REINFORCEMENT DESIGN FOR METAL BUILDING SYSTEMS

By Donald L. Johnson, P.E.

RETROFIT PROJECTS CAN BE NECESSARY FOR ANY NUMBER OF REASONS, though change in use is one of the most common. Change of use can involve equipment loads added to roof, or the addition of sprinkler systems or cable trays. In some cases, buildings need to be expanded in length, width or height.

Understanding the original construction is a critical part of any retrofit or reinforcement project and Metal Building Systems are no exception. Metal Building Systems are characterized by a primary framing system of rigid frames fabricated with bar or plate flanges and tapered webs cut from sheet or coil. The flange-to-web weld is usually made on one side, except when the shear magnitude requires welding on both sides.

Single-span rigid frames can range from 20' to over 200' clear span. "Modular Rigid Frames," rigid frames with interior columns, can be built to almost any width and 800' to 1,200' is not uncommon (see figures 1 & 2). Roof slope will vary with the building end use, though the most common slopes are ¼:12 and ½:12. Slopes of 2:12 and 4:12 are frequently used on recreational facilities and 6:12 and up are used on some church buildings.

The secondary system usually consists of "Z" shaped purlins or in some cases "C" shaped purlins and a metal roof. The purlins are roll formed from continuous coil

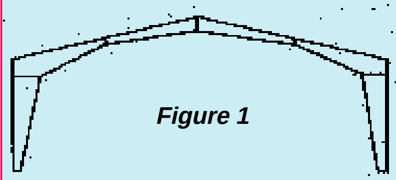

Figure 1

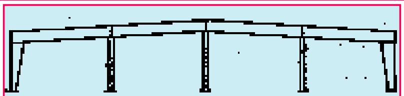

Figure 2

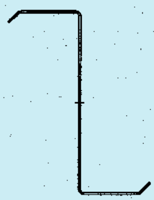

Figure 3

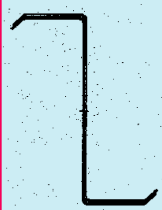

Figure 4

material in thickness from 0.04" to 0.14". Depths can range from 4" to 16", with 8" to 10" the most common. "Z" sections will normally have 45 degree stiffening lips while the "C" sections use 90-degree lips. The 45-degree lips are to permit nesting when used in continuous systems (see figures 3 & 4).

Spacing of the primary frames is a function of the secondaries. Simple span purlins are effective up to 25' while 30' and 33' bays require continuous purlins. Longer spans are accomplished with truss type members fabricated from cold-formed parts or with conventional bar joists.

Flange and web material in Metal Building System rigid frames will in most cases have a minimum yield of 50 ksi. The exception is older buildings (manufactured before 1960) that usually used 33/36 ksi material. Purlin and girt material can be expected to be 50 or 55 ksi yield material unless the building was manufactured prior to 1958. If there is doubt concerning what material was used, coupons should be taken and tested.

Before starting a retrofit project, a designer should obtain as much information as possible. Considerable data can be obtained from the manufacturer if it can be identified. So called "standard" buildings have not been produced for at least 20 years; design to order has been the standard procedure. This means that without an order number, useful data will be difficult to obtain. The building order number and the name of the manufacturer can be obtained from the owner if the original documents were retained. Other sources would be the dealer who sold the building and the local building code office, where documents would have been filed for permit purposes. In some cases, the order number can be found on frame parts.

With the order number and the name of the manufacturer it is often possible that the manufacturer can supply drawings and design calculations, which will greatly simplify the designer's job. If the manufacturer data is unavailable, the only recourse is field measurement.

LENGTH MODIFICATION

Extending the length of a metal building is very easy if the site permits it. Additional units of frames and secondary members identical to the original can be added to the building. The end frames of the original building must be checked. If expansion were contemplated in the original design, the end frame would be the same design as the interior frame, with no modifications necessary. In the case of a "non-expandable endwall," it can be replaced with an intermediate rigid frame and the existing endwall parts can be dismantled and moved to the end of the new addition.

Non-expandable endwalls are commonly beam and post construction. If the endwall footings were designed for this condition, they will not be suitable for an intermediate rigid frame, which can have a significant horizontal reaction. If this is the case, the footings will have to be replaced. The other option would be to reinforce the existing endwall. The footing must be checked for the increased reactions.

WIDTH MODIFICATION

Expanding a metal building laterally is relatively simple with use of a width extension (figure 5). Width extensions (a lean-to type structure) are available from virtually all manufacturers.

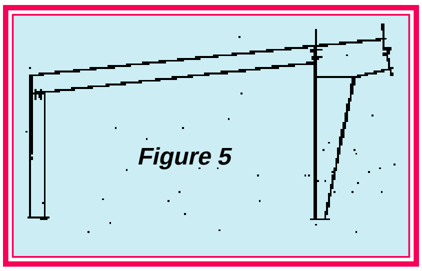
Figure 5

It is preferable to obtain the width extension from the same manufacturer who provided the original structure in order to match panel profiles, otherwise transition flashing will be required. The original wall condition must be addressed. If the sidewall is left intact with secondary and paneling, the only thing that must be checked is additional axial load in the building column. If, however, the secondaries are removed, the column length increases from approximately 7.5' (in many cases) to the full length of the column. With the increased unsupported length, the column will be seriously deficient with or without the additional load. Normal reinforcement consists of plates welded to both flanges. If possible, the plates should be wider than the column to increase the flange radius of gyration. Staggered stitch welding of the cover plates is satisfactory but the width thickness ratios must be checked to prevent local buckling.

HEIGHT MODIFICATION

The height of a metal building system is the most difficult dimension to change. It can be done structurally; however, the associated problems in raising the roof panels and purlins can make the project impractical based on cost. If simple span purlins were used, the roof system can be dismantled a bay at a time, the roof beams raised and the roof replaced.

The bending moment at the rigid frame knee (column-to-roof beam joint) does not change significantly when the height is increased. The bending moment at the ridge or centerline of the frame will need to be reviewed for possible reinforcing.

Two methods of increasing height have been used. The first is to fabricate a new column with a cap plate that matches the splice plate on the roof beam. The second is to fabricate a stub column with a uniform depth and cap plates on each end to match the existing splice plates.

LOADS

There may be the case where an entire structure is required to be retrofitted to a higher load; perhaps the original specification were incorrect or insurance rules may require a higher capacity. To significantly increase the overall capacity of an entire Metal Building System, virtually every member in the structure would require reinforcing. Assuming the load increase is an overall live or snow load increase, there are a few cost effective options.

• **Roof system:** Install additional purlins between the existing purlin. The addition of continuous purlins is usually not practical due to clearance problems, however, simple span purlins of higher stiffness can be installed and effectively double the roof capacity. Care must be taken to provide sufficient lateral bracing to the added purlins, particularly if they are not attached to the roof panel. The stiffness of the roof panel normally provides lateral support to the purlin. In some standing seam roof systems additional braces may be used. If attachment of the purlin to the roof panel is impractical, spacing of discrete bracing can be computed using AISI Section C3.1.2

Another method of increasing the capacity of the roof system is to install an additional frame between the existing rigid frames. This will reduce the purlin span by half. The frame can be other than a rigid frame. A post and beam frame simplifies the foundation requirements, but requires intermediate columns. The purlins will be continuous over the added frame and the reaction will be supported by a single web. This condition will need to be checked for web crippling using AISI Section C3.4, combined shear and bending using AISI Section C3.3, and combined bending and web crippling using AISI Section C3.5.

• **Primary Framing:** The use of added intermediate frames will double the capacity of the primary framing as well as increasing the capacity of the purlin system. These method doses not require any modification to the existing rigid frames.

If intermediate columns can

be made compatible with the usage of the building, an interim column (or columns) can be added to the rigid frames. This method will frequently result in less reinforcing than required for a clear span. There will be moment reversal at the new column locations and bearing stiffeners will be required at the reactions, but the overall reduction in bending moments will reduce the amount of reinforcement.

Analysis

Before a designer can determine the reinforcement details for a Metal Building System rigid frame, an analysis must be made for the required code loadings plus new imposed loads. Before an analysis can be done, the frame dimensions and material sizes must be determined using information from the manufacturer or from field measurements. When taking field measurements it should be remembered that some manufacturers use flange segments as short as 5' and weld seams in the web can be overlooked unless a careful inspection is made.

A number of commercially available computer programs can perform the necessary analysis. Some packages have direct input of tapered members, but even the most basic program can be used by dividing the members into 5' segments and inputting the section properties for each segment. With tapered members, the critical sections cannot be determined by inspection. The objective, when using tapered members, is to minimize material usage by matching each section to the moment and shear at that point. Therefore all sections with increased moment or shear over the original design must be reviewed for possible reinforcement

Frame Reinforcement

• **Flange reinforcement:** The main objective of flange reinforcement is to increase the section modulus of the section. This can be accomplished by adding cover plates. Welding a plate wider than the original inside flange permits the welder to weld in the "down" portion rather than overhead and increases the radius of gyration of the flange. In some cases reinforcement of one flange is all that is necessary. However, the un-reinforced flange can be over stressed due to the neutral axis location. In this case both flanges must be reinforced. Where the inside flange normally has few obstacles to installing a cover plate, the outside flange has purlins attached at approximately 5' for its entire length. One method utilizes a pair of hot rolled angles welded to the inside of the outer flange and the web (See Figure 6).

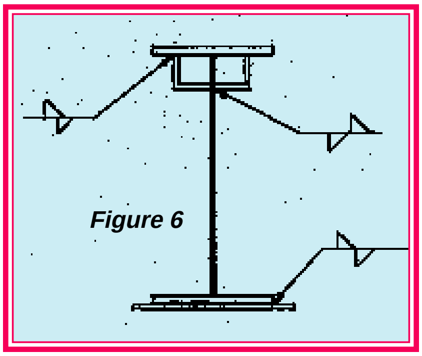

Figure 6

• **Web reinforcement:** Addition of concentrated loads will frequently result in overstress of the web. Rigid frames supporting roof loads are basically flexural members resulting in relatively deep members. Although the use of tension field is not common, frames with stiffeners to utilize tension field action may be encountered. The majority of rigid frame webs will have been designed per AISC Section F4 without the use of intermediate stiffeners. The use of AISC Section F4 results in relatively low values of allowable shear stress and the allowable shear can be doubled or even tripled with the addition of intermediate stiffeners welded to the webs. This is an economical and quick way to solve a web overstress problem. If the web shear exceeds that given by formula F4-1 or $0.4F_y$, the only solution is more area; i.e. doubler plates welded over the existing web.

• **Flange to web welding:** Most Metal Building System rigid frames will have one side welding which works quite well under the low shear from a normal roof load. This is not satisfactory when localized concentrated loads are imposed on the section. Several checks must be made at a section supporting a concentrated load.

1. Is the flexural strength satisfactory?
2. Can the web carry the shear without reinforcement?
3. If the load is attached to the bottom flange, can the web carry the load in tension and the weld must be checked for horizontal shear and shear normal to the weld?
4. If the load is carried by the top flange, can the web carry the load in bearing.?

Overstress due to conditions in 2 through 4 can be eliminated by a combination of added flanges to web welds and/or the addition of a bearing stiffener at the load point. Shear in the web may require the addition of more than one stiffener.

Splices

Metal building rigid frames use splices of two types, the classic cover plate type and the end plate moment splice. The end plate moment splice is the most common.

The cover plate splice uses plates welded or bolted to the top and bottom flanges with the moment carried by bolts or welds in shear. (See Figure 7). The shear on the section is carried by shear plates bolted to the webs. In some cases, the splice is designed for considerably less than the capacity of the section, due to the location of the splice relative to the inflection point. Reinforcement of this type of splice is relatively simple. Welding of the plates to the flanges can increase the capacity or the bolts can be replaced with bolts

of a higher strength. If the cover plates are too small to carry the tension, supplemental plates or angles can be welded to the under side of the flanges. The web splice can be reinforced by replacing the plates and/or bolts with higher-grade bolts and thicker plates.

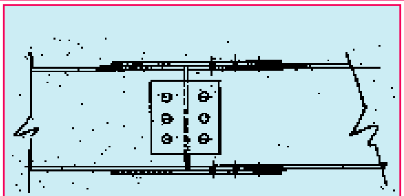

Figure 7

End plate moment connections are more common in rigid frames and many companies use this type splice exclusively (See Figure 8). In retrofitting this type of splice, the bolts, welds, webs and splice plates must be reviewed. A number of different design procedures have been used in the analysis of end plate connections. The AISC Manual contains one method; the "AISC Steel Design Guide #4, Extended End-Plate Moment Connections" is an excellent reference. For retrofit, I have used a procedure where the moment of inertia and neutral axis is computed for a section composed of the bolt areas in tension and the beam compression flange plus a portion of the web in compression. The bolt loads can then be computed using simple flexural theory. Given the bolt loads resulting from the new moment and axial load on the section, the design checks can be made.

1. **Bolts** - If the bolts are overstressed, an A490 bolt can replace the A325 bolts which are the standard. In extreme cases the holes can be reamed and larger bolt diameters substituted.
2. **Webs** – The webs adjacent to the bolts must be checked for tension stress and if the web is overstressed, a stiffener or gusset can be welded to the web and end plate. The gusset must be long enough to transfer the tension from the bolt into the web along the length of the gusset.
3. **Welds** – Web to end plate welds must be checked for the same load that the bolts were transferring into the web. If the web is not overstressed, the welds can be increased in size or a gusset can be added to reduce the load to the weld.
4. **End Plates** – The end plates must be thick enough to transfer the load from the bolts to the beam or column without being overstressed in flexure or shear. As in the previous cases, the most economical solution to the overstress is to add gussets to stiffen the plate.

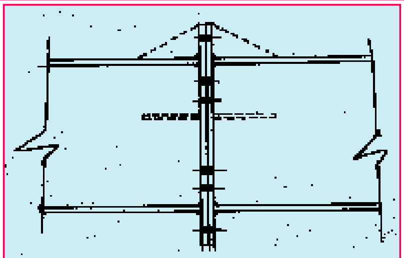

Figure 8

PURLIN REINFORCEMENT

Concentrated loads are a common addition to any building. The list of items that could be hung from the roof is endless; most concentrated loads are supported from a single point and this can be a problem. Point loads on cold-formed purlins should be avoided unless they are quite small in magnitude. Before adding load to a purlin it is necessary to know it's capacity. Most roofs will have been supplied with some amount of collateral load included. A simple rule of thumb to determine what the allowable concentrated load might be is to take the excess uniform load capacity, if there is any, and find the equivalent concentrated load. A 25' purlin with a 3 psf excess capacity on 5' centers has a total load capacity of 5 x 25 x 3 = 375 pounds or a concentrated load of half of 375 or 187 pounds. An additional problem is the method of hanging the load. Many installers make the mistake of

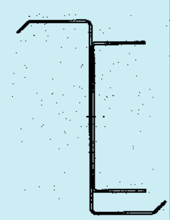

Figure 9

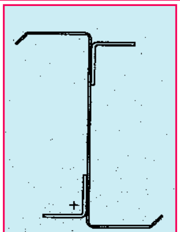

Figure 10

Table 1

Member	Depth (Inches)	Width (Inches)	Lip (Inches)	Thickness (Inches)	Area (Sq. Ft.)
Z purlin	9.0	3.0	0.75	0.08	1.285
C Reinf.	7.0	2.5	0	0.10	1.167
Angle Reinf.	2.0	2.0	0	0.15	0.56

Table 2

Member	Area	Allowable Moment
Unreinf. “Z”	1.285 sq. in.	89.4 in. kips
“Z” Purlin plus “C”	2.45 sq. in.	135.3 in. kips
“Z” Purlin plus 2-Angles	2.41 sq. in.	182.87 in. kips

using proprietary clamping or hanging devices that damage the stiffening lip and or flange. There are devices on the market that allow an attachment to the web of the purlin, which is the proper way to support the load.

If the concentrated load exceeds the capacity of a single purlin, the load can be spread to several purlins or the purlin can be reinforced. Spreading the load is more economical than reinforcing and can be accomplished by attaching a reasonably stiff member to the bottom flange of two or more purlins.

Several methods can be used to reinforce purlins. With simple span purlins, two areas must be checked, flexural stress in the span and shear and crushing at the support.

In span reinforcing: a channel can be attached to the web or angles can be fastened near each flange (See Figure 9 and Figure 10). Attachment of the reinforcing members to the purlins can be done in several ways, welding, bolts or screws. Welding is a good structural solution, but welding to the thin materials can be difficult for a welder not experienced in this type of welding, particularly in the positions that would be encountered. Bolting is most easily done by clamping the members in place and drilling all the parts in place. Self-drilling screws are the most practical fastener, although there will be a larger number of screws than would be required if bolts were used. The member being added should be predrilled or punched with the required number of holes so that the screw will only be drilling through the purlin material. Spacing of the fasteners can be calculated using shear values obtained from the screw manufacturer or from AISI Section E4.3. As an example of the effect of typical reinforcing of a generic 9 inch purlin, 0.08 inches thick, refer to Tables 1 and 2. Table 1 lists the properties of the basic purlin a channel reinforcing member and an angle that can be used for reinforcing, as illustrated in Figure 9 and 10. Table 2 lists the allowable moment capacity (assuming member is fully braced) for the un-reinforced purlin and the purlin with the reinforcing members attached. In this case the top and bottom angles was the minimum material solution for the maximum moment capacity.

Support reinforcing: shear is usually not a problem but should be checked. Web crippling is frequently a problem and should be checked with Section C3.4 of the AISI Specification. If the purlin does not have a clip at the support, one can be added. If the existing clip is inadequate, a stronger clip can be installed. A hot rolled angle is usually the most expeditious solution (See Figures 11 and 12).

With continuous purlins, the problem is compounded by the purlin splice. Splice length can be (on each side of the frame) as little as 6 to 8 inches or as much as 36” to 48”. The review of a cold-formed continuous purlin system can be a challenge to the designer that does not use the AISI Cold Formed Specification on a regular basis. Fortunately, several software packages are available that will perform the analysis and review using the latest AISI Specifications. A list of software packages with description and data can be obtained from the American Iron and Steel Institute, 1101 17th St., NW, Suite 1300, Washington, DC 20036 (ph: 202/452-7100; fax: 202/463-6573).

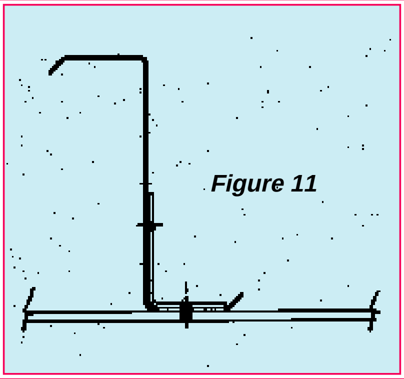
Figure 11

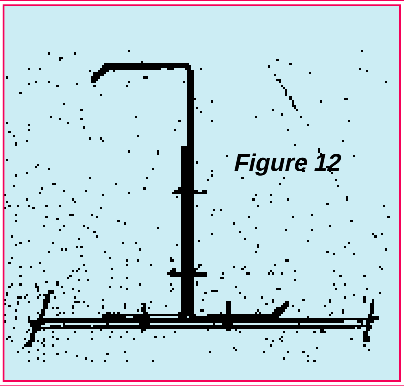
Figure 12

Because of the disparity between the positive and negative moments in a continuous beam system, the positive moment is usually only critical in the end bay which normally will have a heavier member; therefore the positive moment areas in the spans will frequently need no reinforcement for moderate increases in load.

The critical section in a continuous purlin is usually the section at the end of the lap splice but is a function of the lap length. The solution is to increase the lap length to a point where the combined shear and moment stresses are satisfactory. If the existing splice were 2' each side of the frame, a reinforcing member that is 6' to 7' long would probably move the end of the splice out of the critical area. The reinforcing member could be a channel bolting to the web of the purlin (Refer to Figure 13). Relief holes should be drilled in the reinforcing member to clear the existing splice bolts. It is important that at the end of the reinforcing member, a 2 bolt connection be made to the existing purlin and these 2 bolts be spaced vertically as far apart as possible.

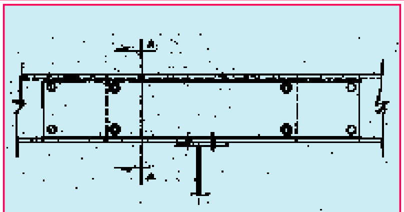

Figure 13

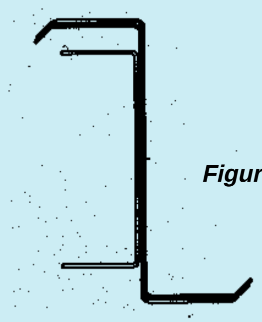

Figure AA

Donald Johnson is a Consulting Engineer from Wolfeboro, NH, specializing in structural steel applications, including cold-formed structures. Previously, he was with Butler Manufacturing Co. and was twice chairman of the MBMA Technical Committee.