Pre-Engineered Metal Buildings
A Guide for Engineers and Architects

December 13, 2002

PURPOSE

Provide an "informal" document as a guideline for engineers and architects who must work with Pre-Engineered Metal Buildings (PEMB) on their projects but have not had design experience working for a PEMB manufacturer. For anyone not familiar with PEMBs, I suggest getting a copy of "Metal Building Systems, Design and Specifications" by Alexander Newman. I found this book to be fairly good and cover a wide range of the MB systems. The balance of this document will be based around the 2002 Metal Building Systems Manual (by MBMA). This book is considered to be a building code itself although it only covers MBs. Had there been a college course in engineering covering Metal Buildings, these 2 books would most likely have been the required textbooks for the course. The term Metal Building (MB) is to be used synonymously with the term Pre-Engineered Metal Building (PEMB).

PREFACE

This is the 2nd version of a rough draft and is not complete at this time. This is offered for discussion. These are predominantly my own experiences with metal buildings. I apologize in advance for any poor grammar or sentence structure. I speak the King's English, the King being Elvis. I am intentionally writing this document loosely because I am not addressing structural engineering in the classical sense. This is more about coordination, planning and project management.

Metal building manufacturers are not all the same although they appear to be on the surface. I know most of you think the term "conservative" does not appear to go with MB systems but "within the world of MB companies, some are more conservative than others". I know there are several really good suppliers and there are suppliers that have lesser services or offer a lesser product. There is a need for a wide variety of suppliers because the projects vary widely. There is a huge difference in a tractor shed and a sophisticated distribution facility. One is very simple and can be supplied by most anyone but the other can be very complex and requires a supplier in the middle to upper end. In addition to this, the local Metal Building Supplier who actually has the building franchise can be a huge factor.

DEFINITIONS

See the Glossary of the 2002 Metal Buildings Systems Manual for terms not contained in this document. The current glossary appears to need more emphasis in some areas. The following additional terms need to be added to the 2002 MBSM. Terms shown below that that are not underlined are used in this document, but the definition is found in the 2002 MBSM.

Actual Clear Height: Distance from Finished Floor to the underside of the lowest point of the roof beam (rafter).

Actual Eave Height: Distance from Finished Floor to the top of the eave member (eave strut) or eave roof structural. This height DOES NOT include the thickness of the roof panel.

Collateral Load: See 2002MBSM

Frame Height: Distance from Finished Floor to the top of the steel beam/rafter at the eave of the building.

Girt: (AKA Wall girt, wall structural, wall secondary) A structural member that spans horizontally between steel frames that the **wall** panels attach to. Can be cold-formed or mill steel. Cold-formed Zs and Cs can be used. Cold-formed Zs can be lapped to create a continuous member whereas Cs are typically simple span. Similar to a Purlin but Girts are used for walls.

Nominal Eave Height: Distance from Finished Floor to some fictitious distance above the eave of the steel frame. For example, one company uses 12″ above the steel frame. This is a 12″ allowance for the roof structural (purlin) and possibly the roof panel. The supplier may use 9.5″ of the allowed 12″ or may use any amount within the 12″ envelope. Other suppliers may have other allowances.

Purlin: (AKA Roof purlin, roof structural, roof secondary) A structural member that spans horizontally between steel frames that the **roof** panels attach to. Can be cold-formed or mill steel. Cold-formed Zs and Cs can be used. Cold-formed Zs can be lapped to create a continuous member whereas Cs are typically simple span. Similar to a Girt but Purlins are used for a roof.

Structure Line (SL) (AKA girt line, sheet line or steel line): **This is possibly the most important term to know if you must function in the PEMB world. It does not exist in the classical architecture or structural engineering world.** It is a term that refers to the exterior face of the wall girt system. This face would be where the insulation or wall panel attaches. The length and width of a PEMB is the distance from this exterior face of wall girt to the opposite SL. The REAL width and length of a PEMB is the distance from opposing structure lines (SL) PLUS twice the thickness of the wall panel. For example, a 100'x150' PEMB with a 1.5" thick panel is REALLY 100'-3"x150'-3". The PEMB supplier will define this as 100'x150' PEMB although the true width is greater. The reason for this is that any depth wall panel could be used without changing the frame spacing, punching, etc. In general, the PEMB supplier does not include wall panel depth in their width, length and height. Also, this term would exist on a project even if the PEMB was not supplying any wall girts.

Architects need to understand this concept because if they intend a building to be 100' wide from outside of wall panel (or brick) to outside of wall panel (or brick) the width of PEMB to order is 100'-(2 x wall thickness allowance). If you are allowing 5" for sheathing and brick, it becomes a 99'-2" wide building. It is best to use 1' increments for the PEMB dimensions. So in this example, make allowances for a 100' PEMB SL to SL and have a 100'-10" final product or use a 99' PEMB SL to SL and get a 99'-10" final product. Fight this concept if you want but the PEMB supplier has almost all their details drafted around this concept when it applies.

OVERVIEW OF PEMB

The term Pre-Engineered Metal Buildings has almost lost its' original meaning since the invention of the computer. All buildings are engineered before they are built and therefore they are "pre-engineered". The original term was applied to PEMB as we know them because "parts" of these buildings were designed one time and these designs were then saved to be re-used provided the design criteria was the same. A more appropriate term would be "Re-used Engineering Metal Building". The steel frames were designed for a given vertical and lateral loading. These frames were reused continuously and each PEMB had literally hundreds of these designs saved. Roof and wall structurals were done in a similar fashion.

Once someone ordered a building of a given size such as 40'wide X 100' Long X 14' tall, the supplier went to the rack and pulled the applicable components for the loading requested. In essence, it was almost like buying a shirt off the rack, "Give me a 32 Large".

With the invention of the computer, most designs by larger PEMB suppliers are done per order now. The use of older "shelved" designs still occur, but it is less frequent now. The reason for this is the supplier wants the ability to take advantage of different material availability and prices and is willing to use engineering resources to get this.

SUPPLIERS

The potential supplier of a PEMB is going to be one of the main factors in having a successful project. For purposes of this discussion, I assume the following items:

- The project will have the involvement of an Engineer and possibly an Architect. These professionals are responsible for specifying/describing a MB that fits the overall project.
- The building must fit in with other Non-PEMB related items such as masonry, mechanical systems etc.
- The project IS NOT a standalone PEMB such as a farm storage shed, small warehouse, etc. that requires no coordination with other "functions" of the building.

Most PEMB suppliers are really in competition with each other for construction projects. Many times I have made the statement, "Yugo and Mercedes, both make cars, but they are not in competition with each other". Anyone considering one of those vehicles, is most likely not even looking at the other brand. IF price is the only criteria, for a successful supplier, then you are going to most likely wind up with one of the lower end suppliers. The upper end suppliers cannot easily compete on price alone. But the design professional needs to seriously think this out on a per job basis. If you do not address Quality, Time and Services in your design, then you have Yugo and Mercedes competing with each other. With the proper criteria in your specifications, only companies that provide the needed criteria will be competing among themselves. You will have Yugo competing with Kia or you will have Mercedes competing with Lexus.

The difference in price, is not always just a higher mark-up on the material. In a lot of cases, it is Quality, Time and Services offered.

Here is a partial list of things to consider:

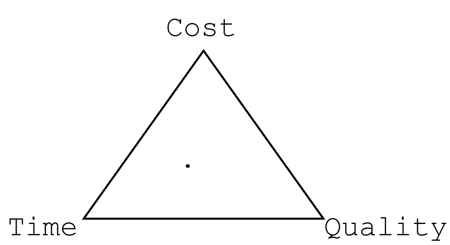
- <u>Do they have service after the sale</u>. This factor is huge when something goes wrong or criteria changes after initial construction. Some companies sell you the original building and after that, have no services for you. The analysis of a PEMB frame or the cold-formed secondary structurals can be horrendous if you do not have the software or have never designed in this area before. Here are some examples of problems that I have been involved in that required service after the sale:

   - Your concrete contractor located the anchor bolts wrong. You decide to modify the steel frames to accommodate the defect. I know some suppliers will give the additional assistance (at a fee) and some will not assist at all.
   - You decide to add a 3 ton hoist to one rafter but need the one frame to be analyzed for the additional load and any retrofit to be designed by the PEMB. Some will help (at a fee), some will not. Addition of loads after the initial design is one of the most common problems Owners encounter. This can occur years after the original sale.
   - Foundation problems have caused the frames to settle down into the ground unevenly and therefore the baseplates are at different elevations. You need someone to model the effect of the settlement on the steel frames and work with you on necessary corrections. Again, some will assist, some will not.
- <u>Do they have a troubleshooter</u> who will come out to assist in correcting problems with erection of the building even if the problems were not caused by the PEMB company? There may be a charge for this service, if the problem was not theirs, but at least you have a professional who is familiar with their product.
- <u>Can you get Single Source responsibility</u>? Some companies do not sell or design all components you may need. Remember, any portion of this building you need but the final PEMB supplier does not provide is one of the responsibilities someone else must take on. This responsibility will be for items such as supplier for the item, how well it fits with the "system", scheduling, effects on the warranty, and other important subjects. A lot of the problems with ANY construction occur where one supplier stops and another one starts. The less suppliers, the less finger-pointing. Here are some of the more common ones I have seen:
   - The "standard" PEMB Insulation. If they do not supply insulation, you have to determine who does. The insulation goes between the PEMB roof/wall sheets and the PEMB purlins/girts. If they do not supply insulation in their bid, I do not generally deal with them. They will tell you that it is easy to get someone else to insulate, and I generally tell them "Then you shouldn't have any problem finding someone to contract this with you, but I want a single source to be responsible." Remember, they build the frame, then you must get it insulated BEFORE they install the roof/wall sheets. Wind/rain can really tear up

insulation if panels do not get installed quick enough.
  - Cranes. Some companies will not even analyze/design their structures for additional loads from cranes. They are typically low-end suppliers who are supplying a "Re-used Engineering Metal Building" and do not have the engineering staff to analyze it for the additional load. I typically want any column brackets or separate crane columns that are tied back to the PEMB to be supplied by the PEMB. The crane beams can come from either the PEMB supplier (if they can build to crane tolerances) or from the crane supplier. If the crane supplier is to do the beams, I want the PEMB to coordinate the connection with them to ensure they match. If they are not willing to coordinate, I don't need them.
  - Mezzanines. If a somewhat "self-supporting" mezzanine is to be tied to the PEMB for stability, will they analyze it for the loads and supply the punching necessary for the connection. I prefer the PEMB supply the mezzanine if they want, but at a minimum, I want them to coordinate with the supplier on the loads and connections. On tapered columns, the vertical load at the PEMB column (assuming the exterior mezzanine support is the PEMB) is pulling down on a welded PEMB flange that may have only a one-sided weld. With coordination, the flange will be reinforced and welded to resist the load from the mezzanine and any needed holes are there. If they will not assist in the mezzanine, then you must supply columns where the mezzanine attaches to their column rather than using their column to support the load.
- <u>How reputable is their actual structural design</u>? You can only learn this one by experience with suppliers over time. The section on "Specifying" should assist in this, but I wanted to include this reference in this section since ANY structural problem tends to affect the Owner and Engineer/Architect even if it was caused by a supplier. Remember, they do not have to cut their structural criteria in half, to get a cheaper price. Assuming they all have the same costs/overhead, they only need to cut 5% to 10% to get the job IF price is the only criteria. Here are some "design practices" I have seen over the years:
  - Ignoring common deflection criteria. #1 difference in price on some jobs. This is especially true if cranes, masonry or other lateral deflection sensitive systems are present.
  - Interpolating the 20/12 Roof Live Load chart to get a roof live load of 13 psf instead of 16 psf. When asked, the supplier said, "Code doesn't say I can't". My specifications clearly state this is not permitted.

- Reducing foundation reactions for wind by 1/3 BEFORE giving them to the engineer for the project (poor little ol' me) and not informing the engineer it was done that way. When asked, the supplier said, "Well, code allows a reduction for wind, so we went ahead and reduced them now." I also found out the salesman for the company was telling my client to buy his building because the foundations would be lighter. He was using the lowered reactions to show my Client how their structures required less foundation when compared to competitors. In general, you can catch this with simple statics. Add up all reactions and compare to what you think the applied forces are. As mysterious as PEMB are, they are yet to defy statics.

Well enough about suppliers in general. I wanted to include the "Time-Cost-Quality Triangle (TCQ Triangle)" that was shown to me one time. When choosing a PEMB company on a per job basis, you as the design professional need to balance out Cost-Time-Quality. Each job is different. Placing a dot inside the triangle below that somewhat indicates the particular job's "criteria" may be helpful in selecting PEMB suppliers that meet your job criteria. Any supplier claiming to have all 3 components to the max, is a liar or working for a company owned by an idiot. If you have the best quality at the fastest delivery, "Why be the cheapest too?" You cannot approach one criterion, without moving away from another. As you deal with PEMB companies and watch what they actually offer you, you can almost write their names around this triangle to coincide with their ACTUAL product offering.



SPECIFYING

Specifying the "layout" of a metal building is probably the closest that most of you will come to the design of a metal building. Specifying in the sense of this document includes the following items:

- Project criteria beyond normal requirements of the building codes, AISC and other typical codes
- Length, width and height
- Spacing of the frames (called bays)

- Spacing of the interior columns if any (called modules)
- Location of bracing and defining acceptable means of bracing
- Defining the loads and/or codes to be designed around
- Defining performance criteria such as lateral deflection, minimum material sizes etc.
- Defining items to be coordinated with other trades
- Defining items concerning coordination of any documents (drawings, submittals etc)

1. <u>Project criteria beyond normal requirements of the building codes, AISC and other typical codes.</u>

One of the more common complaints from engineers is the fact that PEMB are designed to the maximum allowed values at some point in the structure. Assuming the design loads are correct, this does not present a problem in most cases. If this bothers you, or you feel the project should have some "additional strength" allowance there is no problem specifying this. Here are several examples of ways to do this:

- State that no combined stress ratio may exceed some value such as .85, .9 etc. If desired you can use different values for frames, roof structurals, etc.
- Ask for additional loading beyond the code required. "All frames shall be designed for an additional 3 psf LL and 2 psf Wind Load beyond that required by the code".

What you may find out is that some higher end companies may not max out all of the components while other companies do. I know we rarely designed a column beyond .9 to .95 where I worked. If our competitors did, we already were at a disadvantage financially. If you made it a job requirement, it only brought their product up to a more conservative PEMB's standards.

Depending on your project, it may be prudent to require inclusion of a point load on any frame at most any location. For example, one 2 kip load at any point on any frame. Generally, an Owner needs to hang something from the frames later. You cannot predetermine the magnitude and location but it would be prudent to make <u>some allowance</u> in the original design because most any building that is maxed out cannot have much added to it.

2. <u>Length, width and height</u>

First you need to read the definition of "Structure Line", "Actual Clear Height", "Actual Eave Height", and "Nominal Eave Height" before proceeding any further with this category.

To most PEMB companies, the distance that the steel frames span is the WIDTH. If I have a frame that spans 100' and I take 2 of these frames and space them to achieve a 20’ final dimension, I have created a 100’ Wide x 20’ Long building. In other words, width does not indicate the lesser dimension to most PEMB companies. Length tells them how many frames they will need to provide for the building. Generally, “Looking at the width of the building will profile the roof slope”.

Most PEMB companies define their length and width in terms of the Structure Line (SL). This is the single most critical item when it comes to coordinating the PEMB with items such as brick, block, and most other exterior finishes. Architects need to realize this is why most exterior finishes do not come out as planned. If you are designing a project around a MB, the Architect needs to show the relationship between the SL and other items such as brick on their drawings.

The height of the building is normally defined in terms of the “Nominal Eave Height”. So bear in mind, a 14’ tall building is 13’ to the top of the frame for a supplier who uses a 12” envelope. The true eave height of such a building (Actual Eave Height) could be 13’-9” or 13’-10” etc. <u>If it is important on your project</u>, it is best to define your building in either Actual Eave Height or Actual Clear Height rather than Nominal Eave Height. I generally define a minimum clear height under the frame for my projects and only define an Actual Eave Height ALONG with my minimum clear if it is necessary. In defining both clear height and any other height together, you limit the depth of the rafter at the knee area. Be sure you used a reasonable allowance for the rafter depth. Too shallow a rafter will increase the cost. It is best to stay with clear height when possible. You should be clear to the supplier what you are looking for and define these terms somehow in your specifications in case they have another definition for these terms beyond those stated here.

3. <u>Spacing of the frames (called bays)</u>

The distance from one steel frame to the other is called a bay. It is best to use even 1’ dimensions although 6” multiples can generally be handled by most suppliers. Try to avoid anything other than 6” multiples because is messes up standard hole punching in the roof purlins. Typical bays are 20’, 25’ and 30’. Some suppliers can go beyond 30’ with different roof structurals. I have seen many PEMB with a bar joist roof system. These bays can be 60’ long or more.

The last bay of a PEMB is generally measured from the SL of the endwall to the Centerline (CL) of the first steel frame. PEMB companies rarely dimension from the center of the endwall frame to the CL of the first steel frame.

The exterior columns of the steel frame are not dimensioned to the centerline in both directions. The distance from a Steel line or the Centerline of another rigid frame is taken to the centerline of

4. Spacing of the interior columns if any (called modules)

If the steel frame has interior columns, spaces between columns is called a "module" by some PEMB companies. For example, a 100' wide PEMB with a single center column is said to have "2 modules at 50 feet". The modules are generally measured from SL of the sidewall to CL of the interior columns. The interior columns are generally located by centerlines in both directions.

5. Location of bracing and defining acceptable means of bracing

It is best for you to determine where bracing will be allowed and what type of bracing is acceptable. Bracing is generally used to stabilize the PEMB along its length and at any endwalls that are Beam & Post construction. The following are the 4 main types of bracing supplied by PEMB companies:

- Roof Rods and Wall rods; this is the cheapest form of bracing; this generally must be located between exterior frame columns and the plane of the bracing is oriented to the direction the loads are to be resisted. Bear in mind, Architects typically dislike wall rods since it minimizes the use of windows and doors at that area. Wide buildings and really long buildings may require several bays of bracing of ANY type.
- WindPost; the is a fixed base column that is generally located next to a rigid frame column. Windposts cost more than rod bracing and require a large foundation due to the moment at the base of the column.
- Portal Frame; this is rigid frame that is generally located between 2 exterior columns; portal frames cost more than windposts or rod bracing generally, however, the foundations are generally not much larger than required for rod bracing.
- Panel Diaphragm; this is a shearwall made of the PEMB wall panels PROVIDED the panel is capable of diaphragm action. Some panels are not capable of providing this support. Some codes do not allow panel diaphragm and most engineers including metal building designers do not prefer using this type of bracing. It is very common for Owners to install large overhead doors later and therefore cut holes in the diaphragm. Low-end suppliers may rely solely on it.

6. Defining the loads and/or codes to be designed around

It is not necessary to define the loads directly, however you should define which code is to be designed for. You should also require them to meet the current MBMA code in addition to your locally required code. This should not add any cost to your project. For purposes of lateral deflection due to wind, you may need to specify the "Occurrence interval" you want designed for (50 year vs. 10 year occurrence for example). Most building codes use at least a 50 year occurrence for loads. Some PEMB companies will use the 10 year occurrence for lateral deflection calculations although I have never found a sound reference that permits this. The only reference I have seen mentioned is the Canadian Building code. I do not find this to be a good enough reason to allow 10 year winds to be used. I require 50 year winds in my specifications for all deflection calculations based on wind. This is another of the ways some companies keep their price down. This can be a significant amount of money when dealing with cranes, masonry or glass.

The other issue with lateral deflection is whether they are using "bare-frame" calculated drift to compare to the allowable drift or whether they are factoring the bare-frame drift with a chart that relates number of frames to true drift of a building. The times someone has told me this was what they had done, I asked for the chart they used. I was told it was in MBMA. They were talking about a chart on page 231 of the 1986 MBMA code. This chart specifically states it was for conceptual purposes only and NOT to determine real numbers. I generally require bare-frame deflection as the valid number to compare to but do not generally have a problem with someone falling a little short of the value since I do realize the building is stiffer as a unit than as a bare frame. The problem with using too much of this roof diaphragm is it can lead to roof leaks at the fasteners with time. In addition to this, it requires a lot of specific information that most suppliers have not calculated in order to apply this methodology. If I require H/100, I may accept H/93 for example but I will not allow much deviation from the 100. The chart they are citing in MBMA goes as much as 45%.

If cranes are involved, you may want to specify the maximum wheel loads, wheel spacing, number of wheels and other pertinent information rather than merely stating the crane capacity to design for. If you only specify a crane tonnage, some companies will use the "Light-Duty" crane data which will yield less loading than "Medium-Duty" or "Severe Service". **You should define the crane parameters.**

Beware of one trick some suppliers use. Most PEMB have pinned base columns. In some cases, they can supply a fixed base building which may have significantly less cost for the steel frames but have much higher cost for the foundations. In some cases, fixed base may be the better overall alternative but this rarely is true of a small to medium PEMB with common PEMB features. Always compare alternatives based on the same type foundation system. If one submits fixed base, while the others are pinned base, you need to estimate both sets of foundations to get a total job cost. When I feel pinned base is all that is needed, I specify frames must be pinned base in my specifications.

7. <u>Defining performance criteria such as lateral deflection, minimum material sizes etc</u>.

If the code you are requiring does not give adequate specifications for deflection, it would be best to spell it out in your specifications. For example, most codes do not address lateral drift at all. Most PEMB will design for approximately H/50 for a metal sheeted building with no crane. The addition of a crane would normally make this need to be H/100 at a minimum. Masonry and other items can make this even more stringent. If you code does not address this, you need to specify this.

In some cases, the materials supplied by the PEMB company may be satisfactory for the loadings but you may want something thicker for some reason. Here are some examples of things I have required on some projects:

- On a project with a mezzanine, I do not want any column webs thinner than 3/16". Interior columns webs on a PEMB can be as thin as .09 inches. My specs said "Regardless of whether design calculations will permit a thinner web, no column web shall be thinner than .1875 inches." This is assuming I am going to be attaching mezzanine beams to these webs.
- On a project with a <u>lot of small loads</u> being suspended from the roof purlins, I do not want any purlin to be thinner than .074" inches. Some purlins can be as thin as .055" in thickness. Since most small loads are hung from C-clamps over the bottom flange or by all-thread rods drilled through the lower flange, I want a thicker flange to permit a little more leeway in hanging light loads. See the section on "Attaching Point Loads" later in this document.

8. <u>Defining items to be coordinated with other trades</u>

If there is anything that the PEMB must accommodate on the project, then it is best to specify these if possible. Here are some examples:

- If the exterior wall covering is not supplied by the PEMB, then it may be necessary to coordinate how far the roof sheets hang over the SL. One example would be if you had brick on the outside of the SL and the brick went all the way to the roof. You may not be getting wall girts or siding from the PEMB but the roof sheets must extend over farther than normal to accommodate the brick.
- If you have a brick wainscot that is 5' high. It may be necessary to insure there is a wall girt at that elevation to brace your brick to and you may need to determine if the

PEMB supplier is to supply the flashing at the joint of the brick to the PEMB metal panel.

9. <u>Defining items concerning coordination of any documents (drawings, submittals etc</u>)

This probably has more to do with the drawings than anything else. If you do not specify grid markings for them to follow, they are going to use their own system. PEMB companies do not have a standardized grid marking system. If your architectural/engineering drawings have grids 1-10 horizontally to the right and A-G going down the page. You may get J-A going horizontally and 9-1 going down the page. This really adds confusion to a job when you tell someone to change something at Grid B. I generally mark the following grids on my drawing for the PEMB and require them to use the same grids or double-grid their drawing to show mine and theirs.

- Each Structure Line
- Centerline of each steel interior frame
- Reference line for each interior endwall column

I do not mark the following at all:

- The corner column of an endwall
- Centerline of any exterior column of a main frame (a grid that runs perpendicular to the span of the frame.

It is best to require "Design Data" from any supplier. Design Data is generally the following:

- Structural analysis of the steel frames;
- Structural analysis of the steel endwall
- Structural analysis of the roof purlins
- Structural analysis of the wall girts
- Structural analysis of the bracing
- Structural analysis of the roof and wall sheets

There may be and additional charge for the data but it generally not very much and is worth the expense.

For the person designing the foundation, it may be necessary to specify the reactions be presented as individual load cases (DL, LL, WL) and as combinations. Some PEMB companies supply reactions that are combinations only and this is of little use to foundation designers.

The actual written specifications for a job is another issue to consider. Most engineering firms used "canned specifications" as a starting point and modify these specifications per job. One of the selling points of software companies that produce canned specifications for sale is that "each section is written by someone who is an expert in that area". A firm I used to work owned 2 different set of canned specifications at one time or another when I worked there. Both sets were well-known companies. I did not find either specification to be very well-written in some areas. On one occasion I called the company and spoke to their "expert on PEMB". They did not have AISI listed as one of the references in their specifications and I was tired of having to add it every time (they did have AISC 9th edition). The expert did not know what I was talking about when I mentioned AISI or the American Iron and Steel Institute. I told them AISI governs all cold-formed parts in a PEMB. The roof/wall sheets, all roof purlins, all wall girts and many other components are designed by this code and not AISC. The fact he never heard of it made me easily question how much of an expert they were. The following are a list of comments I have on canned specifications and some of the things you should consider rewording, omitting or adding:

- Add a Letter of Certification for Loading (shown below)
- Add the correction of 4.2.1 (shown below)
- Carefully read the roof structural and wall structural parts. I have seen most specifications call out a depth of purlin/girt and in some cases the thickness. I have seen the depth called out as 8" typically, but some suppliers use other depths such as 9.5", 7" and so on. I do not have a required depth although I may call a minimum depth and a minimum thickness. Specifying a set depth limits potential suppliers. Also bear in mind, the design of any cold-formed structural has numerous parameters and just specifying depth will not guarantee any reliable design. It is best to require it meet the loading and deflection criteria and then let the supplier handle it from there. The design of any Z or C shaped member includes the following:
    - Depth
    - Thickness
    - Width of flange
    - Stiffening return lip on flange
    - Steel allowable
    - Other

- Add a section on minimum allowed thickness, depths and other criteria you may feel necessary. Again, anything you specify here will most likely increase cost and may also eliminate some suppliers. You should have a good reason for the requirement. Here are some random examples:
    - All roof purlins shall be have a minimum depth of 7"
    - All roof purlins shall be a minimum .065" thick.
    - All interior columns shall have a minimum 1/8" thick web

- All exterior columns shall have a minimum depth of 10” at the baseplate
- No anchor bolts shall be located closer than 5” to the structure line (PEMB engineers apparently have never designed a foundation). They only care about bolt edge distance, not concrete cover or rebar diameter.

The specifications should always require a “Letter of Certification for Loading” (LOCL). This is a document that basically states what codes and loadings the building is or will be designed for. This letter should be stamped by the PEMB company’s registered engineer. Be cautious of the wording since some companies have the wording VERY non-committal. I generally include my own letter in my specifications and require them to submit it on their letterhead and stamped by their engineer. I have included 2 examples at the end of this document of real LOCL that I have seen. The first is close to the one I include in my specifications and the second is one of the worst ones I have ever seen. This may seem unnecessary to some of you, but the legal departments of some metal building companies do not think it is. If they will not submit the letter at least reasonably close to the first one I have shown, best to back away from that company. You may need to modify the letters to match your local codes and preferences. I require the letter written in future tense at bid time and present tense once they get the project. The 2nd letter I have shown was actually submitted by one company and they would not sign the other letter. We refused their bid. Requiring the first letter cuts down on the manipulating of the codes and gives you a lot more breathing room.

The typical PEMB specification states they are to adhere to the steel code (usually 9th Edition AISC). Section 4.2.1 does not appear to me to be addressing PEMB suppliers since they are supplying engineering, drafting and fabrication services, but some of them contend this section does pertain to them. This section was intended for local fabricators who are making components based on your design but you may be allowing them to design their own connections. Typically, when this is done, they are making the connections and you should approve them. PEMB companies have their own engineers and are designing all of the components and connections. You should change this to eliminate any doubt about responsibility. You should change this to read something like: “adhere to the 9th Edition of AISC Manual of Steel Construction except section 4.2.1. Section 4.2.1 shall be substituted in its entirety with the following”

4.2.1

“Approval by the Owner or his representative of shop drawings or erection drawings prepared by the fabricator indicates the fabricator has correctly interpreted the general layout and framing of the metal building specified in the contract requirements and is released by the Owner to start fabrication. This approval does not constitute the Owner or the Owner's representatives acceptance of any responsibility for the design adequacy of any structural components designed by the metal building supplier or the adequacy of any connections designed by the metal building supplier. The metal building supplier and their engineer are responsible for all structural design services they performed on this project. Approval does not relieve the fabricator of the responsibility for accuracy of detail dimensions on shop drawings, nor the general fit-up of parts to be assembled in the field."

End of 4.2.1

Take care that this only will apply to the PEMB supplier and not to other conventional steel fabricators who did not do the structural design and connections for the project. I have not looked into AISI or MBMA to see if a similar section is present, but if it is, it should be changed. Your responsibility should end with specifying codes, loads and local requirements.

Attaching point loads

Attaching of points loads to PEMB frames and roof structurals needs to be somehow addressed to other trades such as electricians, plumbers and HVAC contractors. Most PEMB roof structurals are Z-purlins. These purlins are inherently in torsion because of their shape and roof panel load point (on the upper flange). While purlins are fairly strong for roof loading, they are not well suited to hanging multiple loads from the outstanding lower flanges.

This subject is well worth a thesis topic to some graduate student but I will try to shed a little light on it. A good PEMB supplier should be able to give you some assistance with this topic. Larger point loads (say 200 lbs or more) should be designed for by the PEMB supplier and can do this provided you tell them the general locations. If they cannot, you are possibly dealing with another Re-used Engineering Metal Building.

One thing to watch for on Z-purlins and C-purlins, is how they calculate section properties and allowed loads. Most of them are calculated with the purlin on a 0:12 slope. The actual roof is generally .25:12 to maybe 2:12. Rolling these purlins up on this shallow of a pitch does not greatly change the strengths, but I have seen them roll them up on a 6:12 pitch and used the same properties and allowables. This inaccuracy is augmented when you have point loads hung through the purlins, even if the loads are hung through the purlin webs. The rest of this section is assuming the roof is a 2:12 or less in pitch. The reason for this is the magnitude of loads I am stating.

Only small loads should be hung by C-clamps from the outstanding bottom flange. The loads would typically be in the range of 50 to 75 pounds depending on the flange thickness. The C-clamp causes the bottom flange to bend down like a diving board. The moment arm to the clamp is one of the bigger factors.

Slightly higher loads can be hung by an all-thread rod drilled through the bottom flange of the purlin. These loads would typically about 100 to 150 pounds depending on the thickness of the purlin. The rod has a lesser moment arm and therefore can have more load.

Loads of 200 pounds or more will most likely need to be attached to a support member that frames between two purlins and would be attached to the webs of the Z-purlins. I have hung loads up to 700 lbs in this fashion onto a properly designed roof purlin.

The point I am trying to make is that the other trades may want a change order or at least cry if they cannot hang these C-clamps or threaded rod for ALL their loads. It is best to coordinate this at bid time and not during construction.

The frames cannot have large loads suspended from the lower flange even if the frame in general can support the load because of the web-to-flange connection of the beam/column. It may be necessary to provide a weld on the other side of the web and a short stiffener that is welded to both the web and flange. This stiffener will pass the load to the web without going through the web-to-flange weld. I will use just the weld for smaller frame loads. The larger loads I will require the weld and the stiffeners on both sides of the web.

The following is a list of some problems I have run into with hanging loads from roof purlins:

- All trades hang from the same few purlins because they are the most easily accessible. These collateral loads are design on a square foot basis but they are not spreading them out over the building. I have seen one purlin with a bunch of loads hung from it and the next 10 have nothing on them.
- Contactor has been told he can only hang 50 pounds from a C-clamp so he uses 8 of them side by side to hang 400 pounds. He acted ignorant of why this cannot be done but he was wanting to play the “technically game”. We had stated that C-clamps and rods could not be any closer than 16” apart unless approved.

- Someone hangs so much load they actually bend the flange over permanently. They move the load to another location and never tell about the damaged purlin.

The hanging of point loads through the flange creates torsion on the purlins that most likely was not designed for at all. Small loads that are spaced out are not much of a problem since the Z-purlin is inherently in torsion from the roof panel dead load and any live load. These loads typically cause torsion in the opposite direction of the inherent roof dead and live torsion. This would be additive for uplift conditions though.

If you have a project with a lot of large point loads or has a lot of loads concentrated in one area, it is best to consult with the PEMB supplier. This can be a problem though, with conventional construction also.

In some cases, I have required the roof structurals to be designed for a higher live load than the steel frames to accommodate some of these problems. I have used 5 psf collateral load for my frames but 10-15 psf for my roof structurals because of the problem with defining the locations of these numerous loads at bid time.

Foundations

There is little to tell about foundation design for metal buildings since this is rarely done by the PEMB supplier. What is generally needed is a clear coordination of where the column loads are and what magnitude they are. PEMB suppliers should supply you with a COMPLETE set of what is typically called "Anchor Bolts and Reactions". A complete set should have the following:

- A drawing of the "footprint" of the PEMB. This should show the overall dimensions, column locations, grid marks and anchor bolt detail callouts.
- A list of reactions at each column or brace connection to the foundations. These reactions should be for load cases and not combinations. Reactions should be denoted by the grid lines they are related to.
- A drawing of the actual anchor bolt details.

Some PEMB suppliers only send the main frame reactions. If you ask for the endwall columns and bracing they may tell you they do not typically give those "but you can figure them on your own". Your specs should require them to provide at least the items I listed above. If they do not, you should not calculate them yourself. They have to know what they designed the components for in the first place. I require them to supply all reactions that their components (frames, bracing, endwall columns etc.) apply to the foundations.

The PEMB will generally ONLY figure the quantity, size and location of the anchor bolts. The embedment length is a function of the foundation and therefore they leave that to the foundation designer. You can make requirements to them about these factors but the embedment will still not be part of their design generally. Bear in mind, any requirement you make that takes them away from some standard details and fabricating practices they have, will tend to add cost to the project.

One other feature of a PEMB is the corner columns. If you do not know the supplier, you will most likely not be able to predict where the corner column sets on the endwall. I always advise Architects to beware of putting toilets and other such items in one of the corners of a PEMB. There is no telling where the column will be UNLESS you know the supplier.

In the foundation design, you will generally find the following on a PEMB:

- Foundation size will most likely be governed by wind uplift on the foundation. Rarely does DL+CL+LL govern the actual size of the foundation but uplift does. This is assuming the building does not have a crane. Beware of a supplier telling you that you only need to design for 70% of the wind for uplift. They may tell you it is in the MBMA manual. This was a "suggestion" in the commentary at one time but I looked into this. What I was told by AISC was that this was preprint article and in the end, they did not approve the practice.
- The outward kick at the base of the column will generally be governed by DL+CL+LL. Column kick can be accommodated with a hairpin for narrower buildings but may require a tension tie across the columns for larger spans. Column kick is affected by span, magnitude of gravity load, height of building (shorter buildings have more kick than a taller building of the same span) and the absence/presence of interior columns.
- Contractors may prefer a monolithic foundation pour where the foundations, grade beams and slab are all poured at once. This can be done for buildings 70' and smaller in width but may be more difficult for buildings greater than this UNLESS they have one or more interior columns.
- Large uplifts on interior columns will generally be more economical to use a foundation slab with pier stem rather than a large thickened slab.
- Contractors generally want you to allow them to use expansion anchors or epoxy anchors for interior column anchor bolts if they are doing a monolithic pour. Interior columns rarely have any horizontal load but do generally have high uplifts.

- It may not be clear from the reactions rather the wind bracing loads (generally occur at a column) have already been added to the column at that location. In most cases, you must add the uplift from bracing to the column the bracing is attached to. The horizontal load from bracing is generally orthogonal to the horizontal column reaction (column kick).
- The endwall column foundations are generally about the same size in your geographic location regardless of how wide, long or tall your building is. Endwall columns are typically spaced 20-30' apart. Their tributary is based on the spacing of the last frame and the column spacing. For normal PEMB dimensions, this will yield roughly the same foundation size.

Cranes

On a lot of projects, the crane may have its own self-supporting runway for vertical loads but rely on the PEMB for lateral stability. This is very common. If your codes do not outline crane requirements (deflection, load application etc) then you should define it or define the controlling publication (CMAA or other).

Underhung cranes should be treated as large point loads on frames. You should require the PEMB company to reinforce their frame for the applied load and it would be advisable to consult them as to how to support the load from their frame.

Lighter top-running cranes can be supporting by crane brackets welded to the PEMB column. I generally want them to include the brackets in their package.

Separate crane columns that are tied back to the PEMB for lateral stability are generally required by me to be supplied by the PEMB supplier.

Measuring Existing PEMB for Assessing Structural Capacity

If you must assess the strength of an existing PEMB, the following points should be remembered at a minimum.

1. Try to find the original supplier and when the building was originally done. If they still have records/drawings, this is well-worth the cost in most cases.

2. The material yield may be 50 to 60 ksi. If you can find the original supplier, they might be able to tell you what yield

stress they used when the building was originally designed even if they no longer have a record of the project.

3. You need to measure the distance to any depth-change or connection in the part (column or rafter).

4. Any place there is a weld between two plates such as the inner flange, you need to measure the material size on both sides of the weld. The plates may be the same width, but most likely are a different thickness. There can be as little as 1/16" difference. The typical distance between welds such as these is 10'-20'. If you encounter a building with multiple short sections such as 7.5' or less, you are possibly dealing with a really low-end supplier. I have seen them as short as 3'-4' long. This building was built from scrap that I believe was bought from a higher end supplier.

5. Typically, the outer and inner flange of any part will have a different thickness. They may both be the same width flange (called balanced flanges) but may be different in thickness. Most suppliers will use the same width because it keeps the part from running down the assembly process setting on an angle.

6. Note where flange braces occur. Flange braces are the diagonal members (usually small angles) that extend from the roof purlin or wall girts to the inner flange of the part. The purlin/girt is typically assumed to brace the outer flange and these flange braces are typically assumed to brace the inner flange. These generally run on about a 45 degree angle. A common spacing for a rafter is 10' for inner flange braces, although the outer flange may be 5'. You will possibly find rafter inner flange braces closer together at columns. The fact you have different brace lengths on the same part, can make it difficult to model on software not equipped with the ability to specify inner flange unbraced length and outer flange unbraced length simultaneously. In that case, it is best to use the large length and then hand check weak points to verify their adequacy.

7. Measure all connections. This would include bolt spacing, size, weld locations, stiffeners and other plate that have been welded to the splice. It WOULD NOT be prudent to assume the splice is as strong as the parts it connects. Most rafter splices are placed in a location that has a lower moment. Some companies only design for the moment at that location even though the rafter at this location is capable of supporting more load. Some companies design for a minimum 50% of the capacity of the member even if the actual loads are less. Other companies may only design for the actual loads.

8. When modeling on a computer, most companies let the width of the modeled structure be the centerline of the part and not the overall width of the building. Due to the width of some

columns tops and rafters, this can be 3’ to 4’ less than the actual building width and 1.5’-2’ less in height. They do have to put the load from this missing 3’-4’ (or 1.5’-2’) on, but the modeled span/height is not the true width/height.

9. Not all of a PEMB is maxed out to 1.03 combined stress ratio. There is generally one location that is maxed out and the rest of that part is stressed to a lower CSR. Unfortunately, this is sometimes the connection of the exterior column to the eave end of the rafter. It is possible to beef up this one area (and other areas) to provide for more load capacity.

Questions for discussion

1. For foundation designers who use the monolithic foundation with a Hair-pin to resist lateral thrust. What is the maximum horizontal reaction you feel comfortable using before abandoning the hairpin for a heavy foundation, shear block or tension tie? See page X1-A3-3 of the 2002 MBSM for information about hair-pins. It is in the appendix in section A3.

2. In what capacity do you usually get involved with a PEMB?

Capacity 1---foundation designer
Capacity 2---Specifying the MB for the Owner only
Capacity 3---Specifying the MB for the Owner and assisting in selecting final MB supplier.
Capacity 4---Specifying the MB for the Owner, assisting in selecting final MB supplier and construction inspection services.
Capacity 5---Specifying the MB for the Owner, assisting in selecting final MB supplier, foundation design and construction inspection services.

Attachments

LETTER OF CERTIFICATION FOR LOADING

Owner name
Owner Company Name
Address1
Address2

Bldg Desc: 80'Wx120'Lx14' Nom Eave
End Customer:
Bldg. Loc:

TO WHOM IT MAY CONCERN:

This letter serves as <PEMB company name's> certification that the above referenced metal building will be designed in accordance with the 1999 Edition of the Standard Building Code and the 1986 edition of the MBMA Low Rise Building Systems Manual. Steel components will be designed in accordance with the 9th edition of the Manual of Steel Construction (AISC) and the 1986 edition of the AISI specification for the Design of Cold-Formed Steel Structural Members.

The governing design code is the 1999 Edition of the Standard Building Code. The following loads are applied in accordance with the governing code:

Dead Load		____psf +frame weight
Roof Live Load	0-200 sf Trib Area	20 psf
	201-600 sf Trib Area	16 psf
	over 600 sf Trib Area	12 psf
Collateral Load		2 psf
Wind Speed		80 mph
Seismic Acceleration %		______
Seismic Velocity%		______
Building Use Category		1
Crane Rating		5 Ton Medium Duty
Max Crane Wheel Load		15 kips

Load combinations are in accordance with the governing code.

These <PEMB name> components, when erected on an adequate foundation in accordance with the erection drawings as supplied and using the components furnished, will meet the above loading requirements. This certification does not cover field modifications or design of material not supplied by <PEMB name>. All facilities that will be manufacturing these components are Category MB certified by the American Institute of Steel Construction.

Cordially Yours,

<name of AL licensed Engineer>

LETTER OF CERTIFICATION FOR LOADING

(THIS IS THE REALLY BAD ONE)

Owner name
Owner Company Name
Address1
Address2

Bldg Desc: 80’Wx120’Lx14’ Nom Eave
End Customer:
Bldg. Loc:

TO WHOM IT MAY CONCERN:

This letter serves as XYZ Metal Buildings certification that the above referenced metal building will be designed in strict accordance with XYZ Metal Building design standards. XYZ standards are based on such guidelines as 1986 edition of the MBMA Low Rise Building Systems Manual, the 9th edition of the Manual of Steel Construction (AISC) and the 1986 edition of the AISI specification for the Design of Cold-Formed Steel Structural Members.

The building will be designed using the 1999 Edition of the Standard Building Code as a reference.

Cordially Yours,

<name of AL licensed Engineer>