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GUIDELINES FOR LIMITING FOUNDATION MOVEMENTS TO REDUCE SYMPTOMATIC CRACKING OF MASONRY

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ABSTRACT

The model building code groups have adopted requirements for limiting deflection in concrete foundations and masonry walls used in residential and light construction. Requirements for foundations were developed by various concrete design standards organizations and similarly, requirements for walls by various masonry institutes. Professional groups such as the American Society of Civil Engineers have also contributed to the development of standards for both. Even with the availability of this information, building designers often fail to appreciate how their foundation systems and their masonry wall systems must work together to reduce the potential for symptomatic cracking of the masonry. This paper presents design solutions for concrete foundations and masonry walls that were developed by the authors in areas of the United States with expansive clay soils where the goal was to limit cracking in the masonry veneers. These recommendations should help architects, engineers, contractors, masonry subcontractors and masonry consultants produce economical, functional foundation and masonry wall systems. The recommendations should be especially useful where differential foundation movements are anticipated.

INTRODUCTION

Cracks in masonry veneers can adversely affect the architectural appearance of a building. They can also be the conduit through which water is driven by wind, pulled by gravity, or sucked by capillary action into interstitial spaces behind the veneers. Water that gets past the outer veneer into the wall cavity can ultimately place other components of the building's structure at risk from damage such as spalling, rot or corrosion.

Cracks in the masonry veneers of residential and light commercial structures are often symptoms of more serious physical or structural problems. They may be the signs of impending structural failure, perhaps leading to structural collapse. The causes of cracks in masonry should be investigated fully when they appear.

There are a number of steps the architect, engineer, or building designer can take to minimize the potential for symptomatic cracking of masonry. *Symptomatic cracking* is used here to mean the cracks caused by physical changes in the dimensions of the masonry materials themselves, or from movements of the supporting foundation or other structural systems. The symptomatic cracks are what we see, and they are the result of the underlying forces.

The purpose of this paper is to present the design criteria that should be considered to minimize cracking in masonry veneer construction for residential and light commercial structures. The paper is divided into four sections. First - a review of the basics on cracks in masonry and the forces that cause them is presented. Second - the varying opinions about acceptable crack widths in masonry are covered. Third - foundation and masonry wall designs that limit cracking in masonry and which have been successfully employed by the authors in areas of the United States with expansive clay soils are presented. Fourth - in the conclusions and recommendations section, alternative foundation and wall designs that can be used to work together to minimize the potential for symptomatic cracking in masonry are presented. With this information designers should be able to select the alternatives necessary to produce economical, yet functional buildings for their clients.

The following information, which includes some structural analyses for foundations and masonry walls, is limited to design and construction recommendations for one-story residential construction. The designer would perform similar analyses for two-story houses and for light commercial structures. Further, the discussion here is limited to the foundation settlement condition. In locations with expansive clay soils, where uplift from soil swelling is also a possibility, provision for the settlement problem will normally accommodate the design requirements for the uplift problem and therefore that phenomenon is also excluded here. Almost any basic structural primer can be consulted to confirm this assumption. Provision for the foundation settlement problem will normally take care of expansion and contraction problems in the masonry materials themselves.

CAUSES OF CRACKS IN MASONRY

There are at least two basic causes of cracks in masonry. First, cracks in masonry may be caused by expansion and contraction of the materials used to manufacture masonry units, or in naturally occurring stone. Such expansion and contraction is the result of changes in moisture content and/or changes in temperature (e.g. thermal load) in the materials. Extensive coverage of these forces and their effects on building materials is not provided here. Instead, the reader is encouraged to refer to publications such as by Drysdale et al. (1994, 530). It must also be noted that with excessive movements in the material leading to cracking, there are two basic conditions: (1) the unrestrained case where the masonry moves back and forth on the supporting structure without physical restraint except for perhaps some bearing friction, and (2) the restrained case, where the bases and/or sides of the masonry are physically attached or restrained to the structure. Both conditions present different types of cracking phenomena and the investigator must know whether or not he or she is looking at a wall design that is restrained or unrestrained when examining cracks. Moisture and/or thermal changes in the material can cause cracks to occur in all three directions of the wall plane: x (horizontal) , y (vertical) and z (front to back).

The second cause of cracks in masonry is movement of the supporting structural system. Bruce Suprenant (1990) has presented an excellent overview of cracks in masonry that are the result of movements in the structural system. Differential settlement or uplift of the foundation, or excessive movement of the structural frame surrounding the masonry or to which the masonry is attached, will often cause cracking in masonry.

Structural settlements or movements typically place cracking forces on the masonry wall in the x direction, in the y direction, or in the z direction of the wall plane. Diagonal cracks are often seen in masonry, but these will normally be within one plane. Seldom will one find the structurally-induced forces resulting in diagonal cracks and cracks in the x, y, or z directions simultaneously.

Cracks may be located (1) in the masonry materials themselves, (2) in the mortar joints, (3) between the masonry units and the mortar beds at their interfaces, and (4) continuously through both the masonry and the mortar (Grimm 1988, 257-258). As noted previously, crack patterns may be (1) horizontal, (2) vertical, and/or (3) diagonal. When vertical, they are often straight or cogged. When they are diagonal they may be either straight or stepped.

ACCEPTABLE CRACK WIDTHS IN MASONRY

An investigation of cracks in masonry leads one to the question of what are acceptable crack widths. The answer to this question can help the investigator determine what needs to be done to repair or arrest the cracks. The original designer can use knowledge of acceptable and unacceptable crack widths to establish the initial design parameters for both the walls and their supporting structures. Repair decisions can also be made with this information. Obviously the type of building, the types of materials used in its construction, the climate, the soil conditions, the owner's budget and other factors can affect decisions about design and repairs. Local building code requirements, recommended practices, and labor skills further impact the decisions.

The concrete foundations and masonry veneers typically found on residential and light commercial structures are considered "brittle" materials and this is why uncontrolled stresses can cause cracking in them. The performance of the masonry is particularly vulnerable to the behavior of the foundation. Movements or differential settlements in the foundation can affect the performance of the masonry. Floors not supporting or attached to brittle elements such as masonry are limited in deflection to L/360, where L is the span of the floor and it's supporting beams. For floors supporting or attached to brittle elements, deflections are limited to L/480, where L is the floor span (French 1994, 36-37). The concrete foundation should be designed according to ACI 318 *Building Code Requirements for Structural Concrete and Commentary (latest edition)*.

Masonry has a different set of criteria for deflections and should be designed according to the latest editions of ACI 530/ASCE 5/TMS 402 *Building Code Requirements for Masonry Structures* (MSJC Code). Deflections in masonry are generally limited to L/600. One can see from these standards that design limits for deflection in the foundations and the masonry wall systems are different. The problem for the designer is how to reconcile the acceptable differences in movements of these brittle materials.

There is also little consistency among the design professions as to what are acceptable crack widths in masonry. One rule-of-thumb says if cracks are observable at twenty feet (6.1 m) by the average person viewing them under diffused lighting, they are unacceptable (ASTM C 90-96, Sec. 7.2). Another rule-of-thumb in the design of movement joints for masonry walls requires that expansion and contraction in joints not exceed one-half the width of the joint (BIA Technical Notes 18A Revised, December 1991). Flexible movement joints in masonry veneers are typically installed in widths of 3/8" (9.5 mm), equaling the width of the common mortar joint. One would expect movements in the wall at the movement joints, therefore, to be no greater than 3/16" (4.76 mm) in either direction. Potential crack widths in the masonry could be limited by similar criteria.

Bruce A. Suprenant (1990, 10) has reviewed the criteria for acceptable crack widths in masonry and provides a good summary of what is acceptable according to certain performance criteria. The Portland Cement Association, for example, states that a crack width that is 0.010" (0.254 mm) to 0.015" (0.381 mm) wide "...neither hurts the surface appearance nor alarms the viewer" (Pp. 10-11; from Portland Cement Association 1982). Tests conducted in Norway found that wind-driven rain will not enter cracks in masonry that are narrower than 0.004" (0.102 mm) (p. 11; from Birkeland O., and Sevendsen, S.D.). Suprenant (Pp. 11-12) has categorized crack widths in masonry greater than 0.016" (0.406 mm) as severe and in need of repair. In areas with high potential for wind-driven rain, where one wants to limit water penetration, the designer should reduce the criteria to 0.004" (0.102 mm) of width, or about 1/265" (0.096 mm). A maximum crack width of 0.016" (0.406 mm), or about 1/64" (0.397 mm) will be used by the authors as the criterion used in determining foundation designs that limit potential cracking in masonry to that width. Visual acceptability, therefore, is being used as the governing design criterion in this paper. Again, in areas with high potential for wind driven rain, one might want to limit the crack width even further. Figure 1 shows a crack in brick that is about 1/64" (0.397 mm) wide.

DEFLECTION IN CONCRETE FOUNDATIONS

The design of the supporting foundation is critical to limiting the potential for cracking of the masonry it supports. In areas with expansive clay soils, the outside corners of the foundation are where shrinkage of the soil typically occurs first. Shrinkage of the soil upon which the foundation rests causes the foundation slab and the supporting beams in it to go into a cantilever condition at some distance back from the corners. Eventually, the beams will engage supporting soil and it is near these locations that deflection in the beams begins and rotation of the masonry walls above also occurs. Near the point of rotation, the stresses in the brick and/or the mortar can exceed their tensile strengths and one or the other, or both, will crack.

Figure 2 shows a typical residential foundation design for a slab-on-grade. This foundation type is common in the Southern United States where basements and crawl spaces are seldom utilized. Figure 2 shows a partial plan view and a section view of the foundation in the vicinity of a typical outside corner. The foundation has a perimeter beam with a grid of interior stiffening beams at approximately twelve feet (3.66m) spacing in each direction. Some designs allow spacing of the interior beams of up to sixteen feet (4.88m), but the analysis here will use the twelve foot (3.66m) spacing. The qualifying assumptions for this configuration are as follows:

- The Plasticity Index (P.I.) of the soil ranges from 25 - 50.
- It is assumed the soil expands and contracts with seasonal moisture changes.
- During extended dry periods, the authors have observed from professional experience that the soil will dry up to within approximately 5 feet inside the exterior foundation beams and shrink away leaving the exterior beams with no soil support. The only legitimate support for exterior beams is the interior stiffening beams cantilevering out over the shrunken soil from the still expanded interior soil upon which they are bearing.

- Live loads on the foundation come from the Standard Building Code and the Uniform Building Code for residences. Dead loads come from assumed roof framing, roof materials, typical drywall installations, and masonry at 40 lbs/ft^2 (1.92 kPa). The dead load shown does not include the weight of the foundation beam because the software used to compute deflections of the beams automatically includes the beam weight.
- The material properties used in the example foundation are: concrete $f'c$ = 3,000 p.s.i. (20.69 MPa); fy of the reinforcing steel = 60 ksi (414 MPa); interior beams are 12" x 30" (305 mm x 762 mm) with 3 - #6 (#19) reinforcing bars top and bottom; exterior beams are 12" x 36" (305 mm x 910 mm) with 3 - #6 (#19) reinforcing bars top and bottom; and, the slab thickness is 4 ½" (114 mm) thick and is reinforced with #3 reinforcing bars at 12" (305 mm) on center each way.

Figure 3 shows a section-elevation of the brick wall for a one-story residence as constructed on the foundation. The brick wall is 8 feet (2.44 m) high and runs the length of the foundation. In this example, no movement joints are provided in the masonry. In effect, the brick wall forms an unrestrained, deep, thin beam. Lateral bracing is provided by the wall ties to the framing behind the wall but this is not factored into the analysis here because very little vertical support of the wall is provided.

As the soil shrinks away from the foundation's exterior beams, their only support is the cantilevered portion of the interior beams. Using the loads indicated in Figure 3 and assuming the support provided by the interior beams, the back span of the exterior beams approximates a fixed end situation for the exterior beams cantilevering toward the corner. The deflection of the exterior beams at the corner will be approximately 0.13" (3.30 mm).

When the typical exterior beam begins to deflect the masonry veneer wall above begins to act as a deep slender beam. Using a 3" (76.2 mm) wide brick and an 8 foot (2.44 m) tall wall with type N masonry mortar, the flexural tensile stress that will cause failure of the mortar as determined by the Masonry Standards Joint Committee (MSJC) Code is 30 p.s.i. (207 kPa). By calculating the moment of inertia and section modulus of a 3" (76.2 mm) wide by 96" (2.44 m) deep cross-section and using bending stress of 30 p.s.i. (207 kPa), the bending moment that the veneer wall can withstand before tensile cracks occur in the top head joints can be calculated. Equating this moment to the moment of a cantilevered beam with a uniform load, $M= WL^2/2$, the maximum cantilever span of the brick veneer beam can be determined. The approximate deflection of the masonry beam can also be determined.

The $f'm$ for masonry will be assumed as 3,000 p.s.i. (20.69 MPa) and Em (modulus of elasticity) = 750 $f'm$ = 2.25×10^6 p.s.i. (2,250 ksi or 15,514 MPa) for this example (values from Uniform Building Code 1994). The maximum span of this cantilevered masonry wall-beam was found to be approximately 8.5' (2.6 m) and the deflection of this 8.5' (2.6 m) span cantilevered beam at the free end was 0.0007" (0.0178 mm).

In Figure 3, the concrete beam's deflection is 0.011" (0.28 mm) at 3.5' (1.07 m) and 0.13" (3.30 mm) at 12' (3.66 m) from the support. The relative difference (0.13" - 0.011" = 0.119") (3.02 mm) is far in excess of the 0.0007" (0.0178 mm) that will cause the masonry to crack and exceed the commonly acceptable crack width of 1/64" (0.397 mm).

This indicates that foundations on expansive soil have to be extremely stiff to prevent cracking in the masonry. For example, an 8.5' (2.6 m) span cantilevered beam using reinforced concrete and supporting the superimposed live and dead loads shown in Figure 3, plus it's own weight and not deflecting over 0.0007" (0.0178 mm), would have to be 18" (457 mm) wide, 96" (2.44 m) deep, and be reinforced with 4 - #ll (#36) bars top and bottom. This type of beam is economically not feasible for residential construction. Other methods of controlling deflections or movement must be employed in both the foundation and the wall designs.

FOUNDATION DESIGN

One method of stiffening a slab-on-grade foundation that will reduce cracking of the masonry is to install a relatively inexpensive diagonal beam at the exterior corners of the slab. The authors have used this beam configuration on approximately 15 foundations in an area of the United States with expansive clay soils. Some of these foundations have been in place for over ten years and the results have been very satisfactory. They continue to perform without excessive symptomatic cracks in the masonry veneers they are supporting. The mathematical calculations in the following paragraphs demonstrate why.

Figure 4 shows the use of the diagonal beam at the typical exterior corner of the foundation. The exterior corners are usually the first locations on a foundation to experience deformations due to soil shrinking. The diagonal beam is most efficient when it extends from the intersection of two interior beams to the exterior corner. This allows the diagonal beam to cantilever over the shrinking soil and to have a support on the back span that approximates a fixed end where it is integral with the intersection of the interior beams. The diagonal beam is usually a 12" x 24" (305 mm x 610 mm) beam with 3 - #5 (#16) reinforcing bars top and bottom. The slab can be used to transform this diagonal beam into a T-beam with even more stiffness. To get an indication of the resistance to the service load this beam can provide at the exterior corner the following values were assumed (see also Figure 4). First, allowable soil pressure on the expanded (high water content) soil = 2,500 p.s.f. (120 kPa). The effective width of the T-beam formed by the diagonal beam and slab = 3'- 0" (0.91 m). A triangular soil pressure distribution resulting from no deflection at the back span support and increasing at further distances toward the foundation corner is also assumed. Idealized as a triangular distribution equal to 2.5 k.s.f. x 3'- 0" of width = 7.5 k/ft. (109 kN/m). Analyzing this beam to determine the maximum value of "R" allowed in terms of service load yields the result of 7 kips (31.1 kN). As the corner begins to move downward as a result of the shrinking soil around the perimeter this beam can conservatively provide a 7,000 lb. (31.1 kN) force upward to resist that movement. The deflection of the beam associated with the 7,000 lb. (31.1 kN) load is approximately 0.08" (2.03 mm), or a little more than 1/16 of an inch (1.6 mm).

The actual interaction between an elastic soil and a foundation slab on that elastic soil is very complex. The simplifying assumptions made for the sake of these examples are considered conservative by the authors. It is apparent from this example, however, that the use of a diagonal beam at the outside corners of the foundation greatly reduces the deflection of the foundation, enough to also minimize the potential for cracking in the masonry beyond acceptable limits.

RECOMMENDATIONS FOR LIMITING SYMPTOMATIC CRACKING IN MASONRY

Assuming the ultimate goal of the designer is to minimize the potential for symptomatic cracking in the masonry veneer, he or she would appear to have following four alternatives:

1. To design the slab-on-grade foundation with extremely deep interior and exterior beams (a costly and problematic alternative);
2. To design the slab-on-grade foundation with the more common beam widths and depths for slab-on-grade construction, and add diagonal beams at the corners;
3. To design the masonry veneer wall as a deep, thin beam with additional reinforcement suitable to augment the capacity of the foundation beams; and/or,
4. To add movement joints at strategic locations in the masonry wall to take up the anticipated movement in the wall caused by settlement of the foundation beams and rotation of the wall; e.g., such rotation being beyond the ability of the masonry to resist the stresses and hence inducing cracks in it.

Option "1" above is often practiced in the design of foundations but the problem observed by the authors is that many designers fail to design the foundation stiff enough to limit the potential crack widths to 1/64" (0.4 mm). Addition of the diagonal beams in the foundation's outside corners shows the economical, yet satisfactory performance benefits.

Figure 5 shows the third option - additional reinforcement in the masonry veneer making it a strong, deep beam. Strands of joint reinforcement placed in the top two courses of the masonry just below the soldier course will greatly reduce the wall's potential for cracking from settlement of the foundation. Installation of reinforcement in the two bottom courses of the wall will reduce the potential for cracking mid-span at the bottom of the wall, a condition often caused by swelling of the soils at the ends of the beams. Addition of a strand of joint reinforcing in the middle of the wall will limit the potential for shear stresses to cause cracking in the middle of the wall (e.g. stepped cracks).

And lastly, the Brick Institute of America, in Technical Note 18 Revised, January, 1991, discusses the topic of movement and volume changes and effects of movement in masonry. .In Technical Note18A Revised, December, 1991, design and detailing of movement joints in masonry are discussed. This approach generally requires movement joints 3/8" (9.5 mm) wide to be installed with backer rods and flexible caulk at the following locations: near each of the outside or inside corners of walls or where the walls change plane or direction; the mid-point of long walls; at each door and window opening; where masonry walls change height; where the masonry abuts dissimilar materials; and, where thermal, moisture, freezing, or structural forces dictate their use. Properly placed movement joints can greatly minimize the potential for cracking in masonry, even with rather extensive differential foundation movement.

The authors perceive that the range of options for the designer that are necessary to make the foundation system and the masonry wall system work together for better ultimate performance (performance being limiting cracks in the masonry to acceptable widths) as presented give the designer greater flexibility than they may have realized. The primary contribution of this paper has been to present a comprehensive way of analyzing the alternatives. The addition of a diagonal beam at the outside corners of the foundation has worked very well and should be considered by designers as a cost-effective solution. Addition of horizontal joint reinforcement at the top, middle and lower portions of the wall will also provide the designer with the desired benefit of minimizing the potential for cracking. The methodology for determining the point of rotation in the masonry wall where unacceptable cracks may occur can be used by designers to better understand the performance of their foundations and masonry walls when they are working in unison.

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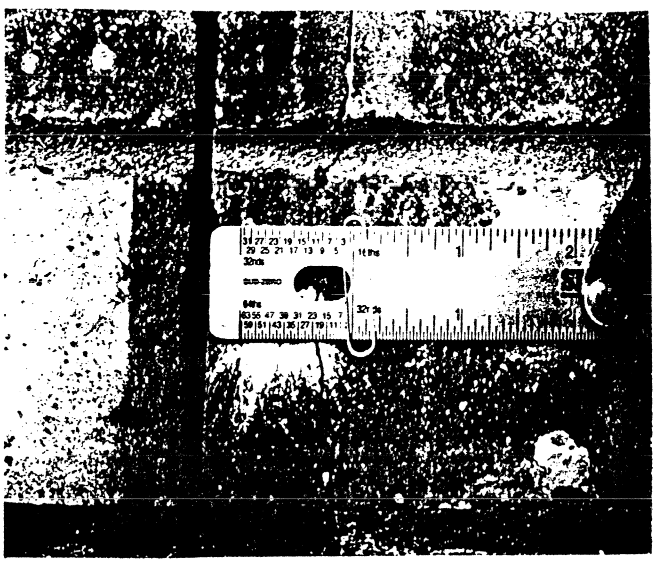

Figure 1: A Crack in Brick Veneer 1/64" (0.397 mm) Wide

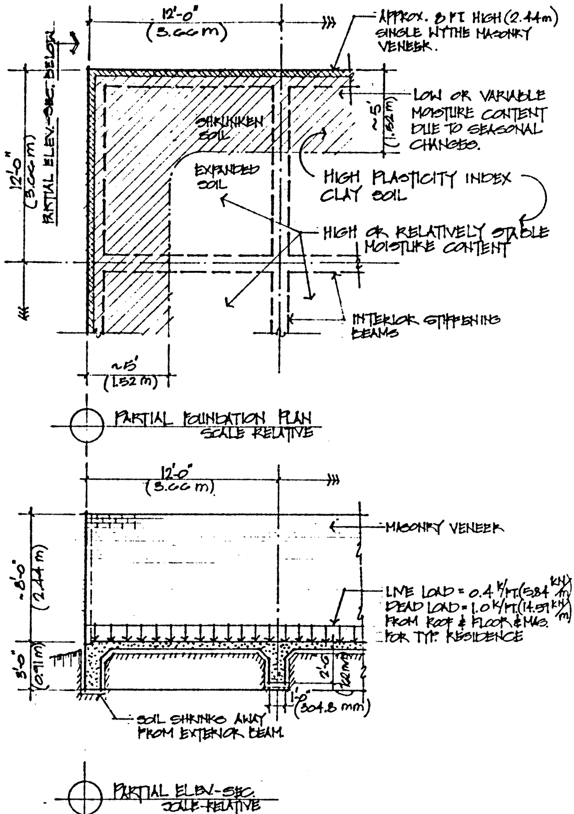
12'-0" (3.66 m)
APPROX. 8 FT. HIGH (2.44 m) SINGLE WYTHE MASONRY VENEER.
LOW OR VARIABLE MOISTURE CONTENT DUE TO SEASONAL CHANGES.
~5' (1.52 m)
SHRUNKEN SOIL
EXPANDED SOIL
HIGH PLASTICITY INDEX CLAY SOIL
HIGH OR RELATIVELY STABLE MOISTURE CONTENT
12'-0" (3.66 m)
PARTIAL ELEV.-SEC. BELOW
INTERIOR STIFFENING BEAMS
~5' (1.52 m)
PARTIAL FOUNDATION PLAN
SCALE RELATIVE
12'-0" (3.66 m)
MASONRY VENEER
~8'-0" (2.44 m)
LIVE LOAD = 0.4 K/FT. (5.84 kN/m)
DEAD LOAD = 1.0 K/FT. (14.59 kN/m)
FROM ROOF & FLOOR & MAS. FOR TYP. RESIDENCE
3'-0" (0.91 m)
2'-6" (762 mm)
1'-0" (304.8 mm)
SOIL SHRINKS AWAY FROM EXTERIOR BEAM.
PARTIAL ELEV-SEC.
SCALE RELATIVE

FIG. 2

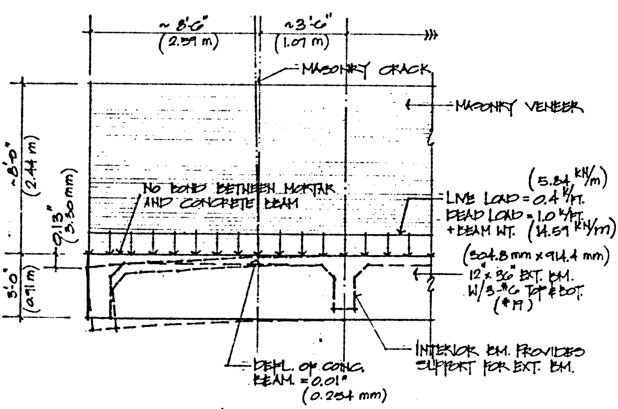
~8'-6" (2.59 m)
~3'-6" (1.07 m)
MASONRY CRACK
MASONRY VENEER
~8'-0" (2.44 m)
0.13" (3.30 mm)
NO BOND BETWEEN MORTAR AND CONCRETE BEAM
LIVE LOAD = 0.4 K/FT. (5.84 kN/m)
DEAD LOAD = 1.0 K/FT. + BEAM WT. (14.59 kN/m)
(304.8 mm x 914.4 mm)
12" x 36" EXT. BM. W/3-#6 TOP & BOT. (#19)
3'-0" (0.91 m)
INTERIOR BM. PROVIDES SUPPORT FOR EXT. BM.
DEFL. OF CONC. BEAM = 0.01" (0.254 mm)

FIG. 3

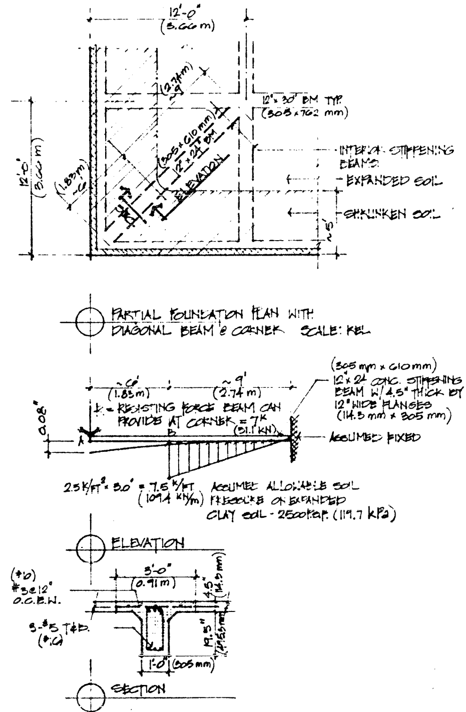
12'-0" (3.66 m)
12'-0" (3.66 m)
12" x 30" BM TYP. (305 x 762 mm)
INTERIOR STIFFENING BEAMS
EXPANDED SOIL
SHRUNKEN SOIL
12" x 24" BM (305 x 610 mm)
ELEVATION
PARTIAL FOUNDATION PLAN WITH DIAGONAL BEAM @ CORNER SCALE: REL.
~6' (1.83 m)
~9' (2.74 m)
(305 mm x 610 mm) 12" x 24" CONC. STIFFENING BEAM W/ 4.5" THICK BY 12" WIDE FLANGES (114.3 mm x 305 mm)
ASSUMED FIXED
RESISTING FORCE BEAM CAN PROVIDE AT CORNER = 7K (31.1 kN)
2.5 K/FT² x 3.0' = 7.5 K/FT (109.4 kN/m) ASSUMED ALLOWABLE SOIL PRESSURE OF EXPANDED CLAY SOIL - 2500 PSF (119.7 kPa)
ELEVATION
3'-0" (0.91 m)
(#10) #3 @ 12" O.C.E.W.
3-#5 T&B (#16)
4.5" (114.3 mm)
19.5" (495.3 mm)
1'-0" (305 mm)
SECTION

FIG. 4

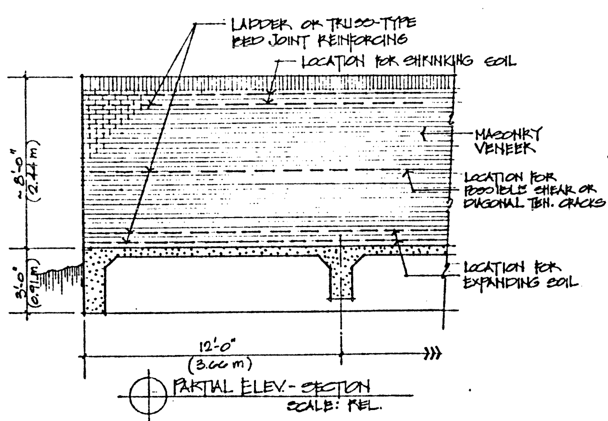
LADDER OR TRUSS-TYPE BED JOINT REINFORCING
LOCATION FOR SHRINKING SOIL
MASONRY VENEER
LOCATION FOR POSSIBLE SHEAR OR DIAGONAL TEN. CRACKS
LOCATION FOR EXPANDING SOIL
~8'-0" (2.44 m)
3'-0" (0.91 m)
12'-0" (3.66 m)
PARTIAL ELEV. - SECTION SCALE: REL.

FIG. 5