the smaller of the distance between center lines of supports, $\ell_c$, or 1.15 times the distance between faces of supports, $\ell_n$</td>
<td>depth is not defined; assumed to be overall member height, $h$</td>
</tr>
<tr>
<td>CSA</td>
<td>span to depth ratios &lt; 2.5 for continuous spans or &lt; 2 for simple spans</td>
<td>span is not defined; assumed to be distance between center lines of supports, $\ell_c$</td>
<td>depth is not defined; assumed to be overall member height, $h$</td>
</tr>
<tr>
<td>EN</td>
<td>effective span to overall height ratio $\leq 2$</td>
<td>effective span is 1.15 times the distance from face to face of the supports, $\ell_n$</td>
<td>overall height is not defined; assumed to be overall member height, $h$</td>
</tr>
<tr>
<td>NZS</td>
<td>clear span to overall depth ratio &lt; 2.5 (continuous spans) or &lt; 1.25 (simple spans)</td>
<td>clear span is the clear horizontal distance between lines of effective vertical support; assumed to mean the distance from face to face of supports, $\ell_n$</td>
<td>overall depth is not defined, assumed to be overall member height, $h$</td>
</tr>
</tbody>
</table>

**Design Method:** The recognition that a deep beam behaves differently than a slender beam have led many countries to include design provisions for these elements into their design codes. These provisions are typically based on simple modes of the actual behavior. A summary of the methods from the different codes or standards is presented in Table 2.

The New Zealand Standards (NZS) requires designers to take into account the non-linear distribution of strain occurring on the beam cross-section but does not provide any guidelines on how to accomplish such a task, a rather non-traditional approach. The Canadian Standards Association (CSA) also requires designers to take into account the non-linear distribution of strain across the beam cross-section but it allows the use of a reduced effective beam depth, $d$. By reducing $d$, the CSA is approximating the value of the actual internal lever arm between compressive and tensile forces. The CSA reduced $d$ approach is a modification of the CEB’s simplified method to calculate the internal lever arm, $z$, of a deep beam. The British Standards, also shown for comparison purposes only and the European Standard are also based on the CEB recommendations. The ASCE standard does not have any design recommendations. Likewise, the Australian Standard, also shown for comparison, does not have any design provisions but it does recognize that there is an interaction between masonry members and that such an interaction may require special attention.
Table 2: Design Methods for Deep Beam

<table>
<thead>
<tr>
<th>Standard</th>
<th>Notes</th>
<th>Expressions</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>ASCE</strong></td>
<td>No provisions</td>
<td></td>
</tr>
<tr>
<td><strong>BS</strong></td>
<td>Provisions are based on CEB recommendations except that the lever arm is equal to 2/3 of the depth with a maximum value of 0.7 times the span.</td>
<td>Eq. 1a: $z = 0.2 \left( L + 2h \right)$ when $1 \leq \frac{L}{h} \leq 2$&lt;br&gt;Eq. 1b: $z = 0.6L$ when $\frac{L}{h} &lt; 1$</td>
</tr>
<tr>
<td><strong>CEB</strong></td>
<td>Simply Supported Beams</td>
<td>Eq. 1a: $z = 0.2 \left( L + 2h \right)$ when $1 \leq \frac{L}{h} \leq 2$&lt;br&gt;Eq. 1b: $z = 0.6L$ when $\frac{L}{h} &lt; 1$</td>
</tr>
<tr>
<td></td>
<td>Continuous Deep Beams</td>
<td>Eq. 2a: $z = 0.2 \left( L + 1.5h \right)$ when $1 \leq \frac{L}{h} \leq 2.5$</td>
</tr>
<tr>
<td><strong>CSA</strong></td>
<td>Design shall take into account nonlinear distribution of strain, lateral buckling, and the increased anchorage requirements for the reinforcement. As an alternative, the effective depth may be taken as 0.67 of the section depth, but not greater than 0.35 times the span or 0.7 times the cantilever length.</td>
<td></td>
</tr>
<tr>
<td><strong>EN</strong></td>
<td>Provisions are based on CEB recommendations except that the lever arm is equal to the lesser of 0.7 times the effective span or 0.4 times the clear height plus 0.2 times the effective span.</td>
<td></td>
</tr>
<tr>
<td><strong>NZS</strong></td>
<td>Design shall take into account non-linear distribution of strain and lateral buckling.</td>
<td></td>
</tr>
<tr>
<td><strong>AS</strong></td>
<td>Where a masonry member and associates slabs, beams or columns are designed for composite action, the magnitude of the stresses likely to occur as a result of the composite action shall be assessed and taken into account in the design.</td>
<td></td>
</tr>
</tbody>
</table>

The EN provisions and CEB recommendations are the same for $L > h$. For $L < h$, however, the CEB recommendations are slightly more conservative—a smaller moment arm requires slightly more flexural reinforcement. There is a remarkable difference between the CSA provisions and the CEB recommendations—the CSA provisions are significantly more conservative. There is, however, no evidence in the literature for such conservatism.

The MSJC deep beam provisions allow designers to take into account nonlinear distribution of strain in any way they deemed appropriate but it also allow designers to do so by reducing the value of the internal lever arm, $z$, of the deep beam. The simplified approach follows the CEB’s recommendation, except that for a continuous beam, the effective span-to-depth ratio is increased to 3 to be consistent with the deep beam definition given above. The members of the Flexural and Axial Load subcommittee did not find any evidence in the literature to modify the original recommendations from the CEB. The provisions are given below:
Unless the internal lever arm, \( z \), between the compressive and tensile forces is determined by a more comprehensive analysis, it shall be taken as:

(a) For simply supported spans

1. When \( \frac{l_{eff}}{d_v} < 1 \)
   \[ z = 0.6l_{eff} \]

2. When \( 1 \leq \frac{l_{eff}}{d_v} < 2 \)
   \[ z = 0.2(l_{eff} + 2d_v) \]

(b) For continuous spans

1. When \( \frac{l_{eff}}{d_v} < 1 \)
   \[ z = 0.5l_{eff} \]

2. When \( 1 \leq \frac{l_{eff}}{d_v} < 3 \)
   \[ z = 0.2(l_{eff} + 1.5d_v) \]

**Flexural Reinforcement:** The amount of main flexural reinforcement can be determined using any acceptable analytical method that takes into account the non-linear distribution of strain or the traditional method using the internal lever arm, \( z \), calculated as described above. The required amount of reinforcement is commonly determined by equating the moment demand to the nominal moment capacity and solving for the required reinforcement; the moment capacity is calculated by multiplying the tension force, \( A_{sfy} \), by the internal lever arm, \( z \).

Building codes also require designers to comply with a minimum amount of flexural reinforcement. A summary of the minimum flexural tensile reinforcement for deep beams from the different building codes or standards is presented in Table 4. The ASCE specifies that the minimum flexural tension reinforced must comply with section 3.3.4.3.2 of the MSJC. The reference to section 3.3.4.3 appears to be incorrect, since that section specifies the requirements for piers; the correct section is presumed 3.3.4.2, which gives the requirements for beams. The EN and NZS codes requirements are also those for all flexural members. The CSA does not have a specific minimum tensile reinforcement requirement for deep beam but does not nullify the applicability of minimum tensile requirement for flexural members to deep beams. In Table 4 the CSA minimum tensile requirement for flexural members is shown.

<table>
<thead>
<tr>
<th>Code</th>
<th>Requirement</th>
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</thead>
<tbody>
<tr>
<td>ASCE</td>
<td>Requires a lower bound on the nominal flexural strength of the beam—1.3 times the nominal cracking moment strength of the beam</td>
</tr>
<tr>
<td>EN</td>
<td>Requires the tensile strain of the reinforcement to be less than 0.01</td>
</tr>
<tr>
<td>CSA</td>
<td>( \rho_{min} = 0.8/f_y )</td>
</tr>
<tr>
<td>NZS</td>
<td>( \rho_{min} = 0.7/f_y )</td>
</tr>
</tbody>
</table>

Although these codes have a minimum tensile reinforcement requirement for deep beams, these values are not really applicable to deep beams but to flexural members. A minimum tensile reinforcement value is specified to prevent the sudden and brittle failure of a flexural member when the computed flexural strength of that reinforced member is less than the bending moment that causes cracking. These codes values have been typically determined by equating the cracking moment of the section, using the modulus of rupture, to the strength
computed for the reinforced member, and solving for the reinforcement. Flexural designs, however, are not often penalized by the minimum requirement because the probability of the cracking moment governing the design of flexural members is small.

The minimum tensile reinforcement specified by these codes is not applicable to a deep beam. The typical observed thin cross-section cracks along the length of a flexural member is significantly different from those observed in a deep beam—few cross-section cracks along the length and a large cross-section crack near the maximum moment location. Both types of members, flexural and deep, if unreinforced, may fail in a sudden, brittle mode; however, it is unclear if the cracking moment limit is applicable to a deep beam. In fact, “arch action” may prevent a deep beam to fail in a brittle like fashion. Deep beams are deeper than typical flexural members and, everything else being equal, a deep beam will have a cracking moment much larger than that of a flexural member; e.g., a beam whose depth is twice as that of another beam will have a cracking moment $2^3$ times greater. It is, therefore, plausible that the main flexural tensile reinforcement of a deep beam will be dictated much more often by the minimum requirement than by load demand. Because of these reasons, the adopted MSJC deep beam provisions do not have a minimum flexural tensile reinforcement requirement. As evidence (through research) that such a requirement is needed becomes available, such a requirement can be incorporated into the code.

Current codes do not provide any guidelines nor have requirements for where to place the main flexural reinforcement in a deep beam. Engineers typically place the main flexural tensile reinforcement as close to the face in tension as possible in order to maximize the internal moment arm and decrease the amount of reinforcement needed. In a simple supported deep beam experiencing positive bending moments, the CEB recommends that the main flexural tensile reinforcement be placed uniformly distributed over a depth equal to $0.25h – 0.05L$, measured from the lower face of the beam.

The adopted MSJC deep beam provisions do not have any requirement pertaining to the placement of the main flexural tensile reinforcement because the flexural and axial subcommittee did not find any explanation for the CEB recommendations. Guidelines and/or requirements may be incorporated into the MSJC as future research expands our understanding of the behavior of lightly reinforced masonry deep beams.

**Shear and Web Reinforcement:** Specimens tested failed in such way that stirrups did not cross the cracks (Leonhardt and Walther 1966). The CEB, therefore, has no recommendation for stirrups placement; this, however, does not mean that there is no recommendation for web reinforcement. A summary of the requirements for both shear and web reinforcement from the different building codes or standards are presented in Table 5.

The CEB web reinforcement recommendations are to control crack width rather than to satisfy load demand, and the placement of a mesh of orthogonal reinforcement near each face of a concrete deep beam is an easy way to accomplishment such a task; such a feat is not easily accomplished in a masonry deep beam. Nevertheless, the purpose of the CEB recommendations is captured on the masonry deep beam requirements of the different codes. The MSJC adopted five requirements to satisfy the purpose of the CEB recommendation.
### Table 5: Shear and Web Reinforcement Requirement for Deep Beam

<table>
<thead>
<tr>
<th>Standard</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>ASCE</strong></td>
<td>Uniformly distributed horizontal and vertical reinforcement shall be provided throughout the length and depth of flexural members such that the reinforcement ratios in both directions are at least 0.001.</td>
</tr>
<tr>
<td><strong>CEB</strong></td>
<td>For beams loaded on the upper portion of the beam: Provide a light mesh of orthogonal reinforcement consisting of vertical and horizontal bars placed near each face. Suggested value in both directions: $A_v &gt; 0.002 b s$ for a deformed bar and $A_v &gt; 0.0025 b s$ for a smooth round bar; $s$ is the bar spacing.</td>
</tr>
<tr>
<td><strong>CSA</strong></td>
<td>$A_v &gt; 0.002 b s$ where $s$ shall not exceed the smaller of $d/4$ and 16 in. $A_{vh} &gt; 0.002 b w s$ and shall be distributed over the depth of the beam; one bar for beams up to 9.5 inches and one bar on each face for wider beam; $S$ shall not exceed 16 in.</td>
</tr>
<tr>
<td><strong>EN</strong></td>
<td>Beams can be designed with or without shear reinforcement contributing to its capacity Reinforcement must be provided in the bed joints above the main reinforcement to a height of $0.5 L_{eff}$ or $0.5d$, whichever is the smaller, from the bottom face of the beam.</td>
</tr>
<tr>
<td><strong>NZS</strong></td>
<td>Shear requirements are the same as those for a concrete deep beam The minimum horizontal and vertical reinforcement shall be not less than the minimum reinforcement required for structural walls, which in each direction is 0.07% of the gross-sectional area of the wall taken perpendicular to the orientation of the reinforcement considered. Also, the sum of horizontal and vertical ratios for running bond wall shall be at least 0.2% of the gross cross-sections area. For stack bond walls the minimum horizontal ratio varies from 0.07 to 0.25% depending on the building importance level. The placement of the horizontal reinforcement must comply with specific guidelines.</td>
</tr>
</tbody>
</table>

Three of the requirements, listed below as numbers 1 through 3, are applicable only when the masonry shear capacity alone is not enough to satisfy the shear demand.

1. The minimum area of vertical shear reinforcement shall be $0.0007 b d_v$.
2. Horizontal shear reinforcement shall have cross-sectional area equal to or greater than one-half the area of the vertical shear reinforcement. Such reinforcement shall be equally distributed on both side faces of the beam when the nominal width of the beam is greater than 8 inches.
3. The maximum spacing of shear reinforcement shall not exceed one-fifth the total depth of the beam, $d_v$, or 16 in.

There is no rationale for requirement No. 1 except the traditional thought that when shear reinforcement is needed there must be a minimum reinforcement amount and for it to be effective and it must be placed perpendicular to the flexural tensile reinforcement. Thus, the same amount required for a flexural element is being required for a deep masonry beam. Requirement No. 2 is due to the understanding that web reinforcement is to control crack width and that such a goal can be best achieved if reinforcement is placed in both directions and as close as possible to the side faces of the beams. For easier construction of a masonry deep beam, the subcommittee decided that such reinforcement need only be distributed on both side faces of the beam when the nominal width of the beam is greater than 8 inches.
Requirement No. 3 is to ensure a well distribution of small-width cracks rather than sparsely distributed wide cracks. Thus, a limit on reinforcement spacing is more important than the actual amount required. The spacing is limited to the smaller of one-fifth the total depth of the beam or 16 in. There is no evidence to support this value.

The other two requirements are listed below as numbers 4 and 5. These requirements ensure that there will always be horizontal reinforcement, which is more effective than vertical reinforcement in controlling crack width.

4. Distributed horizontal flexural reinforcement shall be provided in the tension zone of the beam for a depth equal to half of the total depth of the beam, \(d_v\). The maximum spacing of distributed horizontal flexural reinforcement shall not exceed one-fifth of the total depth of the beam, \(d_v\), nor 16 in. (406 mm). Joint reinforcement shall be permitted to be used as distributed horizontal flexural reinforcement in deep beams. Horizontal flexural reinforcement shall be anchored to develop the yield strength of the reinforcement at the face of supports.

5. The sum of the cross-sectional areas of total horizontal and vertical reinforcement shall be at least 0.001 multiplied by the gross cross-sectional area, \(bd_v\), of the deep beam, using specified dimensions.

Note that the total amount of web reinforcement required is approximately the same in case the masonry shear capacity alone is sufficient to satisfy demand as well as when the masonry capacity alone is not sufficient to satisfy shear demand. The difference between the two cases is the distribution rather than the amount of the reinforcement. When the masonry shear capacity is not sufficient to satisfy shear demand, both vertical and horizontal reinforcement are required while when the masonry shear capacity is sufficient to satisfy shear demand the designer has the option of dividing the total amount required between vertical and horizontal reinforcement or simply requiring horizontal reinforcement. The MSJC is allowing such an approach because of the combination of two factors: the installation of vertical reinforcement in a beam is somewhat cumbersome and the horizontal reinforcement is slightly more effective than vertical reinforcement in controlling crack width.

The MSJC is requiring the placement of the distributed horizontal flexural reinforcement in the tension zone of the beam because that is where cracks initiate and have greater probability of becoming wide. The CSA requires that horizontal reinforcement be distributed over the entire depth of the beam while the EN requires that horizontal reinforcement be distributed along a height, measured from the tension face of the beam, of \(0.5L_{eff}\) or \(0.5d\), whichever is the smaller. To be conservative but not overly conservative, the MSJC code is requiring that distributed horizontal flexural reinforcement be distributed for a depth in the tension zone of the beam equal to half of the total depth of the beam.

**General Considerations:** Although not direct recommendations, the CEB mentions that it is important to use small diameter bars in order to limit the width and development of cracks under service loads and to facilitate anchorage at the supports. Future editions of the MSJC may provide a discussion of the objectives of the different reinforcements and the most effective way to accomplish such objectives.
FINAL THOUGHTS
As new knowledge becomes available, either thought theoretical or experimental research, the provisions described in this article can be verified and/or modified accordingly.

LITERATURE CITED


Masonry Standards Joint Committee (MSJC), Building Code Requirements and Specifications for Masonry Structures, TMS 402-11/ACI 530-11/ASCE 5-11, The Masonry Society, Boulder, CO, American Concrete Institute, Farmington Hills, MI, and Structural Engineering Institute of the American Society of Civil Engineers, Reston, VA.


Minimum Design Loads for Building and Other Structures, ASCE Standard ASCE/SEI 7-11, American Society of Civil Engineers, Reston, VA.

