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The Designer's Responsibility

FOR REBAR DESIGN

Loring A. Wyllie, Jr and
Ronald W. LaPlante
Degenkolb Engineers

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The Authors

Loring A. Wyllie, Jr., P.E., S.E. is a Senior Principal and Chairman Emeritus of Degenkolb Engineers, San Francisco, CA. With over 38 years of design experience, Loring has designed many reinforced concrete structures and seismic resistant structures. He has served on the ACI Code Committee 318 for 30 years and is an Honorary Member of the American Concrete Institute, American Society of Civil Engineers and the Structural Engineers Association of Northern California. He was elected to the National Academy of Engineering in 1990.

Ronald W. LaPlante, S.E. is a Project Engineer of Degenkolb Engineers in San Diego, California, specializing in the design of seismic resistant concrete structures. He earned a bachelor's degree from California State Polytechnic University, Pomona, and a master's degree from University of California, Berkeley. He is a licensed Structural Engineer in the state of California and is an active member of the Structural Engineers Association of California and the American Concrete Institute.

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Introduction

Many years ago, a contractor presented a structural engineer with a framed piece of plywood that had a #11 (#36) reinforcing bar bent into a 180° hook. The caption read: "This is a #11 bar, 1-7/16 inches in diameter. BE IT RESOLVED, I will never again hook #11 bars in an 8-inch wall." The momento was hung on the wall of the engineering firm to remind the engineers that designing for constructibility is an essential part of their job. An 8-inch thick wall with two curtains of typical wall reinforcement and several #11 (#36) bars with 180° hooks is a very congested situation.

As structural engineers, we have many responsibilities when designing a structure. We need to design members to resist the required loads and comply with the applicable building codes. In reinforced concrete members, we need to hook, develop and locate reinforcing bars so that they transfer forces properly and develop the required strength. We also need to detail the reinforcing bars so that they can be placed efficiently and with enough clearance that the concrete can be placed and consolidated properly. In other words, we need to size the members and design the reinforcement so that the structure can be built as designed.

Traditionally we are taught in our engineering classes to minimize the tonnage of reinforcing steel and cubic yards of concrete on the somewhat false premise that minimizing materials results in an economical design. In reality, labor is the most expensive item for construction in the U.S. When we "waste" a little concrete by making members larger so that the reinforcing steel can be placed more easily and the concrete consolidated more efficiently, we are actually achieving true cost savings.

The purpose of this paper is to highlight what we as designers can, and should, do in design and detailing to make reinforced concrete construction easier and thus more economical. The suggestions come both from the authors' own design experiences and their experience in peer reviewing the designs of other engineers. There is also a discussion of several details that are not always well understood by engineers. Although the focus is on issues related to earthquake-resistant construction on the West Coast, similar issues occur in reinforced concrete construction throughout the United States.

Contents

Beams, Columns and Joints

Beam and column frames are a common form of reinforced concrete construction in the U.S. In cast-in-place frames, there is always continuity and moment transfer between the beams and columns that must be accounted for in design. In most reinforced concrete structures, the beams and columns contain the greatest concentration of reinforcing steel, but there is often an attempt to keep the size of these members small, which results in considerable congestion at the joints where beams and columns meet.

When sizing beams and columns, it is usually better not to make the beam and column widths identical. Why? Because beams and columns always need bars close to their faces and at corners to hold the stirrups or ties. When the beam and column are the same width, these bars are in the same plane in the beam and the column, and they conflict at the joint. This requires bending and offsetting one set of the bars, which will increase fabrication costs. Offsetting bars can also create placement difficulties and results in bar eccentricities that may affect ultimate performance. If the beam is at least 4 inches wider or narrower than the column (2 inches on each side), the bars can be detailed so that they are in different planes and thus do not need to be offset. To illustrate this point, Fig. 1 shows congested, but neatly placed, reinforcing bars in a beam-column joint designed for seismic resistance. Since the beams are the same width as the columns, the detailer used a smaller, discontinuous bar to support the stirrups at the edge of the beam. If the beams were not the same width as the columns, the main beam bars could have been used as stirrup supports without conflicting with the longitudinal bars in the columns.

As you continue reading this paper, start thinking of "planes of reinforcement." Every type of reinforcement in a wall, such as the typical vertical, typical horizontal, vertical heavy trim, horizontal heavy trim, diagonal, must have its own dedicated plane or it will conflict with other bars. This applies to all reinforced concrete members; keep this in mind when providing specific reinforcing steel details. Establishing planes for the vertical column bars, then the top and bottom beam bars that pass through or hook into the columns (keeping the top and bottom beam bars in the same vertical plane) reduces the three-dimensional problem of intersecting bars to an exercise in two-dimensional visualization.

Back to beams and columns: try not to layer the reinforcing bars. This is sometimes necessary, but it makes placement very difficult, especially when two or more layers of top bars must be hooked down into the joint at an exterior column. If more than one layer of bars is required, it may be because the beam is too small; if this is the case, it should be made larger.

Some years ago, the authors were asked to do a peer review of a parking garage that had a transfer beam supporting a discontinuous column. The beam spanned between exterior walls and was approximately 3 feet wide and 5 or 6 feet deep. The beam's positive moment reinforcement was six layers of #11 (#36) bars with ten bars in each layer. We questioned this and were told that all was fine. We then suggested that the engineer who stamped the drawings be required to personally supervise and approve the placement of all 60 - #11 (#36) bars. Designs like this should raise a red flag to all involved.

Figure 1
Although congested, the reinforcing bars in this beam-column joint are neatly placed and designed for seismic resistance.

Another factor affecting the way we detail beams is the structural integrity provisions of *Section 7.13 of the ACI Building Code Requirements for Structural Concrete (ACI 318)*, which first appeared in the 1989 Code. These provisions stipulate that a percentage of the positive moment reinforcement required at midspan be continuous or be lap spliced at or near the supports with a Class A tension lap splice. The reason for requiring continuous or developed bottom reinforcing bars is to reduce the likelihood of progressive collapse should a catastrophic event, like an explosion, cause the loss of a column. If a column is suddenly damaged or destroyed, the catenary condition created in the bottom reinforcement will hopefully prevent progressive collapse. One way to address this requirement is to lap splice the bottom bars within the joint. If half of the bottom bars are lap spliced near the point of inflection at each end of the beam framing into the joint, the bars will be of manageable lengths and lap splices in the congested joint region can be avoided. Fig. 2 shows a detail for this. Steel mills generally roll bars in 60-foot lengths. Number 7 (#22) and larger bars can be produced in lengths up to 80 feet by most U.S. mills, but there may be a cost premium for the longer bars, and very long bars may be more difficult to ship and to place.

Likewise, as shown in Fig. 2, some of the top bars must be made continuous with lap slices at midspan. Short bars should be added at the joint to satisfy negative moment requirements. Requiring some continuous top bars is also good insurance against having floors sag from creep deflections. The authors have seen two-way slabs with 3 inches of midspan deflection and one-way slabs with 1 inch of deflection in 8 feet; there was no continuous top reinforcement in these slabs. Continuous top and bottom reinforcement in beams and floor framing is an inexpensive way to ensure serviceable floors.

Let's focus a bit on seismic-resisting beams and columns, or special moment-resisting frames. A special moment frame is detailed to provide ductility for seismic overloads so that it will maintain its strength beyond its yield capacity. A good friend of the authors once described a seismic-resisting frame as a series of joints held apart by beams and columns. When designing special moment frames, joint shear should be checked in the conceptual design phase to ensure that the member sizes are appropriate. The joint (connection) is the critical area; it is important to make the beams and columns large enough that joint shear stresses are kept under control. Joints must also be large enough so that the concrete and reinforcing bars can be placed easily. The ACI Committee 352 report *Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures (ACI 352R-02)* gives more guidance on beam-column joints.

Fig. 3a is a plan view of an exterior beam-column joint in a special moment frame. Longitudinal column bars should be spaced about 6 to 8 inches on center if possible; research has shown that this provides better confinement of the concrete than widely spaced larger vertical bars. To avoid having to offset beam bars vertically at each joint, the top bars in the beams in one direction should be one bar diameter lower than the top bars in the beams in the other direction. If the beams in one direction are made at least two inches deeper than the beams in the other direction, the bottom bars can come into the joints in different horizontal planes. Using different beam sizes is likely to increase forming cost, however; all factors must be considered when finalizing the design.

Some of these issues about beam widths and depths and where bars that intersect should be located may seem trivial, but it is our task as designers to help the reinforcing steel detailers and placers. They cannot change the concrete dimensions we show on our drawings; we need to establish those dimensions so that everything fits well and thus results in an economical design.

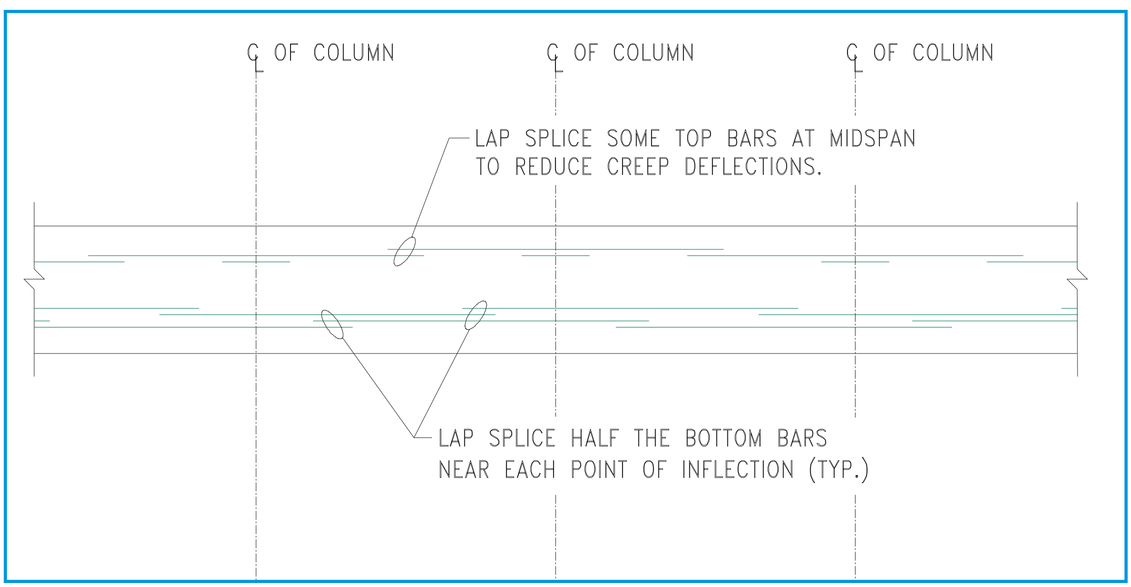

Figure 2
Possible layout of top and bottom slab or beam rebar to stagger splices and meet structural integrity requirements. Note that for equal spans, all bottom bars are the same length as are all the longitudinal top bars.

Confinement ties in beams and columns are relatively straightforward and most engineers understand how to detail them. The 135° hooks are essential for seismic construction; alternating 135 and 90° hooks is a compromise that improves constructability and seems to work. Before some time in the 1960s, ACI 318 required every column longitudinal bar to be tied. Around that time, however, a building was constructed without column ties (hard to imagine), so a column without ties was tested. The initial axial strength of the column was about the same as a nominally tied non-seismic column. The concrete cover on the longitudinal bars acted in tension and served the function of ties until ultimate load was approached. The column then failed dramatically. Based on this "pilot test", the Code was changed to require that only alternate longitudinal bars be tied. This concept carried over to seismic ties where confinement under severe overloads is quite possible. Keep this history in mind when detailing a critical facility where the seismic performance is important. For such situations, tying every bar may enhance performance. Also, keep the ties and stirrups to #4 (#13) or #5 (#16) bars. Number 6 (#19) and larger bars have very large diameter bends and are difficult to place.

One more comment on stirrup and tie hook performance during earthquakes. The concrete cover on beams and columns often spalls off quickly in response to the ground shaking and exposes the stirrup and tie hooks. A 90° hook can easily be bent outward from internal pressure; if this happens, the stirrup or tie will lose its effectiveness. In contrast, a 135° hook will remain anchored in the core of the member when the concrete cover spalls. There is no real cost premium for 135° hooks and their performance in extreme loadings is vastly superior to 90° hooks.

Additionally, these are a few more thoughts on columns in seismic regions. Spirally reinforced columns are unquestionably more ductile than columns with ties and are therefore better for extreme seismic loads. Making the beam-column joint work is more complex because of the geometry but is usually worth the effort. Mechanical splices, especially Type 2 mechanical splices, should be considered for column bars. Type 2 splices develop the specified tensile strength of the bar and are more ductile than most Type 1 splices; this may be significant if stress reversals occur in the columns. Remember that splices take up space, though, and plan for that space (including concrete cover). If there are a large number of bars, consider staggering the splices. Grade 75 bars, where available, can provide some economy in columns by reducing the number of longitudinal bars.

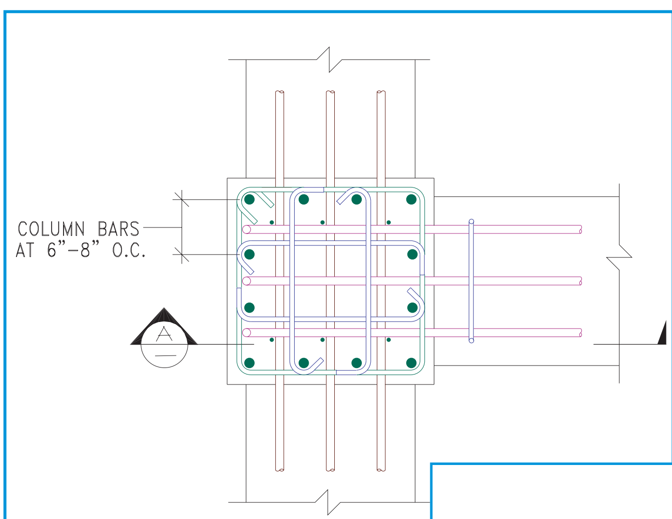

Figure 3a
Plan section of beam-column intersection illustrating the advantage of beams narrower than the column dimension and beams in the two directions being of different depths to avoid rebar conflicts.

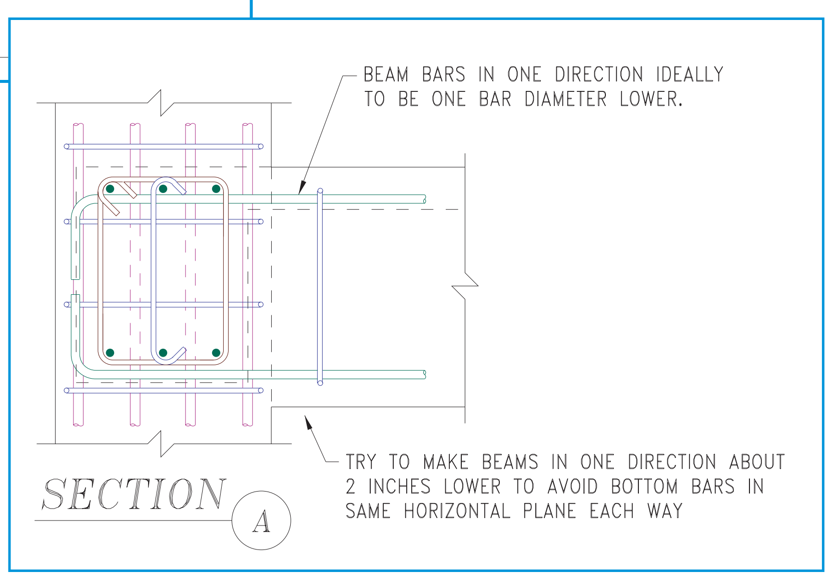

Figure 3b

Shear Walls

Shear walls are very common in reinforced concrete construction, especially where seismic load resistance is required. Historically, shear walls have performed well in earthquakes provided they were of sufficient length and well detailed. Prior to about 1950, the exterior walls in multi-story construction were often cast-in-place concrete with pierced window openings forming an effective shear wall system of piers and spandrel beams. Current shear walls tend to feature relatively tall and slender solid walls that are often separate from the facade and act as vertical cantilevers fixed to the base or foundation of the building.

A few words of caution about designing shear walls. Many engineers estimate the wall length based on what they think is needed for a reasonable shear stress, using only the base shear as a guide. On the basis of this simple calculation, they negotiate with the architect for a certain amount of wall. Later, they calculate the design moment in the wall and determine that they need 100 square inches or more of vertical boundary reinforcement; the congestion and base details quickly become unworkable. Both the design shear and the design moment in the walls should be estimated during the conceptual design phase. Many modern shear wall designs require a substantial amount of reinforcing steel. The walls must be thick enough to allow for reinforcing steel placement with sufficient room left for the concrete placement.

Before getting into a serious discussion of detailing shear walls for constructibility, let's focus on how the wall reinforcement is typically placed. Wall reinforcement is a bit different from reinforcement for slabs and footings, where the most highly stressed or heaviest layer of bars is always placed at the top or bottom layer to increase the effective depth. The placing sequence in walls is somewhat dependent on how the contractor builds the structure. If the wall steel placement is critical to the design, the drawings should make this clear so the contractor has direction when pricing and building the structure.

Typically, the contractor builds the back form to partially support the reinforcing bars. This requires that the far-face vertical bars be the outermost bars and that the far-face horizontals then be tied to the vertical bars. Next, the big boundary or chord bars are placed in the middle of the wall; the near-face bars are placed the same way as the far-face bars, with the horizontal bars either tied to the boundary or chord bars or pulled up into place after the near-face vertical bars are placed.

The vertical bars can be the inner layer, but this requires that the wall reinforcement be placed before the forms so that the outermost horizontal bars can be tied in last on both sides. The contractor may decide to pre-tie or "cage" a lift of wall reinforcement and then hoist it into place. In this case, either the horizontal or vertical bars may be closest to the forms.

Flexural reinforcement is typically concentrated at either end of the shear wall in regions referred to as boundary elements. The longitudinal steel resists the tension forces and the concrete resists the compression forces. When the compression forces are high, the Code requires a "special" boundary element with closely spaced hoops and ties around the vertical bars to confine the concrete and prevent bar buckling, similar to the columns in a seismic frame. This confinement also allows the concrete to resist higher compressive strains. The horizontal wall bars must be anchored inside the confined core. When the compression forces are moderate, the Code allows a larger tie spacing, and the horizontal bars do not have to be hooked within the core, but must be hooked around the edge reinforcement. When compression forces are low, the Code does not require hoops or ties.

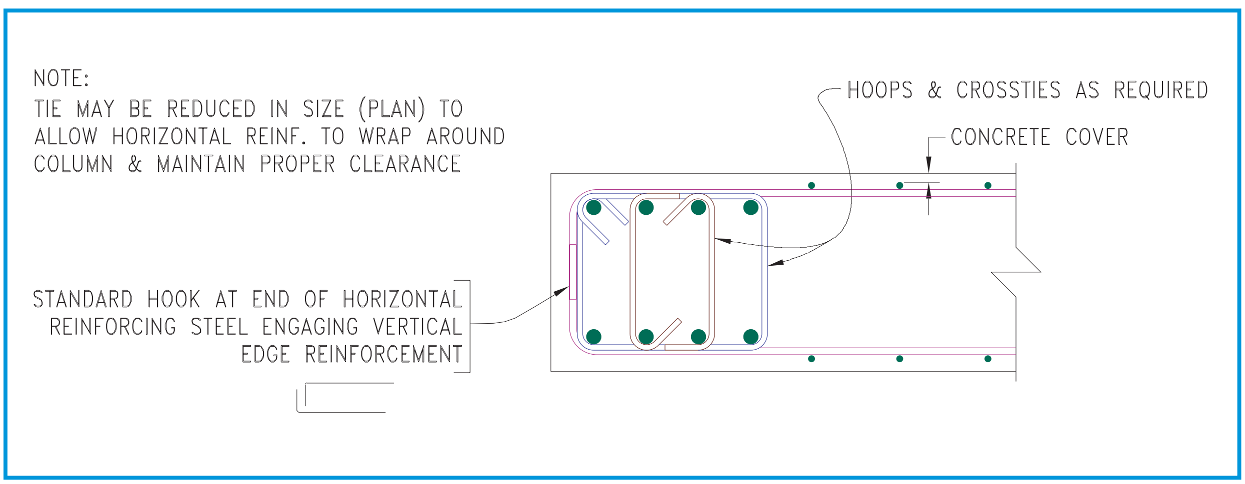

Figure 4a
A typical boundary element, when compression forces are low to moderate; outermost reinforcement placement.

Figs. 4a and 4b show typical boundary elements when the compression forces are low to moderate. In Fig. 4a the typical vertical bars are placed as the outermost reinforcement. The typical vertical bars are discontinued at the boundary element, which is logical since the larger boundary element bars effectively take their place. The horizontal bars end with a standard hook engaging the vertical edge reinforcement.

Fig. 4b shows three rows of longitudinal reinforcement in the boundary element. Note that for a clear spacing of 4 inches between the longitudinal bars, the wall must be 16 to 18 inches thick. Clearance is also needed for both lap and mechanical splices. The Code allows closer bar spacing, but considering the realities of splicing, a clear spacing of 4 inches is desirable. Engineers sometimes use this type of detail in walls that are 10 or 12 inches thick. It is much easier for us to draw the little dots representing the bars in the boundary elements than it is to place those large bars with such tight congestion. Note that if we detail the typical horizontal bars as the outer layers of bars, we can place the vertical boundary bars in the same plane as the typical vertical bars; we thus pick up an extra inch or so for our boundary bar placement.

Figs. 4c, 4d and 4e (shown on page 8) reflect typical "special" boundary elements. As noted before, the Code requires that horizontal bars be anchored in the confined core. This is typically done by hooking the bars into the confined core with a standard 90° or 180° hook as shown in Fig. 4c. Headed reinforcing bars could be used if there was severe congestion. Typically, the boundary element cages are assembled at the fabrication shop, delivered to the construction site, then hoisted into place. The typical horizontal bars are fished through the hoops and ties and anchored. Engineers sometimes specify that the horizontal bars be hooked down into the cores with a 90° hook; however, it can be difficult to install these bars without disassembling numerous ties. The bars can be hooked horizontally rather than down. This allows the typical horizontal bars to slide into the core without having to dissemble ties. Another solution is to use 180° hooks instead of 90° hooks. Since the out-to-out dimension of the hook is smaller for 180° hooks, the horizontal bars can be fished through a smaller width. Using 180° hooks, however, can make concrete placement and vibration more difficult.

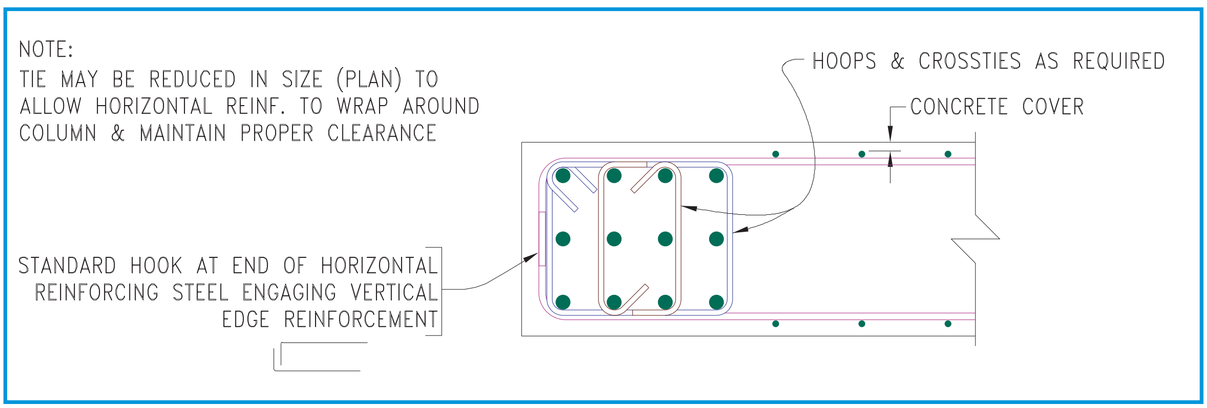

Figure 4b
A typical boundary element, when compression forces are low to moderate; additional longitudinal reinforcement.

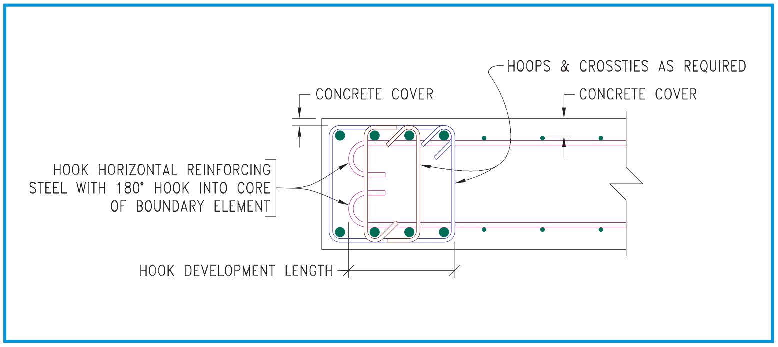

Figure 4c A typical "special" boundary element; reinforcing bars not offset and the most desirable.

One of the problems with anchoring the horizontal bars into the core of the boundary element is that the concrete cover for the typical horizontal and vertical bars is much greater than the minimum cover requirements, which causes congestion between the two curtains. One way to alleviate this problem is to offset the horizontal bars as shown in Fig. 4d. However, this will increase fabrication costs and the out-of-plane force at the bends may cause concerns. A better solution may be to simply provide additional concrete cover for the typical shear wall reinforcement. Another alternative would be to thicken the boundary elements as shown in Fig. 4e. This would allow a thinner wall section for shear, maintain the clearance necessary at the boundary element, and provide minimum concrete cover at all surfaces without bar offsets. Another way to reduce congestion is to design the boundary element so that it is long enough that the typical horizontal bars can be developed within the boundary element without end hooks.

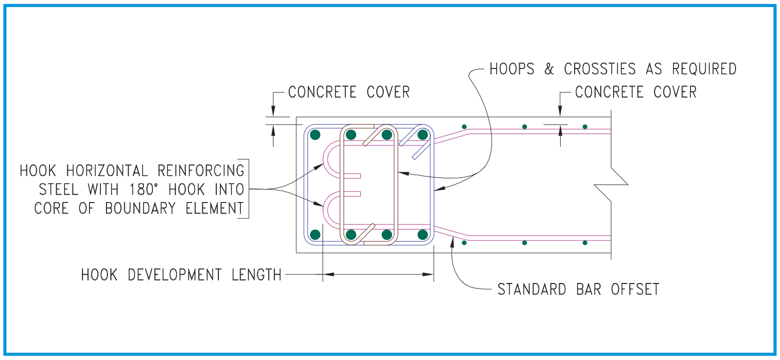

Figure 4d A typical "special" boundary element; offset horizontal bars.

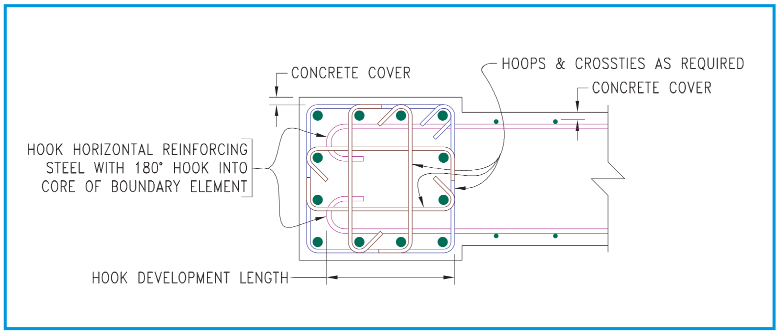

Figure 4e A typical "special" boundary element; shear wall thickened.

Shear Walls With Coupling Beams

Shear walls with coupling beams are a very effective means of providing lateral bracing for buildings subjected to earthquakes. This system is similar to the exterior concrete wall with window openings discussed previously. For simplicity, let's consider a simple wall of two vertical piers connected at each floor with coupling beams. Fig. 5 illustrates this simple wall with seismic forces applied horizontally. If the coupling beams are fully effective and provide complete coupling between the piers, the overturning moments will be resisted by a vertical compression force in one wall pier (the pier to the right in Fig. 5) and tension in the other pier. Under load reversals, the opposite will occur. If we draw a freebody diagram through the coupling beams, the sum of the vertical shears in the coupling beams would equal the tension or compression force at the base of the wall from the overturning moment. The distribution of total vertical shear to the individual coupling beams can be determined by analysis. For preliminary analysis, consider some foundation rocking or base flexibility, and assume equal shear or equal shear stress in each coupling beam for rough sizing. If the coupling beams are more flexible and provide only partial coupling, there will be three base overturning moments – one similar to the model already described above and one at the base of each pier representing the pier acting independently as a vertical cantilever wall. A computer analysis is necessary to determine the wall's response, and it is a good idea to consider variables such as uncracked and cracked concrete sections and some soil or foundation deformation beneath the base of the wall. Minor changes in cracked concrete properties or foundation deformation can have a significant effect on computer results; it is advisable to consider the range of possible conditions.

The Code requires that coupling beams with a length-to-depth ratio less than two and shear stresses of $4\sqrt{f'c}$ or greater be reinforced with groups of diagonal bars. The Code permits diagonal reinforcement in beams with a length-to-depth ratio less than 4, but ratios greater than 3 result in very flat diagonal bars that are not very effective. Diagonal reinforcement is not required if the coupling beams do not affect building stability, which is usually an appropriate assumption for flexible coupling beams with length-to-depth ratios greater than 3. Research following the 1964 Alaskan earthquake demonstrated the superior performance of this arrangement of reinforcement, compared to traditional beam reinforcement. While no one debates the superior performance, some note the reinforcing bar congestion and placement issues that this arrangement of reinforcement creates. Placement can be greatly simplified with proper planning and large enough sections. The wall should be at least 16 inches thick to allow placement of the reinforcing bars; 14 inches is the bare minimum.

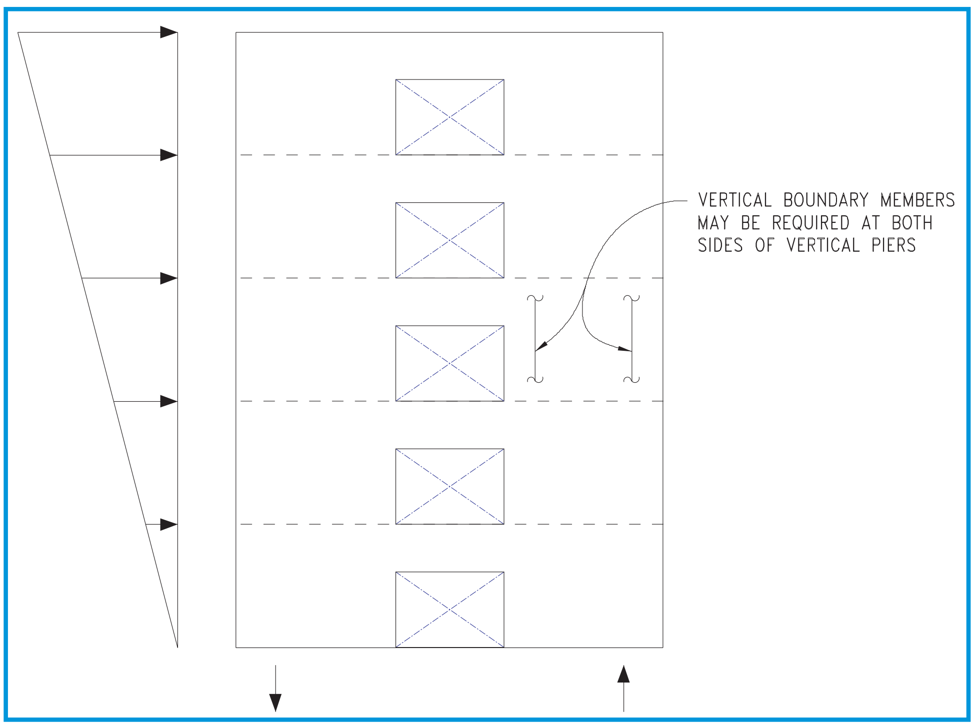

Figure 5
Coupled shear wall.showing typical coupling beams requiring diagonal reinforcement.

Let's consider a 16-inch thick wall with a diagonally reinforced coupling beam similar to Fig. 6. Assume that each diagonal requires 4 - #9 (#29) bars and that there are two layers of #9 (#29) longitudinal reinforcing bars at the edge of the vertical piers. The #9 (#29) diagonal bars must extend at least a full tension development length into the pier at each end. Each group of 4- #9 (#29) bars must also be enclosed in confinement reinforcement as would be specified for column confinement in a special moment frame or the special boundary member of a shear wall. This amounts to something like #4 ties at 4 inches on center, as shown in Fig. 6. In the center where the diagonals overlap, the two groups of ties can either continue with some overlapping, or one set of somewhat wider ties can be placed around both groups of diagonal bars as shown in Fig. 6.

The key to getting the reinforcing bar layout to work is to spend a few minutes on a sketch like Fig. 7 to make sure that all of the bars will fit. Let's look at Fig. 7 using rough sizes for bars and including an allowance for the bar deformations (which take up space). An interior wall needs ¾ inch concrete cover; if the typical vertical and horizontal bars are #4 or #5 bars at 12 inches on center, the reinforcement at each face including the cover will take up about 2.25 inches. Adding another 1.25 inches for vertical wall boundary or trim bars gets us to 3.5 inches from each face. The two layers of bars from each diagonal group add another 2.5 inches and bring us to about 6 inches from each face. If our wall is 16 inches thick, we have 4 inches available open in the middle of the wall for concrete place-

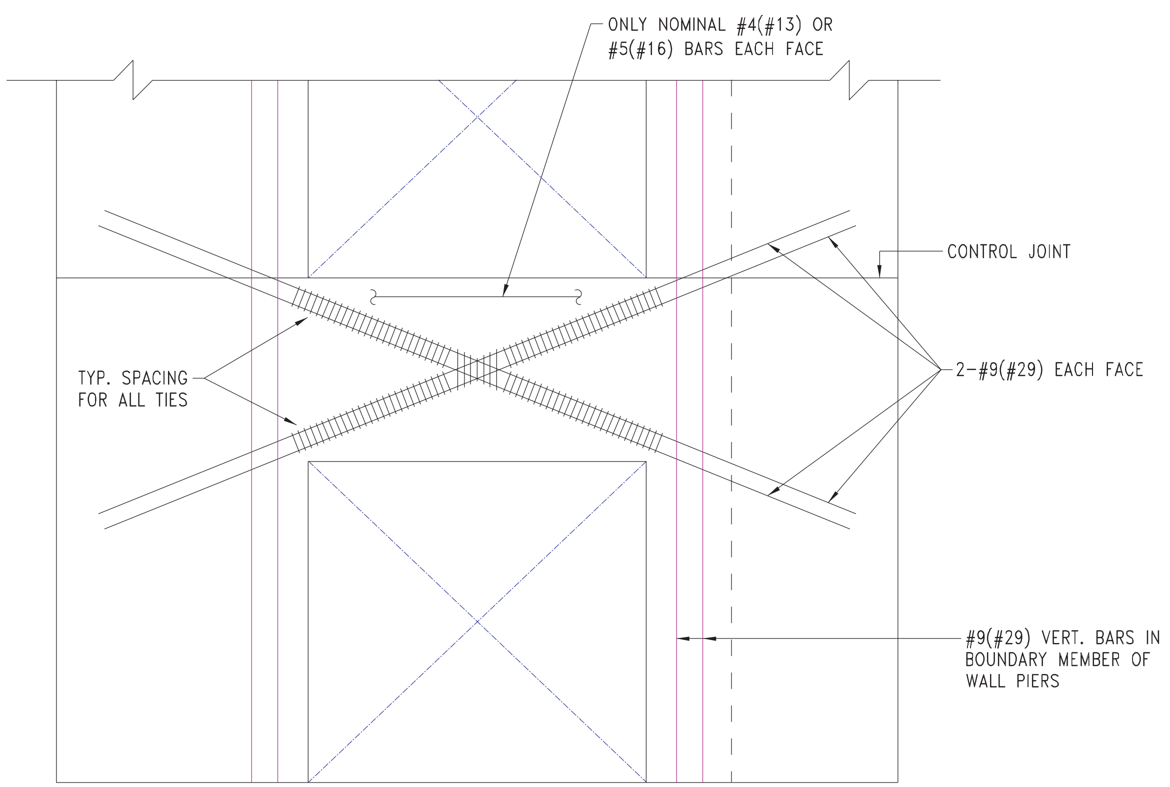

Figure 6
Elevation of coupling beam with diagonal reinforcing bars. Note that each diagonal strut must consist of at least 4 bars with closely spaced ties as shown. Best to use a few vertical ties at central intersection.

ment and vibration. (The ties around the diagonal reinforcement coincide with the vertical boundary element bars and thus do not require a dedicated space of their own.) A 14-inch thick wall leaves only 2 inches clear; this invites concrete placement difficulties and rock pockets at the surface. Note how easy it is to draw the sketch in Fig. 7; you can quickly ensure that you have provided a section that is thick enough to place both reinforcing steel and concrete. The authors use this type of sketch routinely and find it guides them towards a design that the contractor can build without any great difficulties.

Fig. 8 shows reinforcing bars being placed for a coupling beam with diagonal reinforcement. It all goes well with some planning and a wall that is thick enough to accommodate the required reinforcement.

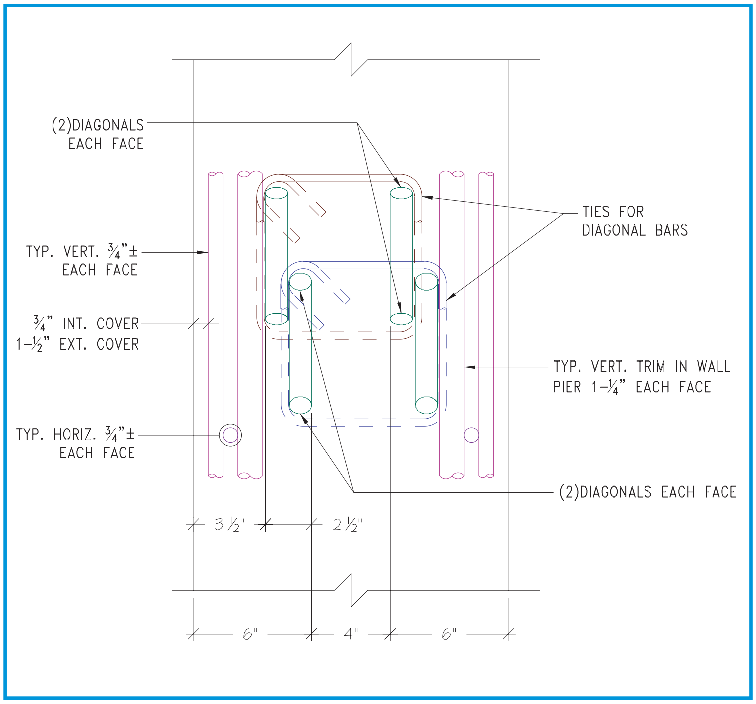

Figure 7
Cross section of a wall in Figure 6 showing rough bar dimensions. Use a sketch like this in your calculations to ensure you make the wall thick enough to allow rebar and concrete to be placed.

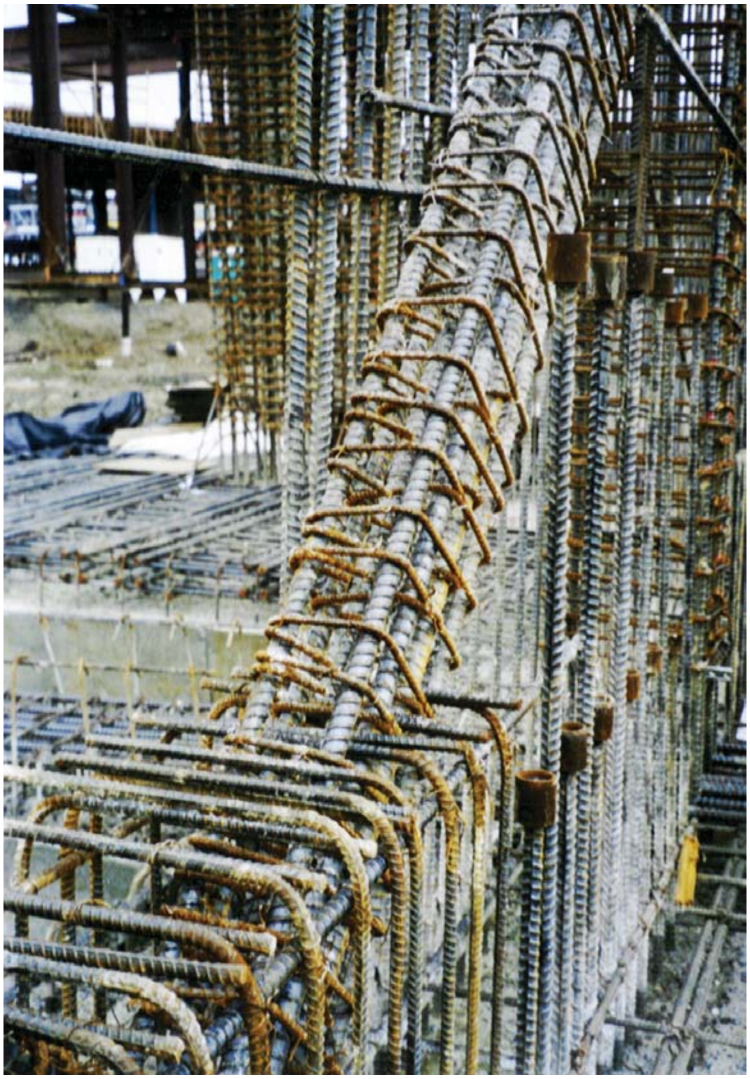

Figure 8
Placement of reinforcing bars for a coupling beam with diagonal requirements.

Headed Reinforcing Bars

A somewhat recent development in reinforcing steel technology is the use of headed reinforcing bars as an alternative to 90° or 180° hooks. The heads may be forged at the bar end, a plate may be welded to the reinforcing bar or a proprietary washer may be threaded onto the bar. The idea is to use this enlarged section to provide anchorage or development in place of a hook that might cause congestion. Using headed reinforcing bars in highly congested areas such as joints and shear walls can also reduce labor costs. The authors are excited about the potential of headed reinforcing bars and believe that it will simplify many reinforcement details. Our office has used headed reinforcing bars on several projects with great success.

An industry consensus on the size of heads had not been reached by mid-2003 when this paper was written, however. The first version of the ASTM standard covering headed reinforcement "Standard Specification for Welded or Forged Headed Bars for Concrete Reinforcement" (ASTM A970-98) requires a head that is 10 times the bar area with a further limitation that the concrete have a 28-day compressive strength of 4300 psi or greater. Recent research, not yet published, suggests a much smaller head, perhaps 5 to 6 bar areas, may be adequate, and one manufacturer is marketing a 4 bar area head. A large enough head can provide anchorage by itself while smaller heads can provide acceptable anchorage with some amount of bar length to the critical plane.

ACI Committee 318 has yet to incorporate headed reinforcing bars into the Code as the product is somewhat undefined and research on anchorage is still being evaluated. Designers should recognize that headed reinforcement is proprietary and technically treated as an alternative material-method by the Code. Building officials will probably accept heads of 10 bar diameters in accordance with ASTM A970-98, but smaller heads may require specific approvals as an alternative material. Headed reinforcement can certainly reduce congestion, so it is often a desirable design solution, but until it is adopted in the Code, designers should verify that the building officials will accept what is specified.

Figure 9
Headed reinforcing bars on the top of longitudinal column bars.

Designers should also check availability and lead time. There are currently a limited number of suppliers in the U.S., and most reinforcing steel fabricators need to special order headed reinforcing bars. If there are likely to be changes and modifications to anchorage details during construction, obtaining the required reinforcement on short notice may be a concern. Designers in seismic regions should be aware that headed reinforcement is intended for tension development; if stress reversals cause compression in headed reinforcement near a surface parallel to the head, the concrete between the head and the surface may pop out.

Fig. 9 illustrates using headed reinforcing bars on the top of longitudinal column bars. The heads were staggered slightly to avoid creating a weakened plane. Imagine the congestion that would have resulted if all of these bars were hooked. For reasons similar to the tank and retaining wall base discussed in the following section, proper design would require that the bars be hooked inward to develop the moment strength of the column.

Notes on Soft Metric Reinforcing Bars

Soft metric designations for the sizes of reinforcing bars are shown throughout this report. This approach follows current industry practice. In 1997, producers of reinforcing bars (the steel mills) began to phase in the production of soft metric bars. The shift to exclusive production of soft metric bars has been essentially achieved. Virtually all reinforcing bars currently produced in the USA are soft metric. The steel mills' initiative of soft metric conversion enables the industry to furnish the same reinforcing bars to inch-pound construction projects as well as to metric construction projects, and eliminates the need for the steel mills and fabricators to maintain a dual inventory.

The sizes of soft metric reinforcing bars are physically the same as the corresponding sizes of inch-pound bars. Soft metric bar sizes, which are designated #10, #13, #16, and so on, correspond to inch-pound bar sizes #3, #4, #5, and so on. The table below shows the one-to-one correspondence of the soft metric bar sizes to the inch-pound bar sizes. More information about soft metric reinforcing bars is given in Engineering Data Report No. 42, "Using Soft Metric Reinforcing Bars in Non-Metric Construction Projects". EDR No. 42 can be found on CRSI's Website at www.crsi.org.

Soft Metric Bar Sizes vs. Inch-Pound Bar Sizes

Soft Metric Bar Size Designation	Inch-Pound Bar Size Designation
#10	#3
#13	#4
#16	#5
#19	#6
#22	#7
#25	#8
#29	#9
#32	#10
#36	#11
#43	#14
#57	#18

T-Configured Joints: Beam-Column and Wall-Footings

T-configured joints are common in structural design, yet engineers often do not design them correctly. Let's consider beam-column joints. Figure 3b illustrates an exterior beam-column joint in a seismic resisting frame; since there is only a beam on one side of the column, we have a T-configuration. Most engineers know they should hook the beam bars towards the center of the beam. This allows the beam bars in tension to impart diagonal compression from the radius of the hooks to close the internal forces within the joint. ACI 318-02 includes a new Appendix A for strut and tie models, which are an excellent way to understand the stress flow through such a joint. Let's now look at a column-to-continuous roof beam, another T-configured joint. Without thinking of the internal joint stress flow, many engineers will hook the column bars out away from the center of the column with the thought that it will be easier to place concrete down into the column. This is probably true but the joint cannot develop the flexural strength of the column that may be needed in frames resisting lateral forces.

A similar condition exists at the base of a retaining wall or tank wall that is resisting significant bending moments. Let's consider the wall-to-base slab detail in Fig. 10a. The wall resists lateral fluid or soil pressure as a cantilever with a large base moment. A basement wall supported by a floor at the top also has large base moments. Likewise, a circular tank wall that is designed allowing horizontal hoop tension to resist the fluid pressure works fine but, the restraint of the base slab also results in a significant moment at the base slab.

Should the vertical wall reinforcing bars be hooked in or out at the base slab in these cases? Many engineers and most contractors would hook the bars out to avoid congestion. This is wrong. A base slab detail with bars hooked out cannot develop the total moment in the wall. Research in Sweden many years ago showed that a T-joint with wall and base slab of about equal thickness, with wall bars hooked out as seen in Fig. 10b, can only resist about 40% of the wall moment. The joint develops diagonal shear cracks and eventually fails. The authors have observed a similar situation where the cantilever wall of an open rectangular aqueduct resisting its internal water pressure deflected out several inches and leaked through the failed base joint. The cracks and failure surface were similar to those in the Swedish tests.

Think of the wall as similar to the beam in a special moment resisting frame at an exterior column. For that T-joint, we know we must hook the top and bottom beam bars towards the center of the joint to make the joint perform properly. It is the same for the wall base joint, as

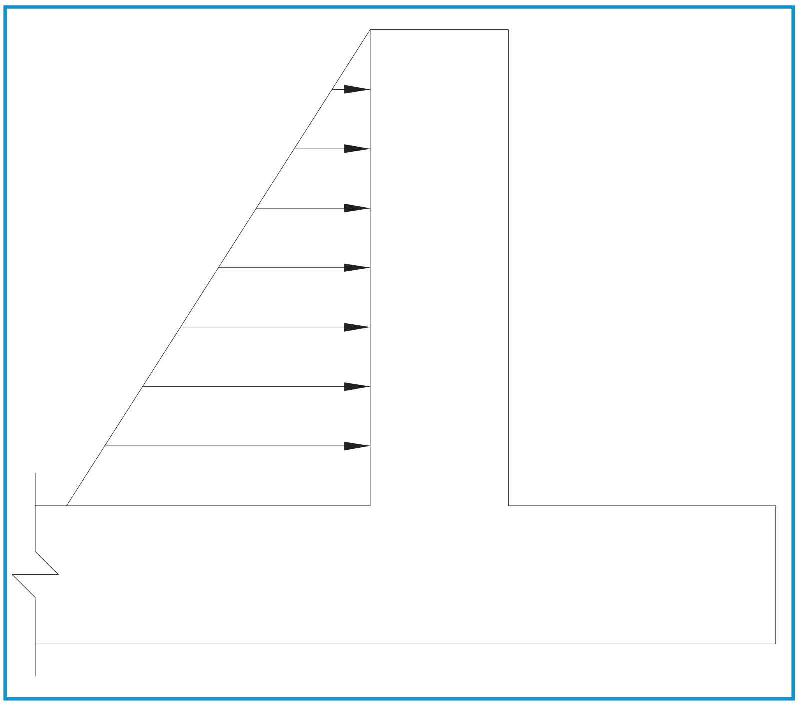

Figure 10a
Retaining or Basement Wall to base slab detail.

shown in Fig. 10c. For tank and retaining wall base joints, hook bars that are in tension towards the center of the joint.

ACI 318-99 had new seismic requirements for foundations. When hooks are required for development in special moment frame columns that are assumed fixed for moment at the base, Section 21.8 (21.10 in ACI 318-02) required the column bars resisting the moment be hooked towards the center of the base joint. This is for the same reason. If the column bars are developed as straight bars in the footing, however, the direction of the hooks solely to facilitate placement is unimportant.

One might say that this detail is contrary to the theme of this paper as these inward hooks may increase congestion. There is some truth here, but we have to detail the reinforcing bars to properly resist the forces. Recognizing this, we then strive to minimize congestion and increase constructibility.

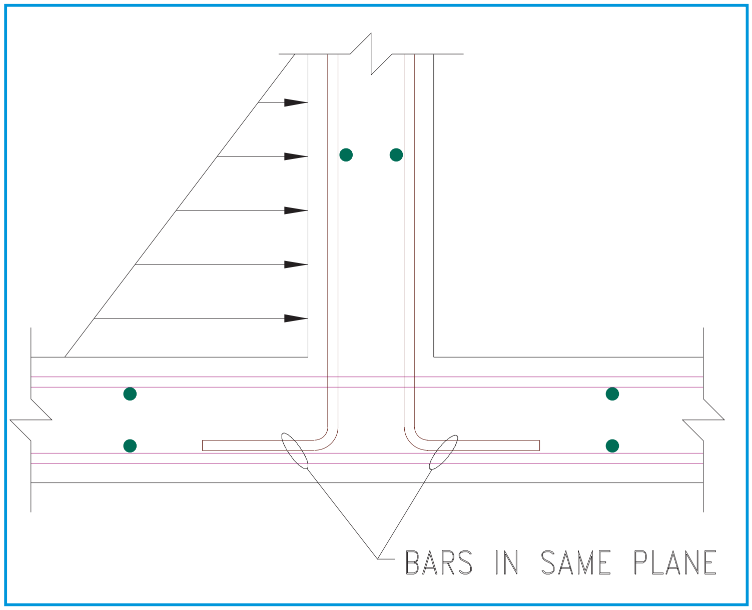

Figure 10b
This T-joint detail is not recommended, as it cannot develop nominal moment strength.

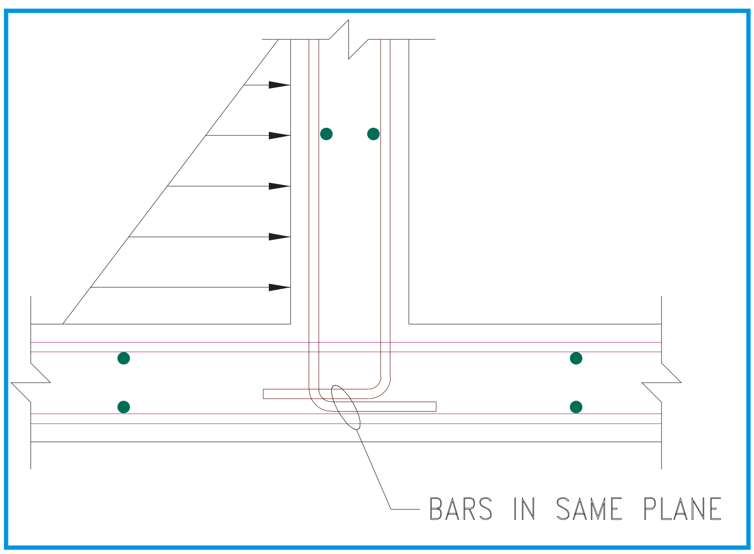

Figure 10c
This T-joint detail represents the proper way to develop nominal moment strength at the base of the wall.

Slabs On Ground

Slabs on ground are slightly different. Typically, the problem is not avoiding congestion but achieving a slab that does not crack excessively. It is common for designers to consider a slab on ground as nonstructural, which they often are, and simply specify a light welded wire fabric for reinforcement. Workers must walk on the small diameter welded wire fabric to place the concrete; tests after the fact almost always show the welded wire fabric at the bottom of the slab except very close to the supports. Having the workers pull up the fabric with claw hammers as they finish the concrete never seems to work. The authors believe it is better to reinforce slabs on ground with #4(#13) bars at perhaps 18 inches on center, 1 to 1.5 inches down from the top of the slab. Having this reinforcement near the top surface of the slab controls cracking and the spacing is large enough for the workers' boots to go between bars without forcing them to the bottom. Also, the bars are stiffer than the welded wire fabric, which is advantageous in case they get stepped on during concrete placement. Reinforcing steel in slabs on ground will not be effective in controlling cracking unless it is properly located within the concrete.

Two other points to consider for slabs on ground: slabs on ground often crack excessively because of restraint to normal slab shortening from the soil or from columns and footings. The shrinkage and temperature reinforcement in ACI 318 is based on unrestrained concrete shrinkage and shortening. When a slab is restrained, two and one-half to three times this reinforcement is needed to control cracking. Slab cracks often cause performance issues and a single curtain of substantial reinforcing steel is a very minor cost for any building. Controlling cracking in slabs on ground can make building owners very pleased with your design abilities.

Finally, slabs on ground often require expansion anchors to anchor equipment, shelving, storage racks, etc. ACI 318-02 contains new requirements for expansion anchors and anchor bolts. It requires that expansion anchor embedment be limited to two-thirds of the slab thickness. This is due to a concern that the anchor expansion might blow out the bottom of the slab; the researchers had actually proposed that the limit be half of the slab depth. A 6-inch thick slab on ground will have a 4-inch anchor embedment limitation; a 5-inch. thick slab with have a 3.5 inch limitation. Slabs in warehouse structures may need to be thicker than many have traditionally designed.

Conclusions

In this paper, the authors have offered some suggestions to their fellow structural engineers on our obligation to design and detail reinforced concrete structures so the contractor and reinforcing steel subcontractor can build them as easily and economically as possible. Most of these suggestions are common sense and simply require giving a little thought to how the design will be built. It has been the authors' experience that a well-detailed set of drawings, where these constructibility issues have been addressed, results in lower bid prices. Once you establish a reputation in this way, contractors will praise your drawings, consistently give your designs lower bid prices, and the word will spread, possibly bringing you new design commissions.

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