engineer's notebook

Designing Cost-Competitive Low-Rise Steel Buildings

Part 1

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STRUCTURE magazine is pleased to present a new Column, The Engineer's Notebook. The purpose of the column is to provide practical information and design aides to assist engineers in their day-to-day tasks. STRUCTURE encourages you to clip these articles and include them in your Notebook.

In this issue, James Fisher provides practical information on cost/benefit decisions for low-rise steel buildings. This is the first installment of a two-part article. Watch for Part 2 in the May 2004 issue of STRUCTURE.

The selection of "the best" framing scheme for a low-rise building is dependent on numerous considerations, and often depends on the owners' requirements. The following article is an assessment of initial costs and the beneficial decisions that can be made for the life of the structure.

Roof Systems

The roof system is often the most expensive part of a low-rise building (even though walls are more costly per square foot). Designing for a 20 psf mechanical load, when only 10 psf is required, adds cost over a large area.

Often, the premise that guides the design is that the owner will always be hanging new piping or installing additional equipment. A prudent designer will allow for this in the system. If this practice is followed, the owner should be consulted and the decision to provide excess capacity should be the owner's. The design live loads and collateral (equipment) loads should be noted on the structural plans, so that future requirements can be more easily addressed.

Steel Deck for Built-up or Membrane Roofs

Decks are commonly 1-1/2 inches deep, but deeper units are also available. The Steel Deck Institute[2] has identified three standard profiles for 1-1/2 inch steel deck (narrow rib, intermediate rib and wide rib), and has published load tables for each profile for thicknesses varying from .0299 to .0478 inches. These three profiles, NR, IR, and WR, correspond to the manufacturers' designations A, F and B respectively. A comparison of weights for each profile in various gages shows that strength to weight ratio is most favorable for wide rib and least favorable for narrow rib deck. In general, the deck selection that results in the least weight per square foot may be the most economical. However, consideration must also be given to the flute width because the insulation must span the flutes. In addition to the load, span, and thickness relations established by the load tables, there are other considerations in the selection of a profile and gage for a given load and span. First, the Steel Deck Institute limits deflection due to a 200-pound concentrated load at midspan to span divided by 240.

Secondly, the Steel Deck Institute has published a table of maximum recommended spans for construction and maintenance loads, and Factory Mutual lists maximum spans for various profiles and gages in its *Approval Guide* [1].

Steel decks can be attached to supports by welds or fasteners, which can be power or pneumatically installed, or self-drilling/self-tapping. The attachment of roof deck must be sufficient to provide bracing to the structural roof members, to anchor the roof to prevent uplift, and, in many cases, to serve as a diaphragm to carry lateral loads to the bracing. While the standard attachment spacing may be acceptable in many cases, decks designed as diaphragms may require additional connections. Diaphragm capacities can be determined using the *Diaphragm Design Manual* [3].

Metal Roofs

Standing seam roof systems were first introduced in the late 1960's, and today many manufacturers produce standing seam panels. A difference between the standing seam roof and lap seam roof (through-fastener roof) is in the manner in which two panels are joined to each other. The seam between two panels is made in the field with a tool that makes a cold formed, weather-tight joint. (Note: some panels can be seamed without special tools.) The joint is made at the top of the panel. The standing seam roof is also unique in the manner in which it is attached to the purlins. The attachment is made with a clip concealed inside the seam. This clip secures the panel to the purlin, and may allow the panel to move when experiencing thermal expansion or contraction.

A continuous single skin membrane results after the seam is made, since through-the-roof fasteners have been eliminated. The elevated seam and single skin member provides a watertight system. The ability of the roof to experience unrestrained thermal movement eliminates damage to insulation and structure (caused by temperature effects which built-up, which through-fastened roofs commonly experience). Thermal spacer blocks are often placed between the panels and purlins in order to insure a consistent thermal barrier.

Because of the ability of standing seam roofs to move on sliding clips, they possess only minimal diaphragm strength and stiffness. The designer should assume that the standing seam roof has no diaphragm capability and, in the case of steel joists, specify that sufficient bridging be provided to laterally brace the joists under design loads.

Roof Pitch, Drainage and Ponding

Prior to determining a framing scheme and the direction of primary and secondary framing members, it is important to decide how roof drainage is to be accomplished. If the structure is heated, interior roof drains may be justified. For unheated spaces, exterior drains and gutters may provide the solution.

Performance of roofs with positive drainage is generally good. Problems (e.g., ponding, roofing deterioration, leaking), which may result from poor drainage, lead to the recommendation that a roof slope of at least 1/4 inch per foot be provided for all building roof systems. Ponding as it applies to roof design has two meanings. To the roofing industry, ponding describes the condition in which water accumulated in low spots has not dissipated within 24 hours of the last rainstorm. Ponding of this nature is addressed in roof design by positive roof drainage, and control of the deflections of roof framing members. Ponding as an issue in structural engineering is a load/deflection situation in which there is incremental accumulation of rainwater in the deflecting structure. The purpose of a ponding check is to insure that equilibrium is reached between the incremental loading and the incremental deflection. This convergence must occur at a level of stress that is within the allowable value.

The AISC Specifications for both LRFD [5] and ASD [8] give procedures for addressing the problem of ponding where roof slopes and drains may be inadequate. The direct method is expressed in Eq. K2-1 and K2-2 of the Specifications. These relations control the stiffness of the framing members (primary and secondary) and deck. This method, however, can produce unnecessarily conservative results. A more exact method is provided in Appendix K of the LRFD[5] Specification, and in Chapter K in the Commentary in the ASD[8] Specification.

The initial stress ("at the initiation of ponding") is determined from the loads present at that time. These should include all or most of the dead load and may include some portion of snow/rain/live load. Technical Digest No. 3, published by the Steel Joist Institute [10], gives some guidance as to the amount of snow load which could be used in ponding calculations.

Joists and Joist Girders

A decision must be made whether to span the long direction of bays with the joist girders that support the joists, or to span the short direction of bays with main framing members that support longer span joists. Since the joists sit on top of the girder, they can be made deeper than the joist girder (by the amount of the seat depth) without infringing upon the clear height requirements. Experience in this regard is that spanning the shorter bay dimension with primary members will provide the most economical system. However, this decision may not be based solely on economics, but rather on such factors as ease of erection, future expansion, location of overhead doors, etc.

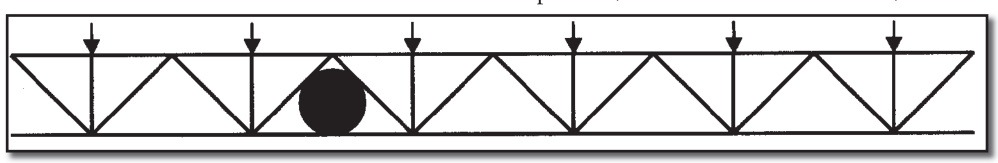

Figure 1. Typical VG Girde

Experience has also shown that wide joist spacing provides very economical roof and floor systems. In fact, the widest spacing for a given deck profile and slab thickness should always be used. The wider joist spacing provides several advantages. Typically erection costs are less, and the wider joist spacing provides a floor system with better vibration characteristics. The joists are deeper, thus allowing larger penetrations through their web openings.

VG type joist girders can often be used to advantage when mechanicals are run between the joists and through the joist girders. This joist girder type aligns an open panel in the girders with the space between joists, thus the joist bottom chord does not impinge on the opening in the girder. A typical VG girder is shown in *Figure 1*.

The wall system can be chosen for a variety of reasons, and the cost of the wall can vary by as much as a factor of three. Wall systems include:

1. Field assembled metal panels,
2. Factory assembled metal panels,
3. Precast concrete panels,
4. Masonry walls (part or full height).

A particular wall system may be selected over others for one or more specific reasons, including:

1. Cost,
2. Appearance,
3. Ease of erection,
4. Speed of erection,
5. Insulating properties,
6. Fire considerations,
7. Acoustical considerations,
8. Ease of maintenance/cleaning,
9. Ease of future expansion,
10. Durability of finish,
11. Maintenance considerations.

Specific advantages and disadvantages of the various wall systems are discussed in *Industrial Buildings, Roofs to Column Anchorage*, AISC Design Guide 7.

Wind Columns

When bay spacings exceed 30 feet, additional intermediate columns may be required to provide for economical girt design. Two considerations that should be emphasized are:

1. Sufficient bracing of the wind columns to accommodate wind suction loads is needed. This is normally accomplished by bracing the interior flanges of the columns with angles, which connect to girts.

2. Proper attention should be paid to the top connections of the columns. For intermediate sidewall columns, secondary roof framing members must be provided to transfer the wind reaction at the top of the column into the roof bracing system. A positive and calculable system is necessary to provide a traceable load path. Bridging systems or bottom chord extension on joists can be used to dissipate these forces, but the stresses in the system must be checked. If the wind columns have not been designed for axial load, a slip connection would be necessary at the top of the column.

Girts

Typical girts for low-rise buildings are hot rolled channel sections, or cold-formed light gage C or Z sections. In recent years, cold-formed sections have gained popularity because of their low cost. Cold-formed Z sections can be easily lapped to achieve continuity, resulting in further weight savings and reduced deflections. Z sections also ship economically. Additional advantages of cold-formed sections compared with rolled girt shapes are:

1. Metal wall panels can be attached to cold formed girts quickly and inexpensively using self-drilling fasteners.

2. The use of sag rods is often not required.

Hot-rolled girts are often used when:

1. Corrosive environments dictate the use of thicker sections.

2. Common cold-formed sections do not have sufficient strength for a given span or load condition.

3. Girts will receive substantial abuse from operations.

4. Designers are unfamiliar with the availability and properties of cold-formed sections.

Both hot-rolled and cold-formed girts subjected to pressure loads are normally considered laterally braced by the wall sheathing.

Cold-formed girts should be designed in accordance with the provisions of the *North American Specification for the Design of Cold-Formed Steel Structural Members*[6]. Many manufacturers of cold-formed girts have provided this design and offer load span tables to aid design.

Section C3.1.2, *Lateral-Torsional Buckling Strength*, of the AISI Specification provides a means for determining cold-formed girt strength when the compression flange of the girt is attached to sheeting (fully braced), or when discrete point braces (sag rods) are used. For lapped systems, the sum of the moment capacities of the two lapped girts is normally assumed to resist the negative moment over the support. For full continuity to exist, a lap length on each side of the column support should be equal to at least 1.5 times the girt depth[7]. Additional provisions are given in Section C3 for strength considerations relative to shear, web crippling, and combined bending and shear.

Section C3.1.3, *Beams with One Flange Attached to Deck or Sheathing*, provides a simple procedure to design cold-formed girts subjected to suction loading.

Bay Size

Bay sizes and column spacing are often dictated by the function of the building. Economics, however, should also be considered. In general, as bay sizes increase, the weight of the horizontal framing increases. This will mean additional cost unless offset by savings in foundations or erection. Studies have indicated that square or slightly rectangular bays usually result in more economical structures. The author has studied various bay sizes to determine the optimum bay. In general, it can be stated that a 40-foot by 40-foot bay, in roof snow load areas greater than 20 psf, are the most economical. In areas where the roof snow is 20 psf or less, the optimum is closer to a 50-foot by 50-foot bay size.

Another factor that may be important is that, for the larger bays (greater than 30 ft), normal girt construction becomes less efficient using C or Z sections without intermediate "wind columns" being added. Additional economic and design considerations include:

1. When steel joists are used in the roof framing, it is generally more economical to span the joists in the long direction of the bay.

2. K- Series joists are more economical than LH- Series joists; thus an attempt should be made to limit spans to those suitable for K- Series joists.

3. For 30-foot to 40-foot bays, efficient framing may consist of continuous or double cantilevered girders supported by columns in one direction, and steel joists spanning the other direction.

4. If the girders are continuous, plastic design is often used. Connection costs for continuous members may be higher than for cantilever design; however, a plastically designed continuous system will have superior behavior when subjected to pattern load cases. All flat roof systems must be checked to prevent ponding problems.

5. Simple-span rolled beams are often substituted for continuous or double cantilevered girders where spans are short. The simple span beams often have adequate moment capacity. The connections are simple, and the savings from easier erection of such systems may overcome the cost of any additional weight.

6. Consideration must be given to future expansion and/or modification, where columns are either moved or eliminated. Such changes can generally be accomplished with greater ease where simple span conditions exist.▪

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"Designing Cost-Competitive Low-Rise Steel Buildings, Part 2" will cover Structural Systems and Serviceability Criteria. Watch for Part 2 in the May 2004 issue of STRUCTURE magazine.

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