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NORTH CAROLINA DEPARTMENT OF ADMINISTRATION  
OFFICE OF STATE CONSTRUCTION

**METAL BUILDING SYSTEMS DESIGN GUIDELINES**

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## 1. INTRODUCTION

Over 50 percent of the nation's annual non-residential low-rise construction involves the use of metal building systems. The State of North Carolina owns a vast inventory of buildings, many of which similarly incorporate metal building systems.

Metal building systems present unique challenges to the Designer in the preparation of drawings and project specifications. Unlike many commodity materials used in construction projects, metal buildings are integrated assemblies of many structural members and related accessories, all of which are custom configured by the manufacturer as required by the nature of each specific project. The structural design work performed by the metal building manufacturer's engineering staff is consequently broader in scope than that required by other engineered construction materials such as bar joists or precast concrete components.

The existence of this additional design team and the numerous conventions unique to the metal building industry require extra effort by the Designer to ensure the effective conveyance of performance criteria and project information.

The aforementioned communication of criteria and information is particularly crucial for work executed under the requirements of bid laws governing public works. In most instances, a specific project's metal building supplier is unknown until bids are received and a qualified low bidder is identified.

## 2. GENERAL DISCUSSION

Although the marketing and manufacturing of metal building systems differ significantly from conventionally produced steel structures, metal building systems are not inherently mysterious nor do their structural systems defy the laws of structural mechanics.

As with any element of a construction project, the specifying design professional is expected to possess a working knowledge of the characteristics, advantages, and limitations of the product in question.

Today's metal buildings are no longer the "pre-engineered" product of decades ago. Virtually any low-rise building configuration, which can be framed in conventional steel, can be framed with a metal building system. However, Designers and Owners are cautioned that attractive square-foot cost estimates, derived from the most basic metal-clad building, will not be applicable to a fully engineered metal building that involves masonry wall systems, complex geometry, etc. Do not start with a basic metal building cost, delete wall panel costs, add masonry wall costs, and expect to have a valid cost estimate for a masonry-clad metal building.

### 3. AISC CERTIFICATION

- 3.1. Possession of certification in Category MB of the AISC Quality Certification Program should be viewed as indication that a manufacturer “has the personnel, organization, experience, procedures, knowledge, equipment, capability, and commitment to produce fabricated steel of the required quality for a given category of structural steelwork.” The State Construction Office recommends that AISC certification be a required qualification for an acceptable manufacturer and that any AISC certified manufacturer be granted reasonable consideration for inclusion as an acceptable vendor.
- 3.2. In recognition of the fact that some small, regional manufacturers may not be AISC certified, but may be capable of producing a quality product for projects of limited size, it is suggested that the potential acceptability of such manufacturers be considered if the Designer or Owing Agency has had satisfactory experience with a particular manufacturer on previous similar projects.
- 3.3. In deciding whether or not to require AISC certification, the Designer should consider that the State Construction Office (SCO) deems AISC certification sufficient to render a manufacturer an “approved fabricator” for purposes of Special Inspections as addressed by Section 1704.2 of the North Carolina State Building Code (NCSBC). The SCO will likely require Special Inspections for larger, more complex metal building projects (see the SCO “Special Inspections Guidelines” for more discussion).

### 4. BUILDING

- 4.1. The specification should state the type of building (rigid frame clear span, multi span, gabled, single slope, etc.). The minimum roof slope shall be restricted to 2:12 (unless the roof is to be of the standing-seam type, in which case the minimum roof slope may be 1/4:12). Endwall framing shall be specified as expandable or nonexpandable. Maximum permissible column depths or minimum clearances should be clearly indicated on the drawings.
- 4.2. Bracing should be stated to be by cross-bracing to the extent possible for the particular project. Moment resisting portal frames and other alternate systems shall be specified when required to accommodate openings in walls. Permissible locations of wall bracing systems shall be indicated on the drawings.
- 4.3. Designers should avoid restricting design and fabrication by specifying particular structural shapes and cold-formed member dimensions unique to a particular manufacturer.
- 4.4. Wall and roof panels shall be specified to be formed from a minimum of 26 gage steel coil material (standing seam roof panels shall be 24 gage minimum).
  - 4.4.1. The nature of a specific project may warrant a greater minimum panel thickness. Designers are reminded that panel costs may increase disproportionately for thicknesses that are not chosen from a manufacturer's normal inventory.

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4.4.2. The finish on both roof and wall panels shall have a 20-year manufacturer's warranty. The roofing system shall have a 10-year manufacturer's warranty against leakage as well as an independent 2-year contractor's warranty against leakage.

4.4.3. The panels should be specified as to panel type, color, and/or finish on the availability of equals from three different manufacturers.

4.5. Building Nomenclature and Layout:

4.5.1. Building width and length shall be from inside to inside of wall panels. The building eave height shall be measured from the top of finished floor to the top of the eave strut. The top of the eave strut is the point of intersection of the inside surfaces of the roof and wall covering.

4.5.2. The bay spacing shall be measured as follows:

4.5.2.1. Interior bays are measured from centerline to centerline of interior frames.

4.5.2.2. End bays are measured from the inside of the endwall sheets to the center-line of the first interior frame.

4.5.3. Other dimensional relationships:

4.5.3.1. The Designer shall clearly define the outer face of foundation concrete with respect to the building girt line (inner face of wall panel).

4.5.3.2. The Designer should dimension interior partitions and other features with respect to constant building dimension lines rather than to locations that may vary among manufacturers (such as a variation of wall girt depth or anchor bolt centerline).

## 5. DESIGN CRITERIA

The Designer of Record must furnish structural design information and performance requirements that are sufficient to enable the metal building manufacturer to effect a complete and proper design of the metal building.

### 5.1. Specification of Design Loads:

Specification of design loads shall follow the basic requirements of NCSBC Section 1603. The Designer should keep in mind that some of the code-required data, such as Seismic Base Shear and Seismic Analysis Procedure, are determined by the metal building manufacturer in the course of final building design.

The following paragraphs amplify certain design load information that is uniquely important in the design of metal building structures. The bid documents must contain the following information underlined below:

- 5.1.1. Collateral Loads: The project drawings shall clearly show the location and nature of any concentrated roof loads such as roof-top HVAC units or underhung air handling units. Provide a project-specific uniform loads (psf) for widespread hanging loads such as suspended ceilings, lights, sprinklers, etc. Do not require the manufacturer to determine the weights of these systems; the Designer is the entity most capable of determining this information. [Code reference: Section 1606.2]
- 5.1.2. Snow Loads: Provide sufficient data for the design.
  - 5.1.2.1. Ground Snow: Interpolated from the NCSBC Ground Snow Loads map. For some mountainous areas, local records should be consulted and, if necessary, a higher ground snow load should be specified.
  - 5.1.2.2. Exposure Factor & Thermal Factor: These factors shall be determined and stated by the Designer.
  - 5.1.2.3. Snow Drift: If the project is subject to drift *from* an adjacent building, provide sufficient information (existing building eave height, roof slope, roof plan dimensions, etc.) to permit drift calculation by the metal building manufacturer. If the project will *subject* an existing structure to snow drift, the Designer must evaluate the existing structure and design any necessary reinforcement; this is not the responsibility of the new metal building manufacturer.
- 5.1.3. Wind Design Data: Metal buildings designs are sensitive to small variations in wind load. Therefore, it is important to correctly prescribe wind design loads.
  - 5.1.3.1. Basic Wind Speed: Provide a wind velocity *interpolated* from the NCSBC Basic Design Wind Speeds map. "Rounding-up" to the nearest higher 10-mph wind contour may impose a significant cost penalty upon the design.

5.1.3.2. Wind Exposure Category: The metal building design engineer, likely located in a remote manufacturing plant, cannot assess the ground surface roughness proximate to the project site. The Designer shall determine and specify the Wind Exposure Category. Keep in mind that, in the current NCSBC wind provisions, Exposure Category D is not applicable to any NC project site.

5.1.3.3. Building Enclosed or Partially Enclosed: The Designer shall define the building accordingly.

5.1.4. Further Discussion of Design Loads:

5.1.4.1. Dead Load: This is the actual self-weight of the metal building. The metal building manufacturer will calculate this.

5.1.4.1.1. Do not specify a “dead load” or confuse it with the collateral load discussed above. [Code reference: Section 1606.1]

5.1.4.1.2. Designers are cautioned that metal building self-weight dead loads are quite low, often on the order of 3-psf. This may significantly impact foundation designs due to the possibility of significant net uplift at the bases of columns.

5.1.4.2. Roof Live Loads: Specify roof live loads to be “As required by the North Carolina State Building Code”. Period. [Code reference: Section 1607.11]

5.1.4.2.1. Roof Live Load Reductions: NCSBC Section 1607.11 permits the reduction of roof live loads based upon the tributary area of individual elements of the roof structure.

5.1.4.2.1.1. The SCO does not prohibit such reductions. Hence, the SCO recommends the wording stated above.

5.1.4.2.1.2. If, for project-specific reasons, the Designer does not want to permit roof live load reductions, the bid documents must *explicitly* state so.

5.1.4.2.2. Do not specify roof live loads with vague or confusing terminology.

5.1.4.2.2.1. Statements such as “Roof live load shall be a minimum of 20 psf” suggest that tributary live load reduction is not permissible.

5.1.4.2.2.2. Do not confuse roof live load with snow load. Do not add a collateral load to the code-required live load to create an inappropriate “live load”. List loads individually as addressed by the NCSBC.

5.2. Specification of Performance Criteria: Metal buildings are relatively flexible structures in comparison to conventional steel framed buildings. Metal building horizontal and vertical deflections are seldom of concern for basic metal-clad buildings used in traditional commercial and light industrial applications. However, as these buildings become more

commonly used as the basis for more complex and more attractive institutional facilities, building deflections and the support of finished ceilings and/or masonry cladding become significant design issues.

Failure to recognize the behavior (deflections) of typical metal building structures (as well as many conventional steel structures) may result in the distress of brittle finishes that are applied indiscriminately to the building system. Conversely, overly stringent performance criteria may more than offset the economy sought with the original selection of a metal building system for a particular project.

#### 5.2.1. Vertical Deflections:

5.2.1.1. The Designer's attention is directed to NCSBC Table 1604.3.

5.2.1.2. Be advised that the code-specified deflection limits do not ensure that deflection-related difficulties are avoided.

5.2.1.2.1. The Designer should keep in mind that deflections within the code limits could still have problematic absolute values. For example, a building with a 120-foot clear-span primary rigid frame has a deflection limit of  $l/240$  in response to snow loads while supporting a non-plaster ceiling. This computes to a vertical deflection of *6-inches*. An acoustic tile ceiling, suspended from the structure and abutted by ceiling-height partitions below, will exhibit dramatic deformation under applied roof load. Such deformations can panic building occupants and damage ceiling grids, etc.

5.2.1.2.2. Absolute deflections of individual structural components may be additive. For example, a roof purlin spanning 25-feet between the above example's frames could add another inch of deflection to the total amount; while remaining in compliance with code requirements.

5.2.1.2.3. A dramatic tightening of deflection limits, say to  $l/720$ , might appear expedient from the Designer's standpoint. However, such deflection limits would likely render the metal building structure (and most other long-span systems) unworkable. The necessity of the long clear-span might need re-evaluation along with more forgiving wall-to-ceiling details.

5.2.2. Horizontal Deflections: Aside from limits prescribed for seismically induced lateral drift, the NCSBC is largely silent regarding lateral drift of buildings and lateral deflection of building components.

5.2.2.1. General Horizontal Deflection Discussion: Lateral drifts are most pronounced when lateral forces are applied parallel to the moment-resisting frames (generally normal to the building ridgeline).

5.2.2.1.1. If lateral drifts are resisted by the primary moment frames, some *general* relationships should be kept in mind:

- 5.2.2.1.1.1. As clear-span distances increase, gravity loads cause rafter and column sections to increase in size. The added stiffnesses of these members tend to reduce lateral drift caused by wind and seismic forces.
- 5.2.2.1.1.2. Conversely, as rafter spans decrease in multi-span frames, the rafters tend to become more limber and lateral drifts increase.
- 5.2.2.1.1.3. Frames with steep roof slopes tend to drift more than low-slope frames of the same span.
- 5.2.2.1.2. While moment-resisting frames are ubiquitous to metal building construction, the Designer should remember that vertical cross-bracing or shearwalls can be used in conjunction with horizontal roof cross-bracing to greatly reduce drifts parallel to the frames. This does require thoughtful planning (perhaps consultation with one or more manufacturers) and careful specification of the Designer's intent.
- 5.2.2.1.3. Rigid frame lateral drifts can be dramatically reduced by the use of fixed-base columns. However, the Designer must understand that the resultant savings in steel frame weight may be more than offset by the demands placed upon the building foundations. Fixed-base columns should be used only after a thorough analysis of related foundation costs.
- 5.2.2.2. Special Cases of Lateral Drift: Although lateral building drift is often associated with applied lateral wind or seismic forces, certain frame geometries can result in unanticipated frame movements when a frame is subjected to gravity loads only.
  - 5.2.2.2.1. Haunch spread of clear-span frames: Symmetric clear-span rigid frames can exhibit spread at the haunch (column-to-rafter) areas, near the building eaves, in response to applied gravity loads. In general, this effect is more pronounced with steeper roof slopes. It is not unusual for rigid frame haunches to move laterally by more than 1-inch in longer-span frames (120-feet and greater).
  - 5.2.2.2.2. Unequal-span multi-span frames: Multi-span frames with significantly differing spans (particularly the two outer-most spans) can drift horizontally as gravity loads are added. For example, a two-span frame, with spans of 60' and 30', will tend to drift toward the side with the 30' span.
- 5.2.2.3. Masonry Wall Systems: Most lateral drift concerns, related to metal buildings, involve lateral support of masonry wall systems. In designing and specifying the complete building envelope, the Designer must consider the interaction of the wall materials and metal building's structural framing system; just as he/she would for any conventionally framed building.
  - 5.2.2.3.1. Overall lateral drift of the building, often referred to as frame drift, must be compatible with the design and detailing of the masonry walls.



- 5.2.2.3.1.1. Depending upon the details (intentional flexibility at wall bases, etc.) utilized in the design of the masonry walls, the walls may be capable of tilting with the frame deflections. This capability is known only to the Designer and should be reflected in the prescription of any lateral frame drift limitations. Wall movement at building corners and interior partitions bears thoughtful consideration when utilizing this approach.
- 5.2.2.3.1.2. As with *any* steel framing system, the Designer must be aware of the frame's response to lateral loading and ensure that rigid non-loadbearing elements do not inadvertently assume loads.
- 5.2.2.3.2. Support of masonry walls *between* main columns is often a source of confusion and dispute. The metal building industry generally assumes that wall materials, by others, are self-supporting between columns. This assumption may be entirely valid in some instances and overly optimistic in others. The Designer must determine the support requirements for the specified wall materials and ensure that the contract documents clearly prescribe responsibility and criteria for any manufacturer-supplied wall components.
  - 5.2.2.3.2.1. Light-gage cold-formed wall girts and eave struts are typically designed to support only the *wall panel area* tributary to the member.
  - 5.2.2.3.2.2. If the metal building manufacturer is to provide structural girt members to support masonry wall systems, the contract documents must define this role for the girt members and prescribe performance criteria for the members (deflection ratios, etc.).
  - 5.2.2.3.2.3. With suitable design methodology, relatively short partial-height masonry "wainscot" walls can be economically cantilevered from perimeter foundations. In such cases, the Designer should thoughtfully detail the masonry-to-superstructure interface to avoid imposition of lateral loads *upon* the top of the wall as a result of lateral drift of the building frames.
  - 5.2.2.3.2.4. Vertical crack control joint locations, in the exterior masonry, must be chosen with consideration of the supporting wall girt arrangements and specified wall-to-girt connections.

## 6. FOUNDATIONS:

Metal building foundations are often addressed either indifferently or with excess anxiety by the general design community. With reasonable care and effort, a Designer can provide correct and efficient foundation designs that need little or no revision when the successful metal building bidder supplies final reactions.

### 6.1. Estimation of Building Reactions:

6.1.1. Designers should seek preliminary reaction values from at least two metal building manufacturers.

6.1.2. Reasons for disparities between estimated preliminary and final reactions:

6.1.2.1. Miscommunication of design loads: See Section 5.1 above.

6.1.2.2. Frame stiffnesses vary among manufacturers, depending upon the frame optimization strategy used by each manufacturer: Indeed, frame reaction distributions react to changes in rafter or column properties. However, such variations seldom prove significant (variations exceeding 10 percent), all other building criteria being equal and correct.

6.1.2.3. Changes in roof slope: For a particular symmetrical gabled frame clear-span, horizontal reactions may increase greatly with a change in roof slope from 1:12 to 4:12, for example.

6.1.2.4. Failure to consider the effects of longitudinal vertical bracing: Vertical force components of diagonal bracing elements can be several times larger than uplift reactions that would occur at primary frames not involved with such vertical bracing.

6.2. Base-Bid Foundation Design: The construction documents shall depict a foundation system that is fully capable of resisting all forces imposed by the specified metal building structure.

6.3. Adjustment of Contract Sum in Response to Final Reactions: Upon receipt of anchor bolt setting plans and final reactions from the contracted metal building supplier, the Designer shall review these reactions and adjust the final foundation design if necessary.

6.3.1. This evaluation and redesign are not the responsibility of the metal building manufacturer.

6.3.2. The contract Form of Proposal shall solicit unit prices for reinforced foundation concrete as a means to adjust the contract if any change is required from the base-bid foundation configuration.

6.4. Foundation Considerations: With their low self-weight dead load and continuous primary rigid frames, metal buildings may produce column reactions that markedly differ from the reactions seen for conventionally framed buildings.

- 6.4.1. Wind Uplift: Metal buildings often possess self-weight dead loads as small as 3 psf. With so little dead load to resist wind uplift forces, net uplift column reactions can be quite large, particularly at columns that are part of a bay of vertical bracing. As a result, foundation sizes may be controlled by wind uplift rather than by soil bearing capacity in response to gravity loads. In such cases, top-of-footing reinforcing steel may be necessary.
- 6.4.2. Horizontal Reactions: Moment resisting rigid frames can exert substantial horizontal forces upon foundation structures. Large clear-span rigid frames often have very large horizontal reactions induced by gravity loads. The foundation system must provide some means of resistance for these base-of-column forces acting outward and normal to the foundation perimeter.
  - 6.4.2.1. Horizontal Ties: Below-slab concrete encased reinforcing bars may extend across the building, tying the exterior column bases together.
    - 6.4.2.1.1. While this is a seemingly simple solution, the Designer must remain aware of axial deflection of these very long tie elements.
    - 6.4.2.1.2. The ties must be suitably developed into the column piers.
    - 6.4.2.1.3. The tie details and locations must consider the jointing and other attributes of the slab-on-grade.
  - 6.4.2.2. Hairpin Bars and the Slab-on-Grade as a Tension Member: Wire mesh, in the slab-on-grade, is often developed by hairpin bars that are anchored into the column piers. This widely used approach must maintain continuity of the slab-on-grade's continuity as a tension member.
    - 6.4.2.2.1. Slab-on-grade crack control joint details and construction joint details often explicitly depict a complete discontinuity of the wire mesh or at least the partial severing ("cut every other strand") of the mesh at the joint. In other cases, the prescribed saw-cut joint depth inadvertently exceeds the prescribed depth of the wire mesh.
    - 6.4.2.2.2. Trench drains and floor slab elevation changes can also invalidate the assumed floor slab continuity across the span of the building.
    - 6.4.2.2.3. In instances of floor slab continuity, as described above, exercise caution when assuming that the local slab-on-grade elements generate sufficient friction against the slab subgrade to permit resistance of significant horizontal reactions. Keep in mind that floor slabs are often placed over plastic vapor barriers that have very low coefficients of friction.
  - 6.4.2.3. Moment Resisting Footings: In cases where horizontal tie systems are undesirable or unworkable, the horizontal column reactions may be resisted by moment resisting footings. Such footings will likely be significantly larger than ordinary spread footings.

- 6.4.2.3.1. The distance from column base plate to top-of-footing will greatly effect the size of the foundation.
- 6.4.2.3.2. The Designer must consider subgrade deformations in response to long-term overturning forces from permanent dead loads and short-term overturning forces from snow or live loads.
- 6.4.2.3.3. Nothing mandates that columns must be centered over moment resisting footings. Provided that all load cases are thoughtfully considered, eccentric location of the footing may greatly reduce overturning effects of the controlling load combination.
- 6.4.2.4. Use of Passive Earth Pressure: Grade beams can be designed to transfer horizontal column reactions into the earth surrounding the perimeter of the building. However, this approach is valid only if the soils and foundation-to-soil interface are conducive to the proper development of reliable passive earth pressure. For example, the perimeter grade beam concrete would have to be cast directly against undisturbed soils or against very well compacted backfill soils (difficult to achieve without clear specifications and careful construction testing). The perimeter soils would have to be permanent; a future building expansion could not disturb those soils or the lateral capacity would be lost. Also, the perimeter soils would have to be protected from erosion or saturation.

## 6.5. Foundation Details:

- 6.5.1. Anchor Bolts: Metal building anchor bolts are often supplied by the manufacturer or based upon sizes and configurations recommended by the manufacturer. Manufacturer-designed anchor bolts are sized for shears and tension loads computed from reactions for the building design load combinations.
  - 6.5.1.1. These shear and tension loads are assumed to be applied at the top of the concrete pier that contains the anchor bolts. There is typically no allowance for bending forces that might occur in the anchor bolts due to placement of the column bases on grout beds. The Designer must thoughtfully consider the load path from the base plate to the main column pier.
  - 6.5.1.2. Anchor bolt lengths are normally selected to develop the full tensile capacity of the bolt, computed for pull-out from monolithic plain concrete.
    - 6.5.1.2.1. The Designer must review his proposed foundation arrangement to determine if any conditions would violate the pull-out capacity assumptions.
    - 6.5.1.2.2. The Designer must ensure that column piers are adequately connected to footings. Relying upon long anchor bolts, to replace pier-to-footing dowels, is only acceptable if the contract documents explicitly prescribe anchor bolt length and if dowels and associated ties are not required by code. If anchor bolt length is critical to the Designer's foundation design, then the metal building manufacturer's anchor bolt setting plans and shop drawings

must be meticulously reviewed to verify that the anchor bolts are as specified.

- 6.5.2. Slab-on-Grade Control Joint Arrangements: In conventional buildings, control joints frequently occur on frame lines and terminate at diamond-shaped piers at main columns. This may not be a suitable arrangement for some metal building projects.
  - 6.5.2.1. Many small metal buildings are founded on thickened-slab footings. These are spread footings that are cast monolithically with the slab-on-grade. Because the footing is integral with the slab and the column base plate bears directly in top of the footing, there is no isolation between the column and the main slab-on-grade. A saw-cut joint, on the frame centerline, would extend into the thickened slab "footing".
    - 6.5.2.1.1. This increase in effective slab thickness would tend to negate the reduced-section mechanism that is supposed to promote the formation of a controlled crack in the slab. Therefore, the crack is likely to follow the perimeter of the footing; an undesirable outcome.
    - 6.5.2.1.2. In the event that the crack follows the saw-cut joint, then the crack will extend into the very center of the footing, directly beneath the column. Again, this is not desirable.
    - 6.5.2.1.3. If thickened-slab footings are proposed for a project, then control joint layouts should be arranged to avoid the footings.
  - 6.5.2.2. A foundation system that utilizes horizontal ties to resist horizontal column reactions needs careful detailing to avoid conflicts with control joints. As with the thickened-slab footings, if the horizontal ties' concrete encasement is cast integrally with the slab-on-grade, keep in mind that saw-cut control joints, on the frame centerlines, will be negated by the added thickness of the ties' encasement.

## 7. DESIGN DETAIL CONSIDERATIONS:

- 7.1. **Suspended Ceiling Systems:** As mentioned in Section 5.2.1 “Vertical Deflections”, all long-span structures, including long-span metal building structures, deflect in response to applied vertical loads. Suspended ceiling systems will deflect with the roof structure. Wherever such ceilings are supported by partition walls, the ceiling grid will deform. If long-span framing is needed to fulfill project objectives, then the architectural details must anticipate and accommodate movements associated with such framing systems.
- 7.2. **Column Flange Braces Versus Architectural Finishes:** Metal building column inner flanges are commonly braced, in the horizontal plane, by diagonal struts (often small structural angles) that connect to the wall girts. If wall girts are likely to occur below the elevation of a finished ceiling, the Designer must remember that flange braces may very well occur at one or more of these girts.
  - 7.2.1. While the unformed may see these braces as minor accessories that can be discarded without further thought, column flange braces significantly effect the structural capacity of the columns.
  - 7.2.2. If flange braces are not desirable below a particular elevation, this must be clearly specified in the construction documents. The metal building manufacturer can then design the inner flange as an unbraced element (at some increased cost).

## 8. RETROFIT ROOF SYSTEMS:

The use of metal roof systems as “retrofits” over existing roof structures has become increasingly common over the last two decades. The most well known retrofit projects are those that create a sloped roof system, supporting new metal panels, over top of an existing flat roof. The “retrofit” name is also used to describe the application of new metal roof panels over an existing sloped roof surface.

8.1. Basic Description: Metal retrofit systems usually use structural standing-seam roof panels supported by light-gage purlins that are in-turn supported by variable height posts (which impart the desired roof slope). The posts bear upon the existing roof structure and are anchored to the roof structure to resist uplift & lateral loads. Diagonal straps or rods are used in the vertical planes to provide lateral stability for the elevated roof system.

### 8.2. Conceptual Considerations:

8.2.1. Although metal roof systems’ first-costs typically exceed the first costs of conventional built-up or membrane roofing systems, the metal panel industry cites the favorable life cycle costs of such systems.

8.2.2. As with any roof system, minimizing transitions, valleys, and penetrations improves the chances for long lasting and leak free roof.

8.2.3. Roof-top fans, HVAC equipment, etc. will have a significant impact upon the viability of a sloped metal retrofit roof system for a particular project. Such elements can be elevated and mounted on roof curbs, but each piece of equipment will have direct structural costs and potential leakage issues at the attendant roof penetrations.

### 8.3. Responsibilities of the Designer-of-Record:

#### 8.3.1. Existing Roof Capacity and Loading Criteria

8.3.1.1. The Designer must evaluate the existing building’s roof condition and load capacity. These tasks are not the responsibility of the Contractor.

8.3.1.1.1. The existing roof deck and structure must be carefully inspected and evaluated by the Designer.

8.3.1.1.2. Retrofit roof systems are quite light. Total additional load will not usually present a problem. In some cases, existing stone roof ballast can be removed to more than offset the added weight of the retrofit roof system.

8.3.1.2. The Designer must determine and specify parameters that yield acceptable load patterns upon the existing structure.

8.3.1.2.1. While total loading may be well within the capacity of the existing structure, the applied point loads (at the bases of the structural posts) must not overload individual members or create local distresses.

- 8.3.1.2.1.1. For example, light gage retrofit purlins may be able to span 10-feet or more. However, if existing roof joists occur at a spacing of 5-feet, the 10-foot purlin spans could effectively double-load every other roof joist while adjacent joists pick up no additional load.
- 8.3.1.2.1.2. Roof deck elements must be analyzed for their ability to withstand point loads imposed by the post bases.
- 8.3.1.2.2. The prescribed load pattern limitations shall be based upon a thorough and thoughtful review of industry practices, including typical purlin spans, post-to-deck fastening details, etc.
- 8.3.2. Consider Unusual Configurations Yielding Increased Lateral Wind Loads: In most cases, the added height of retrofit roof systems is relatively small in comparison to the existing building height. Therefore, increases in total lateral wind loads, imposed upon the existing building structure, are typically negligible. However, the Designer should remain cognizant of the effects of steep roof slopes and overall height increases that can occur (even with low-slope systems) if a building is quite wide. In these cases, overall lateral wind forces can increase significantly versus the building's original total lateral wind forces.
- 8.3.3. Provide Sufficient Data to Facilitate Design By Retrofit System Supplier: Although final design of the light-gage metal framing is usually done by the system supplier, the Designer must provide project specific information.
  - 8.3.3.1. Define the elevations, above existing finished-floor or grade, of existing rooftops and other roof elements such as parapets, etc.
  - 8.3.3.2. Define the project site's Wind Exposure Category.
  - 8.3.3.3. If adjacent roofs might subject the new retrofit roof to snowdrifts, provide information regarding the elevation of those roofs and their plan dimensions.
  - 8.3.3.4. If local conditions yield ground snow in excess of that mapped in the Code, define that ground snow value.
  - 8.3.3.5. Define any special corrosion resistance requirements for the new-to-existing connection fasteners. These fasteners often penetrate damp or wet existing insulation materials that may contain corrosive chemicals.
- 8.3.4. Structure Contract Documents to Yield Integrated System: Unless the Designer possesses considerable knowledge of metal roofing and framing systems, the Designer should specify the project in a manner that assures a single point of supplier-performed design responsibility. Indiscriminate definition of light-gage metal framing in a Division 5 specification and metal panels in a discreet Division 7 specification may lead to deficiencies in structural performance of the final complete system.



- 8.3.5. Ventilation of the Attic Space: The new sloped roof system will create an attic space above the existing roof. This space should be carefully evaluated for ventilation requirements. Ventilation design is the responsibility of the Designer - the retrofit system supplier's sealed shop drawings typically certify only the structural design of the system.
- 8.3.6. Insulation of the Roof Deck and Endwalls: Existing roof insulation may continue to provide the primary thermal barrier for the building. However, several issues may prompt the use of insulation beneath the new roof panels and inside the new endwall panels.
  - 8.3.6.1. Weather and attic ventilation conditions may necessitate the installation of insulation to prevent condensation problems.
  - 8.3.6.2. Minimal insulation is sometimes installed to prevent objectionable wind-induced buzzing or flutter of the roof panel flats upon the supporting purlins.

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## 9. SUGGESTED READING

- 9.1. "Metal Building Systems, Design and Specifications"  
by Alexander Newman 2004 McGraw-Hill

*Offers a comprehensive and knowledgeable overview of metal building systems.*

- 9.2. "Serviceability Design Considerations for Steel Buildings", 2<sup>nd</sup> Edition  
American Institute of Steel Construction (AISC) 2004

*Supercedes the 1990 "Serviceability Considerations for Low-Rise Buildings". Very good discussion of serviceability issues, as opposed to the more commonly considered strength aspects of structures. Because state buildings must have long service lives, this office disagrees with AISC's recommendation of a 10-year recurrence frequency wind event when evaluating structural drift.*

- 9.3. "Concrete Masonry Walls for Metal Buildings"  
National Concrete Masonry Association (NCMA) 1996

*Generally complements the above AISC Serviceability publication and provides good details. Ironically, the artwork on the book cover depicts several questionable masonry-to-metal-building conditions.*