MBMA
METAL BUILDING MANUFACTURERS ASSOCIATION®
Research • Leadership • Education

RETROFITTING/REINFORCING

METAL BUILDING SYSTEMS

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Background

Metal building systems offer flexibility and adaptability to an owner. This is an advantage if the building's use or function changes from the original design, or the enclosed space is enlarged. However, metal buildings are designed very efficiently as a system, and cost-effective structural upgrades may require specialized knowledge. This technical publication is intended to provide some ideas and insight into how a metal building can be retrofitted or reinforced as part of typically encountered upgrades or modifications. Note that any structural modifications should be reviewed by an experienced licensed professional engineer.

Some of the reasons that a metal building system may need to be retrofitted or reinforced are:

- Building code changes requiring a higher snow load or wind load when other structural changes require a building permit
- Construction of adjacent structure with higher roof resulting in added snow drift or sliding load to the existing building
- Unbalanced snow load may be present or larger than the original design required
- Changed end use of the building, requiring expansion or moving or eliminating columns
- Addition of equipment, such as sprinkler systems or other equipment suspended from the roof, solar panels on a roof, or new or upgraded cranes

Note that this technical publication is focused on retrofits or reinforcements to the metal building structure, but a review of the foundation is also needed and can affect the direction one proceeds with any modifications or expansion.

Characteristics of a Metal Building System

Primary Structural System

The primary framing of metal building systems is normally rigid moment frames, comprised of column and rafter beam members, fabricated with welded “I” shapes using plate or bar flanges and sheet or coil steel webs. Members are typically web-tapered to optimize the use of material and they also utilize bolted end plate connections for quicker field erection. Spacing of the frames, i.e. bay spacing, can range from 12 to 35 feet and can vary along the length of the building. Frames spaced in excess of 35 feet normally require the use of truss type secondary members as described in the next section. Frames can be single span (clear span frame) or may be multi-span frames with interior columns (modular frame). A clear span frame can range up to several hundred feet (See Figure 1). The width of multi-span frames is virtually unlimited, with widths of 800 to 1200 feet, not uncommon (See Figure 2). Roof slopes typically vary from ¼:12 to 6:12.

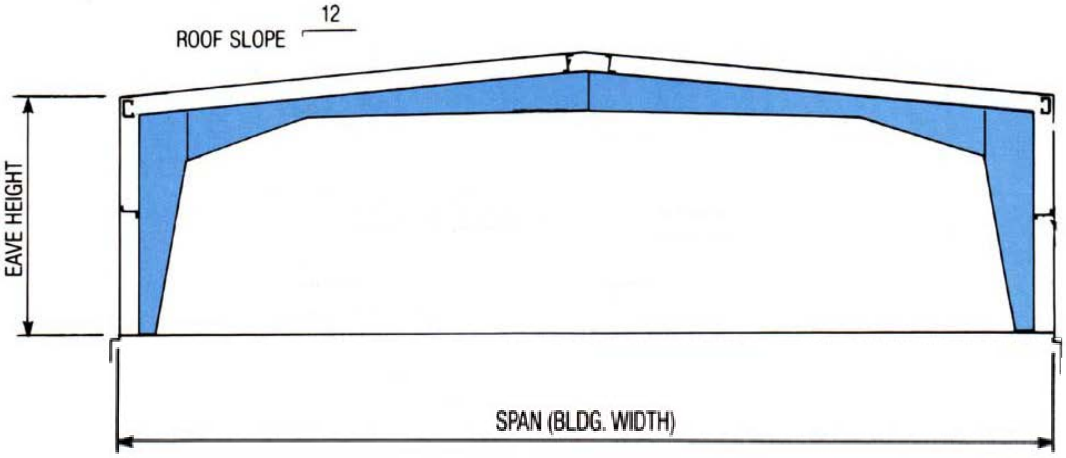


Figure 1: Single Span Frame

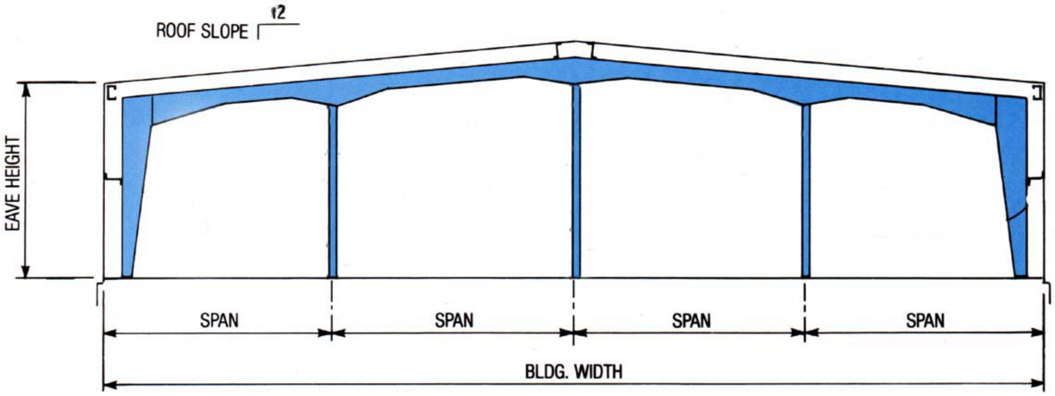


Figure 2: Multi-Span Frame

Frames may be configured as gable frames (as shown in Figures 1 and 2), or they may be single slope (See Figure 3). A lean-to frame (See Figure 4) is commonly used to extend the building width as discussed later.

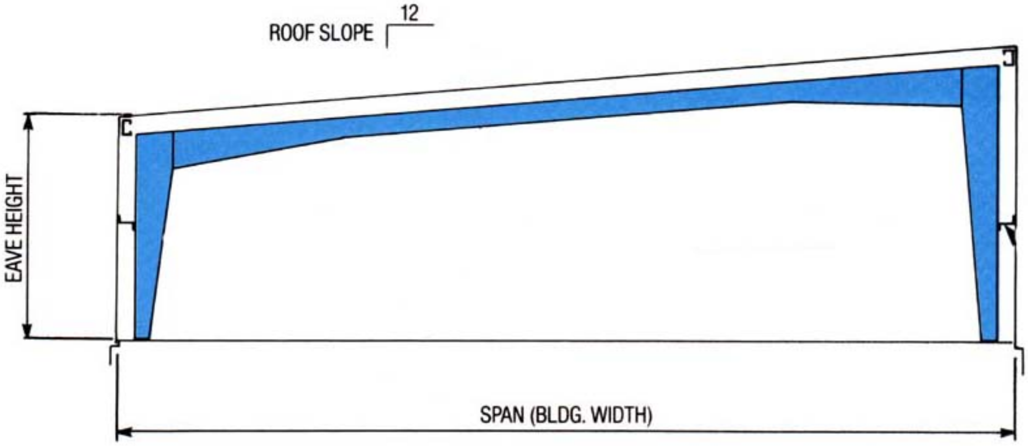


Figure 3: Single Slope Frame

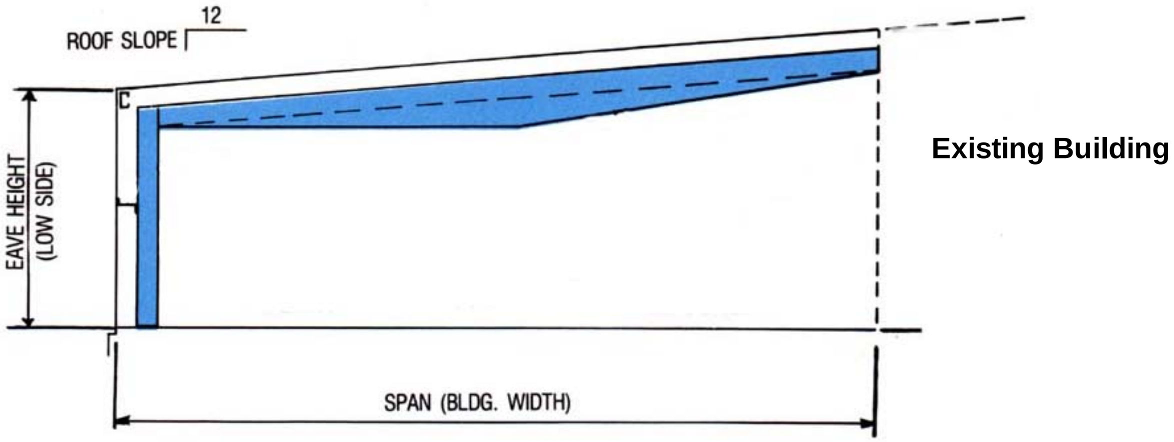


Figure 4: Lean-To Frame

Secondary Structural System (Purlins/Girts)

The typical secondary members used in a metal building are cold formed Z or C shaped sections, ranging in depth from 6 to 15 inches, with thicknesses from 0.04 to 0.15 inches (See Figure 5). They are designed to span from 12 to 25 feet in a simple span condition and up to 35 feet in a continuous system. Continuity is achieved by lapping the members. For Z shapes, this involves nesting them together and for C shapes they are arranged back to back (See Figure 6). The length of the lap varies widely between manufacturers, but is typically between 1 to 4 feet (See Figure 7).

Figure 5: Cold Formed Z and C Shapes for Secondary Members

Figure 6: Nested Z Shapes and Back to Back C Shapes for Laps

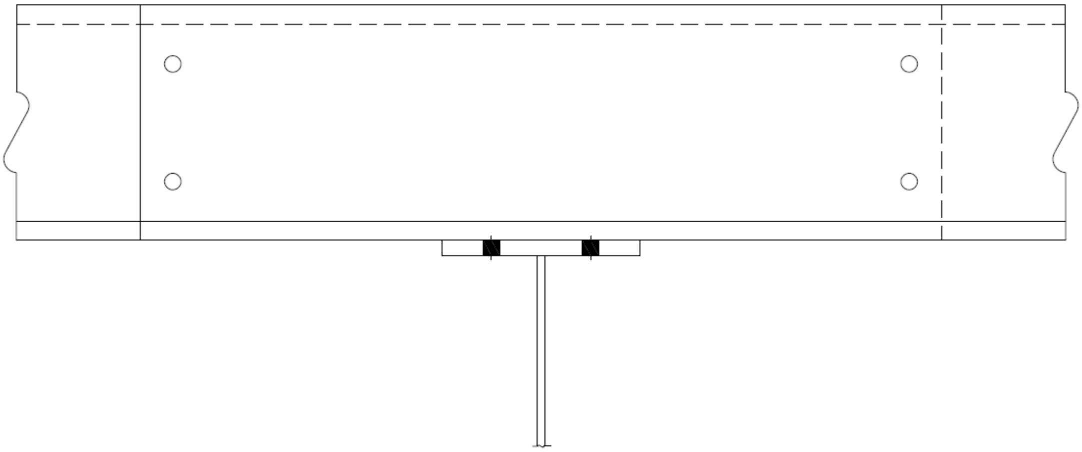


Figure 7: Typical Purlin Lap

Purlin and girt thicknesses can vary considerably within a roof or wall system; variable bay spacing can require different thicknesses. Also, the first two bays from the end will frequently have different thicknesses than the interior bays. Some manufacturers utilize reinforced purlins in the end bays or thicker purlins to transfer lateral loads from the endwall to the roof bracing system.

Web crushing or web crippling are the critical design factors at supports. Angle clips are normally used to prevent this failure mode (See Figure 8).

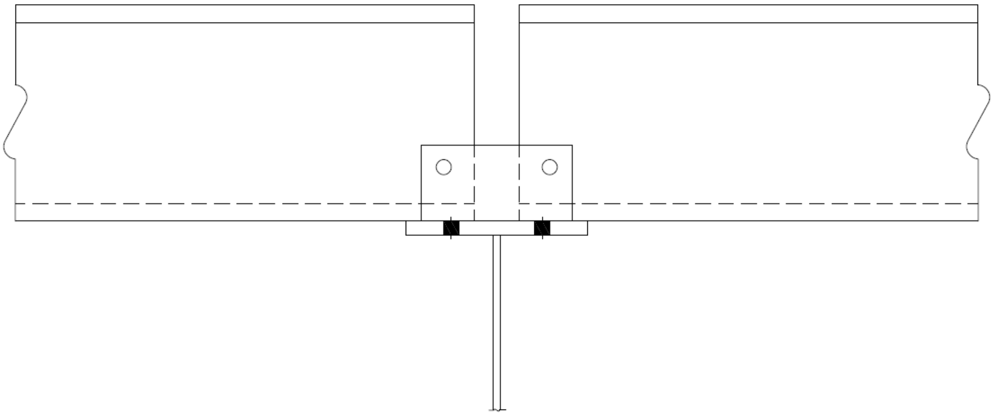


Figure 8: Angle Clips Used to Prevent Web Crushing or Crippling

At least one manufacturer fabricates roll-formed Z shapes with unsymmetrical flanges to enable them to nest more easily without angled stiffener lips (See Figure 9).

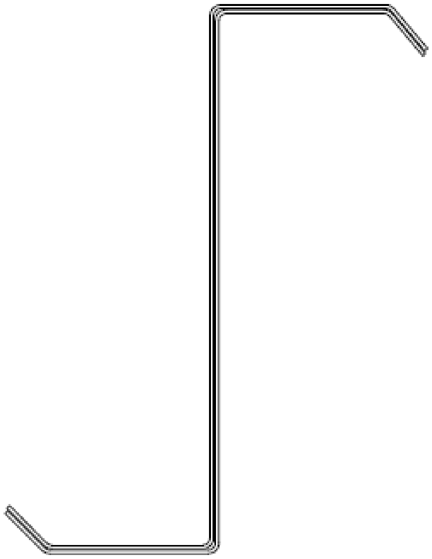


Figure 9: Unsymmetrical Z Purlin Flanges for Nesting

Bay spaces that exceed 35 feet usually require truss type secondary members in the roof system instead of cold formed Z or C shapes. Some manufacturers use standard open web steel joists and some may fabricate their own secondary truss members.

Metal Roofing

Metal buildings typically have a metal roof, either standing seam or through-fastened. Standing seam roofs utilize sliding or fixed concealed clips, that connect the roof panels to the purlins, and help to accommodate thermal expansion and contraction. However, a through-fastened roof is directly connected to the purlins, and thermal expansion and contraction is accommodated through the connection and the inherent flexibility in the roof system. EPDM washers are typically utilized with the fasteners in a through-fastened roof to help maintain weather tightness. Note that replacing a through-fastened roof with a standing seam roof has additional considerations and is not covered by this publication. Insulated metal panels, are also utilized, especially where this is preferred solution to provide energy efficiency.

Lateral Load System

While the primary structural system (moment frames) carries the lateral loads in the transverse direction, a separate system carries the lateral loads in the longitudinal direction. This utilizes longitudinal bracing in the form of x-bracing in some of the bays (sidewalls and roof) or portal frames in the sidewall. Figure 10 shows typical tension only x-bracing in a metal building. Longitudinal lateral loads on the end wall would be transmitted through strut-purlins that align with the x-bracing attachment locations. Strut-purlins may be heavier than the other purlins, multiple adjacent purlins or even a different cross-sectional member. In some cases, diaphragm strength/stiffness of the metal roof and/or wall is utilized.

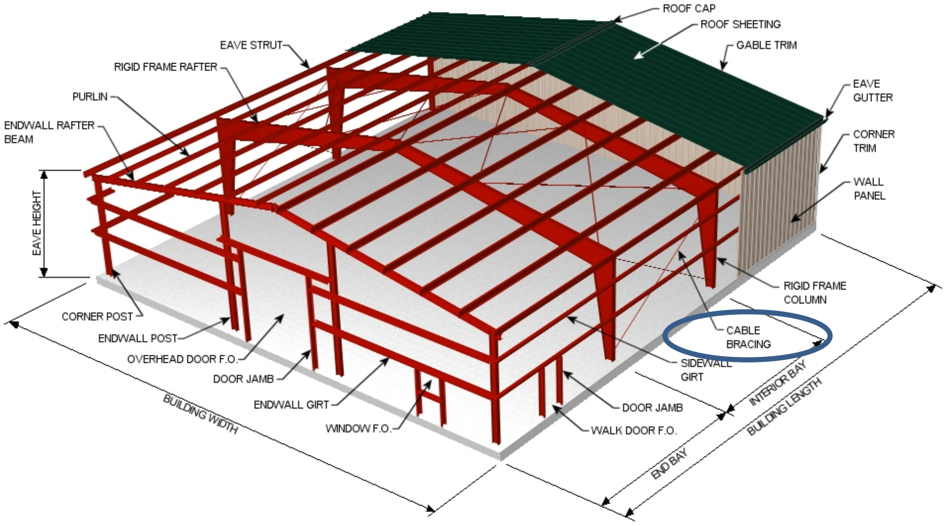


Figure 10: Typical X-Bracing for Longitudinal Lateral Loads

Preparation for a Retrofit/Reinforcing Project

When approaching a retrofit or a reinforcing project, it is essential to gather as much information as possible. The material properties and dimensions of the structure and all of its parts must be determined. The following steps can be followed to try to assemble this information:

1. Can you determine the name of metal building manufacturer? Most manufacturers have a logo somewhere on the building, typically on the ridge cap at either end of the building.
    a. Is the manufacturer still in business? The Metal Building Manufacturers Association (MBMA) website, www.mbma.com, may be of assistance. If the manufacturer is not listed as a member, contact MBMA to see if there is any information on the manufacturer. Sometimes another manufacturer may have acquired the project files in a merger or buyout.
    b. Can you determine an order number? Many manufacturers attach a plaque with the order number on one of the building columns or the order number is marked on many of the building parts. With the order number, contact the manufacturer to

obtain, if available, drawings, specifications and even structural calculations. Note that if an order number cannot be ascertained, the manufacturer might be able search their database using the building address.

2. Who was the building contractor? Is that company still in business? The contractor may have, in their file, the drawings and/or specifications that you need.
3. An additional source is the local building department. If the building is located in an incorporated area, there was probably a permit requirement and drawings and data may be in the building department files.
4. The last resort is field measurement. This is not uncommon in dealing with metal buildings. It must be kept in mind that frames will <u>not</u> necessarily be spaced uniformly, i.e. bay spacing can vary.
    a. Frame cross sections will vary over its length.
    b. Frame flanges will vary in width and thickness, inner and outer.
    c. Web thickness will vary over the length of the frame (columns and rafters). Web thickness can be determined at a hole in the web or use the depth measurement rod on the micrometer to measure from each edge of the flange and subtract to determine web thickness.
    d. Keep in mind when measuring that the paint or metallic coating thickness must be accounted for. The design is based on the base metal thickness, not the coated thickness.
    e. Purlin thickness may vary from eave to eave and from endwall to endwall. Some manufacturers use thicker members to transfer wind load reactions to the bracing system.

5. Without specifications, the steel properties need to be ascertained. If the building is newer than 1960, the frame material (webs and flanges) would most likely have been at least 50 ksi minimum yield strength but some manufacturers may have continued using 36 or 42 ksi beyond that time. Current designs are more likely to utilize 55 ksi minimum yield strength steel for the frame material; however heavy flanges (width/thickness greater than 12” x 1”) and heavy webs (thickness greater than 1/2”) may be 50 ksi due to availability. It is hard to generalize when individual manufacturers may have started making the transition to 55 ksi. However, buildings from the 1940s or 1950s, would probably have used frame material that is 36 ksi minimum yield strength. Steel material for cold formed purlins and girts follows a similar transition over the years. Prior to 1960, the minimum yield strength would likely have been 36 ksi. Between 1960 and 1985, a minimum yield strength of 50 ksi would have been common, and buildings more recent than 1985 would likely have 55 ksi minimum yield strength for cold-formed steel. However, a recent survey showed that a small number of manufacturers might still prefer 50 ksi steel for purlins and girts. If there is a question, or a need to verify the material, one or more samples should be removed and sent to a test lab.

Common Retrofit Projects

Before any decision can be made on the need to modify the building framing, the frame must be analyzed with the added loads or dimensional changes. The tapered members require that the software must allow the direct input of tapered members or a way to input multiple properties along the member length. Commercial programs are available that have these features.

Extending Length of Building

This is normally fairly easy if the site permits it. Frames and bays of secondary members can be added to the building. However, the end framing of the existing building must be reviewed as there are three possible framing conditions.

1. The endwall was designed as an "expandable" endwall, i.e. a rigid frame was designed for a future full bay load. Note that an expandable endwall would be designed for vertical loads and lateral loads in the plane of the endwall and that future changes to longitudinal loads from the new expansion are not considered. An expansion joint may be needed to eliminate the transfer of these loads into the existing building.
2. The endwall was designed as a post and beam endwall that requires diaphragm action of attached wall sheeting or bracing to support lateral loads.
3. The endwall is a rigid frame but designed to only a half bay load for the existing building.

In cases 2 and 3, the existing endwall can be moved to the end of the extension and a rigid frame designed for a full bay load put in its place. However, revisions to the footings will likely be required.

Extending Width of Building

The width of a building can be increased by the addition of a "Lean-To", which is basically a simple beam attached to the existing sidewall column (See Figure 4). The existing sidewall column, which will become an interior column, must be reviewed for several conditions:

1. The column must be checked for the additional axial load.
2. If the sidewall girts will be removed, the column may be deficient due to lack of lateral bracing. If the lack of bracing is critical, the column will require reinforcement. This is commonly done by welding plates to both flanges. The plates should be wider than

the flanges to maximize the radius of gyration about the weak axis; plates can be welded to the flanges using staggered stitch welding, but the width thickness ratio must be checked relative to local buckling (See Figure 11).

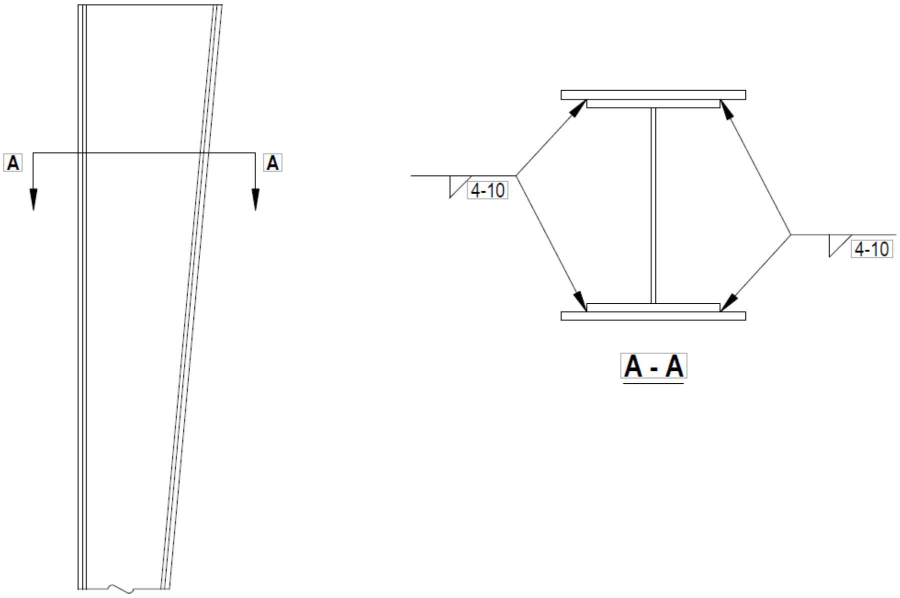


Figure 11: Reinforcement for Laterally Unsupported Column

Increasing Height of Building

In most cases this is very difficult, particularly with continuous purlins. However, with simple span purlins, the adjacent bay of panels and purlins are removed; the column to rafter beam splices on both frames are unbolted and using multiple cranes the two rafter beams with purlins and panels are lifted free and column sections of the desired length with splice plates to match the frame splices are inserted and bolted in place (See Figure 12). This is the repeated for the length of the building

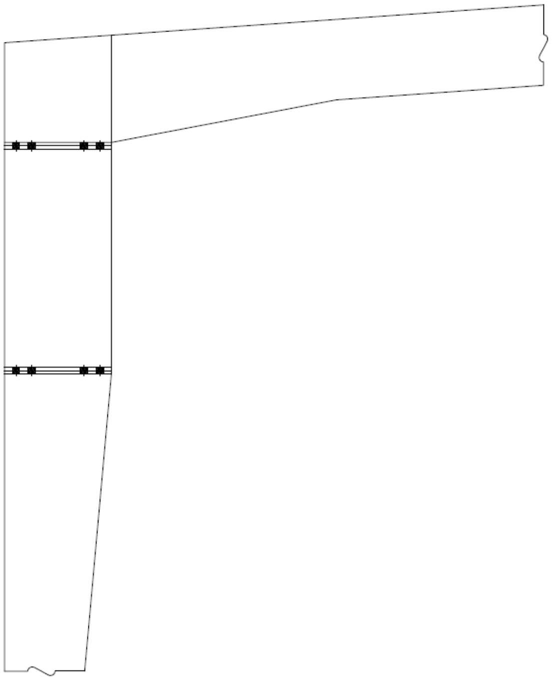


Figure 12: Column Insert to Increase Building Height

Addition of Equipment to Building Roof Structure

Several modifications might include loads from added equipment that will be supported by the roof system. In some cases, the loads will be suspended from below the roof system, and in other cases, the loads will be on top of the roof system. A common example of each situation is listed below.

1. Sprinkler System

   A sprinkler system can add approximately a 3 psf dead load that is suspended below the roof system. If the sprinkler system is suspended from the purlins, care must be taken in selecting hangars that do not distort the purlin lip/flange as discussed in a later section. Of course, the purlins need to be evaluated for the additional dead loads, as well as the primary framing.

2. Solar Panels

Solar panels are an increasingly important feature being added to existing building and new construction to achieve energy savings. The roof system, primary and secondary members must be reviewed for the additional imposed load. Solar panels are normally a minor additional dead load on a pounds per square foot basis – but if the frame or purlin are already close to their design load they should be checked. The method of mounting the panels needs to be reviewed with respect to the integrity of the roof and also any potential impact on a roof panel warranty. Also, the added solar panels can cause significant drifting snow as well as additional wind loads, including uplift, that need to be evaluated and can require strengthening of the roof. (see reference, *Snow Loads on Solar Paneled Roofs* by Michael O'Rourke).

Cranes

Cranes can be added or replaced with a higher capacity as part of a metal building retrofit. The following types of cranes may be used and each has specific design considerations when evaluating an existing building for the loads.

1. Bridge Cranes

Bridge cranes are a common addition or upgrade to metal buildings when a change in use occurs. Figure 12 shows a bridge crane mounted on brackets welded to exterior columns. The frame analysis will give the new moment and axial load on the column. Reinforcement with plate or mill channels welded to both flanges will normally be required. The channels have the advantage of increasing the radius of gyration for the critical buckling case.

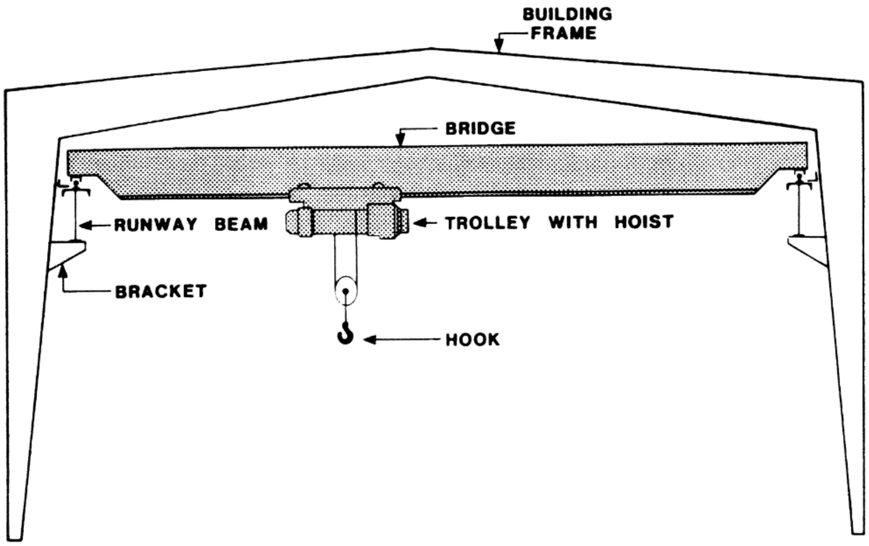
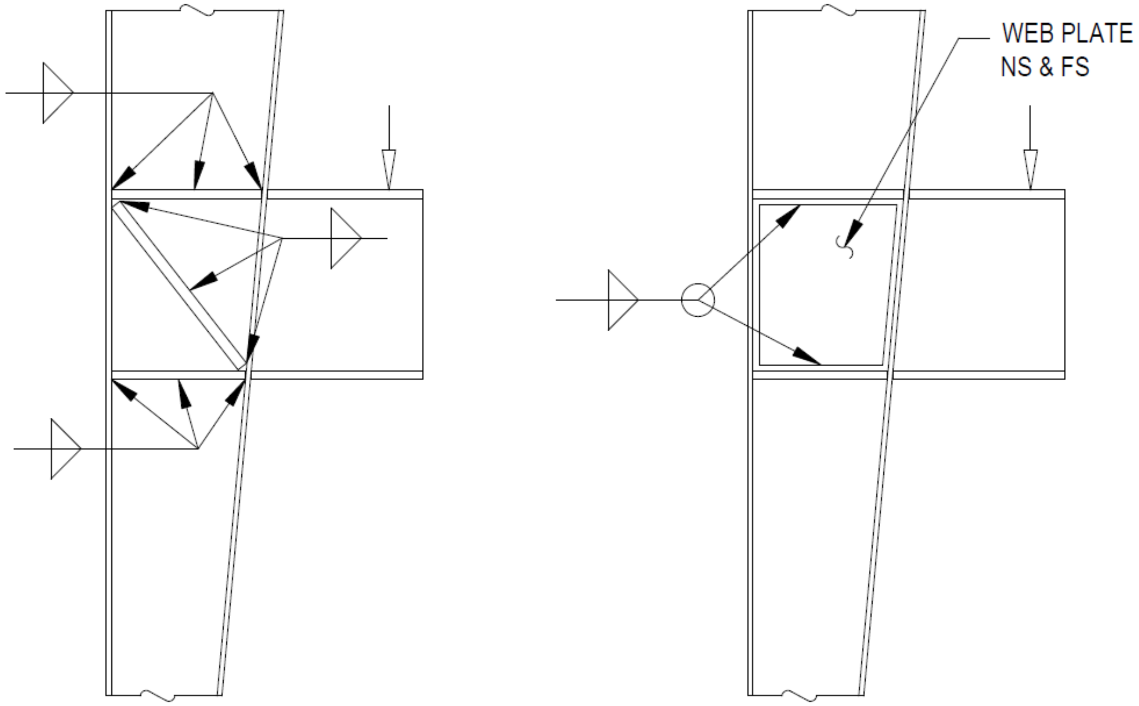


Figure 12: Top Running Bridge Crane Supported by Brackets on Building Frame

Brackets are normally fabricated from mill sections, but can be welded plate – the high shear dictates a thick web. The moment in the bracket results in high shear in the column web. The tension in the top flange of the bracket must be transferred through the column flange to a stiffener to avoid tension failure in the column web. A compression stiffener is similarly required opposite the compression flange of the bracket. In addition, the shear/moment capacity of the bracket can actually be limited by the weld design when considering fatigue.

Shear in the column web will still need to be checked. Even using tension field action, the web may be overstressed. Two solutions are possible as shown in Figure 13, (1) a doubler plate welded to the flanges and the stiffeners or (2) a diagonal stiffener – the stiffener is usually the more practical solution. Note that the web must be checked for shear below and above the bracket, there have been cases, where, under load the bracket rotated when a virtual hinge formed in the column due to shear failure in the webs.



(a) Diagonal Stiffener **(b) Web Doubler Plates**

Figure 13

An alternate solution to the bridge crane support is to add a column adjacent to the frame column as shown in Figure 14. The bridge crane beam is supported by the column rather than a bracket attached to the frame. A tie is welded between the frame and the top of the column and between the frame and beam supporting the crane. Lateral loads will be transferred to the frame. The column can be designed for axial loads only; cross-bracing is required between the columns to support the longitudinal loads from the crane.

In both of these methods to accommodate bridge cranes, the column reactions must be checked against the existing footings. Bridge cranes do not normally have a significant effect on the horizontal reactions. If the design roof load is equal or more than the crane load, the vertical reactions do not cause problems with the footings if the cranes are not operated when the roof load is at its maximum.

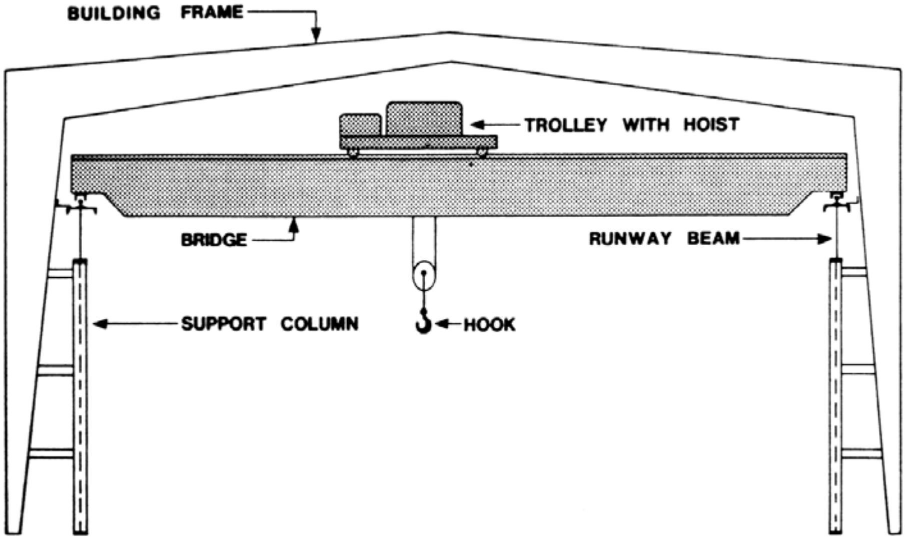


Figure 14: Top Running Bridge Crane on Support Column

2. Underhung Cranes

Underhung cranes that are suspended from roof rafter beams, as shown in Figure 15, can be a single or two-point support. Connections can be made to the lower flange, but refer to the discussion of concentrated loads in the next section. The saddle over the outer flange would be the preferred approach. For the lateral loads from the crane, bracing must be provided from the crane beam to the roof frame.

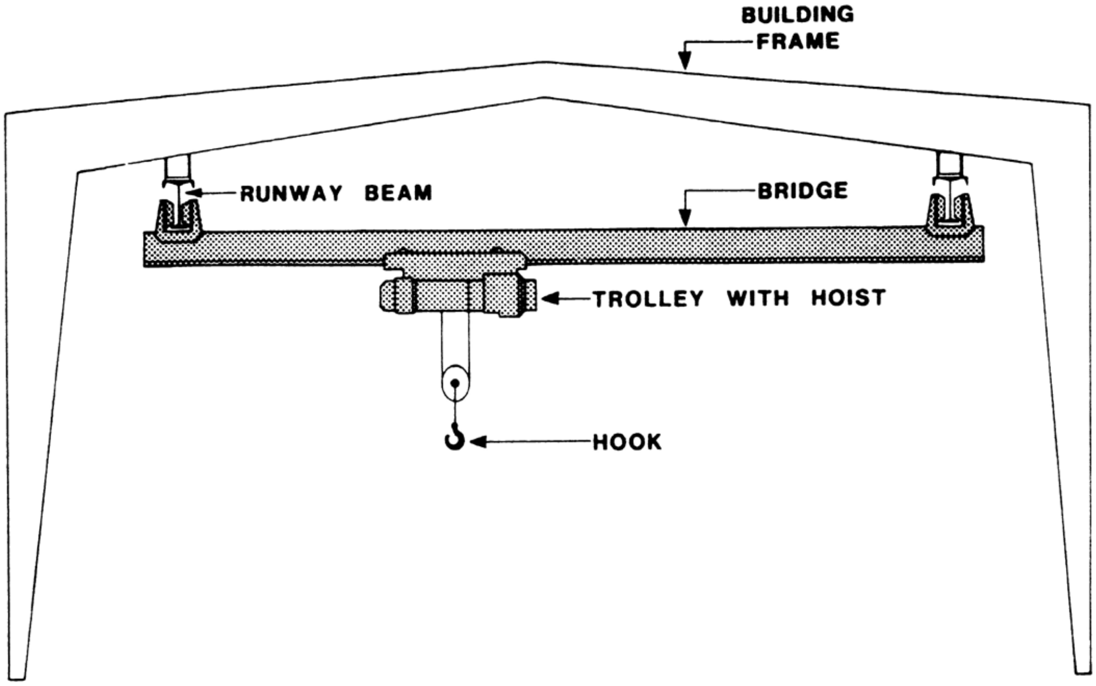
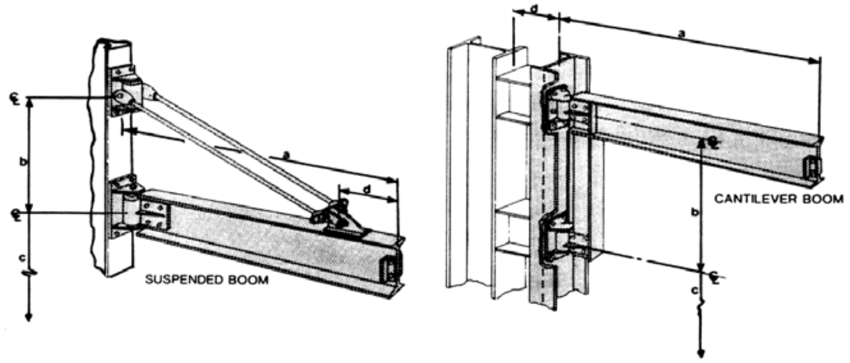


Figure 15: Underhung Bridge Crane

3. Jib Cranes

Jib cranes are mounted on one column and can be stationary or rotate on the vertical axis. One type of jib crane has a suspended boom as shown in Figure 16(a) which uses a tension rod or member attached to the beam and connected to the column above the beam connection. Another type of jib crane utilizes a cantilever boom as shown in Figure 16(b) and can use a supplemental column designed to attach to the existing column. Since most jib cranes can rotate, the support column must be reviewed for bending about both axes. This means most columns must be reinforced in the weak axis, which can be accomplished with a mill channel welded to the flange supporting the boom of the jib crane.



(a) **(b)**

Figure 16: Column Mounted Jib Cranes

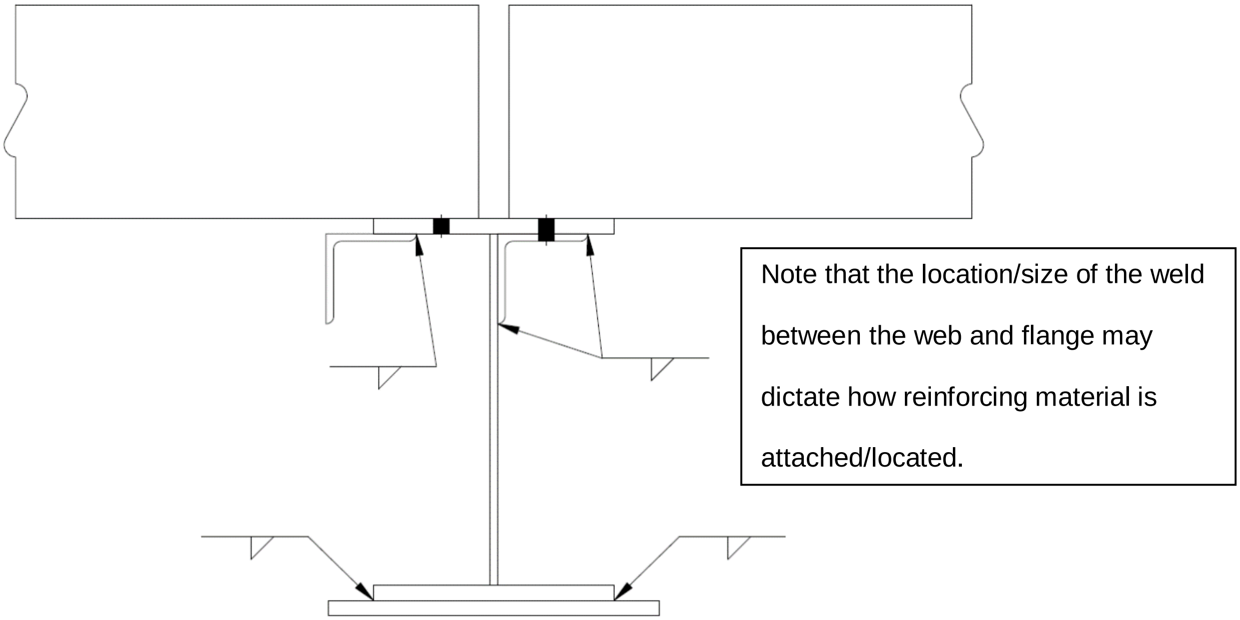
Structural Reinforcement

Primary Frames

Having analyzed the frame with the added loads, the sections must be reviewed to determine if the various sections of the frame are overstressed. Was the allowable bending capacity reduced due to changes in bracing? If so, adding lateral bracing is more economical than welding reinforcement to the flanges.

The usual objective in reinforcement is to increase the section modulus of the column or rafter section under review. This is most commonly done by welding cover plates to the flanges. In some cases, adding a plate to the inner (lower) flange will be sufficient (See Figure 17). This has the advantage of being able to weld in the "down" position for a rafter beam. However, there will be cases where the unbalanced section modulus between the outer and inner flange can result in the tension flange being overstressed.

When reinforcing the inner flange is insufficient, reinforcement must be added to the outer flange. The presence of purlins attached to the outer flange of rafters makes this difficult. One alternative is to weld angles to the flange and web (See Figure 17). Another alternative to the angle locations shown in Figure 17 would be to use angles to create a “box section” with the web and top flange on both sides. Note that in all cases where material is added to the cross section, the yield strength of the reinforcement controls the design, e.g. if the only plate material available has a yield strength of 36 ksi, the allowable stress in the design check must be based on that material.



Note that the location/size of the weld between the web and flange may dictate how reinforcing material is attached/located.

Figure 17: Weld Angles to Reinforce the Outer (Upper) Flange and Cover Plate on Inner (Lower) Flange

In some cases, where the frame is severely overstressed, an economical solution may be to add an interior column if the floor plan can accommodate it. There could be a wall that would allow the addition of a column without interfering with the function of the space. In this case the rafter web over the column must be checked for crushing and shear and in most cases a stiffener would be required. It is very important to ensure that adequate lateral support is provided for the rafter bottom flange at the column connection. In addition, the column support must be checked. The floor slab may have sufficient strength to support the column load, especially if it was reinforced, and the base is spread to distribute the load, otherwise a separate footing must be provided.

When concentrated loads are involved such as for underhung cranes, or isolated hanging loads, the distribution of the concentrated load into the web, as well as shear in the web, must be checked. A web stiffener is typically needed to distribute the load into the web. With regard to shear strength, AISC 360-16 introduced changes in Chapter G to recognize post-buckling shear strength and more situations where tension field action can be considered. But, if the existing shear capacity is still insufficient, a web doubler can be added over the existing web and welded to both flanges to increase the shear capacity.

An alternate method to handle concentrated loads is with a saddle member on the outer flange (See Figure 18). In this case the web must be checked for crippling and a stiffener added, if necessary, and the web must be checked for shear as in the previous case.

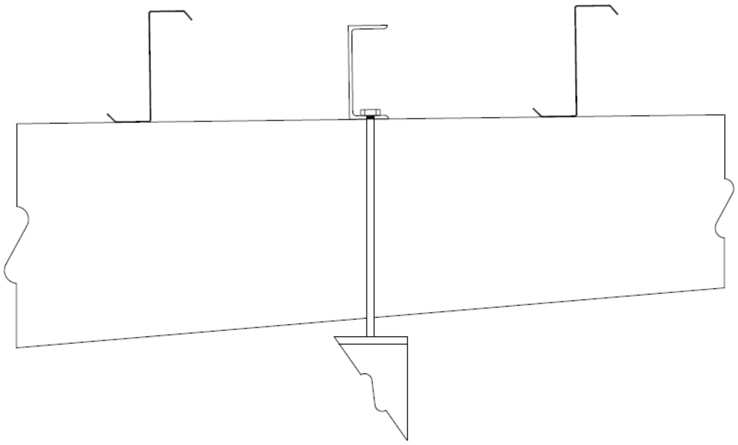


Figure 18: Saddle Member on the Outer Flange

Splices in primary frame members must also be checked for the new moment and thrust conditions. Metal building frames commonly use two types of splices, the classic cover plate type (See Figure 19) and the end plate moment splice (See Figure 20).

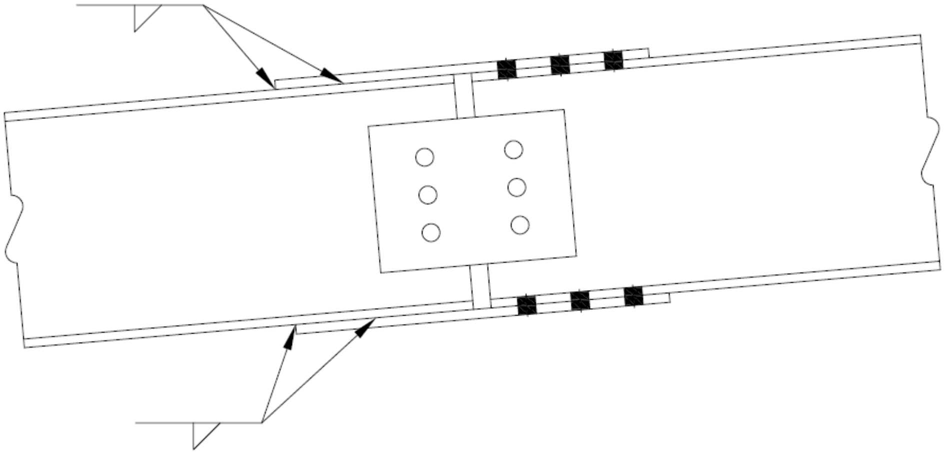


Figure 19: Cover Plate Splice

The cover plate splice (See Figure 19) uses plates welded or bolted to the top and bottom flanges with the moment carried by shear in the bolts or welds. In some cases, the cover plate splice would have been designed for less than the section capacity due to the location of the inflection point (this may change under the new loads). Reinforcement, if required is fairly straight forward; with the cover plate design weld can be added and/or higher strength bolts can be substituted or holes can be enlarged and larger size bolts used. The net section area needs to be evaluated for adequacy. Cover plate area can be increased by added plates or angles welded to the existing plates. The web splice can be reinforced by replacing the plates and/or increasing strength or size of the bolts.

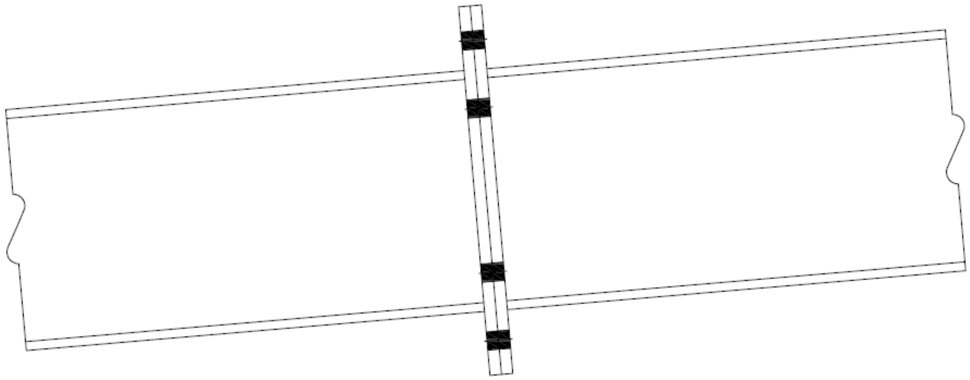


Figure 20: End Plate Splice

End plate splices (See Figure 20) are more common in metal buildings and many manufacturers use this type splice exclusively. With an end plate splice, the moment is carried by the bolts in tension. The end plate can be the same depth as the beam/column (flush end plate) or the end plate can extend beyond the depth (extended end plate) as shown in Figure 20. An extended end plate can be unstiffened, or the end plate can be stiffened with gusset plates in line with the beam/column web. The splice must be analyzed to determine if its components are overstressed. There are two AISC Design Guides for end-plate moment connections, (1) Design Guide No. 4 Extended End-Plate Moment Connections Seismic and Wind Applications (2003), 2nd Edition and (2) Design Guide No. 16 Flush and Extended Multiple-Row Moment End-Plate Connections (2002). Using the bolt loads resulting from the new moment and axial load on the splice, the necessary design checks can be made.

1. <u>Bolts</u>: If the bolts are overstressed, A490 bolts can replace the A325 bolt which are normally used. Otherwise, the holes can be reamed and larger bolts substituted. Note that A490 bolts are not permitted to be snug-tightened and would have to be fully tensioned. Alternately, F2250 twist off bolts (equivalent to A490) can be used.

2. Webs: The webs adjacent to the bolts must be checked and the gusset, in an extended stiffened connection, must be long enough to transfer the tension into the web along the length of the gusset.
3. Welds: The welds between the end plate and web must be sufficient to transfer the bolt load to the web, or to the gussets in an extended stiffened connection.
4. End plates: The endplates must be thick enough to transfer the loads to the rafter beam or column without being overstressed in flexure or shear. As in the case with webs, the most practical reinforcement is addition of welded gussets to stiffen the plate. For tension stress, stiffeners or gussets can be welded to the end plate and the web.

Secondary Members

Reinforcing the secondary system can be accomplished by adding purlins between the existing rows of purlin or reinforcing the existing purlin. Any work on the secondary system can be complicated by access problems. Ceilings, multiple partitions, HVAC equipment, and electrical systems can present serious access problems. Attempting to use continuous purlins between the existing rows is problematical and probably not feasible in many situations. Adding simple span purlins to a continuous purlin system will result in differential deflection problems unless the simple span purlins are much stiffer then the continuous purlins. Purlins inserted between existing rows of purlins are, in most cases, not attached to the roof panel and lateral support must be provided such as braces between the purlin webs as shown in Figure 21.

Figure 21: Braces Between Purlin Webs

One of the most common retrofit problems is concentrated loads on purlins. The list of possible loads is endless, lighting, piping, hoists etc. Concentrated loads on a single purlin should be avoided. Spreading the load to multiple purlins can be accomplished by a support member attached to the bottom flange of several purlins and is more economical than reinforcing a single purlin. Devices connecting to the stiffener lip of the purlin should not be used to support piping or other utilities (See Figure 22). These connections cause excessive twist to the section. There are commercial devices that attach to the purlin web which is the correct way as shown in Figure 23.

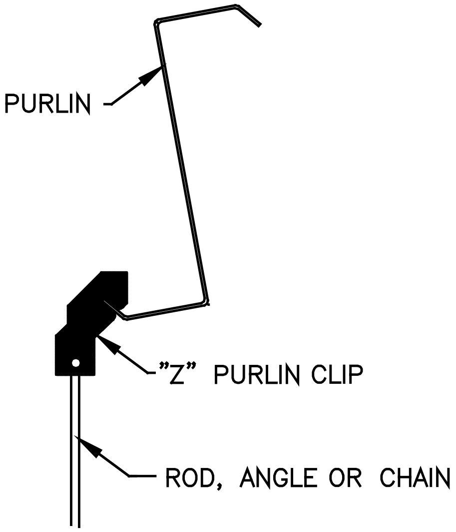


Figure 22: Incorrect Way to Hang Loads on Purlin

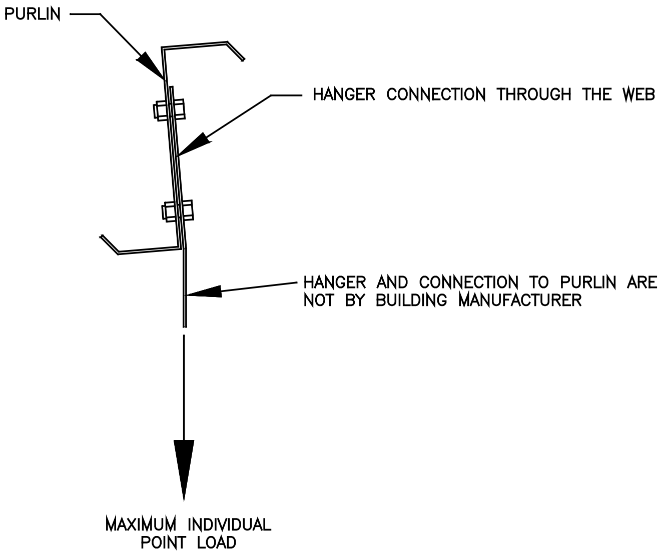


Figure 23: Correct Method to Hang Loads on Purlin

Simple span purlin design is controlled by lateral torsional buckling at mid span and/or web buckling/crippling at the end support. At the support, a clip can be added if there is no clip or the existing clip can be replaced with one that has more capacity. At midspan, the purlin can be reinforced with angles or channels welded, bolted or screwed to the web (See Figure 24). When using self-drilling screws, the reinforcing part should be predrilled/punched so that the screw is drilling through only one web.

Figure 24: Angle Reinforcement for Simple Span Purlin

Continuous purlins must be checked for all of the applicable limit states in AISI S100 including, (1) flexural torsional buckling at midspan or the point of maximum positive moment, (2) combined shear and bending at the end of the lap and (3) web buckling/crippling at the support. For in span reinforcement, the same method as with simple span purlins can be used. At the support a reinforcing clip can be added. The end of the lap failure situation can be eliminated by moving the end of the lap. A channel can be bolted to the web and extended past the end of the existing splice to a point where the combined shear and bending is acceptable. New holes must be drilled for the new splice bolts; oversize holes must be pre-punched for the reinforcing member to fit over the existing bolt heads (See Figure 25).

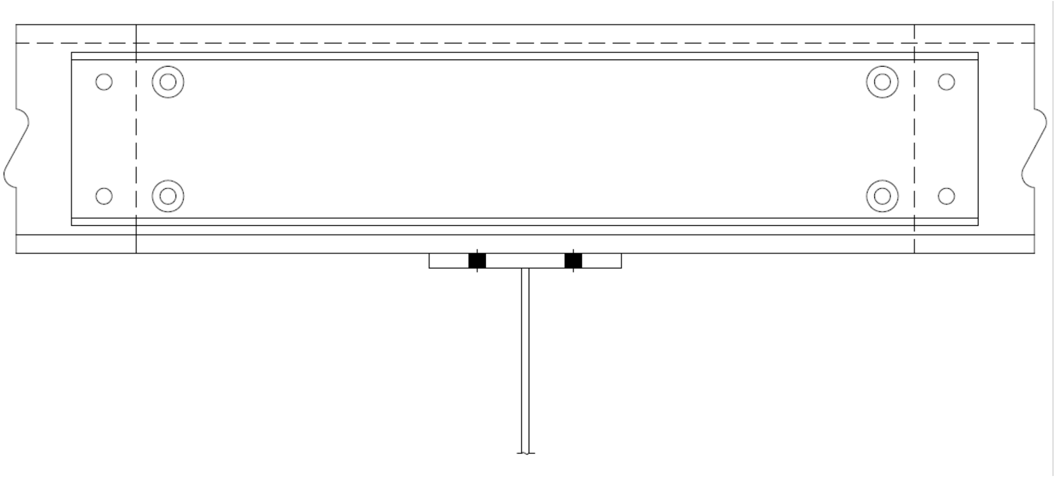


Figure 25: Channel Reinforcement for Continuous Purlins

Metal Roofing

The EPDM washers used with a through-fastened roof may need to be replaced with aging or elongation of the fastener holes that occurs with normal thermal expansion and contraction. If a screw is replaced in the same location, the next largest diameter should be selected. For example, if a #12 diameter screw is replaced, it should be done with a ¼" diameter screw. If the hole in the roof panel is elongated significantly, a screw with an "oversized" sealing washer may be required.

Metal roofs have been documented to perform well and have a long service life. However, when an existing metal roof does reach the end of its service life, or is experiencing problems, it can either be repaired, reroofed, or retrofitted. There are several resources available that discuss available options that are listed in the Resource section of this paper.

Conclusion

A metal building system can have several functional changes over its lifetime or other need for structural modification. This paper provides some ideas and insight that should prove useful information for engineers who are involved in a metal building upgrade or modification. The following section provides details on resources that are available.

Resources

1. *Metal Building Systems Manual*, 2018, Metal Building Manufacturers Association, Cleveland, OH.
2. *Guide for Inspecting Metal Building Systems*, 1st Ed., 2016, Metal Building Manufacturers Association, Cleveland, OH.
3. *Seismic Design Guide for Metal Building Systems (Based on 2015 IBC)*, 2019, Metal Building Manufacturers Association, Cleveland, OH.
4. Kaehler, R. and White, D. (2011) *Frame Design Using Web-Tapered Members*, AISC Design Guide No. 25, American Institute of Steel Construction, Chicago, IL.
5. Murray, T. M. (1990) *Extended End-Plate Moment Connections*, AISC Design Guide No. 4, American Institute of Steel Construction, Chicago, IL.

6. Murray, T.M. and Shoemaker, W.L. (2002) *Flush and Extended Multiple Row Moment End-Plate Connections*, AISC Steel Design Guide No. 16, American Institute of Steel Construction, Chicago, IL.
7. *Cold-Formed Steel Design Manual*, 2017 (Includes AISI S100-16 Specification and Commentary), American Iron and Steel Institute, Washington, D.C.
8. Retrofitting Metal Roof Resources
    a. Gardner, Brian, *Comparison of Retrofit Systems Over Existing Metal Roofs*, White Paper for the Metal Building Manufacturers Association, Cleveland, OH (www.mbma.com/media/MetalRoofRetrofits.pdf)
    b. Nelson, D., Sagan, V., and Shoemaker, W.L., *Metal to the Metal, Re-covering Existing Metal Roof Systems Requires Diligence*, Professional Roofing, National Roofing Contractors Association, Chicago, IL. (http://www.professionalroofing.net/Issues/Contents/194)
    c. Sagan, V., *Comparison of Metal and Non-Metal Roofing Systems*, presented at the 2018 Building Enclosure Science and Technology Conference (BEST5), Philadelphia, PA. (https://www.brikbase.org/content/comparison-metal-and-non-metal-retrofit-systems)
    d. *Retrofit Metal Roofing*, 2019, Metal Construction Association, Chicago, IL.
9. Suggested Software
    a. CFS, Cold Formed Steel Design Software, RSG Software, Inc., Lee's Summit, MO. (https://www.rsgsoftware.com/)
    b. MBS, Metal Building Software, Inc., Fargo, ND. (http://www.mbsweb.com)

Acknowledgement

Donald L. Johnson, P.E, was consultant and contributing author for this publication. Don retired from Butler Manufacturing in 1996 as the Buildings Division Chief Engineer, after a 44-year career with Butler and he currently resides in Wolfeboro, NH. He served two terms as Chairman of the MBMA Technical Committee. He is a wealth of knowledge and his input to this publication was invaluable.