engineer's notebook

Designing Cost-Competitive Low-Rise Steel Buildings

Part 2

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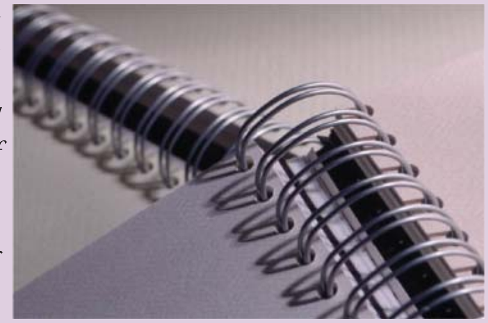
STRUCTURE magazine is pleased to present a new Column, The Engineer's Notebook. The purpose of the column is to provide practical information and design aides to assist engineers in their day-to-day tasks. STRUCTURE encourages you to clip these articles and include them in your Notebook.

In this issue, James Fisher provides practical information on cost/benefit decisions for low-rise steel buildings. This is the second installment of a two-part article. Look for Part 1 in the April 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 is an assessment of initial costs and the beneficial decisions that can be made for the life of the structure.



Structural Systems

Roof Bracing

Roof Diaphragms

The most economical roof bracing system is achieved by use of a steel deck diaphragm. The deck is provided as the roofing element, and the effective diaphragm is obtained at little additional cost (for extra deck connections). A roof diaphragm used in conjunction with wall X-bracing or a wall diaphragm system is probably the most economical bracing system that can be achieved. Diaphragms are most efficient in relatively square buildings; however, an aspect ratio up to 4:1 can be accommodated.

Cold-formed steel diaphragms are analogous to the web of a plate girder. That is, its main function is to resist shear forces. The perimeter members of the diaphragm serve as the "flanges".

The design procedure is quite simple. The basic parameters that control the strength and stiffness of the diaphragm are:

1. Profile shape,
2. Deck material thickness,
3. Span length,
4. Type and spacing of the fastening of the deck to the structural members,
5. Type and spacing of the side lap connectors.

The profile, thickness, and span of the deck are typically based on gravity load requirements. The type of fastening (i.e., welding, screws, and power driven pins) is often based on the designers' or contractors' preference. Thus, the main design variable is the spacing of the fasteners. The designer calculates the maximum shear per foot of diaphragm and then selects the fastener spacing from the load tables. Load tables are most often based on the requirements set forth in *Reference 3*.

Deflections are calculated and compared with serviceability requirements.

The calculation of flexural deformations is handled in a conventional manner. Shear deformations can be obtained mathematically using shear deflection equations, if the shear modulus of the formed deck material making up the diaphragm is known. Deflections can also be obtained using empirical equations such as those found in *Reference 3*. In addition, most metal deck manufacturers publish tables in which strength and stiffness (or flexibility) information is presented.

Braced Frames vs. Rigid Frames

Generally, braced frames are more economical than rigid frames. There are a few situations for which a rigid frame system is likely to be superior to a braced frame system.

1. Braced frames may require bracing in both walls and roof. Bracing frequently interferes with plant operations and future expansion. If either consideration is important, a rigid frame structure may be the answer.
2. The bracing of a roof system can be accomplished through X-bracing or a roof diaphragm. In either case, the roof becomes a large horizontal beam spanning between the walls or bracing which must transmit the lateral loads to the foundations. For large span to width ratios (greater than 4:1) the bracing requirements become excessive. A building with dimensions of 100 feet by 400 feet, with potential future expansion in the long direction, may best be suited for rigid frames. This would minimize or eliminate bracing that would interfere with future changes.

Experience has shown that there are occasions when braced frame construction may prove to be more economical than either standard metal building systems or special rigid frame construction when certain sacrifices on flexibility are accepted.

Braced Frames

An option usually exists as to whether the vertical bracing system should consist of steel members or be made of concrete or concrete block walls. No clear-cut answer exists as to which system is best. Using steel bracing has the distinct advantage that the frame is totally dependent only on the steel frame for its stability. Scheduling problems are often minimized, since the frame can be constructed independently. Using concrete or masonry shear walls eliminates some steel tonnage from the structure; however, the cost savings may be offset by scheduling delays, increased cost of reinforcement in the walls, and the details of connections between frame and shear walls.

As with any steel frame, most basic bracing configurations can be used with joist and joist girder framing. Chevron, K, single diagonal and X bracing are all practical and common.

Where beams are substituted for joists or joist girders eccentric bracing can be used as well.

Frame bracing is almost always located at the perimeter of the structure. Generally only a few windows exist and their locations can be avoided. Only overhead doors and exits must typically be avoided.

Much like roof diaphragms, the economical use of vertical bracing is dependent a great deal on the building geometry. When the length-to-width ratio between braces exceeds about 4 to 1, bracing forces become quite large. Strut forces transferring the forces to the braced bays also become large. With this ratio, significant column uplift forces are also developed, affecting foundation costs. To avoid uplift forces, it is recommended that bracing be placed in adjacent bays rather than separated so that uplift forces can be minimized. When the length-to-width ratios exceed 4 to 1, the designer should discuss the cost advantages of interior braced bays with the client. The need for interior bracing also occurs in large structures that are quartered by expansion joints. The client's first reaction is almost always negative to allowing interior vertical bracing. Sketches should be prepared of K bracing or eccentric bracing so that the client can see that forklift trucks or pedestrian traffic can be permitted thru the braced bays. If interior bracing is simply not permitted, an alternate lateral force resisting system such as rigid frames must be considered.

Rigid Frames

There are many considerations involved in providing lateral stability to low-rise steel structures. If a rigid frame is used, lateral stability parallel to the frame is provided by the frame. However, for loads perpendicular to the main frames and for wall bearing and "post and beam" construction, lateral bracing is not inherent and must be provided. It is important to re-emphasize that future expansion may dictate the use of a rigid frame or a flexible (movable) bracing scheme.

Since low-rise steel structures are normally light and generally low in profile, wind and seismic forces may be relatively low. Rigid frame action can be easily and safely achieved by providing a properly designed member at the sidewalls. If joist girders are used as a part of the rigid frame, the designer is cautioned on the following points:

1. The design loads (wind, seismic, and continuity) must be given on the structural plans so that the joist manufacturer can provide the proper design. The procedure must be used with conscious engineering judgment, and full recognition that standard steel joists are designed as simple span members subject to distributed loads. [9] Bottom chords are normally sized for tension only.

The simple attachment of the bottom chord to a column to provide lateral stability will cause gravity load end moments that cannot be ignored. The designer should not try to select member sizes for these bottom chords, since each manufacturer's design is unique and proprietary.

2. It is necessary for the designer to provide a well-designed connection to both the top and bottom chords to develop the induced moments, without causing excessive secondary bending moments in the joist chords.
3. The system must have adequate stiffness to prevent drift related problems such as cracked walls and partitions, broken glass, leaking walls and roofs, and malfunctioning or inoperable overhead doors.

Designing joist and joist girder structures as rigid frames is no more difficult than designing rigid frames with wide flange beams and girders. To obtain a cost effective design, the engineer must be aware of the inter-relationships between the framing elements, i.e. joists, joist girders, columns and connections. In general, the most economical design is one that minimizes fabrication and erection costs of the connections, and one that reduces the special requirements (seat stiffeners, chord reinforcement, etc.) for the joists and joist girders.

The first consideration relative to the design of the structure is to determine if rigid frame action is required in both framing directions. When rigid frames are required in only one direction, the joist girders should be selected to resist the lateral loads. If rigid frame action is required in both directions, the framing scheme that creates the smallest end moments in the joists should be examined first.

Selection of the Optimum Lateral Load System

The various methods of resisting the lateral loads have been discussed. The systems can be mixed to provide the optimum structure; for example, rigid frames in one direction and vertical bracing and diaphragms in the perpendicular direction.

The choice of the most economical lateral load system is dependent on several parameters. These principally include:

1. The building geometry.
2. Expansion joint requirements.
3. The type of roofing system.
4. Future expansion requirements.

As a general rule, braced frames with horizontal roof or floor diaphragms provide the most economical framing system for joist and joist girder buildings. This should be the designer's first choice as a system. The four parameters listed above can cause a different framing system to be used.

Building Geometry

As mentioned in the discussion on diaphragms, when the length to width ratio of the structure between vertical braces exceeds approximately 4 to 1, the structural requirements placed on the diaphragm become severe. In addition, uplift forces become significant at vertical bracing locations. For these structures, the most economical approach is to create rigid frames with joist girders. The most efficient rigid frame is to connect the joist girders to the exterior columns only. *Figure 2* is a suggested rigid frame connection. For lateral loads in the long direction of the building, the first choice would be to transfer the lateral loads to the sidewalls using diaphragm action with vertical wall bracing.

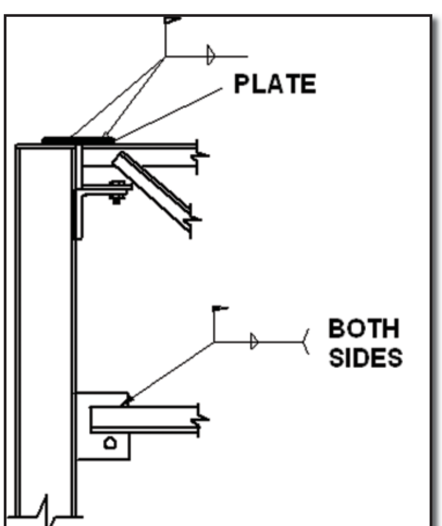


Figure 2: Rigid Frame Connection

Expansion Joints

When the structure is of such a size that expansion joints are required, and these expansion joints destroy the integrity of the roof diaphragm, a rigid frame solution is necessary. If the diaphragm shears can be transferred across a singular expansion joint and the resulting diaphragm has a span-to-width ratio of less than 4 to 1, then the diaphragm solution should still provide the most economical system.

Roofing System

When a standing seam roof is used, either a horizontal roof bracing system with vertical bracing or rigid frames must be used. Again, if the length-to-width ratio is greater than 4 to 1, the rigid frame system will most likely be the least expensive. The rigid frame solution will most likely have heavier columns than the horizontal bracing system, but the erection cost of the bracing is likely to be more expensive than the extra cost for the columns

Future Expansion

Usually, future expansion considerations only influence the lateral bracing system since the vertical bracing may not be permitted where the expansion will occur. If this is the situation, then a rigid frame may have to be used.

HSS Columns vs. W Shapes

The design of columns in industrial buildings includes considerations that do not apply to other types of structures. Interior columns can normally be braced only at the top and bottom; thus, square HSS columns are desirable due to their equal stiffness about both principal axes. Difficult connections with HSS can be eliminated in single-story frames by placing the joist girders over the tops of the HSS. Other advantages of HSS columns include the fact that they require less paint than equivalent W shapes, and they are pleasing aesthetically.

W shapes may be more economical than HSS for exterior columns for the following reasons:

1. The wall system (girts) may be used to brace the weak axis of the column. It should be noted that a stiffener or brace may be required for the column if the inside column flange is in compression and the girt connection is assumed to provide a braced point in design.
2. Bending moments due to wind loads predominate about one axis.
3. It is easier to frame girt connections to a W shape than to a HSS. Because HSS have no flanges, extra clip angles are required to connect girts.

Serviceability Criteria

The design of the lateral load envelope (i.e., the roof bracing and wall support system) must provide for the code-imposed loads, which establish the required strength of the structure. A second category of criteria establishes the serviceability limits of the design. These limits are rarely codified and are often selectively applied, project-by-project, based on the experience of the parties involved.

In AISC Design Guide No. 3 [4], several criteria are given for the control of building drift and wall deflection. These criteria, when used, should be presented to the building owner as they help establish the quality of the completed building.

To be useful, a serviceability criterion must set forth three items: a) loading, b) performance limit, and c) an analysis approach. Concerning lateral forces, the loading recommended by Design Guide No. 3 is the pressure due to wind speeds associated with a ten-year recurrence interval. These pressures are approximately 75% of the pressures for strength design criteria, based on a fifty-year return period. The establishment of deflection limits is explained below, with criteria given for each of the wall types previously presented. The authors recommend that frame drift be calculated using the bare steel frame only. Likewise, the calculations for deflection of girts would be made using the bare steel section. The contribution of non-structural components acting compositely with the structure to limit deflection is often difficult to quantify. Thus, the direct approach (neglecting non-structural contribution) is recommended, and the loads and limits are calibrated to this analysis approach. The deflection limits for the various roof and wall systems are as follows:

Roofs

In addition to meeting strength criteria in the design of the roof structure, it is also necessary to provide for the proper performance of elements and systems attached to the roof, such as roofing, ceilings, hanging equipment, etc. This requires the control of deflections in the roof structure. Various criteria have been published by various organizations.

The Steel Joist Institute [9] presents deflection limits for steel joists supporting structural steel roofs (both through fastener types and standing seam types). A limiting deflection of span over 240 for snow loading is recommended.

Mechanical equipment, hanging conveyors, and other roof supported equipment has been found to perform adequately on roofs designed with deflection limits in the range of span over 150 to span over 240, but this criteria should be verified with the equipment manufacturer and building owner. Consideration should also be given to differential deflections and localized loading conditions.

Metal Wall Panels

Relative to serviceability, metal wall panels have two desirable attributes: 1) Their corrugated profiles make them fairly limber for out of plane distortions, and 2) their material and fastening scheme are ductile (i.e., distortions and possible yielding do not produce fractures). Also, the material for edge and corner flashing and trim generally allows moment and distortion without failure. Because of this, the deflection limits associated with metal panel buildings are relatively generous. They are:

1. Frame deflection (drift) perpendicular to the wall surface of frame: eave height divided by 60 to 100.
2. The deflection of girts and wind columns should be limited to span over 120, unless wall details and wall-supported equipment require stricter limits.

Precast Wall Panels

Non-load bearing precast wall panels frequently span from grade to eave as simple span members. Therefore, drift does not change the statics of the panel. The limitation on drift in the building frame is established to control the amount of movement in the joint at the base of the panel as the frame drifts. This limit has been proposed to be eave height over 100. A special case exists when precast panels are set atop the perimeter foundations to eliminate a grade wall. The foundation anchorage, the embedment of the panel in the soil and the potential of the floor slab to act as a fulcrum mean that the frame deflections must be analyzed for compatibility with the panel design. It is possible to tune frame drift with panel stresses, but this requires interaction between frame designer and panel designer. Usually the design of the frame precedes that of the panel. In this case, the frame behavior and panel design criteria should be carefully specified in the construction documents.

Masonry Walls

Masonry walls may be hollow, grouted, solid, or grouted and reinforced. Masonry itself is a brittle, non-ductile material. Masonry with steel reinforcement has ductile behavior overall, but will show evidence of cracking when subjected to loads which stress the masonry in tension. When masonry is attached to a supporting steel framework, deflection of the supports may induce stresses in the masonry. It is rarely feasible to provide sufficient steel (stiffness) to keep the masonry stresses below cracking levels. Thus, flexural tension cracking in the masonry is likely and, when properly detailed, is not considered a detriment. The correct strategy is to impose reasonable limits on the support movements and detail the masonry to minimize the impact of cracking.

Masonry should be provided with vertical control joints at the building columns and wind columns. This prevents flexural stresses on the exterior force of the wall at these locations from inward wind. Because the top of the wall is generally free to rotate, no special provisions are required there. Most difficult to address is the base of the wall joint. To carry the weight of the wall, the base joint must be solid not caulked. Likewise, the mortar in the joints makes the base of the wall a fixed condition until the wall cracks.

Frame drift recommendations are set to limit the size of the inevitable crack at the base of the wall. Because reinforced walls can spread the horizontal base cracks over several joints, separate criteria are given for them. If proper base joints are provided, reinforced walls can be considered as having the behavior of precast walls; i.e., simple span elements with pinned bases. In that case, the limit for precast wall panels would be applicable. Where wainscot walls are used, consideration must be given to the joint between metal wall panel and masonry wainscot. The relative movements of the two systems in response to wind must be controlled to maintain the integrity of the joint between the two materials.

The recommended limits for the deflection of elements supporting masonry are:

1. Frame deflection (drift) perpendicular to an unreinforced wall should allow no more than a 1/16-inch crack to open in one joint at the base of the wall. The drift allowed by this criterion can be conservatively calculated by relating the wall thickness to the eave height, and taking the crack width at the wall face as 1/16-inch and zero at the opposite face.
2. Frame deflection (drift) perpendicular to a reinforced wall is recommended to be eave height over 100.
3. The deflection of wind columns and girts should be limited to span over 240 but not greater than 1.5 inches.■

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Do you have a design aide or technical overview that you refer to time and again? Is there a section in your Notebook that is "dog-eared" from use? Please share your "tools-of-the-trade" with other engineers by submitting an article to STRUCTURE magazine's Engineer's Notebook. Send your ideas and/or articles to ***publisher@structuremag.org.***

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