PDHonline Course S217 (3 PDH)

Steel Support Framing for Facades, Curtainwalls and Cladding

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Steel Support Framing for Facades, Curtainwalls and Cladding

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COURSE CONTENT

Support Framing for Façades, Curtainwalls and Cladding

In order to protect the interior spaces of a building from the outside environment, buildings are clad with number of different types of materials. These materials can include masonry, precast, metal siding, exterior insulation finishing systems (EIFS, i.e. synthetic stucco system) and a number of other different types of similar materials. In most cases, such as with brick and metal panels, the cladding is really nothing more than a veneer that must be supported via a structural backup (typically either light gage metal studs or CMU).

In some situations the structural backup is not able to span vertically between the adjacent framed levels. In this case it is necessary to introduce an intermediate horizontal support member which is typically referred to as a girt. The girt serves as a lateral support to the backup or cladding and in turn spans from column to column. If the supporting column for a girt is used to reduce the horizontal span of the girt and the same column does not serve to support any gravity load other than its own weight and that of the girt it is typically referred to as a wind (or soldier) column. In almost all cases at an opening in a brick veneer or other similar masonry cladding, it is necessary to provide a lintel in order to support the gravity load of the veneer or cladding above the opening. In some cases a loose steel angle in the plane of the brick is used as a lintel at narrow punched openings. In other cases in which the opening is very long (i.e. a ribbon opening) it is necessary to support the shelf angle from a continuous steel lintel that is hung from the floor or roof framing above.

Other claddings, such as precast concrete panels, are capable of spanning either vertically between each framed level or horizontally from column to column, therefore no structural backup is required. There are also other self-supporting, premanufactured systems available for stone, GFRC (Glass Fiber Reinforced Concrete) and other similar materials.

Examples of a girt supported cladding are shown in Figures 1, 2 and 3. All three of these figures relate to a metal panel clad roof top mechanical penthouse space in which the backup structural system is a vertically spanning light gage metal stud. Figure 1 is a partial elevation of one side of the penthouse. Figure 2 is a section at an interior space of the penthouse. Figure 3 is a section at a screen wall that surrounds an adjoined exterior cooling tower space.

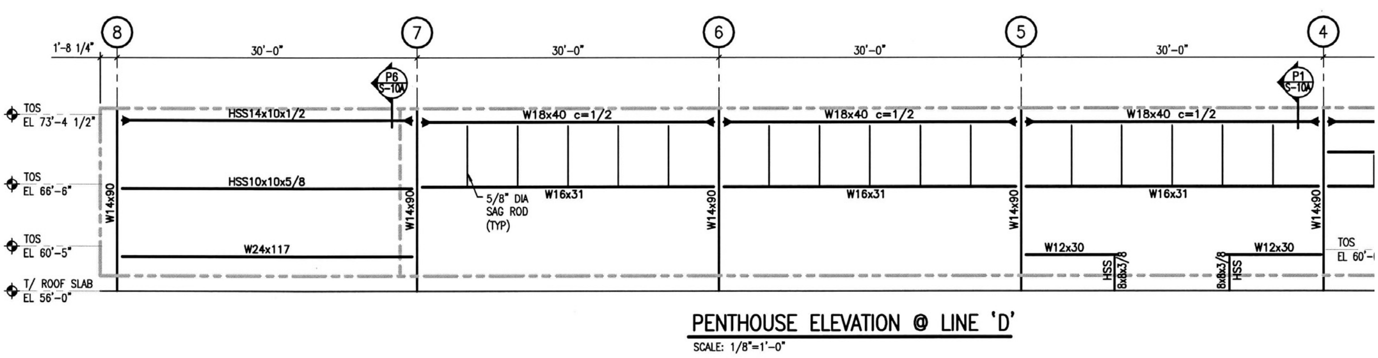


FIGURE 1

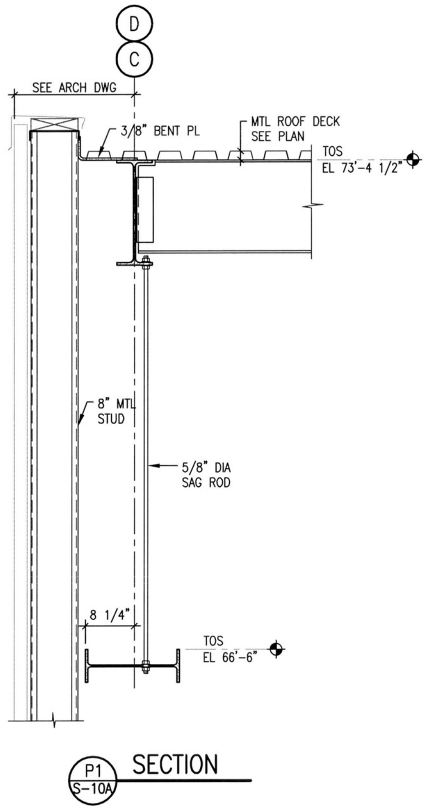


FIGURE 2

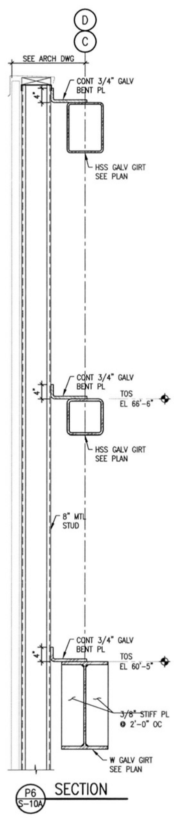


FIGURE 3

Key items that should be noted in Figures 1 through 3 include:

1. Sag rods are required to support the interior W16 girts because these members are turned on their sides to expose the strong axis to the imposed lateral loads, which in turn exposes the weak axis to the dead load of the member. Therefore sag rods are used to reduce the horizontal span of the member. A further discussion of sag rods will be provided later in this lecture.

2. The girts used at the exterior screen wall in Figure 3 are HSS in order to avoid the type of deterioration that would result if a wide flange section laid in its side were exposed to the exterior, i.e. the pocket formed by the web and upturned flanges would fill up with rainwater and snow, unless drainage holes are drilled in the web.

An example of a girt used to support an insulated metal sandwich wall panel is shown in Figures 4 (elevation) and 5 (section). In this case the girt is integral with a vertically X-braced bay. Note that the girt is continuous between the supporting columns. The girt could have been designed and detailed to be discontinuous at the intersection or midpoint of the vertical bracing, however, this would have required that one of the braces be designed and detailed as a continuous member between the supporting columns as required to resist the horizontal reaction of the girts at the middle of the bay.

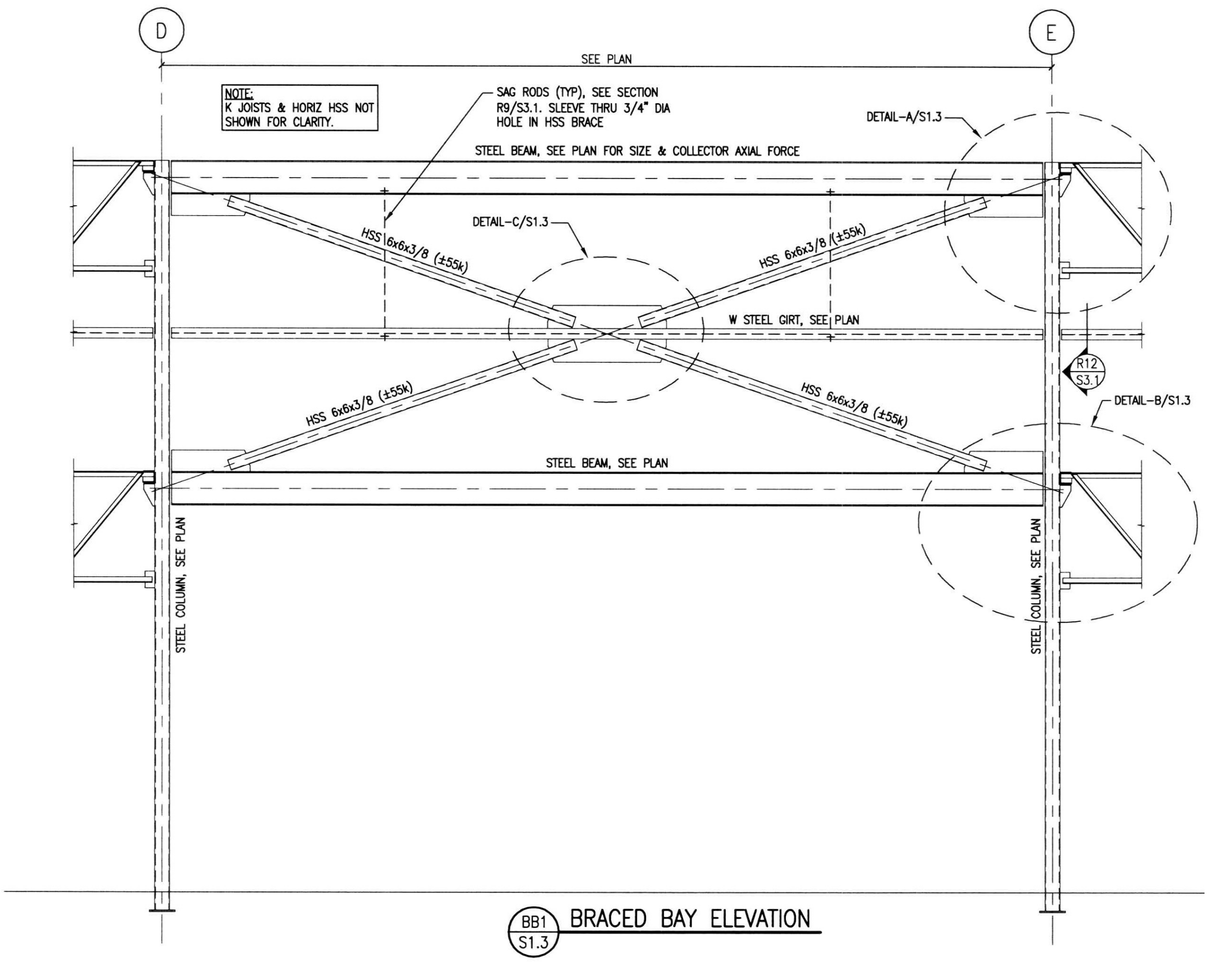


FIGURE 4

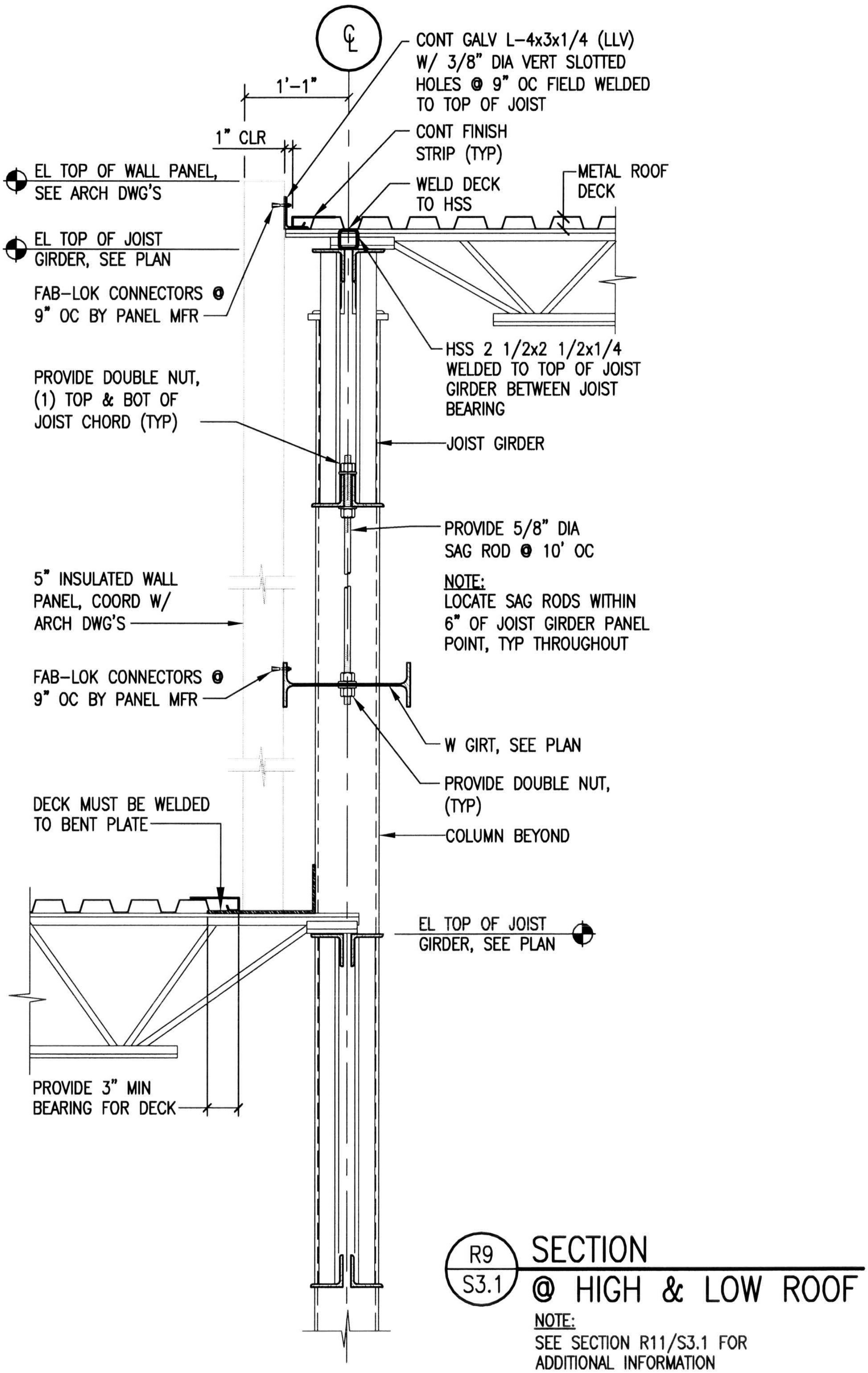

CONT GALV L-4x3x1/4 (LLV) W/ 3/8" DIA VERT SLOTTED HOLES @ 9" OC FIELD WELDED TO TOP OF JOIST
1'-1"
1" CLR
CONT FINISH STRIP (TYP)
EL TOP OF WALL PANEL, SEE ARCH DWG'S
WELD DECK TO HSS
METAL ROOF DECK
EL TOP OF JOIST GIRDER, SEE PLAN
FAB-LOK CONNECTORS @ 9" OC BY PANEL MFR
HSS 2 1/2x2 1/2x1/4 WELDED TO TOP OF JOIST GIRDER BETWEEN JOIST BEARING
PROVIDE DOUBLE NUT, (1) TOP & BOT OF JOIST CHORD (TYP)
JOIST GIRDER
PROVIDE 5/8" DIA SAG ROD @ 10' OC
NOTE:
LOCATE SAG RODS WITHIN 6" OF JOIST GIRDER PANEL POINT, TYP THROUGHOUT
5" INSULATED WALL PANEL, COORD W/ ARCH DWG'S
FAB-LOK CONNECTORS @ 9" OC BY PANEL MFR
W GIRT, SEE PLAN
PROVIDE DOUBLE NUT, (TYP)
DECK MUST BE WELDED TO BENT PLATE
COLUMN BEYOND
EL TOP OF JOIST GIRDER, SEE PLAN
PROVIDE 3" MIN BEARING FOR DECK
R9
S3.1
SECTION
@ HIGH & LOW ROOF
NOTE:
SEE SECTION R11/S3.1 FOR ADDITIONAL INFORMATION

FIGURE 5

Figure 6 is an example of a girt associated with an exterior stand alone screen wall system. Note that all of the joints of the HSS members are to be seal welded thereby preventing moisture intrusion which would result in freeze-thaw damage and deterioration.

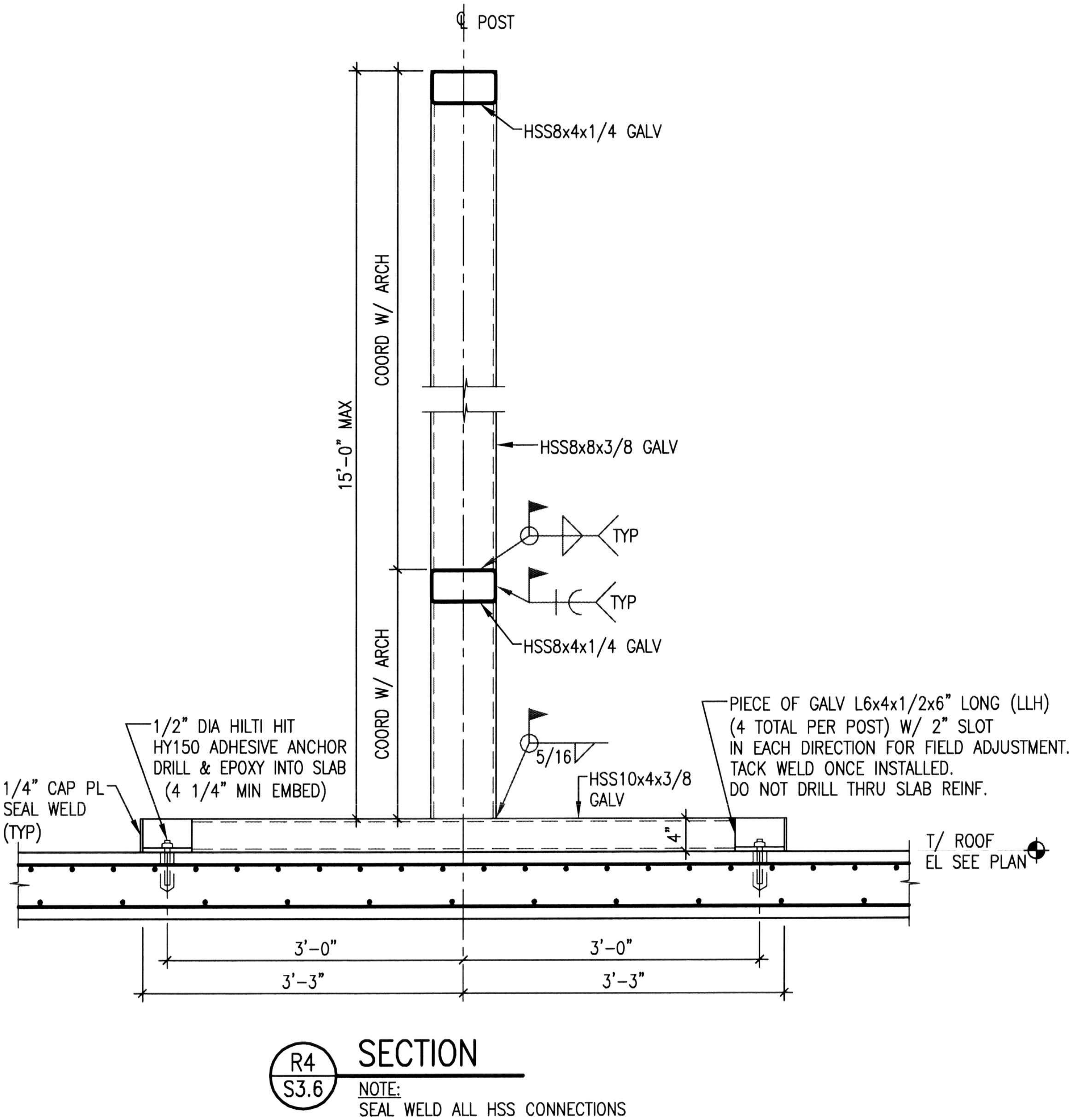


FIGURE 6

Figures 7, 8, 9, 10 and 11 are examples of a premanufactured stone cladding system in which the stone is attached to a self-supporting backup truss that is capable of spanning from column to column.

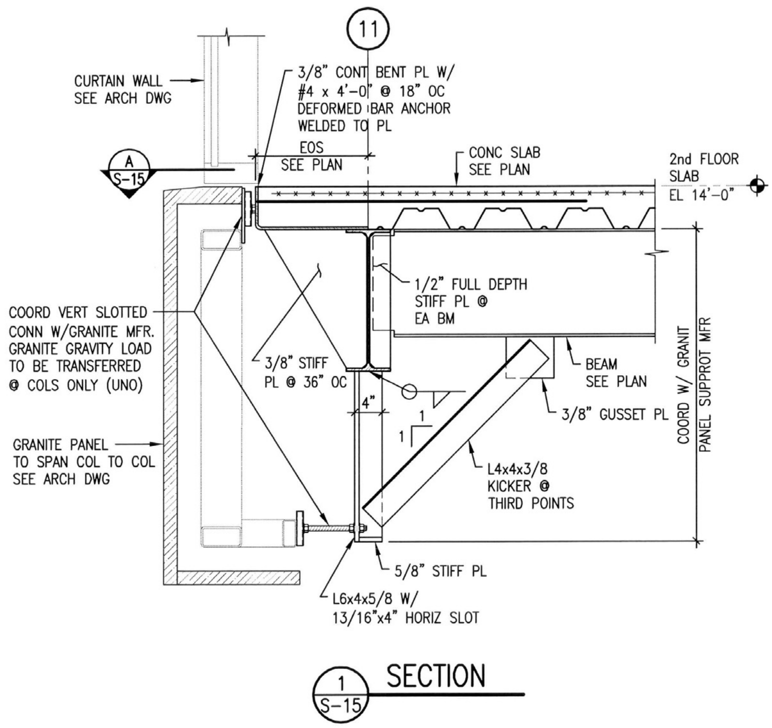


FIGURE 7

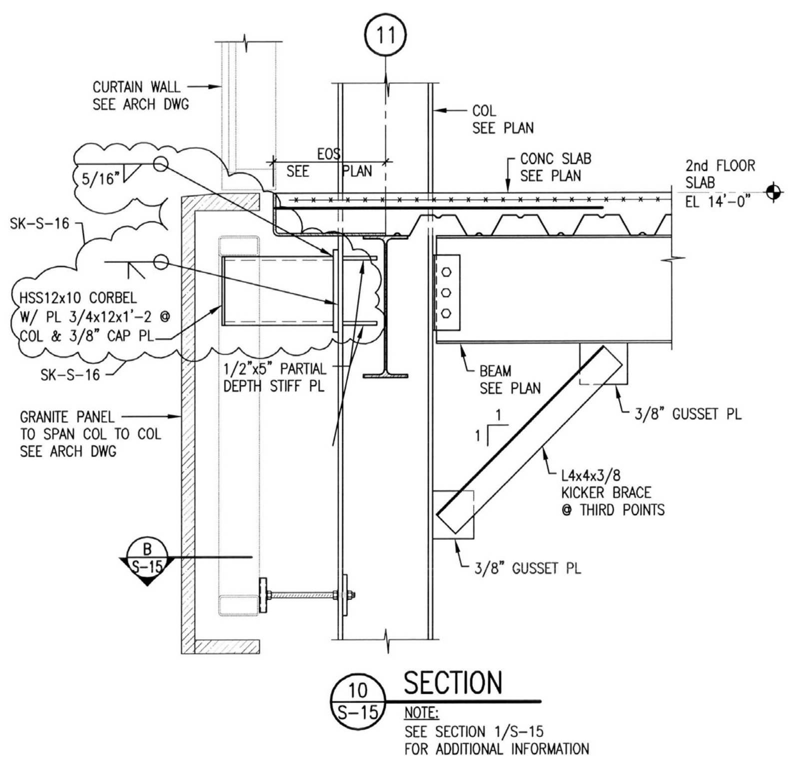


FIGURE 8

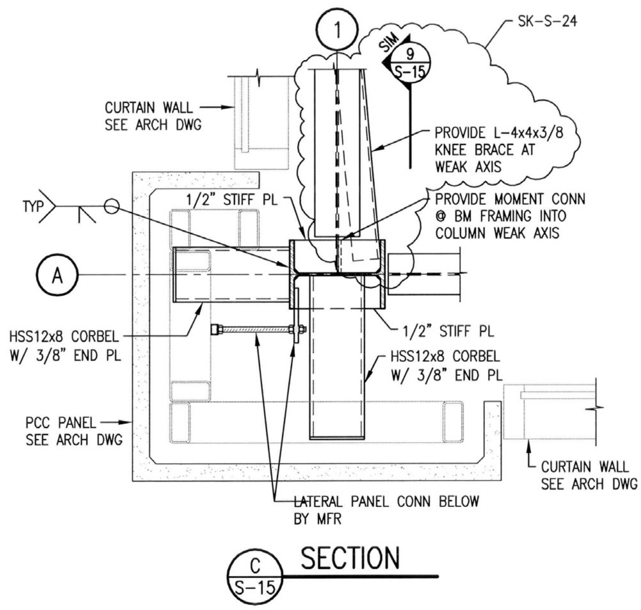


FIGURE 9

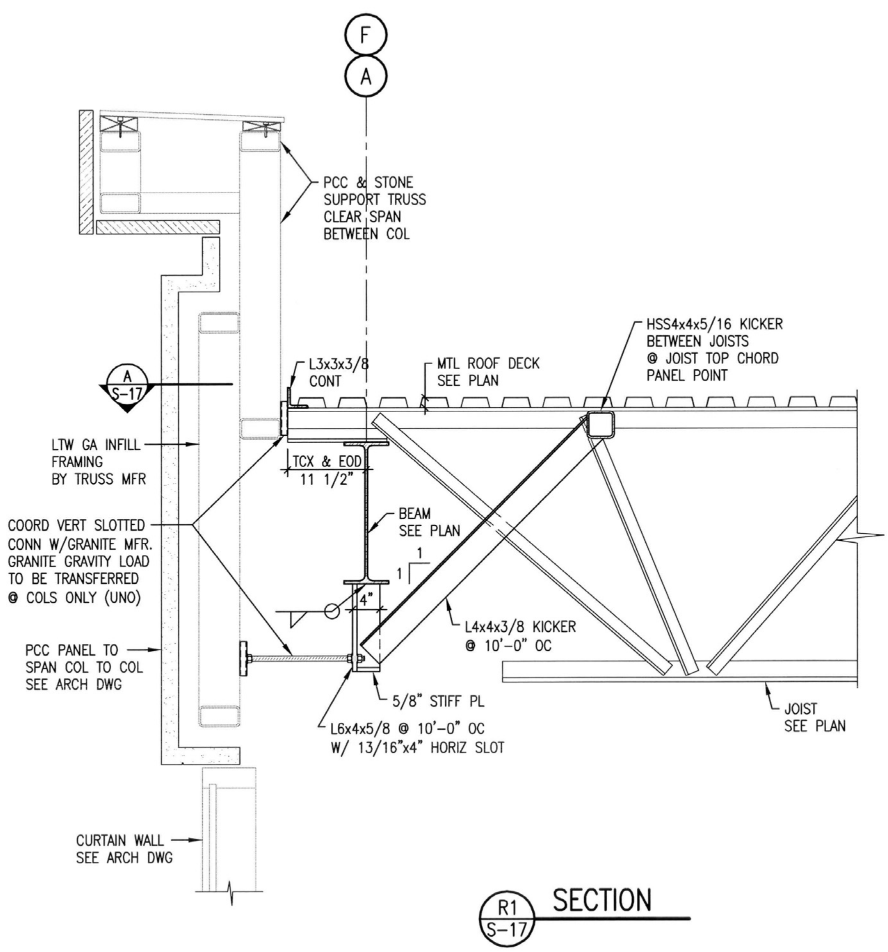


FIGURE 10

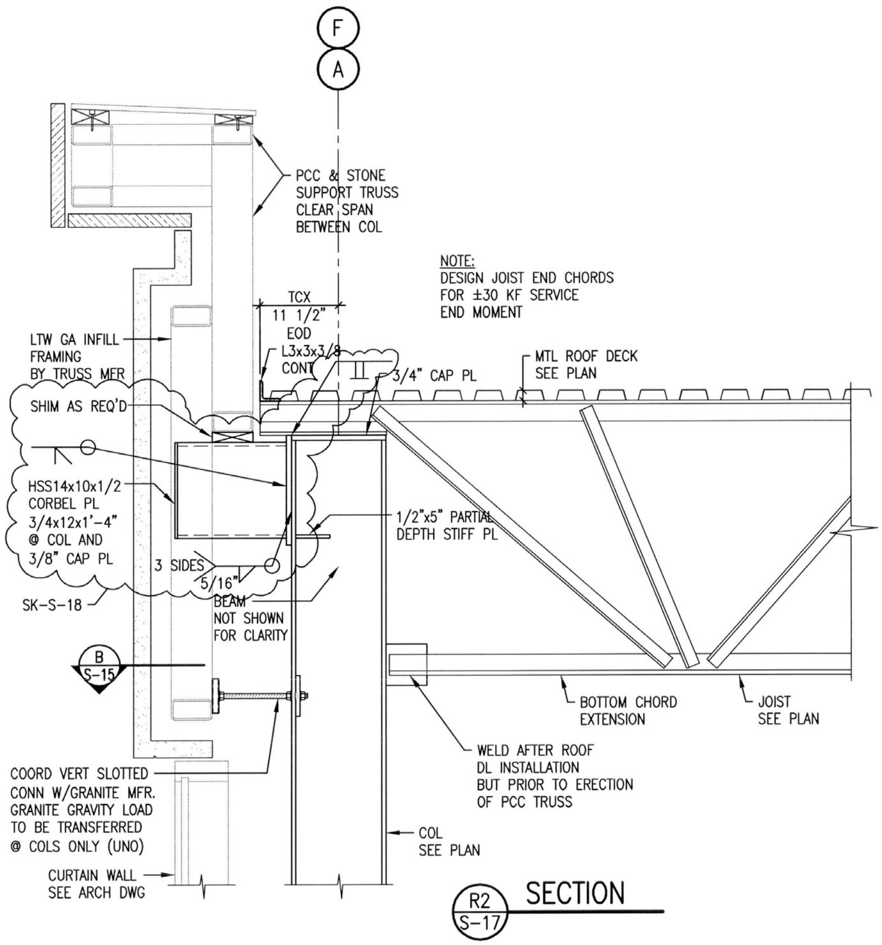


FIGURE 11

Key items that should be noted in Figures 7 through 11 include:

1. Figures 7 (at a floor level) and 10 (at the roof) indicate that vertically slotted connections are required in order to prevent imposing any gravity load from the cladding system on to the beams, i.e. the connections are intended for lateral support only and the self-supporting backup truss is allowed to deflect vertically in between the column support locations.

2. Figures 8, 9 and 10 show how all of the gravity load of the system (and a portion of the lateral load) is supported at the columns only.

Figures 12, 13 and 14 are examples of a metal panel clad roof top mechanical penthouse space in which there is no backup structural system required, i.e. the panels are capable of spanning from the floor of the penthouse to the girt located at the roof of the penthouse. Figure 12 is a partial elevation of one side of the penthouse. Figure 13 and 14 are sections at the roof of the penthouse which show how the wind columns that support the girts are in turn laterally braced by the adjacent roof framing but do not resist any gravity loads from the roof because of the vertically slotted hole at the wind column to underside of beam connection.

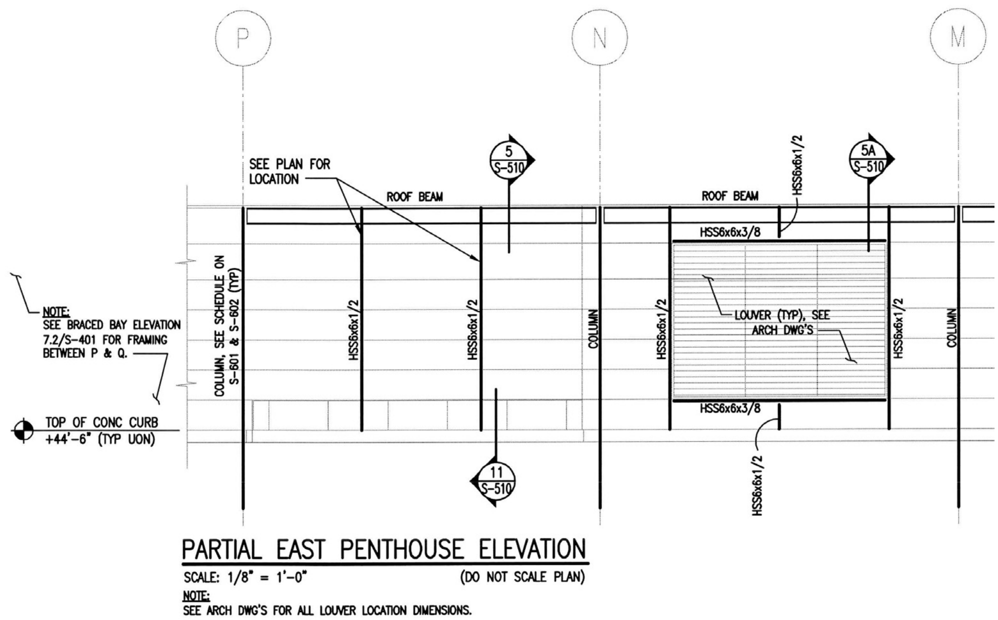


FIGURE 12

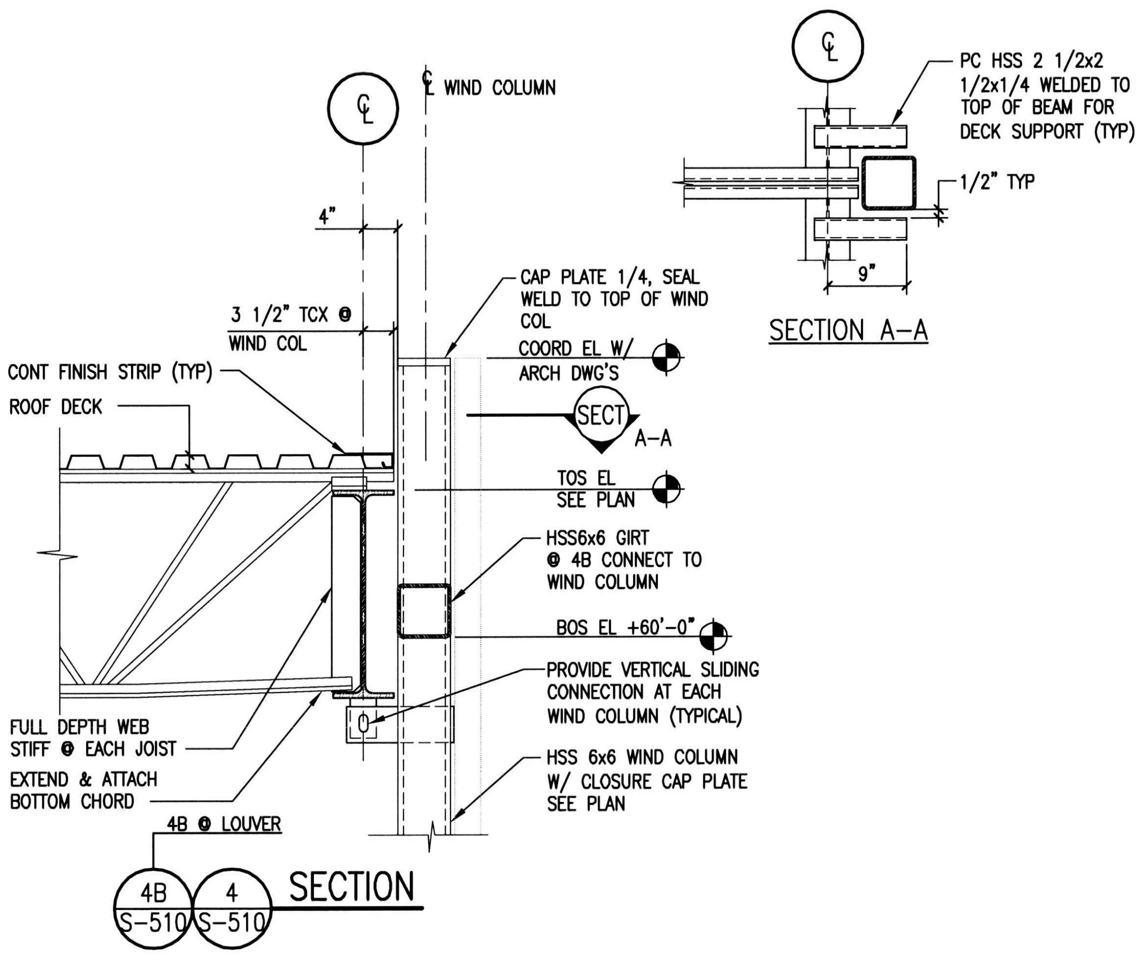


FIGURE 13

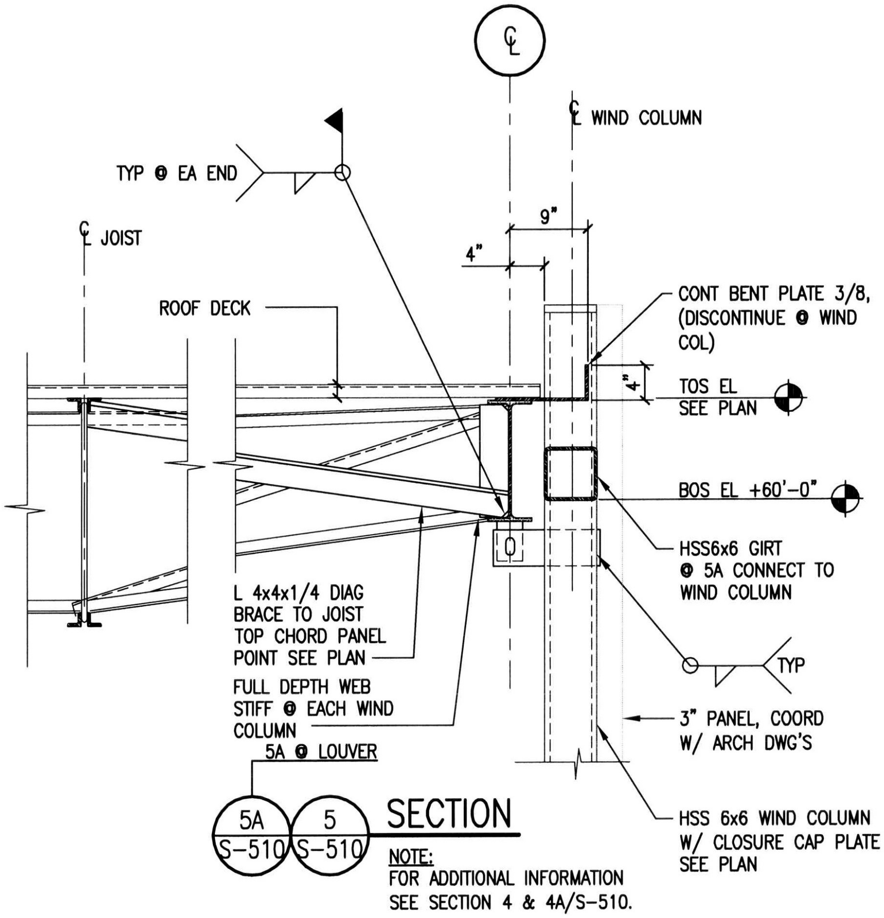


FIGURE 14

Figures 15, 16 and 17 are examples of typical supports provided at a curtainwall system. A further discussion of curtainwall systems will be provided later in this lecture.

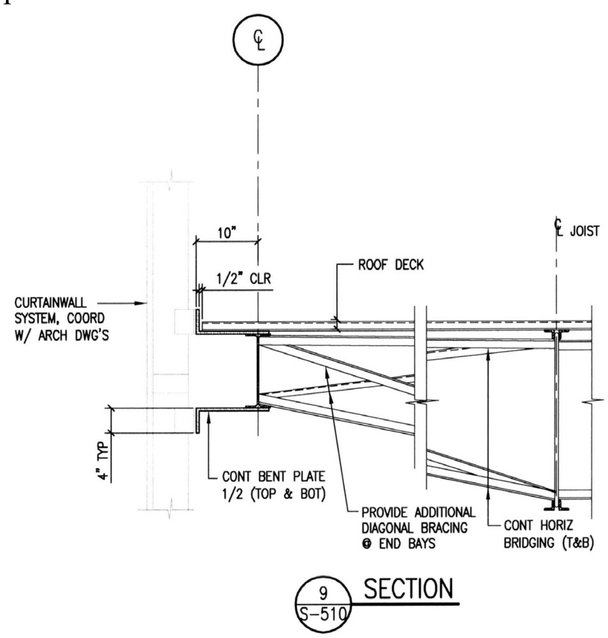


FIGURE 15

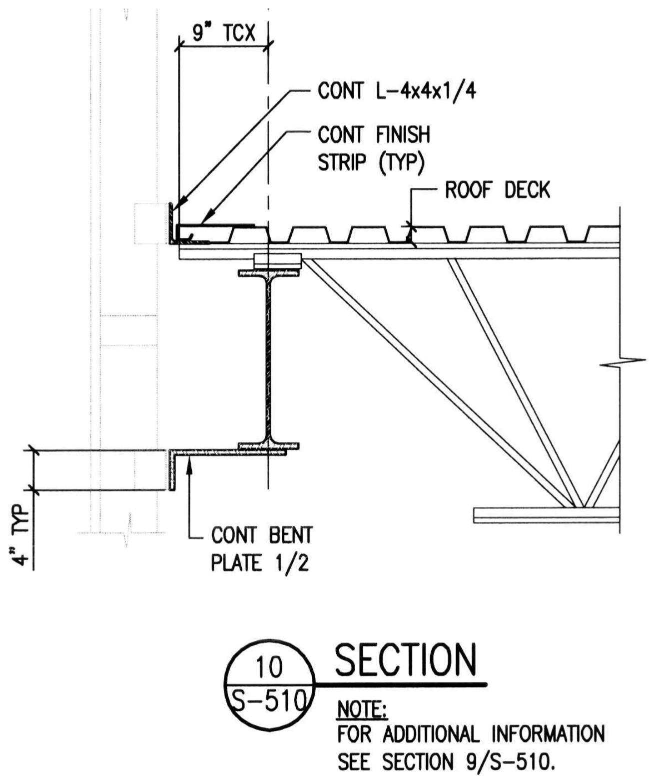


FIGURE 16

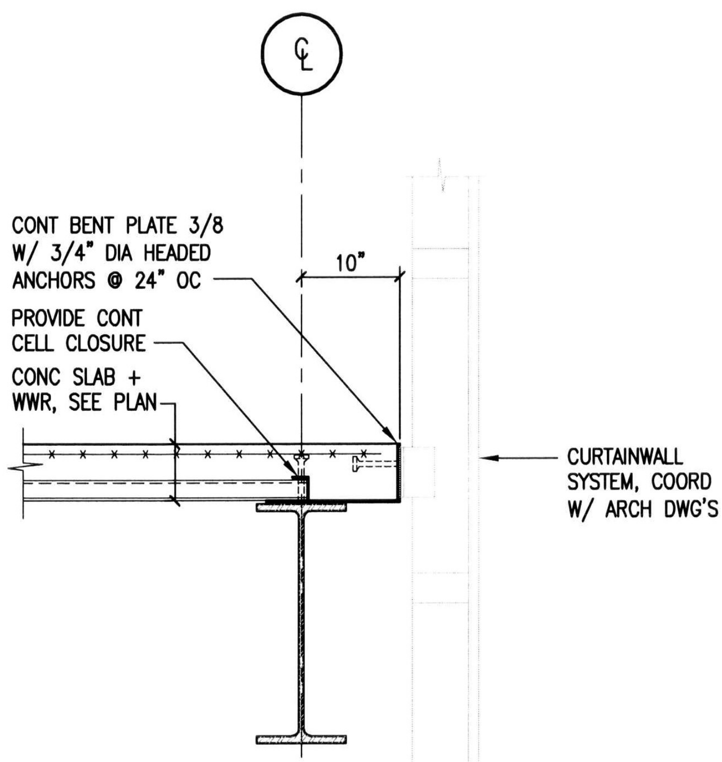


FIGURE 17

Girts can also sometimes be required for a curtainwall system. Figure 18 shows an example of a large horizontal trussed girt at the Philadelphia airport that is being used to laterally brace a large Curtainwall.

FIGURE 18

Figures 19, 20, 21 and 22 are examples a premanufactured precast system in which the panels are capable of spanning from column to column. As was the case with the premanufactured stone support system, the intermediate connections at the beams are vertically slotted so that only lateral loads can be transmitted from the panels to the adjacent framing while gravity loads (and portion of the lateral load) from the panels is supported directly from the main building columns. It should be noted that for both the stone panel examples shown above and the precast panel examples shown below that if the location of the connections to the column occur between the upper and lower framing levels the imposed mid-story loads and eccentricities must be accounted for in the design of the building columns.

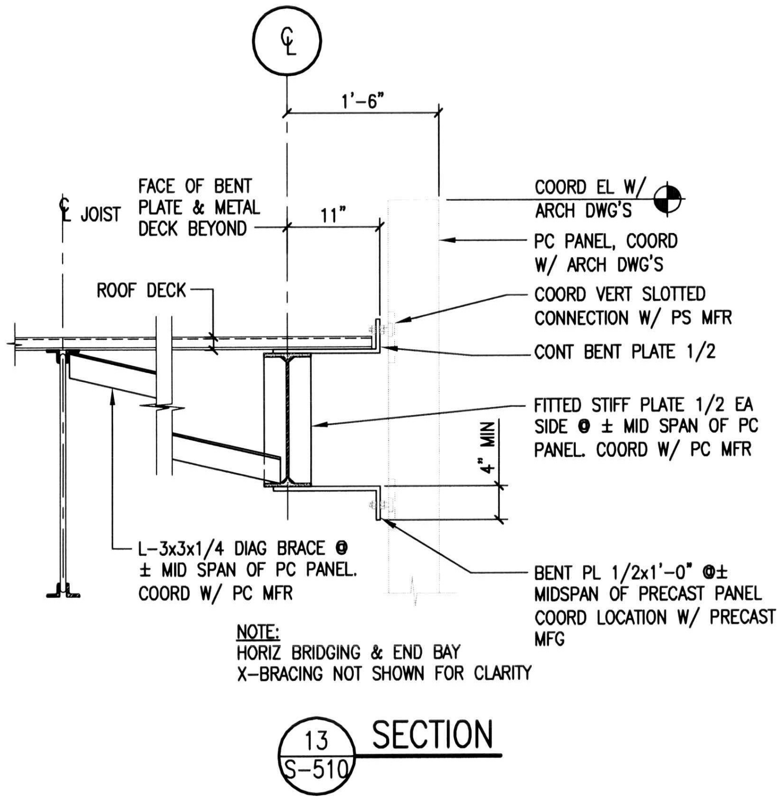


FIGURE 19

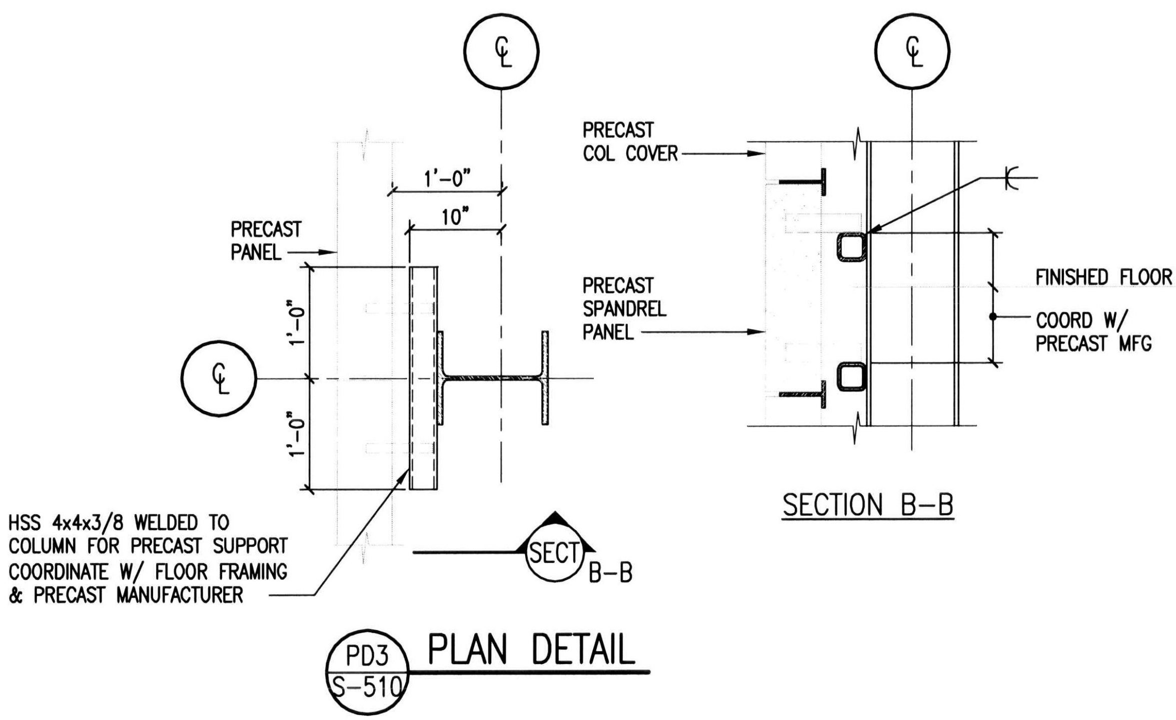


FIGURE 20

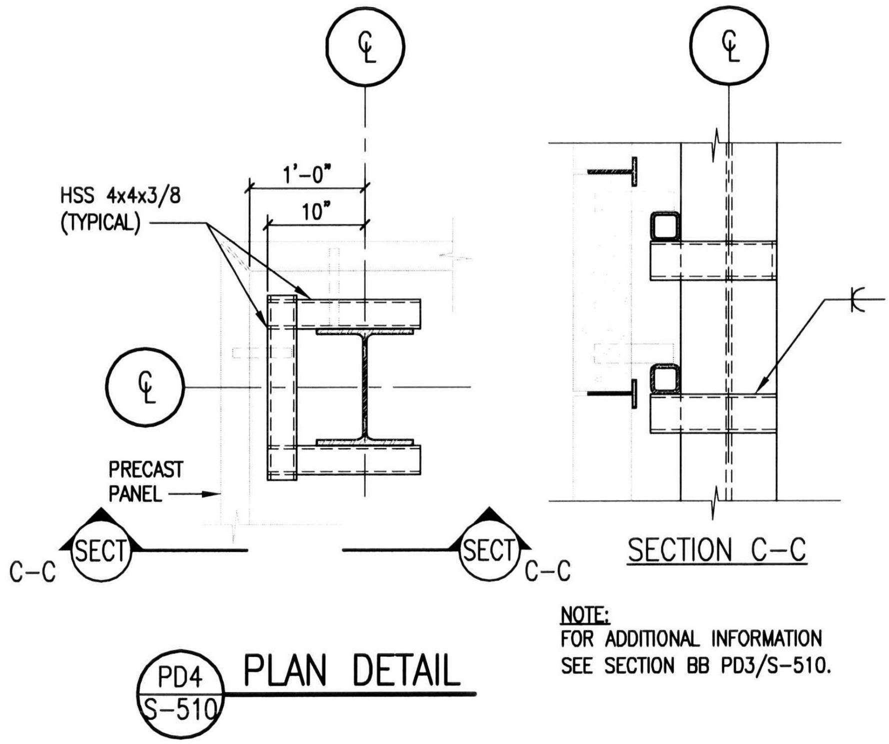


FIGURE 21

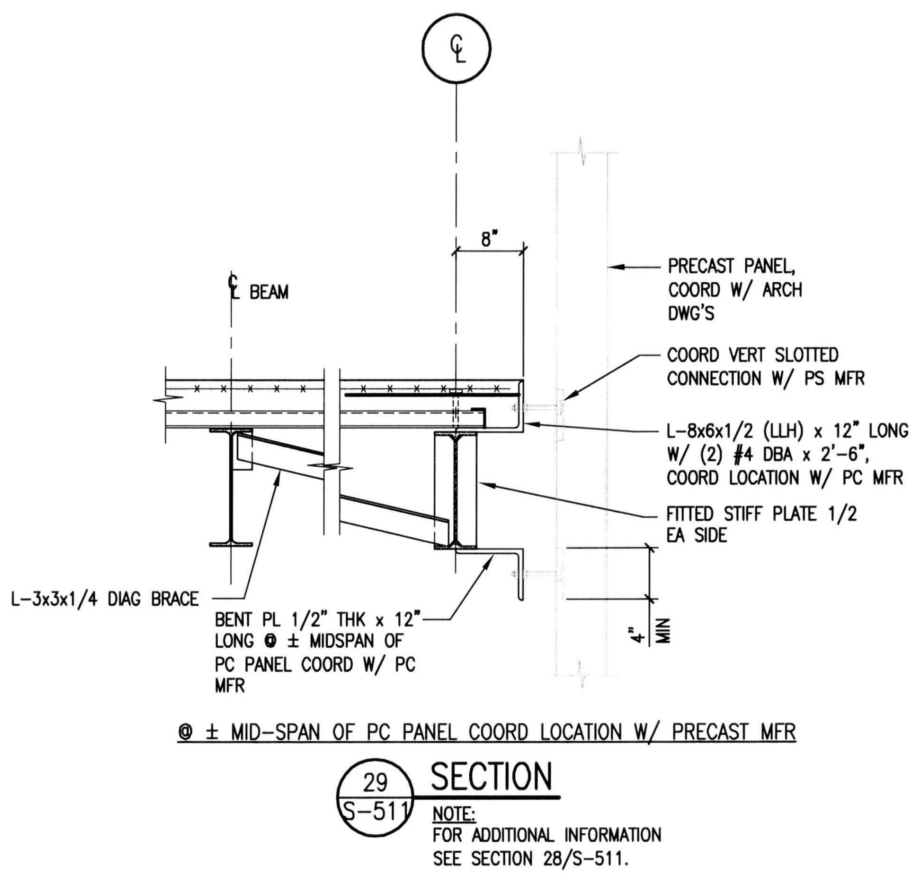


FIGURE 22

Examples of hung lintel systems are provided in Figures 23, 24, 25, 26 and 27. In figures 23 and 24 the HSS lintel is concentric to the supporting beam above. Never the less the torsion induced by the eccentric masonry veneer dead load, which is resisted by the HSS section, is resolved via the flexural resisting moment of the channel or angle hanger. At the same time the hanger sections must also resist the lateral load reaction from the HSS lintel via a resisting flexural cantilever moment at the intersection point of the kicker brace angle. The load path mechanism for Figure 25 is similar to Figures 23 and 24. Likewise the load path mechanism for the hanger and hung lintel in Figure 26 is also similar, however the eccentric gravity load reaction on the supporting beam at the top of the hanger must be resolved via the torsional resistance of the wide flange beam. Figure 27 shows the wind column connection at the end of the hung lintel from Figure 26.

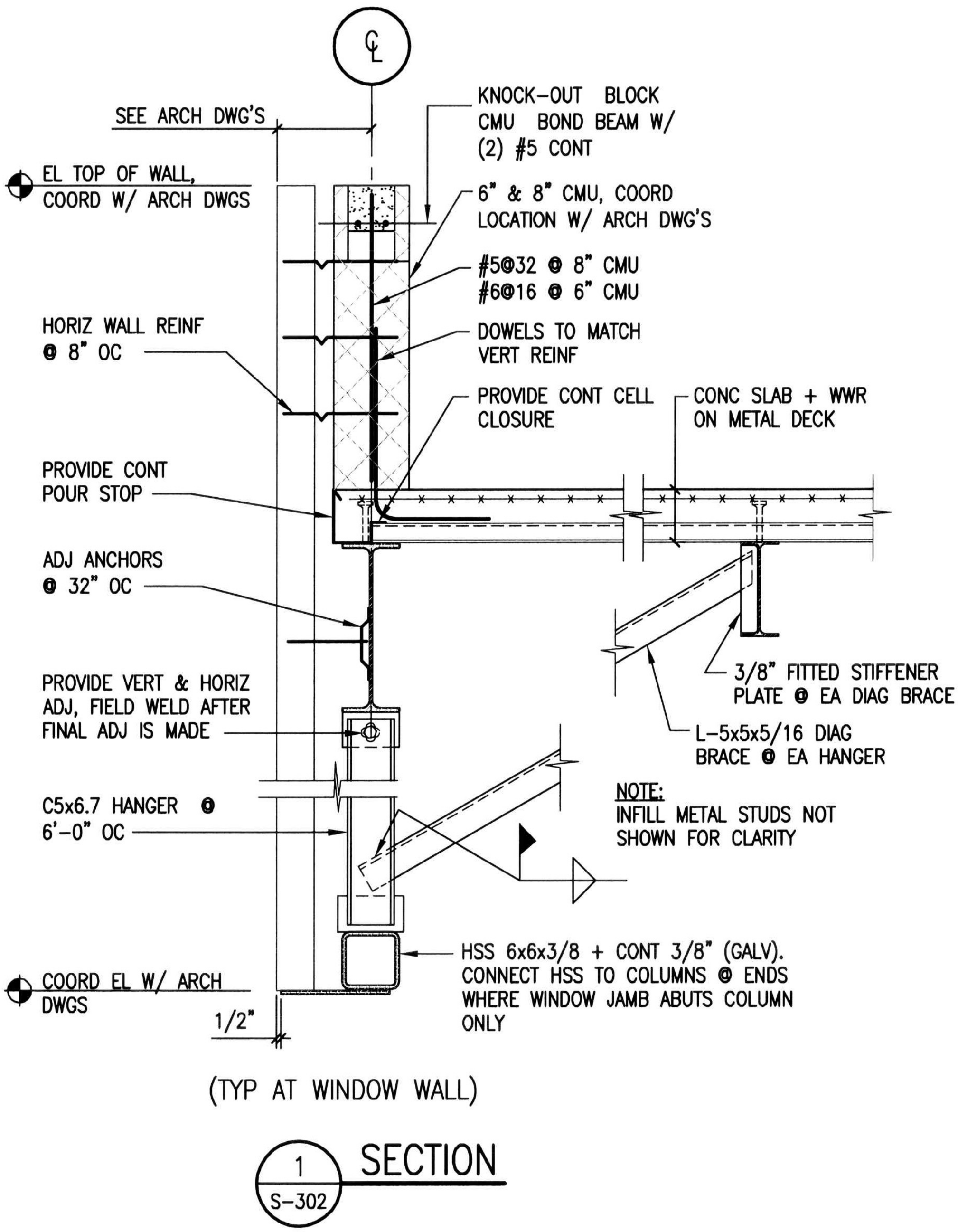


FIGURE 23

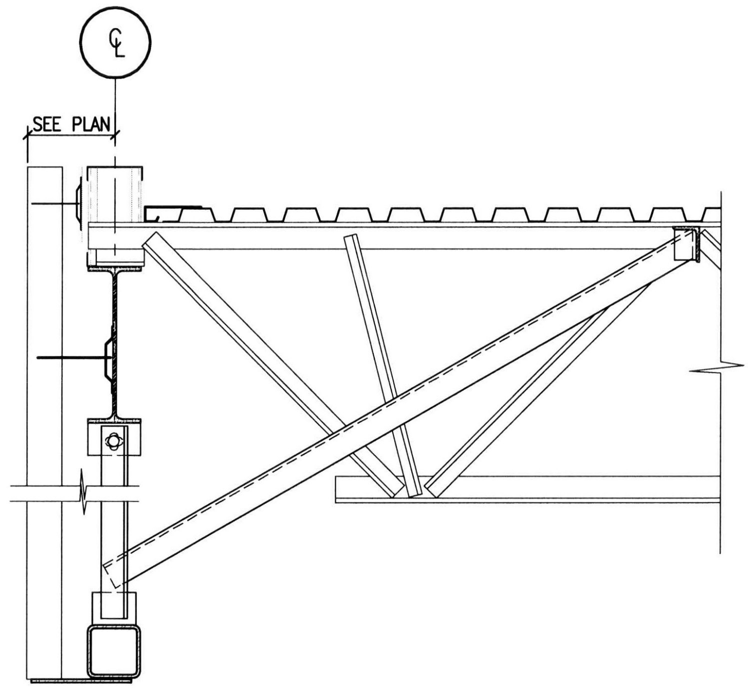


FIGURE 24

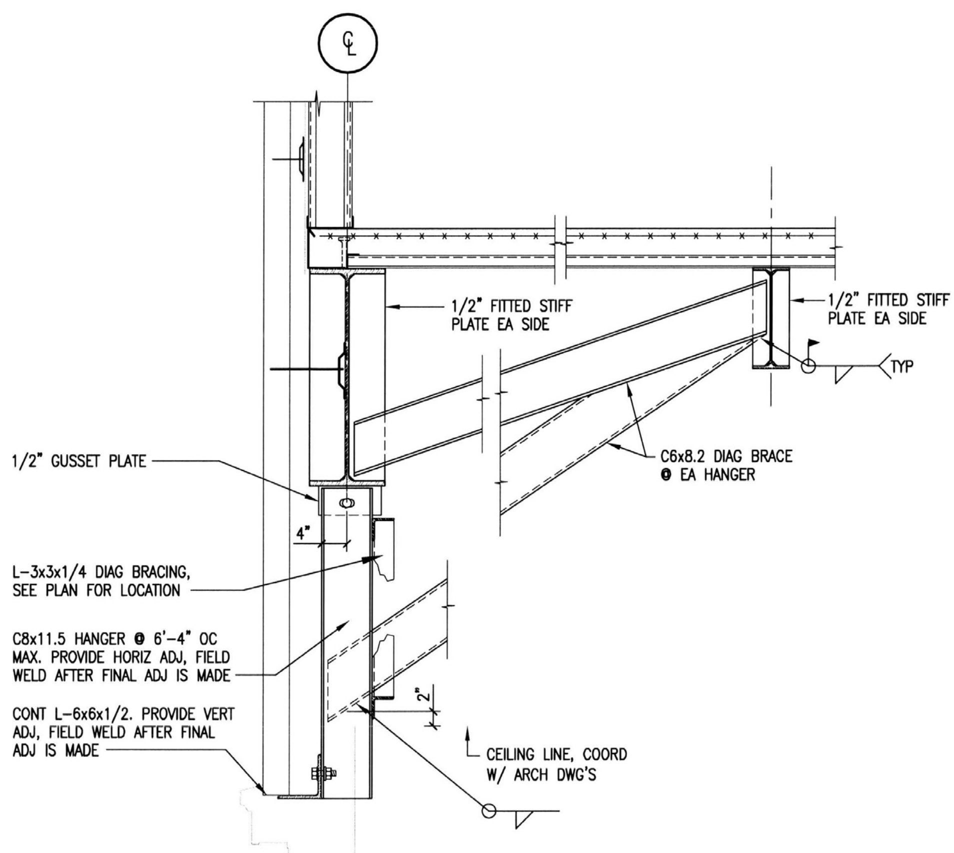


FIGURE 25

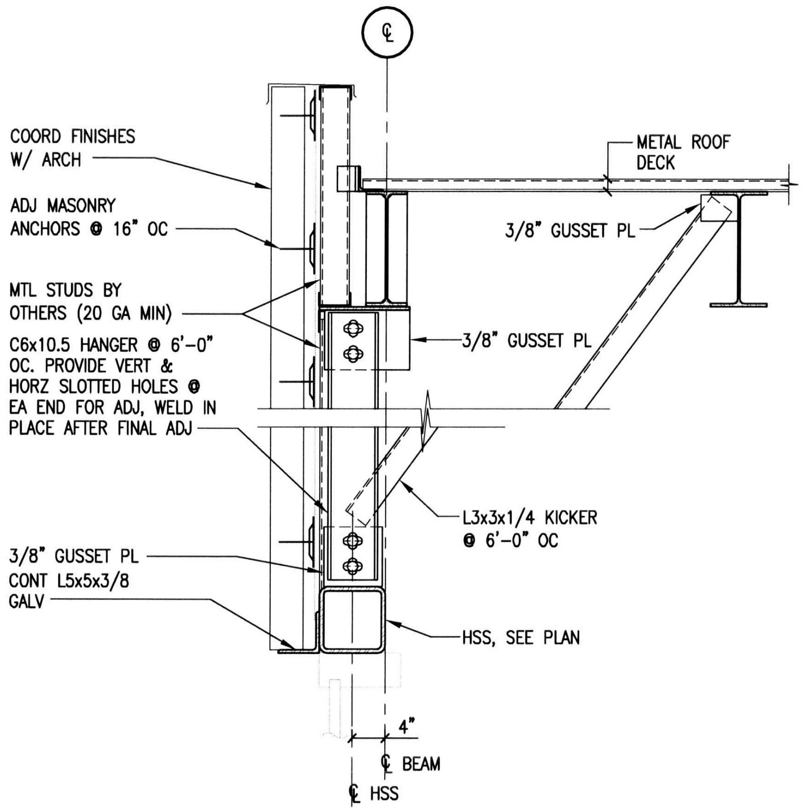


FIGURE 26

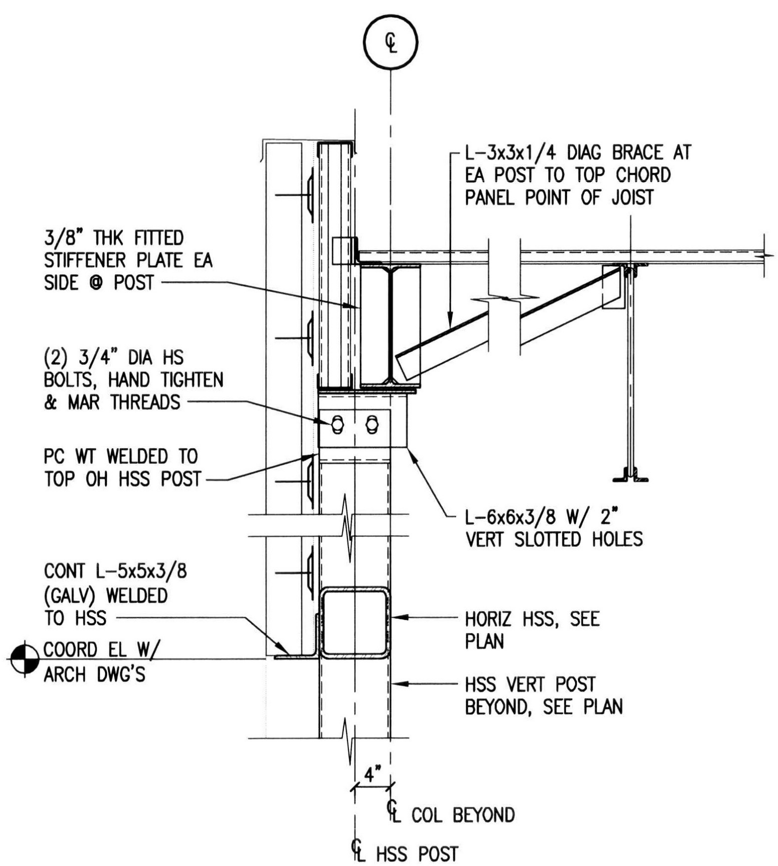


FIGURE 27

Another type of hung lintel is shown in Figure 28. This detail shows a proprietary system that is constructed integrally with the masonry and is often used with suspended faux masonry brick arches. This type of concealed system is manufactured by Halfen. Information concerning this system can be found through the following link; http://www.halfenusa.com/masonry01_lintel00.html.

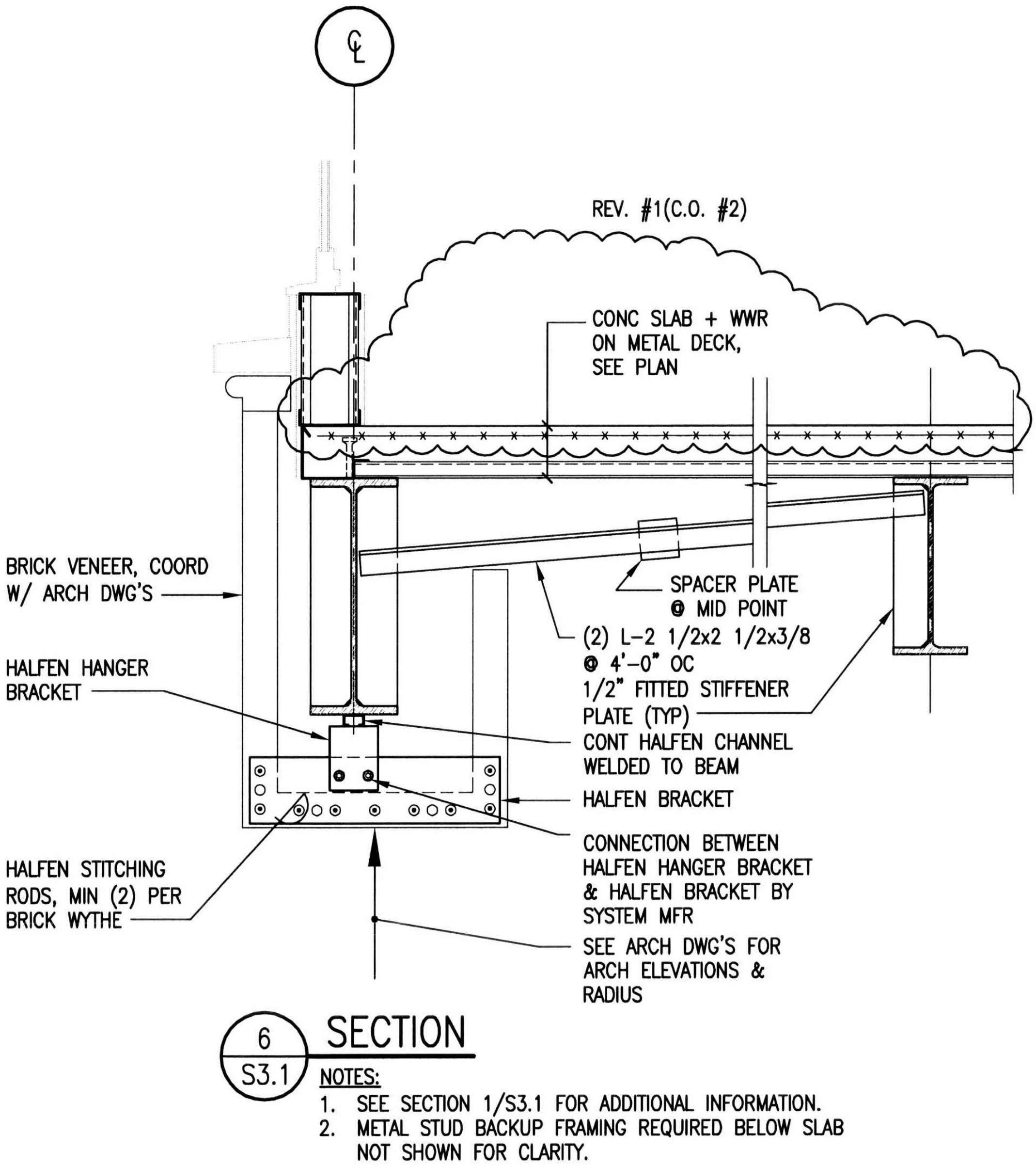


FIGURE 28

Steel is often used for masonry lintels, for both brick and CMU. Steel angles are used for brick lintels almost exclusively across the entire U.S., however, the use of steel as a CMU lintel appears to be more common in the northeastern U.S. than the rest of the country. Steel angles are typically used as self supporting brick lintels for openings up to 9 feet wide, however, beyond this width of opening the brick is supported from either continuous plates attached to a backup or hung lintel or shelf angles intermittently supported by a precast lintel in a CMU backup wall. In all cases, an exterior brick lintel should be galvanized to prevent long term deterioration. Load tables for laterally unsupported angles that can be used to size steel angle lintels can be found in a paper published in the AISC Engineering Journal, 1st Quarter, 1984. In addition Technical Note 31B (Revised) published by the Brick Industry Association provides additional material on the design of brick shelf angle lintels. A further discussion of the design of all types of lintels will be provided later in this lecture.

An example of a steel beam lintel in a CMU wall is shown in Figure 29 and 30. It should be noted that the steel beam is embedded in the wall above the top of the masonry opening in order to provide a measure of fire resistance for the steel member. The support of the masonry below the bottom of the beam is provided by a continuous plate supported from hanger plates connected to the bottom of the beam.

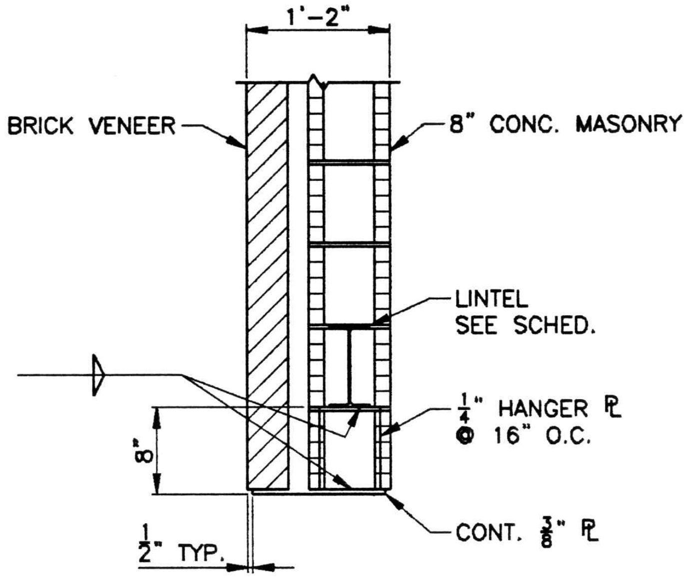


FIGURE 29

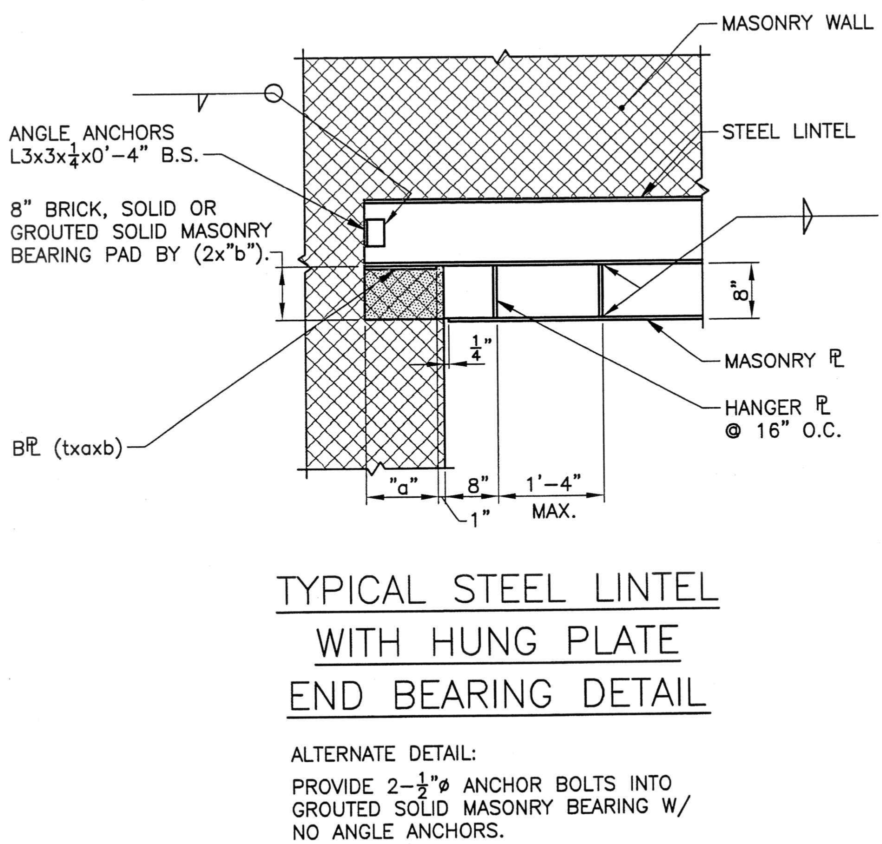


FIGURE 30

Steel beams (primarily channels) are also often used to support masonry (both brick and CMU) at an existing wall where it is necessary to construct a new opening. An example of this type of condition is provided in Figure 31. As indicated by the notes provided in this referenced detail, it is possible to install this type of lintel before the opening is actually created.

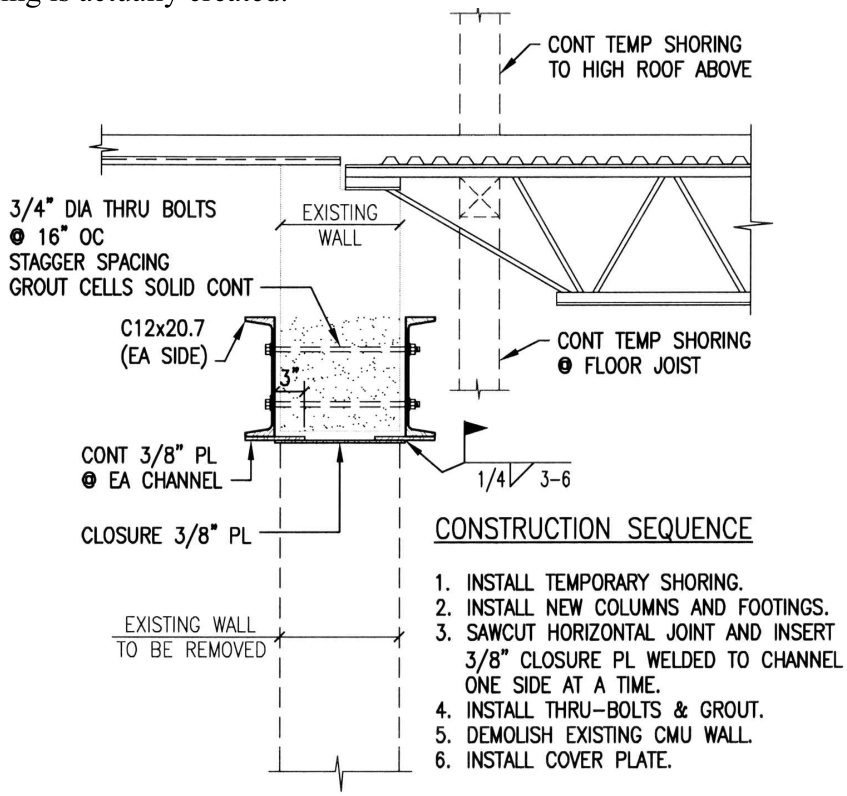


FIGURE 31

It is also possible to install new opening at existing masonry walls using precast or masonry bond beam lintels. An example of this type of condition is shown in Figure 32. Figure 33 (Section X cut at Figure 32) also shows how a new "concealed" steel angle lintel can be installed at a new opening in an existing brick veneer wall.

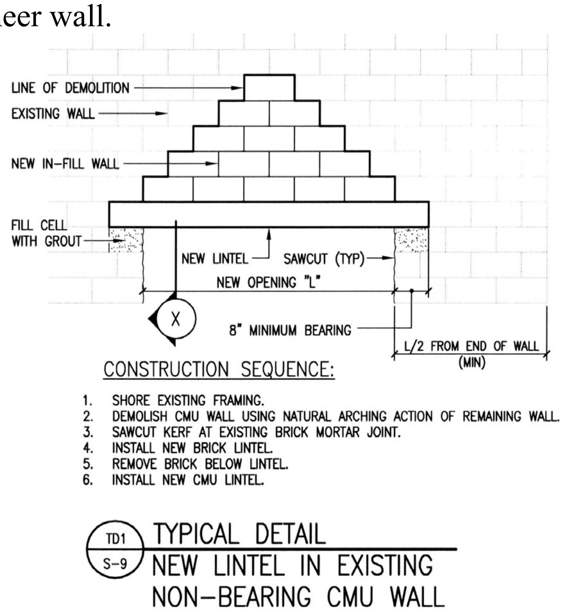


FIGURE 32

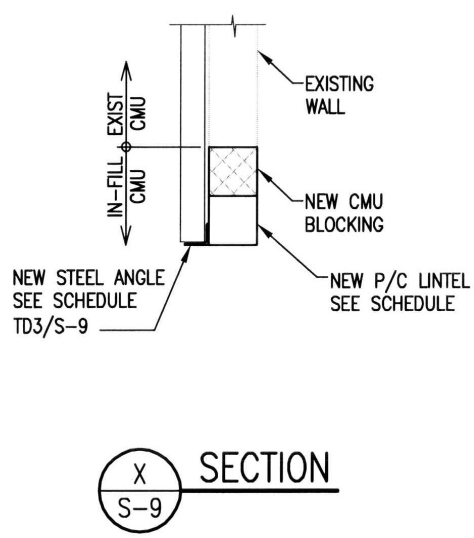
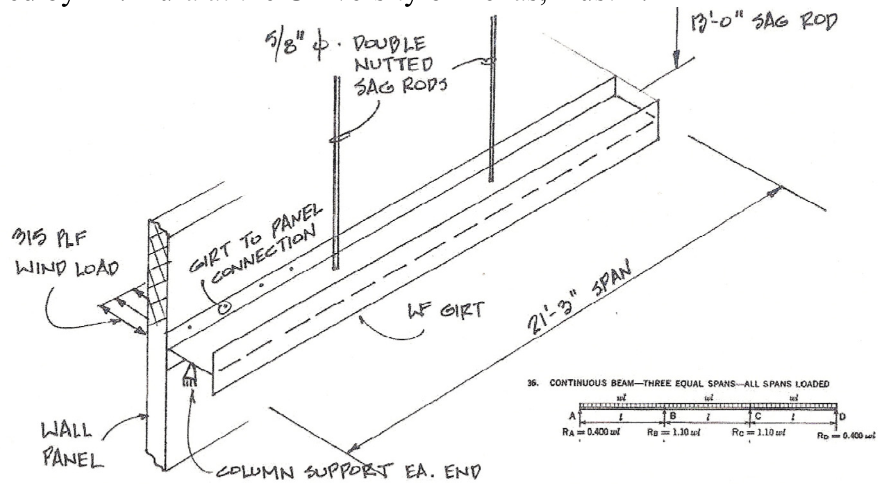


FIGURE 33

Girt Design:

Although there are exceptions with heavier cladding, for the most part girts (and wind columns) are designed to resist wind loads on the building exterior rather than lateral seismic forces. In my opinion, the magnitude of wind load (both pressure and suction) imposed on a girt should always be based on component and cladding criteria, even if the total tributary area of the girt allows for a decrease in the design pressures. Although just about any steel cross section can function as a girt, as indicated above it is typical for a wide flange to be laid on its side to allow the strong axis of the section to resist the horizontal forces. Channel sections are also sometimes laid on their side and used as girts. In either case, this situation results in condition whereby the weak axis of the member must resist all imposed dead loads (typically just the self weight of the member and not the cladding). To overcome this condition sag rods are used to support the beam at intermediate points across the horizontal span of the member.

Although the sag rods do not participate in the resistance to the imposed lateral wind loads they do serve the additional function of laterally bracing the compression flange of the member when the supporting wall is exposed to suction or negative wind forces. When the wall is exposed to positive wind pressures, the compression flange of the beam is typically braced by the attachments to the cladding. An example of the analysis of the bracing capabilities of sag rods as a part of the allowable stress design of a girt is provided below. The magnitude of bracing force imposed on the sag rod (i.e. 1% of the flange compression force) was established by Dr. Yura at the University of Texas, Austin.



CHECK: W8x13 GIRT $\quad M_{WIND} = (315)(21.25)^2/8 = 17.78^{KF}$

EXTERIOR COMPRESSION FLANGE BRACED BY WALL PANEL FOR WINDWARD

INTERIOR COMPRESSION FLANGE – CHECK FOR LEEWARD LOAD

$F_{COMP.\ FLANGE} = M/d = 17.78(12)/8 = 26.67^K$

EQUIVALENT LATERAL BUCKLING FORCE = $1\% \times F_{CF}$ = 267 #

RESISTING FORCE: $(13/2)(21.25/3)(1.10)\,2$ = 101 #

SAG ROD = $(.15\ KSI)(.3068)\,2$ = 92 # (F_a, A)

193 # < 267 # N.G.

TRY: W10x17 $\quad F_{CF} = 17.78(12)/10 = 21.34^K \times 1\%$ = 213 #

$F_R = (17/2)(7.083)(1.10)\,2 + 92$ = 224 # > 213 # ✓

5/8"φ SAG ROD: $Kl/r = 998.4 > C_c = 126 \quad F_a = 12\pi^2 E / 23(Kl/r)^2 = .15$ KSI

It should also be noted that it is necessary to double nut the sag rod to the girt web to assure that the rod can resist buckling of the beam in either direction (i.e. up or down). In addition, mobilization of the dead load of the beam (typically only one half of the uniform dead weight of beam is assumed to account for any support provided by the connections to the cladding) also assists to prevent any buckling of the beam in the upward direction. When multiple layers of girts are employed due to the height of the wall it is necessary to articulate the sag rods (as opposed to running the rods continuously through the upper girts down to the lower girts) to assure that both the upper and lower girts are properly braced by the rods, and to aid with the constructability of the vertically stacked girts.

As was shown in some of the example details above, HSS sections are also used as girts. In this situation, in which the member is symmetrical about both axis, the girt is designed to span the entire distance between the end supports for both lateral and gravity loads. For this scenario the girt must be checked for combined bi-axial bending. Biaxial bending stresses must also be checked for a sag rod supported wide flange or channel girt, however, because of the relatively insignificant minor axis bending moments, this is typically not a controlling factor. For a PEMB it is typical to use cold formed metal sections such as a Z-purlin laid on its side, and supported by sag rods, as a girt.

With the exception of masonry cladding (both brick and CMU), which is governed by ACI 530, the limiting deflection for any exterior cladding should be provided and established by the manufacturer of the system. This information is then used to establish the minimum section properties of the supporting girt and or wind column as it is not unusual for serviceability criteria to control over strength criteria when designing backup supports for building claddings.

Curtainwalls:

A curtainwall is an exterior cladding system which utilizes glass (either transparent, spandrel – i.e. opaque, or both) and integral horizontal and vertical structural mullions to transfer wind and gravity forces to the building structure. Curtainwall systems can weight anywhere from 8 to 15 PSF. There are two types of curtainwall systems, as described below. Non-curtain wall systems include strip (or ribbon) windows and punched window openings.

Stick System: A curtainwall in which the mullions are installed first then the glass panels are inserted into the mullion framing in the field. The vertical deflection criterion for this type of system is more stringent than that required for the other type of curtainwall system.

Unitized System: A curtainwall system in which the mullions are prefabricated with the glass panels already installed with the individual panels then later erected in the field. The advantages of a unitized system over a stick system include:

a. Field erection time is less.

b. Panels are more tolerant of vertical deflection as a result of the gasketed panel joints.

c. The gasketed panel joints allow the curtainwall to be formed out of a number of incremental expansion joints.

d. The joints between the panels allow for almost unlimited field adjustments during erection.

As with all cladding, the design of the support members for a curtainwall should be based on component and cladding pressures. Also, in addition to obtaining the limiting vertical and horizontal defection criteria from the manufacturer, the connections between the curtainwall and adjacent supporting structure should be detailed to prevent the imposition of any gravity loads or corresponding deflection from the building frame on to the curtainwall system. Therefore, the Contract Drawings should clearly indicate where the gravity load of the curtainwall is to be supported.

It is recommended that for full height curtainwalls on a building of 5 to 6 stories that the gravity load of the wall be supported at the lowest or foundation level with the connections at all of the upper framed levels providing lateral support only. On taller building and shorter building in which the curtainwall system starts at an upper floor, the gravity load support of the curtainwall should occur at regularly spaced intermediate floors or at the lowest level that the curtainwall begins, respectively.

In a tall building the joint between the bottom of the upper gravity supported curtainwall section and the top of the lower laterally connected wall should allow for the deflection of the framed support level so that no gravity loads will be imposed on the lower section. In addition, the joint should allow for any differential deflection between the lower and upper supported walls so that the horizontal expansion joint between the two different sections of walls can be properly maintained. Also, the manufacturer of the curtainwall system should account for any differential interstory building sway and thermal movement of the wall in the design of the system and the connections to the main building supports.

For a further discussion of curtainwalls see *Curtainwall Primer for Design Professionals*, Course S119.

Lintels:

Openings in masonry walls (both brick and CMU) are typically spanned by horizontal structural members known as lintels. The purpose of a lintel is to support the weight of the wall above the opening, it's self weight and any gravity loads imposed on the wall if it is loadbearing. In addition, in some cases the lintel must also resist any lateral loads imposed on the head of the opening as a result of any infill material such as glazing.

For masonry walls constructed in running bond (see Figure 34) it is common to take advantage of the natural arching action (see Figure 35) of the masonry wall above the opening when determining the magnitude of dead load that is to be supported by the lintel. In addition to the fact that masonry walls constructed in stack bond (see Figure 36) cannot exhibit arching action, the ability of a masonry wall to arch above an opening is also limited by a number of other factors. The two primary factors are:

1. The height of wall above the head of the opening. If the height of wall above the opening is insufficient to allow arching action to develop then the full load associated with the wall above the lintel must be supported.

2. The location of the opening relative to the end or corner of the wall. If there is insufficient in plane longitudinal stiffness in the wall to resist the horizontal thrust associated with the arching action then the wall cannot exhibit arching action above the opening and the full load associated with the wall above the lintel must be supported. Locating a vertical control joint in the wall adjacent to the opening can have a similar effect as well.

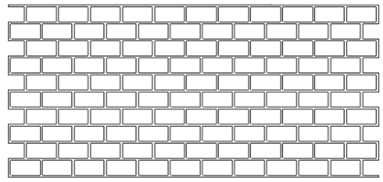


FIGURE 34

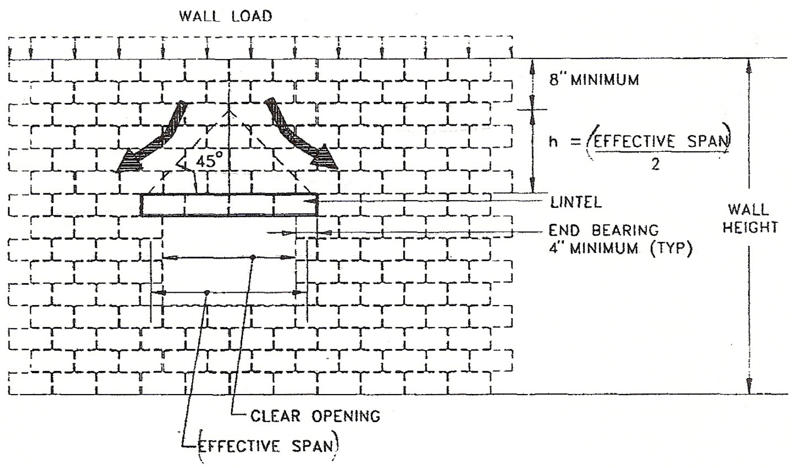


FIGURE 35

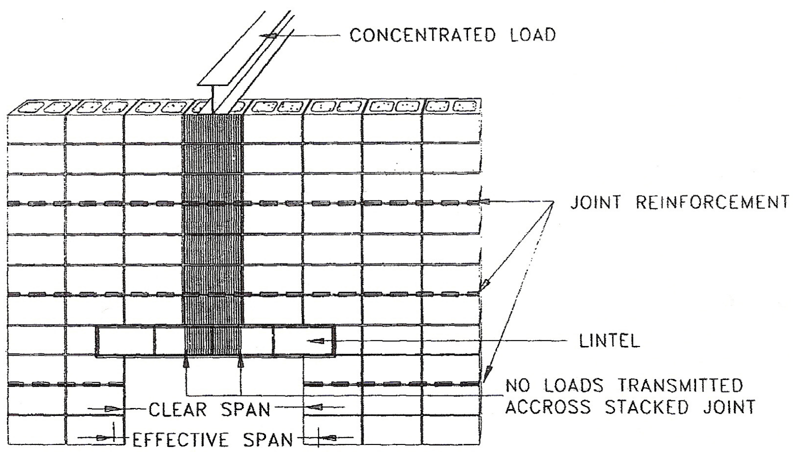


FIGURE 36

Another associated design concern with openings in masonry walls, particularly with exterior or loadbearing CMU walls, is the interruption of the vertical reinforcing in the wall. This must be accounted for by examining the impact of the redistributed forces into the vertical jamb elements of the wall on either side of the opening. Figure 37 provides an example of the supplemental reinforcing that is often required in such a scenario.

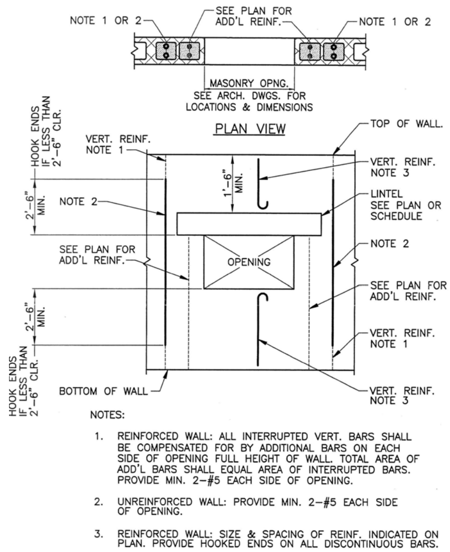


FIGURE 37

Finally, as indicated in previous lectures, the allowable deflection of a lintel is governed by ACI 530. The limiting deflection, as required to prevent cracking (particularly at the mortar joints), is L/600 or 0.30 inches maximum (Section 1.10.1; ACI 530-05). For a further discussion of curtainwalls see *The Design of Reinforced Masonry and Precast Concrete Lintels*, Course S126.