PDHonline Course S215 (5 PDH)

Steel Framing Systems

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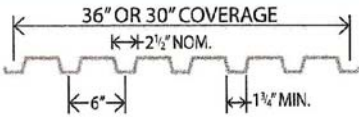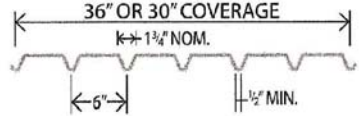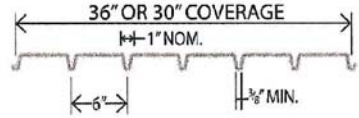www.PDHonline.com
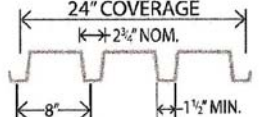
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Steel Framing Systems

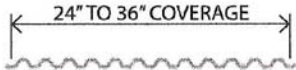### *D. Matthew Stuart, P.E., S.E., F.ASCE, SECB*
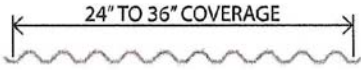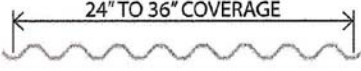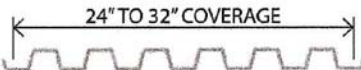
COURSE CONTENT

Roof and Floor Deck Systems Supported by Steel Framing

Metal floor and roof deck is used on almost every type of steel framed building. Metal deck is provided as either galvanized or painted, with galvanized deck recommended for most applications. Typical roof deck profiles include:

Roof Deck Profiles	Name	Nominal Thickness Range	Weight Range	Comments
36" OR 30" COVERAGE — 2½" NOM. — 6" — 1¾" MIN.	1 ½" X 6" Wide Rib (WR)	.03" to .06"	2 psf to 4 psf	May be referred to as "B" deck. Sidelaps may be flat adjustable or button punchable. Acoustical deck will have perforated webs.
36" OR 30" COVERAGE — 1¾" NOM. — 6" — ½" MIN.	1 ½" X 6" Intermediate Rib (IR)	.03" to .06"	2 psf to 4 psf	May be referred to as "F" deck.
36" OR 30" COVERAGE — 1" NOM. — 6" — ⅜" MIN.	1 ½" X 6" Narrow Rib (NR)	.03" to .06"	2 psf to 4 psf	May be referred to as "A" deck.
24" COVERAGE — 2¾" NOM. — 8" — 1½" MIN.	3" X 8" Deep Rib (DR)	.03" to .06"	2 psf to 4 psf	May be referred to as "N" deck. Sidelaps may be flat adjustable or button punchable. Acoustical deck will have perforated webs.

Roof decks are also available in 2 inch depths from certain manufacturers. Acoustical roof and floor (including composite) decks are also manufactured. This type of deck is normally used for exposed applications in which esthetic and noise reduction features are required. The following is a link to one manufacturer (Metal Dek Group) of acoustical decks; http://www.metaldek.com/.

Floor deck profiles come in three basic categories; form deck, composite deck and cellular deck. Form deck is generally used as a stay in place metal deck formwork for poured in placed framed concrete slabs in which the supporting beams are not designed as composite members. Form deck can also be used as roof deck for normal roofing substrates and poured in place regular weight and lightweight concrete applications. Typical form deck profiles include:

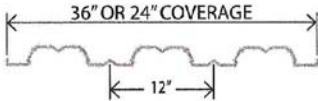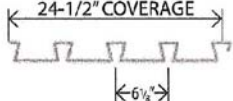| Form Deck Profiles	Name	Nominal Thickness Range	Weight Range	Comments
24" TO 36" COVERAGE	9/16" X Varies Form Deck	.014" to .030"	0.8 psf to 1.5 psf	Standard form deck. Used as centering.
24" TO 36" COVERAGE	15/16" X Varies Form Deck	.017" to .040"	1.0 psf to 2.0 psf	Heavy duty form deck. Used as centering
24" TO 36" COVERAGE	1-5/16" X Varies Form Deck	.017" to .047"	1.0 psf to 2.8 psf	Extra heavy duty form deck. Used as centering.
24" TO 32" COVERAGE	1-1/2" or 2" X Varies Form Deck	.023" to .047"	1.4 psf to 2.8 psf	Super duty form deck. Used as centering.

Note: All profiles may be used as roof deck (for a patented assembly)

Composite metal floor deck is decking in which deformations formed on the deck or the deck cross-sectional configuration (such as Epicore Deck manufactured by Epic Metals Corporation; http://www.epicmetals.com/) allows for the concrete slab (once it has achieved adequate strength) to act compositely with the metal deck. In other words the deck acts as the bottom tensile flexural reinforcement for the composite concrete slab section.

The following requirements for composite deck can be found in the 13th Edition of AISC, Section I3.2c:

1. Maximum rib height (h_r) = 3 inches.

2. Minimum average width of rib (w_r) = 2 inches (however the value used in composite beam calculations shall not exceed the clear width at the top of the deck).

3. For purposes of uplift resistance, the deck must be attached to the beam flange at intervals no greater than 18 inches on center by either studs or puddle welds.

Typical composite deck profiles include:

Composite Floor Deck Profiles	Name	Nominal Thickness Range	Weight Range	Comments
36" OR 24" COVERAGE (12")	1 ½"X 12" 2"X 12" 3"X 12" Composite	.03" to .06"	2 psf to 4 psf	Embossment patterns will vary from manufacturer to manufacturer. Side laps are flat adjustable or button punchable.
24-1/2" COVERAGE (6⅛")	2"X 6" Composite	.03" to .06"	2 psf to 4 psf	
36" OR 30" COVERAGE (6")	1 ½"X 6" Composite	.03" to .06"	2 psf to 4 psf	Embossment patterns will vary from manufacturer to manufacturer. Side laps are flat adjustable or button punchable.
24" COVERAGE (8")	3"X 8" Composite	.03" to .06"	2 psf to 4 psf	Embossment patterns will vary from manufacturer to manufacturer. Side laps are flat adjustable or button punchable. This profile is not generally suitable for use with shear studs.

Cellular metal floor deck is composite decking in which linear voids are formed beneath the upper flutes of the deck via an integral continuous gage metal soffit plate as required for use as electrical, telephone, cable and or computer data raceways, and in some cases for esthetic and noise reduction requirements. Caution should be observed when using cellular or electrified decks (such as Walkerdeck manufactured by Cordeck; http://www.cordeck.com/products/electrified_deck/Walkerdeck-CutSheet.pdf) because the trench ducts that are sometimes required within the deck result in full depth voids in the concrete deck/slab cross-section thereby eliminating the possibility of designing the support beams impacted by these same in slab utility chases as composite beam/slab sections. Typical cellular deck profiles include:

Cellular Floor Deck Profiles	Name	Nominal Thickness Range	Weight Range	Comments
24" OR 36" COVERAGE (12")	3" X 12" Composite Cellular	.03" to .06"	4 psf to 7 psf	Bottom plate may be perforated for acoustical.
24" OR 36" COVERAGE (12")	2" X 12" Composite Cellular	.03" to .06"	4 psf to 7 psf	Bottom plate may be perforated for acoustical.
24" OR 36" COVERAGE (6")	1 ½" X 6" Composite Cellular	.03" to .06"	4 psf to 7 psf	May also be used as a roof deck. Bottom plate may be perforated for acoustical.
24" COVERAGE (8")	3" X 8" Composite Cellular	.03" to .06"	4 psf to 7 psf	May also be used as a roof deck. Bottom plate may be perforated for acoustical.

Because of the restraint caused by metal decking and headed-stud shear connectors, dry shrinkage and thermal contraction of the concrete can cause cracking, just as restraint from the subgrade base can cause cracking in slabs-on-grade. Sawn contraction joints that help control random plastic shrinkage and restraint cracking in slabs-on-grades, however, aren't as likely to be helpful in suspended slabs. For elevated slabs, flexural and shrinkage/temperature reinforcement control cracking while at the same time tying the slab together to assure that it behaves as assumed by the structural design. Because this reinforcement can restrict movement at a control joint, reducing the slab cross section by sawing a joint isn't likely to force the crack to appear at the joint. This condition makes random cracks in framed slabs likely to occur even if sawn joints are provided. Therefore it is recommended for all framed slabs that no contraction joints and as few construction joints as possible be used. As a result of this approach, random cracks will form, however, any cracks wide enough to affect floor finish serviceability can be easily patched and repaired.

Slab cracking in a composite beam and slab system also has a tendency to occur directly over the main girders as the supported bays on either side deflect causing tensile stresses in the concrete perpendicular to the direction of the slab and deck span. In order to help control this type of cracking it is recommended that additional slab reinforcement perpendicular to the girder span be placed in the slab. For a typical combined 6 inch deep deck and concrete slab, #5 bars at 18 inches on center are recommended. The bars should extend beyond the girder, on each side, at least one third of the supported bay span.

Most of the manufacturers of metal decks are members of the Steel Deck Institute (SDI). Information concerning this industry organization and sources of technical material can be found at; http://www.sdi.org/. All of the manufacturers provide technical publications for their own products as well. These publications typically include product manufacturing and installation specifications, material and section properties of the deck, load tables and reinforcing required for both formed and composite slab construction. It should be noted that the load and reinforcing tables are generally categorized into single span, double span and triple span (or more) applications to reflect the advantages of continuous, multi-span installations of the deck over simple single or double span conditions. In addition, for formed deck, it is also possible to provide temporary shoring of this stay in place formwork material if the slab to be cast on top of the deck (after it has reached adequate strength) is capable of spanning further than the metal deck. However, due to the additional time and expense involved with this approach it is seldom if ever used.

Most metal deck manufacturers also provide technical information on their products, particularly for roof deck, for support and sidelap attachment patterns using various methods (self-tapping screws, power-actuated fasteners and or welds) of achieving in plane shear resistance in order to resist imposed diaphragm loads and to provide uplift resistance (due to wind loads). An excellent example of the all of the above technical information is United Steel Deck's website at; http://www.njb-united.com/.

Metal deck roofs are considered as flexible diaphragms, therefore the distribution of the diaphragm forces to the resisting elements (i.e. vertical bracing, shearwall, moment frames, etc.) is based on tributary area. See *Flexible Metal Deck Roof Diaphragms*, Course S184 for a further discussion of this topic. Formed or composite metal deck slabs are considered as rigid diaphragms, therefore the distribution of the lateral forces to the superstructure resisting elements is based on the center of mass of the slab, the center of rigidity of the resisting elements and the location of the applied loads.

One type of metal deck that is not capable of functioning as a diaphragm is standing seam metal roofing. Although this product is typically only encountered with pre-engineered metal buildings (PEMB), it is often specified by architects for all types of sloped roof steel buildings. If a standing seam roof is required then it is necessary to support the material on either a conventional fluted metal deck or install horizontal diagonally

bracing beneath the plane of the roofing in order to provide a diaphragm capable of resisting the applied lateral loads.

Another type of acoustic deck commonly used in buildings with exposed structures is Tectum. Tectum consists of wood fibers bonded together with hydraulic cement to form tongue and grove panels of various thicknesses and widths. Tectum is also available as sandwich panels with a layer of either Styrofoam or expanded polystyrene insulation topped with a thin layer of oriented strand board (OSB). Tectum is often used in facilities such as gymnasiums and is typically supported by open web steel joists. Tectum should never be used in high moisture environments such as a natatorium. Tectum, when properly attached per the manufacturer's recommendations, is capable of behaving as a flexible wood deck diaphragm. The following link will provide access to the manufacturer's website and the available material and section properties, load/span tables and diaphragm shear capacity tables; http://www.tectum.com/.

Another wood roof deck system that is commonly used with open web steel joists in the western U.S. involves the combination of panelized (i.e. prefabricated) wood deck and sub-purlin components connected to open web steel joists. The entire panelized unit (wood deck, sub-purlins and steel joist) is assembled on the ground and then lifted into position at the roof, where the steel joists are welded or bolted to the primary steel beams or joist girder supports. The free edge of the wood decking for each panelized unit is nailed to the framed joist edge of the previously placed unit. Steel joists used in this hybrid panelized system are typically placed eight feet on center. Because of the longspan capabilities of steel joists, the panelized units can be prefabricated in lengths up to 72 feet (see Figure 1). The following link will provide access to a panelized structure contractor's website; http://www.panelized.com/

FIGURE 1

The remaining type of floor and roof deck system used predominately with steel framed structures is precast, prestressed hollowcore slabs or planks. This product comes in a number of different thicknesses and widths and is typically manufactured under a number of different proprietary licenses using both wet cast and dry cast extruded systems. The many different types of products available are listed for the most part in the Prestressed/Precast Concrete Institute's (PCI) Design Handbook, however, it is recommended that information be obtained directly from the local manufacturers that will likely be providing the products for

the specific project in the specific location or region. Hollowcore plank can be designed and constructed with or without field cast composite concrete topping slabs. If no composite topping is required it is common to place a flowable, self-leveling topping on the planks to help facilitate the installation of the required floor finishes in the building. Precast hollowcore plank is also manufactured for use as wall panels.

Most manufacturers provide technical data (including load and span capacity tables) concerning all of their precast products on line. The following link will provide access to one local manufacturer's website (Nitterhouse Concrete Products, Inc.) and the available on line technical material;
http://www.nitterhouse.com/DrawingSpecs/DrawingsSpecs.html

Precast hollowcore plank is a pre-engineered, pre-manufactured product. As such, the Structural Engineer of Record (SOR) only needs to provide a performance specification to the manufacturer (along with framing plans, typical support sections and general notes) as a part of the Contract Documents and is not responsible for the actual design of the component pieces. Instead signed and sealed shop drawings and calculations are provided by the manufacturer's engineer for review and approval of the SOR. It is necessary, however, for the SOR to have a clear understanding of the specified hollowcore system in order to adequately design both the gravity support beams and walls that support the planks, and the lateral resisting system for the superstructure. In the case of the later, it is common for the SOR to specify the minimum in plane shear capacity of the hollowcore plank joints as required to satisfy the floor and roof diaphragm loads.

Although it is more common to analyze hollowcore planks (including slabs with composite toppings) as flexible diaphragms, some manufacturers assume that their precast concrete floor systems behave as a rigid diaphragm. In either case it is recommended that the SOR coordinate his lateral load design assumptions with the manufacturer to assure that the behavior of the product provided coincides with the assumptions made for the distribution of the lateral loads to the main lateral resisting elements of the building.

Typical details associated with a steel beam and concrete masonry unit (CMU) wall supported hollowcore system are provided below: It should be noted that when the hollowcore plank is supported on both ends from beam supports noted in Figures 2 and 3, special erection procedures are required because the plank has to be diagonally positioned in plan before it can be rotated into the bearing pocket formed by the steel beam. It should also be noted that Figure 7 shows the correct method of construction when the plank spans parallel to a wall in order to prevent problems created between differences in the plank elevation resulting from the camber when compared to the fixed horizontal position of the CMU coursing.

FIGURE 2

FIGURE 3

FIGURE 4

#4@32 2'-0" 2'-0"
GROUT SOLID INTO WALL
AND PLANK CORE KEY

5" MIN BRG

PROVIDE KNOCKOUTS IN PCC
PLANK @ 16" OC FOR VERT
WALL REINF PLACEMENT

CONCRETE
TOPPING

8" PCC PLANK-SEE PLAN

T/PLANK
SEE PLAN

STAIR/ELEV
SHAFT

CONT BEARING STRIP
PER GENERAL NOTES

8" KNOCK-OUT BLOCK
CMU BOND BEAM W/
(2) #5 CONT (TYP)

8" CMU

SEE FOUNDATION SECTIONS
FOR VERT WALL REINF

FIGURE 5

REINF CONC TOPPING W/
6x6-W2.0xW2.0 WWR x 4'-0"
WIDE CENTER OVER WALL

CONCRETE
TOPPING

3" MIN
BEARING

8" PCC PLANK-SEE PLAN

T/PLANK
SEE PLAN

#4@32 2'-0" 2'-0"
GROUT SOLID INTO WALL
AND PLANK CORE KEY
(ALTERNATE)

8" CMU

CONT BEARING STRIP
PER GENERAL NOTES

8" KNOCK-OUT BLOCK
CMU BOND BEAM W/
(2) #5 CONT (TYP)

SEE FOUNDATION SECTIONS
FOR VERT WALL REINF

FIGURE 6

FIGURE 7

A proprietary system that also involves hollowcore plank supported on steel beams is the Girder-Slab system. This system uses castellated inverted T steel beams that are placed in the plane of the hollowcore slabs to support the same (see Figure 8). The inverted T beam is grouted into the end joints of the adjacent precast plank allowing for composite action of the beam with the slab system. The foll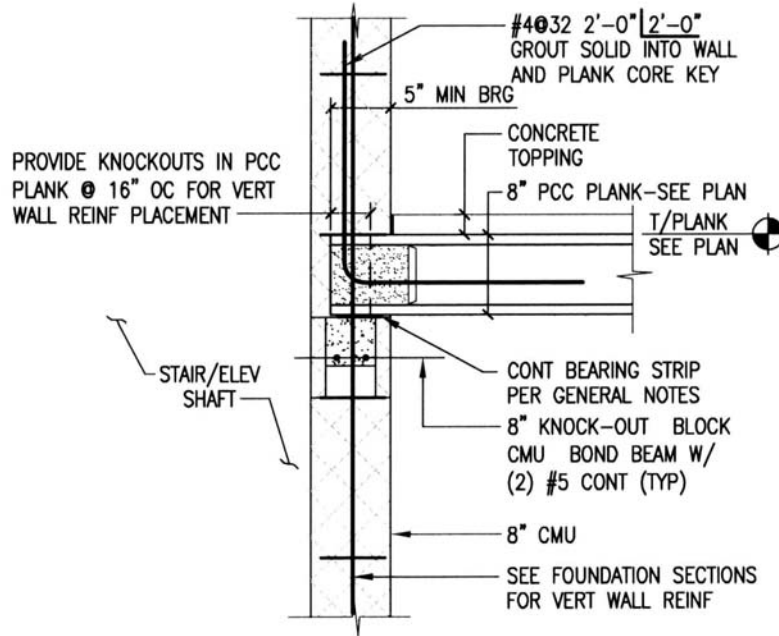owing link will provide access to the manufacturer's website and the available on line technical material; http://www.girder-slab.com/homepage.asp.

FIGURE 8

Open Web Steel Joists and Joist Girders

Since the Steel Joist Institute (SJI; http://www.steeljoist.org/) adopted the first standard specification and first load table in 1928 and 1929, respectively, structural engineers have been able to specify standard joist

designations rather than design each joist and joist component as if were a steel truss. The current SJI Standard Specification Load Tables and Weight Tables for Steel Joists and Joist Girders contains three standard specifications for three distinct series of members; K-Series, LH and DLH-Series, and Joist Girders. The following link provides access to a manufacturer's (Vulcraft) website and the available on line technical material for their joist and joist girder products; http://www.vulcraft.com/downloads.htm.

<center>K-Series:</center>

Open web steel K-Series joists are defined as simply supported, simple span uniformly loaded trusses that can support a floor or roof deck. The load tables for the K-Series assumes that the top chord of the joist is braced adequately by the deck to prevent lateral buckling. The K-Series ranges in depth from 8 to 30 inches with a maximum span of up to 60 feet. The standard bearing seat depth is 2½ inches. The ends of K-Series joists must extend at least 2½ inches over a steel support.

The standard load tables found in most manufacturer's catalogs provide the total safe uniformly distributed load-carrying capacities (PLF) for both LRFD and ASD loads. The weight of all dead loads, including the joist, must be deducted to determine the live load carrying capacity of the joist. This is because, in addition to the total safe load-carrying capacity listed, load-carrying capacities are also listed separately for live loads (PLF) which will produce an approximate deflection of L/360 of the span. Live loads that will produce a deflection of L/240 of the span may be obtained by multiplying the live load capacity figures listed for an approximate deflection of L/360 by 1.5. In no case, however, shall the total load-carrying capacity of the joist be exceeded.

The approximate joist dead load (PLF) shown in the manufacturer's tables do not include accessories. In addition, it should be noted that the maximum uniform load-carrying capacity for K-Series joists is 550 PLF. For sloped parallel-chord joist it is sufficient to use an equivalent span that is equal to the length along the slope of the joist. Most manufacturers also include a K-Series economy table so that the lightest joist for a given load can easily be selected.

The approximate moment of inertia (in^4) of a K-Series joist can be calculated from the load tables as;

$$Ij = 26.767(W_{LL})(L^3)(10^{-6})$$

<center>Where; W_{LL} = Live load listed that will produce an approximate deflection of L/360</center>

<center>L = (Span - 0.33) in feet.</center>

It should be recognized that because open web joists (or any truss member for that matter) exhibit some degree of shear distortion (i.e. elongation or compression of the diagonal web members) in addition to flexural bending. Therefore the calculated gross inertia of the section (Ij) should be adjusted to an effective inertia (Ieff) such that Ieff = Ij/1.15.

The KCS joist is a K-Series joist that was developed to allow a joist to support uniform loads, concentrated loads and other non-uniform loads that exceed the shear and moment capacity of a standard K-Series joist. This is possible because the KCS joist chords are designed for a constant moment capacity across the entire span of the joist. Also, all of the webs of the joists are designed for a vertical shear capacity equal to the maximum end shear capacity. In addition, all webs (except the first web from the end supports, which remains in tension under all simple span gravity loads) are designed for 100 % stress reversal. This allows

for the webs to support the maximum shear capacity as either a tension or compression load as dictated by the location of the moment and shear envelope for a given loading. To select the appropriate KCS joist from the KCS joist load tables all that is required is the maximum moment and maximum shear for the given loading. If a KCS joist with an adequate capacity cannot be selected, or if any uniform load exceeds 550 PLF, or if any concentrated loads exceeds the shear capacity of the joist, it is recommended that two side by side KCS joists be used, or a LH-Series joist can be selected.

LH and DLH-Series Joists:

Long Span and Deep Long Span steel joists (LH and DLH-Series joists, respectively) are simply supported, simple span uniformly loaded trusses. LH-Series joists may support a floor or a roof deck. DLH-Series joists are intended to support roof loadings. The load tables for both LH and DLH-Series joists assume that the top chord of the joists are braced adequately by the deck to prevent lateral buckling. LH-Series joist are available in depths of 18 to 48 inches with a maximum span of 96 feet and a maximum uniform load-carrying capacity of up to 1000 PLF. DLH-Series joist are available in depths of 52 to 72 inches with a maximum span of 144 feet and a maximum load-carrying capacity of 700 PLF.

The standard bearing seat depth for LH and DLH-Series joist is 5 inches, although 7½ inches deep seat is sometimes required for the larger joist designations. The ends of LH and DLH-Series joists must extend a distance of no less than 4 inches over a steel support.

As with the K-Series joist, the standard load tables found in most manufacturer's catalogs for LH and DLH-Series joist provide the total safe uniformly distributed load-carrying capacities (PLF) for both LRFD and ASD loads. The weight of all dead loads, including the joist, must be deducted to determine the live load carrying capacity of the joist. This is because, in addition to the total safe load-carrying capacity listed, load-carrying capacities are also listed separately for live loads (PLF) which will produce an approximate deflection of L/360 of the span. Live loads that will produce a deflection of L/240 of the span may be obtained by multiplying the live load capacity figures listed for an approximate deflection of L/360 by 1.5. In no case, however, shall the total load-carrying capacity of the joist be exceeded.

The LH and DLH-Series load tables apply to joists with either parallel chords or standard pitched top chords. When top chords are pitched, the load-carrying capacities are determined by the nominal depth of the joists at the center of the span. Standard top chord pitch is 1/8 inch per foot. If the actual required pitch exceeds this standard, the load table typically does not apply. As most building codes now require a minimum roof slope of ¼ inch per foot, the SOR should account for this condition when using the available load tables. The approximate moment of inertia (in⁴) of a LH and DLH-Series joists can be calculated from the load tables as;

$$Ij = 26.767(W_{LL})(L^3)(10^{-6})$$

Where;

WLL = Live load listed that will produce an approximate deflection of L/360

L = (Span - 0.67) in feet.

It should be recognized that because open web joists (or any truss member for that matter) exhibit some degree of shear distortion (i.e. elongation or compression of the diagonal web members) in addition to

flexural bending. Therefore the calculated gross inertia of the section (Ij) should be adjusted to an effective inertia (Ieff) such that Ieff = Ij/1.15.

Joist Substitutes:

Joist substitutes are 2½ inch deep sections intended for use in very short spans (less than 8 feet) where open web steel joists are impractical. They are commonly specified to span over hallways and short spans in skewed bays.

Joist Girders:

Joist Girders are simply supported, simple span primary load carrying trusses. The loads are typically applied through the reaction of the supported steel joists and are typically equal in magnitude and evenly spaced along the joist girder top chord. The ends of joist girders must extend a distance of no less than 6 inches over a steel support.

Joist girder tables found in the SJI Specifications include member depth, number of joist spacings, loading at each joist location and an approximate weight of the joist girder. The SJI Weight Table for Joist Girders includes approximate weights for joist girders with depths from 20 to 72 inches and spans up to 60 feet. The standard bearing seat depth is 7½ inches.

When joist girders support equal, uniformly spaced concentrated loads, the joist girder designation provides a complete specification for the member. For example, the joist girder designation of 60G10N12K indicates the joist girder is 60 inches deep, G indicates that it is the Joist Girder series, 10N indicates the number of joist spaces, and 12K indicates the magnitude of the concentrated loads in kips. It is necessary to include the self weight of the joist girder in the panel point loads. The joist manufacturer will design the joist girder using the most economical web configuration, which typically includes placing the diagonals directly under the concentrated loads.

As the depth to span ratio increases for a given joist girder, it is more economical to load both the diagonal panel points and the vertical panel points (see Figure 9). While the designation VG is not shown in the SJI Specifications, joist manufacturers will recognize this designation as an indicator to locate the vertical panel points underneath the concentrated load (see Figure 10). Because the joists align with the web verticals and do not block the open panels above the bottom chord, this later loading configuration provides the largest amount of unobstructed openings to accommodate mechanical ductwork through the joist girder.
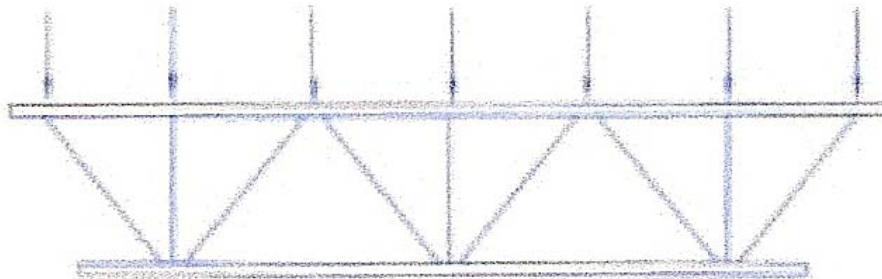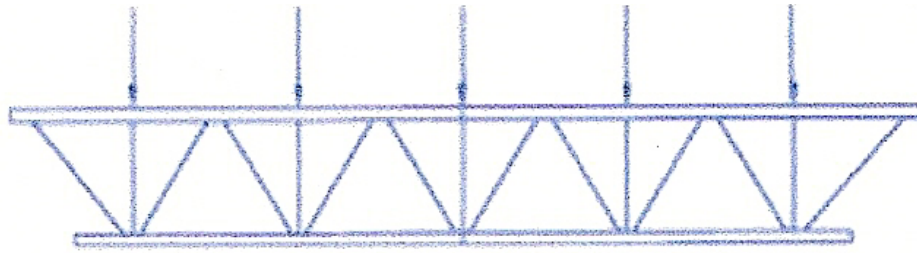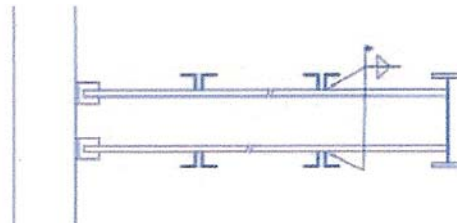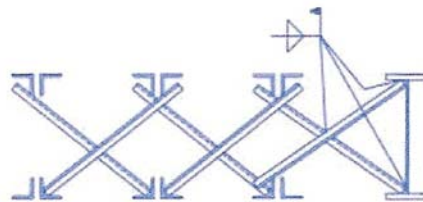

FIGURE 9

FIGURE 10

Bridging:

The purpose of joist bridging is to:

- Help align and provide stability for the joists during erection.

- Control the slenderness ratio of the bottom chord.

- Assist in stabilizing the joist webs.

The size, type and number of rows of bridging depend on the span of the joist, spacing between the joists and the standard joist designation. Joist bridging may also be required to brace the bottom chord for wind uplift and axial loads. There are two types of bridging; horizontal and diagonal. Horizontal bridging consists of continuous angles connected to the top and bottom chords (see Figure 11). Diagonal bridging consists of two angles that cross diagonally from the top chord to the bottom chord between each joist and are connected at their point of intersection (see Figure 12).

FIGURE 11

FIGURE 12

For typical situations, the number of rows of bridging required is given in tabular form in the SJI Specifications or manufacturer's catalog. These specifications also indicate when bolted diagonal erection stability bridging is required during construction. The ends of all bridging should be positively contacted to a rigid structure (such as a wall or beam) that is capable of resisting the applied bridging reaction. However, when a joist is at the end of a bridging line (such as an expansion joint) diagonal X-bridging should be used between the last two joists.

Bridging is required to laterally stabilize the joist against torsional buckling until the permanent roof or floor deck is attached. Construction loads should not be applied to the joist until the bridging is attached to the joist and anchored at its ends. Diagonal bridging between the last joist and a rigid end wall may cause the diagonal bridging to act like a vertical support and will therefore resist the joist's deflection when any vertical loads are applied. In this situation, when the joist tries to deflect and the end wall prevents any vertical movement, damage may occur to the bridging, which in turn will detrimentally impact the effectiveness of the bracing. This condition can be avoided by substituting horizontal bridging for diagonal bridging in this last space after the erection process is completed.

Roof joists and joist girders can be subjected to net uplift loads when the suction from the applied wind forces on the roof exceeds the permanent dead loads. Uplift loads can affect the design of the joists' components as well as the bridging. This is because, due to gravity loads the top chord of the joist is in compression and the bottom chord is in tension. However, when subjected to a net uplift loading, the bottom chord of the joist will be in compression. Due to this load reversal, the bridging design will typically need to be adjusted to properly brace the bottom chord.

The SJI Specifications require that joists subjected to net uplift must have a line of bridging near each of the first bottom chord panel points. Depending on the actual magnitude of the net uplift, additional bridging may also be required. Uplift also causes a stress reversal in the joist webs, and the manufacturer will be required to design them accordingly. The SJI Specifications require the net uplift on joists and joist girders be provided to the joist manufacturer. This can sometimes be handled by providing a note on the Contract Drawings that simply states that the joists, joist girder and bridging be designed for a uniform net uplift of some specified amount, such as 20 PSF.

However, because building corners and edges are subject to increased wind loadings, it is more common for a diagram to be provided on the drawings that clearly shows the magnitude and extent of the net uplift. Joist girders should be considered primary members (i.e. greater tributary area) when determining uplift loads, therefore a lower net uplift on the joist girders can sometimes be specified. The manufacturer designs the bottom chord and the bottom chord braces for the joists and joist girder as required by the net uplift loadings.

<center>End Moments:</center>

When joists or joist girders are used as part of a rigid frame, moment frame or other similar bracing system, axial loads and or end moments may be induced into the joists or joist girders. For this condition a load diagram or schedule of loads should be used to inform the joist manufacturer of the magnitude and the direction of the moments and forces, and the required load combinations that should be considered for each load. Similar loading information should also be provided for any top chord extension or joist that has special gravity load conditions beyond that which is covered by the standard SJI or manufacturer's load tables. As a part of the design and fabrication of the affected members, the joist manufacturer will investigate the impact of the loads, moments and chord forces and adjust the design of the member and or quantity and spacing of the chord braces accordingly.

Appropriate connections are also required for the transfer of end moments and chord forces from the bracing system or columns into the joists and joist girders. If no other load path is provided, the top chord must be designed to accommodate the axial force and bending moment that results due to the eccentricity of the load path (see Figure 13). A more appropriate solution to accommodate this condition is to provide a top plate or knife plate to connect to either the adjacent joist or the top of the supporting column as required.
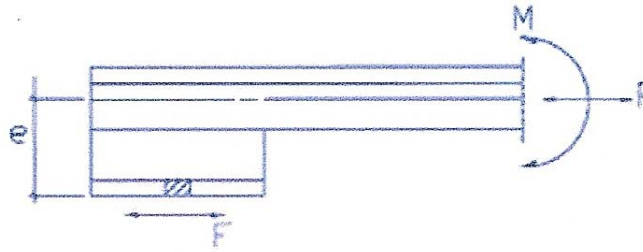
FIGURE 13

When the roof deck is used as a diaphragm, then the collector or chord forces must sometimes be transferred from the deck into the beam or joist girder. The lateral stiffness of the joist seat may not be adequate to transfer this force. When this occurs, either a steel tube or channel section can be used to transfer the lateral load from the deck to the supporting member.

Sprinkler and Roof Top Unit (RTU) Loads:

Loads from small ducts, cable trays and sprinkler systems can be accounted for by including a uniform collateral load of sufficient magnitude to cover all these same loads. A joist can then be selected to resist this collateral load in addition to all the other uniform loads. Although in actuality these loads are delivered to the joists at distinct locations, this uniform collateral load approach is both reasonable and economical for most buildings. However, heavy sprinkler pipe hanger loads should be located at joist panel points so that local bending will not be induced into the joist chords. Therefore it is recommended that in special instances, the magnitude and location of large ducts and sprinkler main concentrated loads should be specified on the design documents.

It is common practice for mechanical units to be placed on the roofs of buildings. RTU's can vary in weight from 100 pounds to more than 45,000 pounds. The footprint of these units can cover an area of a just a few square feet or much more. Therefore it is important to designate on the drawings the magnitude and locations of all RTU's even if the affected supporting joists have been designed for the imposed loads. This information is required to both provide a record of the significant design loads on the building and to serve as a check against the weights of the actual equipment installed.

Joist Span Direction and Joist Spacing:

For floor systems, it is usually more economical to span the joists in the long direction and joist girders in the short direction to take advantage of the fact that the joists sit on top of the joist girder and can therefore be made deeper than the joist girder by the amount of the seat depth without encroaching into the clear height requirements. Wide joist spacings provide a more economical floor system than closer spacings. The widest spacing for a given deck profile and slab thickness should always be used. This is because the erection costs are typically less (fewer pieces to handle) and the floor will usually have better vibration characteristics. Deeper joists also allow larger penetrations through their webs. While wider spacing are also preferred for roofs, the most economical framing will vary based on the roof design requirements and equipment layout.

The optimum joist girder depth in inches is approximately equal to the span of the girder in feet. The joist depth however should be selected based on the economy tables described above. The bridging requirements should also be considered when selecting the joists. This is because it may be more economical to pick a

slightly heavier joist if a line of bridging can be eliminated, particularly if the heavier joist eliminates the need for any diagonal bridging.

Camber:

The SJI Specifications indicate that the any required camber should be provided for LH and DLH-Series joists on the Contract Documents. Camber is optional for K-series, however, if camber is required on a K-Series joists, it should be specified. Long LH and DLH-Series joists will have a significant amount of camber which can cause problems in connecting the deck to an end wall if the camber has not been considered as a part of the detailing of the same connection. If a joist is placed directly adjacent to an end shear wall, and the deck must be connected to the wall, then some allowance must be provided in either the camber or in the location or adjustment of the end wall supports so the deck can be attached to the wall adequately. If the edge joist can be eliminated and the camber of the first interior joist is not significant enough to cause the deck to buckle, the deck can often be warped into contact with the end wall support so that the proper connection can be made. If a special camber is required to make this connection, it must be indicated on the drawings.

Sloping Joists:

The span of a parallel chord sloped joist is the length along its slope. The minimum depth, load-carrying capacity and bridging requirements are determined using this sloped length of span. The load capacities from the SJI Standard Load Tables will be the component normal to the joist. The SOR should therefore specify the load component parallel to the joist so that the manufacturer can account for any axial forces.

Vibration:

Unlike steady state vibration, vibration due to human impact is inconsistent in amplitude and frequency and therefore, more difficult to control. Floor vibration is measured in terms of acceleration amplitude, displacement amplitude, and frequency. These factors are not objectionable to all people at the same level since human sensitivity varies. Acceleration amplitude is the maximum acceleration caused by a force excitation. Displacement amplitude is defined as the magnitude or total distance traveled by each oscillation of the vibration. Frequency is the term used to describe the speed of the oscillations and is expressed in cycles per second or Hz. Damping is defined as the rate of decay of amplitude. Acceleration is the only vibration factor which humans can sense. SJI Technical Digest #5 provides methods for calculating vibrational properties for joist supported floors.

Open floor areas framed with steel joists are sometimes more subject to vibrational problems than systems framed with rolled steel beam sections. This problem is more common in modern offices which tend to have lower live loading and damping, and therefore can potentially be more prone to floor vibration. Partitions, file cabinets, book stacks, heavy furnishings and even crowds of people provide additional damping and minimize complaints. Thicker floor slabs are an economical solution to floor vibration with open web steel joists. Additional thickness of the slab can increase the floor system stiffness transverse to the joists, thus reducing the vibration. The resulting additional mass of the system will help to reduce any objectionable vibration. As already indicated, wider joist spacing will improve the vibrational characteristics of a floor system, only when combined with thicker floor slabs.

Partitions introduce damping and usually help to eliminate vibration problems. They can be effective either above or below a floor as long as they are connected to the floor above. Partitions below a joist supported floor ideally should be in direct contact with the steel deck, however, this can result in buckling of the stud

wall if loading on the slab and adjacent joist result in significant vertical deflections. If partitions below a joist supported floor are in direct contact with the joists, the joist bottom chord and webs must be designed for such intermediate support conditions. Support joist girders or beams can sometimes contribute to floor vibration problems. The natural frequency and amplitude for both the joist and supporting joist girders or wide flange beams need to be calculated when analyzing and entire floor system.

Increasing joist stiffness above that which is required by live load deflection may be beneficial in reducing vibration problems. A higher frequency floor is generally a better floor for most applications. Increasing the stiffness of the steel joists themselves results in increasing the frequency and slightly decreasing the acceleration or displacement of the floor vibration. The SJI provides software for the vibrational analysis of an open web steel joist supported floor at their website (see link above).

Miscellaneous:

See *Special Vertical & Lateral Load Considerations for Steel Joists & Joist Girders*, Course S116 for a further discussion of joists and joist girders.

Additional notable information concerning joists and joist girder:

- OSHA regulations mandate that the bottom chord of a joist or joist girder that frames into a column must be extended to envelop a stabilizer stiffener plate attached directly to the face of the supporting column. This is required in order to provide additional stability to the column during the erection process. The bottom chord extension (BXC) does not have to be welded to the column, however, if the permanent bracing affects of the lower connection has been assumed as a part of the design of the column, the BCX is typically welded to the stabilizer plate after all dead load has been applied to the structure.

- When the modification of an existing roof involves reducing the dead load on the structure (i.e. converting a ballasted roof to as adhered membrane roof) then the existing roof joists should be analyzed for the resulting additional net uplift forces.

- Joists and joist girders can be specially fabricated to provide gabled, mono sloped or curved top chords.

- Open web joists are also available for composite floor construction. The following link provides access to a manufacturer's (Canam Hambro) website and the available on line technical material for their composite steel joist; http://www.hambro-floors.ws/www/v4/ehambro.nsf. In the New York metropolitan area, this type of composite joist in conjunction with load bearing light gage metal stud walls is used extensively in multi-story residential construction. An example photo of this type of construction is provided below:

FIGURE 14

- It is possible to frame a conventional wide flange beam into the side of a joist or joist girder. A typical detail (Figure 15) indicating how this is accomplished as well as some other typical joist framing details (Figures 16 – 19) are provided below.
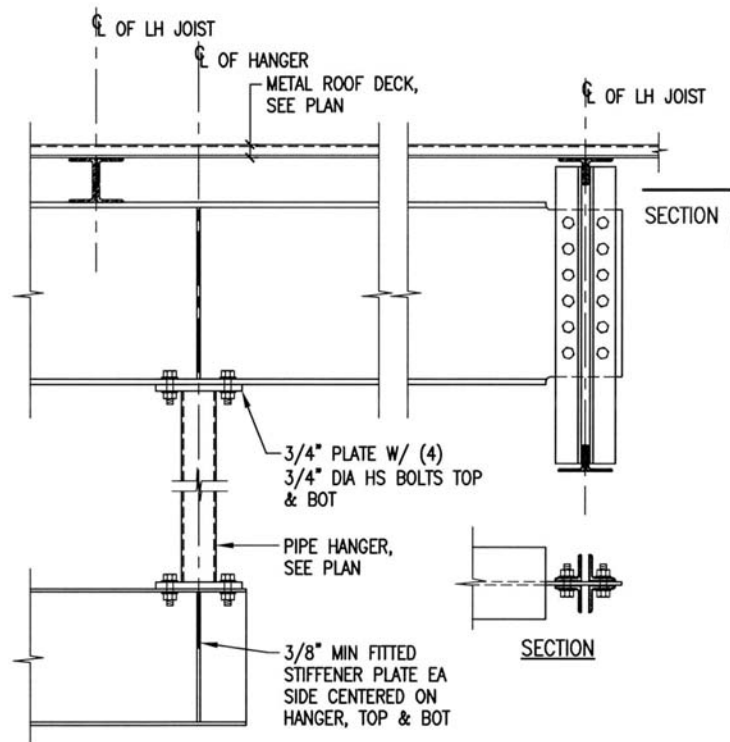

FIGURE 15

FIGURE 16

FIGURE 17

1/2"

METAL ROOF DECK

JOIST

JOIST GIRDER

UPLIFT BRACING @ MID-SPAN OF ALL JOIST GIRDERS AND ADDITIONAL LOCATIONS IF REQUIRED FOR UPLIFT

1/8

R14
S3.1
SECTION @ JOIST
GIRDER UPLIFT BRACING

NOTES:
1. BOTTOM CHORD BRACE TO BE DESIGNED AND FURNISHED BY THE JOIST GIRDER MANUFACTURER.

FIGURE 18

EM-3 MIN (2) PER PANEL (MIN EDGE DIST FROM PANEL END =1'-0")

3/16

L4x4x1/2 CONT, SEE TDR5/S3.1 FOR SPLICE DETAIL

METAL ROOF DECK

TILT-UP PANEL, SEE ELEVATION FOR THICKNESS & REINF

PROVIDE ADDITIONAL DIAGONAL BRACING @ END BAYS

CONT HORIZ BRIDGING (T&B)

L-3x3x3/8x3" LONG W/ (1) 3/4" DIA HILTI HY150 ADHESIVE ANCHOR (4" MIN EMBED) @ EA BRIDGING/BRACE

FIGURE 19

Metal Building Systems

Another example of a pre-engineered, pre-manufactured steel system is a metal building or Pre-Engineered Metal Building (PEMB). This type of system involves specially fabricated structural steel and cold-formed metal sections and components that are designed by the manufacturer's own structural engineer. However, it is common for the foundations that support these types of buildings to be designed by a separate EOR.

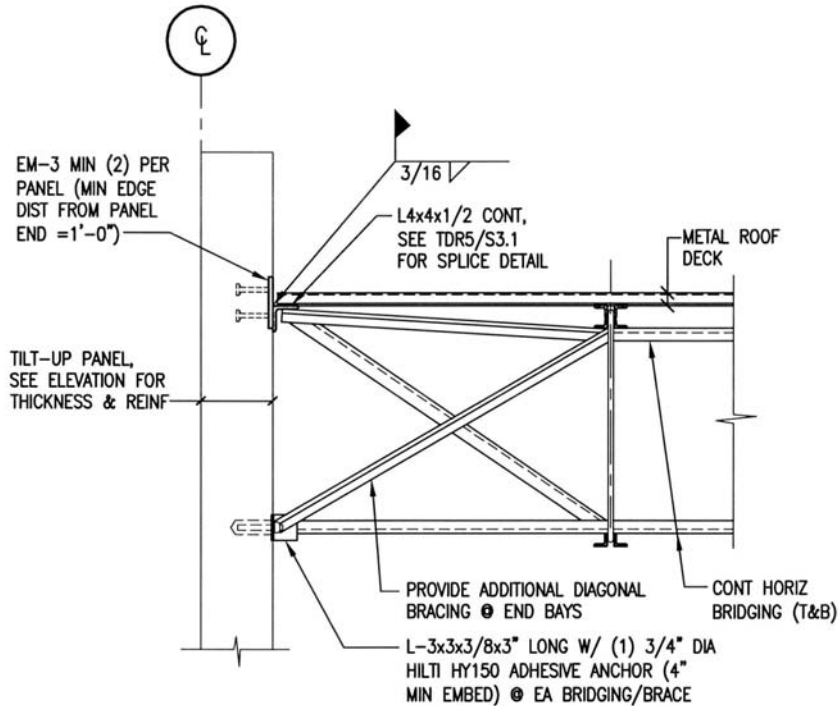As a result of this arrangement in which the design responsibilities are divided between two different engineers, it is important to have an understanding of the common problems that can occur as well as terms that are frequently used in the PEMB industry. See *Metal Building Systems*, Course S120 for a further discussion of this topic. Please also see *The Case for and Engineer of Record for a Metal Building System*, STRUCTURE Magazine, March 2007.

Common problems that arise during the design and development of a PEMB project as a result of the differing responsibilities of the EOR and the manufacturer's engineer include:

- The typical flexibility of a PEMB building (H/120) results in a situation in which any CMU cladding can be adversely affected by the amount of movement allowed by the PEMB superstructure and in some cases by any cladding support members (girts) designed by the manufacturer. As the PEMB manufacturer is typically only interested in the performance of their product and not necessarily with any attached architectural components or cladding design by the EOR, it is often necessary to insist that the PEMB comply with serviceability requirements (such as the ACI 530 for masonry) that are outside of the design criteria normally used by the Metal Building Manufacturers Association (MBMA). The insistence by the EOR of adherence to design criteria outside of the normal practices of the MBMA can often result in cost increases to the Owner because it is common for the PEMB manufacture to have already quoted a construction cost to the Owner for the building structure prior to the involvement of the selected EOR.

- A similar problem with the interface between the design responsibilities of the PEMB manufacture and the EOR can also occur with the selection of anchor bolts (anchor rods). In this situation the design of the column base plates and associated anchor bolts are clearly within the realm of the manufacturer's design responsibility. Never the less, more often than not, the PEMB's engineer will defer the selection of the anchor bolts to the EOR in order to avoid accepting the responsibility for determining the correct amount of embedment in the foundation designed by the EOR. In this situation it is common for the manufacturer's engineer to provide column base reaction forces to the EOR and ask the EOR to size the actual anchor bolts. The appropriate response from the EOR in this situation is to insist that the manufacturer size and provide the anchor bolts whereas the EOR will assure that the foundation into which the anchor bolts are embedded will be sufficient to resist all applied forces from the anchor bolts and base plate.

- Typically the EOR must complete the design of the foundation before the actual final design of the PEMB is completed. This results in a situation in which assumptions concerning the building column reactions must be made by the EOR prior to the design of the foundation. In some instances, if the PEMB manufacture has already been selected, preliminary reactions can be obtained directly from the manufacturer's engineer. In other situations it is necessary for the EOR to make assumptions concerning the reactions on his or her own. In this later situation, because of the variability in the different ways in which different PEMB manufacturer's approach solutions to the gravity and lateral framing needs for a building, it is often necessary for the EOR to produce schematic gravity and

lateral framing plans in order to document the assumptions made for the design of the foundation in order to provide guidance to the bidders for the specific project. It is important that the schematic plans identify the location of the longitudinal bracing systems (i.e. vertical X-braced bays or portal frames) because the foundations at these locations will typically be larger than that required at the typical transverse rigid frame.

In addition, because of the variable inertias of the fabricated framing members typically used in a PEMB, it is very difficult to accurately predict the base column reactions, particularly with regard to the magnitude of the horizontal "kickout" forces associated with the structural behavior of the rigid frames. In this situation it is recommended that a PEMB manufacturer be contacted to provide assistance in establishing the likely magnitude of the reactions.

In either case, it is important that the EOR provide a note on the foundation Contract Drawings which indicates that the construction of the foundations should not occur until after the reactions assumed by the EOR are confirmed with the actual PEMB manufacturer designing and erecting the building. In addition, the EOR should indicate to the Owner that if the actual PEMB reactions are substantially different from that assumed that the EOR should be compensated for the foundation redesign effort required.

The magnitude of the base horizontal kickout force associated with PEMB rigid frames can be challenging to deal with when designing the support foundations. Trying to resist the typical magnitude of horizontal force with the footing alone can result in very large uneconomical solutions. In order to avoid extensive foundation costs it is common to offset the outward kickoff forces by connecting opposing columns associated with the same rigid frame located on opposite ends of the clear span. This is accomplished in two ways. The first method involves the use of a continuous tie rod embedded in a thickened portion of the slab on grade in order to physically connect the bases of both columns. Tie rods are best suited for large horizontal forces in the range of 40 to 50 kips. Tie rods are typically designed to resist the service or working stress reaction up to 60% of their yield strength. Ties should be galvanized or coated to prevent deterioration. Tie rods embedded in the slab are susceptible to future renovation work in a building if the retrofit work involves demolition of the floor.

The second method of resisting the horizontal forces is through the use of hairpin bars embedded in the slab. This solution is similar to the use of tie rods except the transfer of the load is made possible through the physical connection of the hairpin to the base of the column and the subsequent transfer of the force in the hairpin to the transverse reinforcement embedded in the slab. For this solution there cannot be any longitudinal construction joints located on the slab that would interrupt the continuous transverse slab reinforcement. A further discussion of this method of reinforcement can be found in *Metal Building Systems Design and Specifications*, Alexander Newman.

Castellated Steel Beams

Castellated is commonly used to describe a type of "expanded" steel beam. A castellated section is made by first cutting the web of a wide flange beam in a "zigzag" pattern then shifting one portion of the cut section relative to the other to create a regular pattern of holes in the web. The term is derived from the pattern formed by the web holes, because castellated means "built like a castle, having battlements, or regular holes in the walls, like a castle". Figure 20 illustrates a castellated beam. The two halves of the beam are joined by welding the offset portions so that the high points of the web pattern match up. Some castellated beams are

increased in height even further by adding supplemental web plates between the high points of the tee sections. These supplemental plates are called "increment plates".

Cellular "castellated" beams are also manufactured. This type of castellated beam is fabricated with round or circular web openings. This form of castellated beam is required if the member is to be hot-dipped galvanized because conventional castellated beams are susceptible to stress risers at the reentrant corners of the openings as a result of the hot-dipping process.
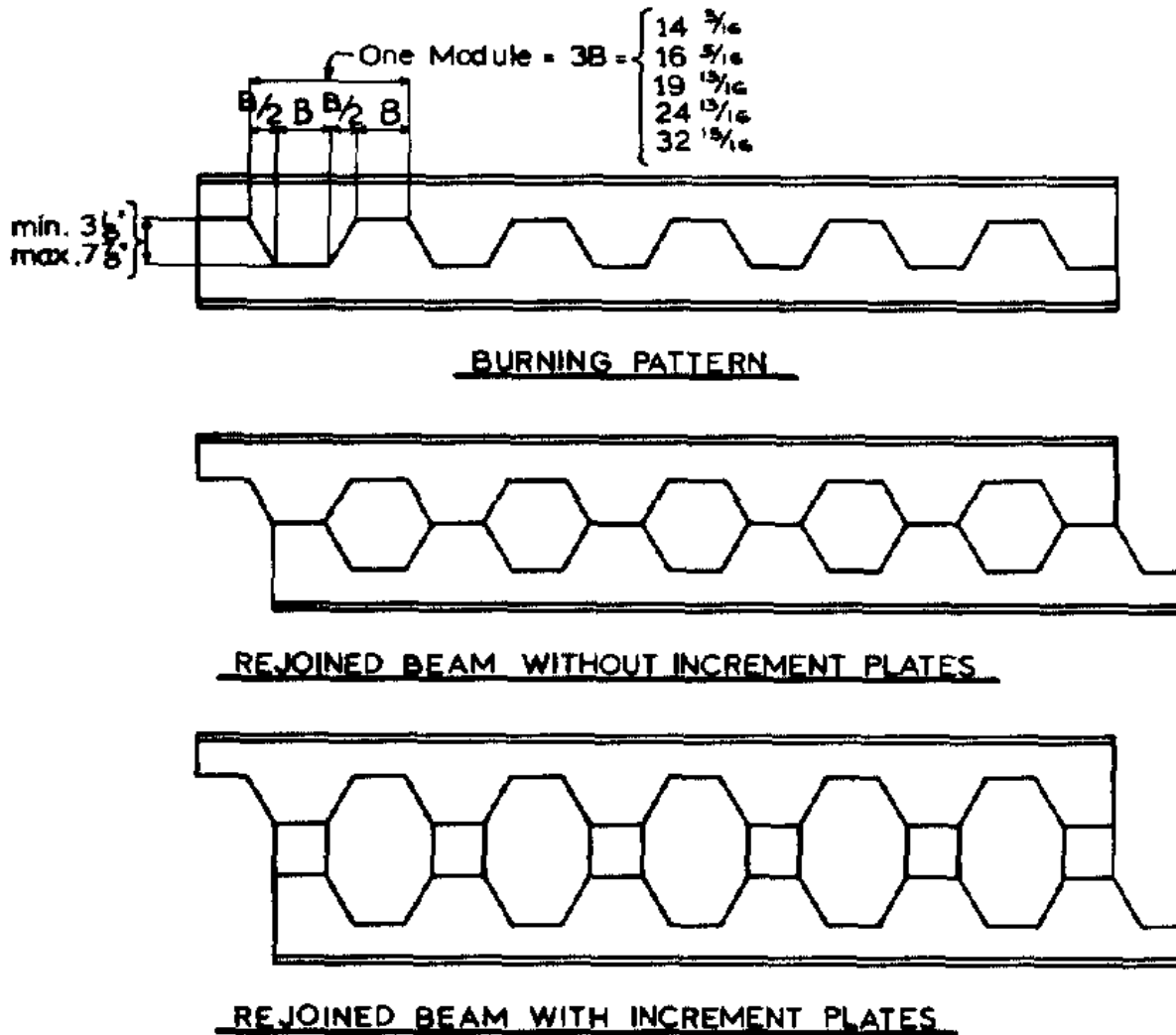

FIGURE 20

The patterns of holes in the web created in a castellated beam are considered by some to be architecturally attractive. As a minimum the web holes provide a way of routing piping, conduits and ductwork through the beam thus reducing the need for additional clearance between the bottom of the beam and the ceiling. The greatest advantage, however, is the economy provided by the increased load carrying capacity and stiffness of the castellated section over that provided by the same non-castellated beam.

Although castellated sections are thought of as beams, these types of members are analyzed as a Vierendeel truss. The ASD (Allowable Stress Design) design of a castellated beam consists of analyzing the effect of the forces and calculating the stresses as shown in Figure 21. Typically the maximum longitudinal stresses occur

in the tee section above and below the openings. The stresses in the tee sections are computed on the basis of the following which assumes that the upper and lower tee sections are symmetrical:

1. The vertical shear is divides equally between the upper and lower tees.

2. Bending in the tees due to shear assumes that there are points of contraflexure at the vertical centerline through each opening.

3. Stresses are distributed as illustrated and can be computed by the formulas shown in Figure 21.
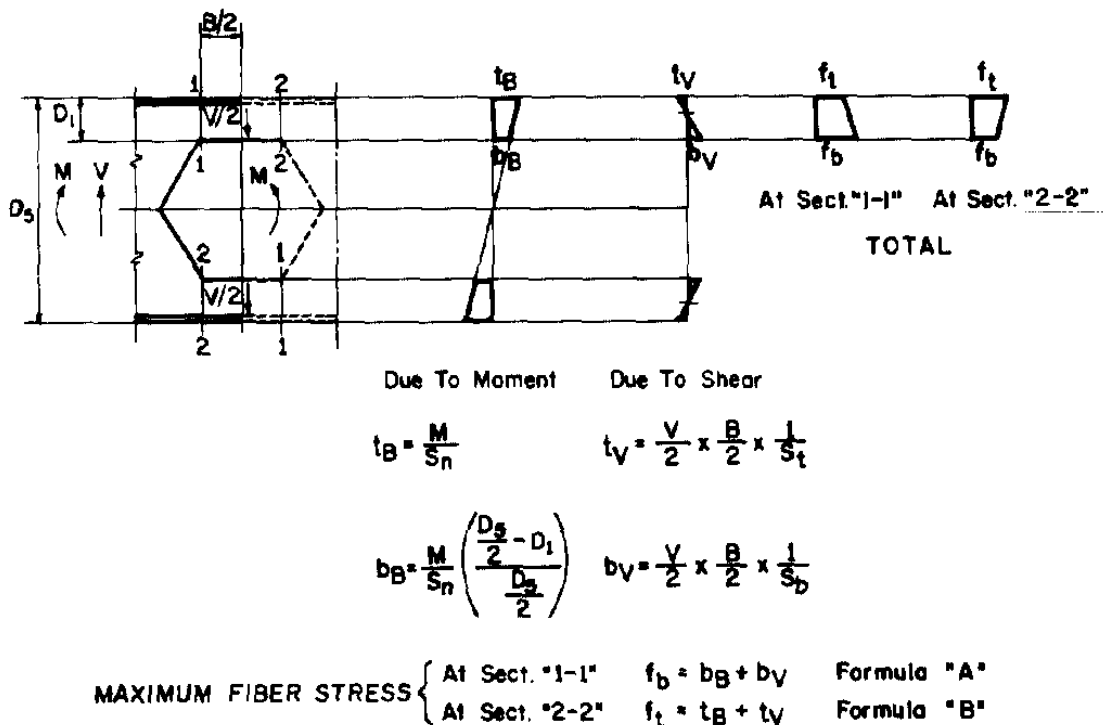

$$t_B = \frac{M}{S_n} \qquad\qquad t_V = \frac{V}{2} \times \frac{B}{2} \times \frac{1}{S_t}$$

$$b_B = \frac{M}{S_n}\left(\frac{\frac{D_s}{2} - D_1}{\frac{D_s}{2}}\right) \qquad b_V = \frac{V}{2} \times \frac{B}{2} \times \frac{1}{S_b}$$

$$\text{MAXIMUM FIBER STRESS}\begin{cases} \text{At Sect. "1-1"} & f_b = b_B + b_V & \text{Formula "A"} \\ \text{At Sect. "2-2"} & f_t = t_B + t_V & \text{Formula "B"} \end{cases}$$

FIGURE 21

Because the location of the maximum stress is typically not readily apparent (i.e. it can occur at any point along the length of the beam) the design of castellated beams is time consuming. This condition occurs because *fb* will be at its maximum in areas of higher shear while *ft* will be at its maximum in areas of high moment. Therefore, for a uniformly loaded, simple span castellated beam the location of maximum stress can occur at any point between the end of the beam (high shear) and mid-span (high moment).

The most practical method for designing a castellated beam is to start with a specific beam section and compute its capacity. However, even this approach can be time consuming. Fortunately manufacturers of castellated beams have produced load and span capacity tables or design software for use by the design engineer. The following link provides access to a manufacturer's (CMC Steel Products; Smartbeam) website and the available on line technical material for their castellated and cellular (i.e. round web openings), composite and non-composite sections; http://www.cmcsteelproducts.com/design_progs.html.

The maximum longitudinal stress generally governs the selection of a castellated beam section, but just as with the design of any steel beam there are other conditions which must be examined including lateral

torsional buckling, local buckling, web buckling and web bending and shear stresses. However, if the compression flange is continuously supported laterally by a deck or slab, design restrictions associated with lateral torsional buckling do not need to be considered.

Web buckling of a castellated beam due to the vertical shear involves a special analysis procedure because of the influence of the adjacent web openings. The following equation provides a method of calculating the allowable shear stress in the solid web section for Fy = 46 ksi steel. This equation is based on a modified version of a column analysis and has been verified by experimental work.

$$fv = 18,000/(1 + ((1/2,666.75)((D_s-2D_l)/T_2)^2))$$

The allowable vertical shear is therefore equal to the unit shear, *fv*, multiplied by the cross-sectional area of the solid web panel at its least section. If the vertical shear at any solid web in the beam exceeds this value, then a stiffener is required. Usually a single vertical bar stiffener added on one side of each solid web section is provided as web reinforcement if required. In general, castellated beams without increment plates do not require stiffeners except for conditions of heavy loading.

Bending and shear stresses in the web elements that act as the verticals of the Vierendeel model should also be checked because in some cases they can be critical. The web stresses in these areas can be calculated using the information provided in Figure 22.
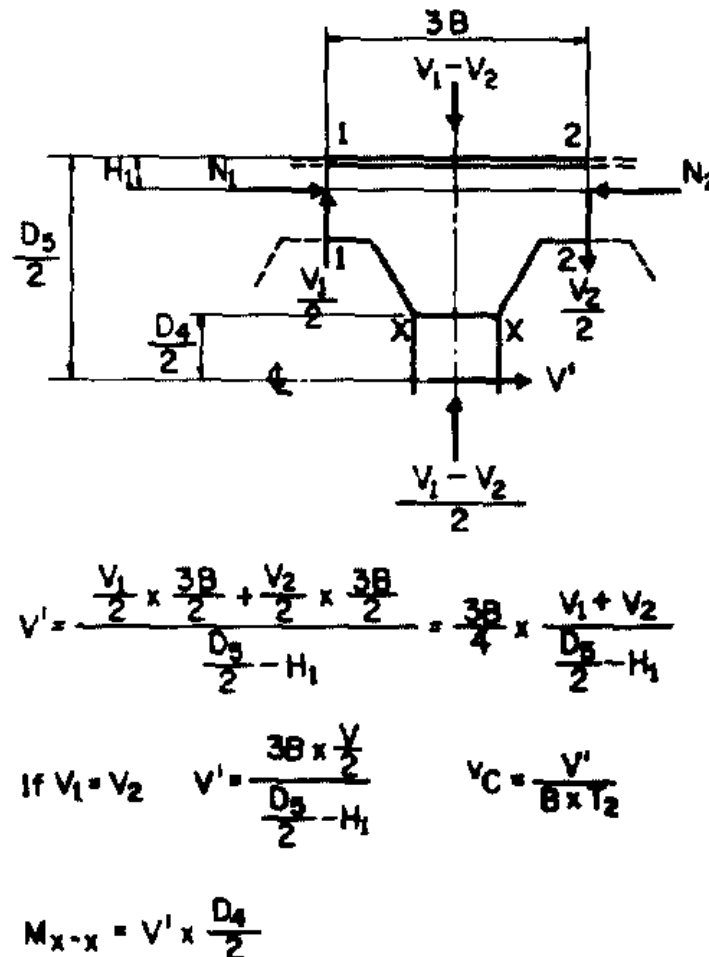
$$V' = \frac{\frac{V_1}{2} \times \frac{3B}{2} + \frac{V_2}{2} \times \frac{3B}{2}}{\frac{D_5}{2} - H_1} = \frac{3B}{4} \times \frac{V_1 + V_2}{\frac{D_5}{2} - H_1}$$

$$\text{If } V_1 = V_2 \quad V' = \frac{3B \times \frac{V}{2}}{\frac{D_5}{2} - H_1} \qquad {}^vC = \frac{V'}{B \times T_2}$$

$$M_{x-x} = V' \times \frac{D_4}{2}$$

FIGURE 22

Conventional composite and non-composite steel beams with web openings are analyzed in a manner similar to castellated beams. A further discussion of this topic can be found in AISC Design Guide #2, *Steel and Composite Beams with Web Openings*.

Staggered Truss System

The staggered-truss framing system was developed by a United States Steel sponsored research team working at Massachusetts Institute of Technology in the mid 1960s. The object of the study was to arrive at a new, efficient, structural steel system which would also provide architectural benefits. The result was the staggered-truss system which has since been used in buildings greater than 30 stories in height.

The basic element of the staggered-truss system is a story-deep truss which spans the full transverse width of the building at alternate floors on each column line. These trusses are supported only at their ends on two longitudinal rows of exterior columns and are arranged in a staggered pattern on adjacent column lines (see Figure 23). The gravity loads are delivered to the truss from the floor slab system (typically precast hollowcore plank) which spans from the top chord of one truss to the bottom chord of the adjacent trusses. Therefore, each truss is loaded at its top and bottom chord and the total gravity load of the building is transferred to the building's exterior columns. In addition, it is common to omit any diagonal web members in the center panel of the truss in order to allow for a central corridor to be constructed down the longitudinal axis of the building.
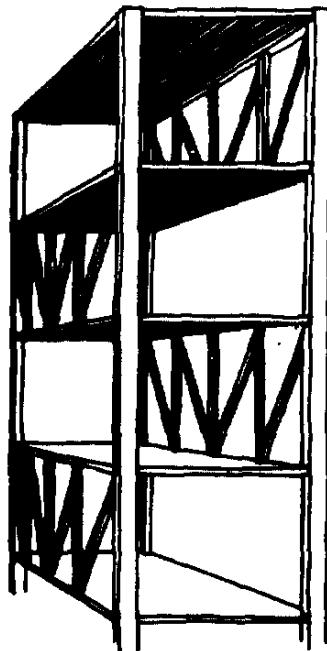

FIGURE 23

The main structural benefit and subsequent efficiency is the system's resistance to lateral loads acting parallel to the trusses (i.e. the transverse direction of a narrow rectangular building where lateral resistance is often a problem since the wind forces developed on the larger face of the building are substantial and must be resisted by the smaller building dimension). The inherent benefit of the staggered truss system in this situation is that the entire building weight is mobilized in resisting the overturning moment. This is because

the large gravity loads on the exterior columns are often sufficient to counteract the uplift forces associated with the applied lateral loads on the building.

Other advantages of the staggered truss system include:

1. Columns have minimum bending moments due to gravity and wind loads because of the cantilever action of the double-planar system of framing.

2. Columns can be oriented with their strong axis resisting lateral forces in the longitudinal direction of the building.

3. Maximum live load reductions can be realized because of the extent of tributary area supported by each truss.

4. Large clear span open areas for lower ancillary or parking garage areas are possible because the columns are located only on the exterior faces of the building.

5. Foundations can be located on the exterior column lines only and can consist of strip footings.

6. Drift is relatively small because the total frame is acting as a stiff truss with direct axial loads only acting in most structural members. Secondary bending occurs only in the chords of the trusses.

7. High strength steels may be used to advantage, because all truss members and columns are subjected, for all practical purposes, to axial loads only.

A further discussion of this topic see AISC Design Guide #14, *Staggered Truss Framing Systems*.