Low-and Medium-Rise Steel Buildings

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Design Guide for Low- and Medium-Rise Steel Buildings

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PREFACE

This booklet was prepared under the direction of the Committee on Research of the American Institute of Steel Construction, Inc. as part of a series of publications on special topics related to fabricated structural steel. Its purpose is to serve as a supplemental reference to the AISC Manual of Steel Construction to assist practicing engineers engaged in building design.

The design guidelines suggested by the author that are outside the scope of the AISC Specifications or Code do not represent an official position of the Institute and are not intended to exclude other design methods and procedures. It is recognized that the design of structures is within the scope of expertise of a competent licensed structural engineer, architect, or other licensed professional for the application of principles to a particular structure.

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DESIGN OF LOW- AND MEDIUM-RISE STEEL BUILDINGS

BASIC DESIGN RULES FOR ECONOMY

A few basic design rules for economy will be presented herein. These rules should be considered in the conceptual phase in the design of a project. There are, of course, many other considerations, but these suggestions are simple and can help in producing a good economical design.

The cost of a filler beam and/or girder beam simply consists of the cost of the mill material, the cost of fabrication, and the cost of erection. The cost of fabrication and erection for a single beam is essentially the same for a heavy beam or a light beam. The real savings for a light member compared to a heavier one is simply the difference in the cost of the mill material. Thus, beams should be spaced as far apart as practical to reduce the number of pieces which must be fabricated and erected.

Rigid moment connections and special connections for bracing are expensive. Care should be taken to minimize the number of these types of connections in a project—that is, reduce the number of moment resisting and braced bents to the minimum. Where practical, one may consider the use of only spandrel moment resisting frames to resist wind loads. Deeper, more efficient sections may be used thus minimizing the number of moment resisting connections required.

Where appropriate, high strength steel (F_y = 50 ksi) should be used in lieu of mild steel (F_y = 36 ksi) for both columns and beams. The reason is simple—the price to strength ratio is about 25% lower for the higher strength steel beams and 10% to 15% lower for columns depending upon their length. For example, a W21x44 (F_y = 36 ksi) simple filler beam is the equivalent of a W16x26 (F_y = 50 ksi) composite filler beam. The difference in the cost of the mill material to the fabricator is about $3.90 per linear foot. The cost of the studs in place at a cost of $1.50 each is about $1.30 per linear foot. The cost of cambering or shoring is considerably less than the $2.60 per foot difference. The floor vibration ratings for the two beams are comparable. The required critical damping using Murray's criterion (Murray, 1991) for the W21x44 and W16x26 spanning 30'-0" spaced 10'-0" o.c. with 10 psf ambient live load is 4.00 and 3.46 respectively. The higher strength steel beam is less costly and functionally equivalent. It should be kept in mind that there are situations where the use of high strength steel is inappropriate. Small inconsequential filler beams, channels, angles, etc., should be of F_y = 36 ksi steel, as this material is readily available from a fabricator's stock or a steel supply warehouse. Members for which strength is not the controlling design consideration, obviously F_y = 36 ksi material should be used.

Repetitive use of members and/or the same shape size is an important factor in the design of an economical project. Repetitive use of members reduces the detailing, fabrication, and erection costs. As an example, in composite construction where beam spacing for non-typical areas is reduced, consideration should be given to the use of the typical size beam section with a reduction in the number of studs. The simpler the framing, the lower the final estimated cost is likely to be at bid time and, as a result, the lower the total square foot cost of the project.

Use live load reductions for the design of members where possible. While live load reduction may not result in any substantial reduction in filler beam weights, a change of one size, perhaps a reduction from a W16x31 to a W16x26, will result in a 16% savings in the filler beam mill material required. The savings in girder and column weights and the cost of foundations are likely to be significant.

The level of inspection specified should be consistent with that required to insure that the completed structure will be functional. Except in unusual circumstances, visual inspection should be adequate for fillet welds. The extent of nondestructive testing of butt welds may be finally determined during the construction period. If the results of tests are marginal, the number of tests can be increased. If the results of the tests are consistently good, the number of tests may be reduced. Especially for large projects, it may be prudent to require AISC certified fabricators in order to insure good quality control and a more trouble-free project.

Finally, paint only members required by the AISC Specification. Unpainted surfaces should be used when in contact with concrete. Fireproofing material more readily adheres to unpainted surfaces. While painting costs may only be $.15 to $.20 per square foot, for a 200,000 square foot project the cost saving of $30,000 to $40,000 is real and is there for the taking.

LIVE LOAD AND BAY SIZE SELECTION

Most buildings are economic machines of one sort or another. In particular, many office building structures are built on a speculative basis. The success of the venture may be a function of the building's planning and serviceability potential. Larger bay sizes increase the flexibility in space planning. Higher design live loads also increase the flexibility in the uses permitted in office space. Buildings with higher live load capacities and larger bay sizes are obviously more attractive to potential building tenants and more valuable to building owners. It will be shown that larger bay sizes and higher

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Table 1. Typical Interior Column Load Comparison			
Design	50 PSFLL +20 PSFPART	80 PSFLL + 20 PSFPART	100 PSFLL
ASD	100%	110%	100%
LRFD	100%	111½%	105%

than promulgated minimum live loads can be achieved with no significant increase in cost.

Live Load Selection

Sometimes developers and/or designers select the minimum live load permitted by the building code. This is a seemingly obvious choice if the costs are to be kept to an absolute minimum. It is possible to upgrade from the minimum permitted design live load of 50 psf plus 20 psf partition load to a 100 psf live load capacity (with no additional partition load allowance) at virtually no increase in cost.

As an example, we will compare the differences for a typical office building with 30 ft square bays and 10 stories in height (Fig. 1). Comparisons will be made for 50 psf live load plus 20 psf partition load, 80 psf live load plus 20 psf partition load, and 100 psf live with no partition load loadings. Column load comparisons are shown for a typical interior column for the AISC Allowable Stress Design (ASD) Specification (AISC, 1978) and the AISC Load and Resistance Factor Design (LRFD) Specification (AISC, 1986).
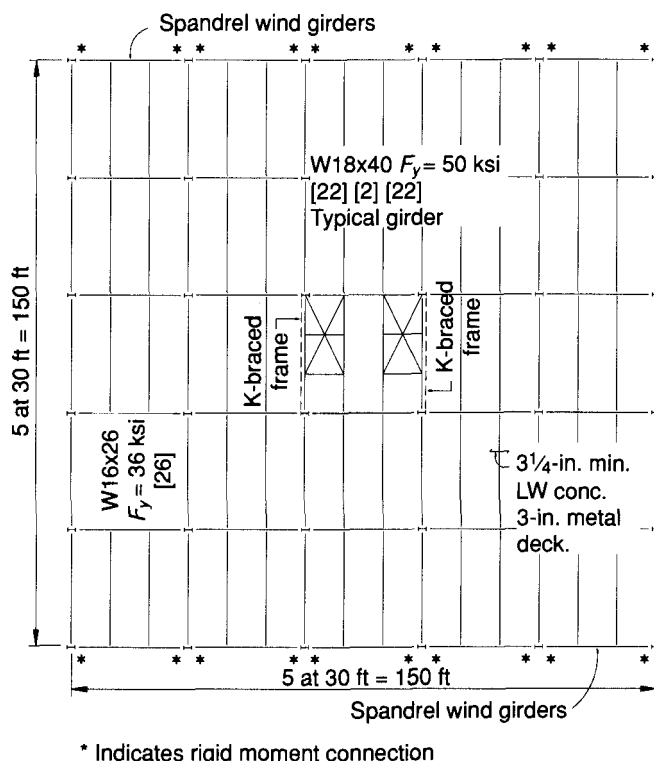

* Indicates rigid moment connection

Fig. 1. Typical office building floor plan.

Live load reductions are made in accordance with ASCE 7-88 (formerly ANSI A58.1). Table 1 is a percentage comparison of the tabulated column loads at the base of the ten story building for the three design load combinations. For ASD design, the column load is identical for that of the 50 psf live load plus 20 psf partition load and the 100 psf live load. Due to the maximum live load reduction of 60%, the 50 psf reduced live load plus the partition load is equal to the reduced 100 psf live load. For the 80 psf live load plus 20 psf partition load the column and foundation loads are increased by 10%. For LRFD the results change due to the difference in the live load and dead load factors. For this case, the column loads are increased by 5% for the 100 psf live loading and 11.5% for the 80 psf plus 20 psf partition loading. The increase in costs for the column mill material for the 100 psf live loading is $.016 per square foot for the ten story building. For either loading case, LRFD will result in lighter column loads because, essentially, the LRFD dead load factor is smaller (1.2) than a comparable ASD factor (1.67).

Tables 2 and 3 tabulate the comparative costs of a typical bay floor system for the 30 ft square bay designed for the three loadings used for the column load comparison for both ASD and LRFD designs. The comparison is made for a difference in mill material costs and the cost of studs. The cost of fabrication and erection remain essentially constant for the six conditions. It is for that reason that the mill material plus the stud costs will give a reasonably good comparison. The cost of mill material is taken as $.25 per pound for $F_y = 36$ ksi and $.28 per pound for $F_y = 50$ ksi steels. The unit prices for both $F_y = 36$ ksi and $F_y = 50$ ksi mill material change periodically. If one desires to make this type of cost comparison, representative mill material prices may be obtained from local fabricators. As would be expected, the 50 psf live load plus 20 psf partition load is the least expensive loading condition. However, the premium for the higher live load capacity (100 psf) condition is only $.09 per square foot. Compared to the total cost of the structural system, the added cost is probably less than 1%.

Knowing these facts, many owners may well wish to select the higher live load capacity. The real difference in the structures in reality may be semantics, but as a practical matter the higher load capacity enhances the building's value and, most of all perhaps, its rentability.

Bay Size Selection

The selection of a smaller bay size to reduce costs may be a fallacy when applied to steel buildings. For economy, it is important to reduce the number of pieces to be fabricated and erected. As noted earlier, the cost of fabrication and erection for a small beam is essentially the same as for a large beam. The savings involved in reducing the member weight is primarily savings in the cost of mill material. When the number of pieces is reduced, the actual cost of fabrication

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Table 2.
Framing Cost Comparison—ASD

Section	Loading		
	50 + 20	**80 + 20**	**100**
Filler BM Studs Cost	$W16 \times 26 F_y = 36$ 26 $702	$W16 \times 26 F_y = 50$ 26 $772	$W16 \times 26 F_y = 50$ 24 $763
Girder Studs Cost	$W21 \times 44 F_y = 50$ 44 $436	$W21 \times 50 F_y = 50$ 54 $501	$W21 \times 44 F_y = 50$ 56 $454
Ave. col. wt/story Cost	1027 $288	1079 $302	1027 $288
Total cost Relative cost Premium	$1426 1.0 —	$1575 1.10 $.17	$1495 1.06 $.09

Table 3.
Framing Cost Comparison—LRFD

Section	Loading		
	50 + 20	**80 + 20**	**100**
Filler BM Studs Cost	$W16 \times 26 F_y = 36$ 12 $639	$W16 \times 26 F_y = 36$ 22 $684	$W16 \times 26 F_y = 36$ 26 $702
Girder Studs Cost	$W18 \times 35 F_y = 50$ 48 $366	$W18 \times 40 F_y = 50$ 62 $429	$W18 \times 35 F_y = 50$ 54 $375
Ave. col. wt/story Cost	913 $256	1036 $290	965 $270
Total cost Relative cost Premium*	$1261 $.88 ($.18)	$1403 $.98 ($.03)	$1347 $.94 ($.09)

* Savings show ($.09).

and erection is reduced. To make a cost comparison of different bay sizes or beam spacings, the cost of mill material, fabrication, and erection must be considered. To illustrate this point we have obtained real prices from two fabricators (one east coast, one midwest) for the four bays shown in Figs. 2 through 5. Table 4 tabulates the relative square foot costs for the selected bays. The LRFD Specification was used to design the members. Absolute minimum sizes were selected for the comparison. In particular, the selection of the W12x14 for the 25 ft bay may not be realistic. It is assumed that the beams are shored or cambered as required. Tabulated costs include the structural steel frame, steel deck, and headed steel studs in place ready for concrete.

Scheme I (Fig. 2) is a 25 ft square bay designed for a 100 psf live load and 65 psf dead load. The unit weight of the steel is 4.10 psf. The total cost per square foot for the typical bay structural steel, headed studs, and composite steel deck is $5.15 per square foot. This value is used as the base price percentage (100%) for the comparison. This cost is not representative for the total cost of the building frame provi-
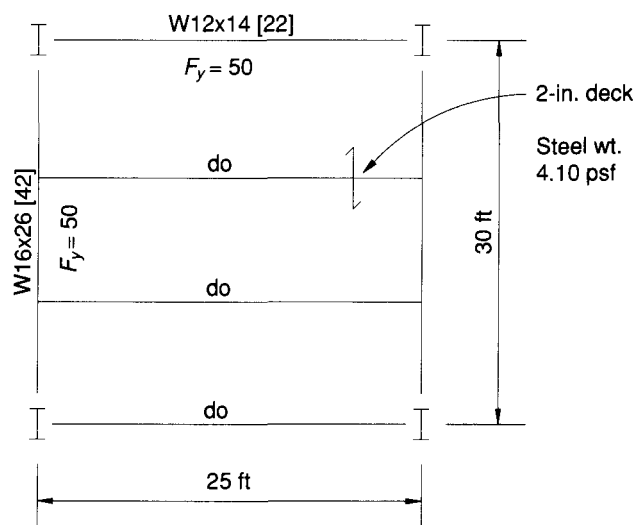

Fig. 2. 25 ft × 25 ft bay.

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	Table 4. Percentage Comparison Square Foot Costs				
Bay Size	Mill Material	Fabrication & Delivery	Erection & Studs	Composite Deck	Total
25 x 25 ft	21%	14%	34%	31%	100%
30 x 30 ft	25%	14%	32%	32%	103%
30 x 30 ft (alt.)	31%	16%	35%	31%	113%
30 x 40 ft	31%	13%	33%	32%	109%

sions for non-typical framing, spandrel conditions, and lateral load resistance systems have not been included.

Scheme II (Fig. 3) is a 30 ft square bay designed for the same loads as Scheme I. The unit weight for the steel is 5.07 psf and the cost is 103% of the base price. The 30 ft
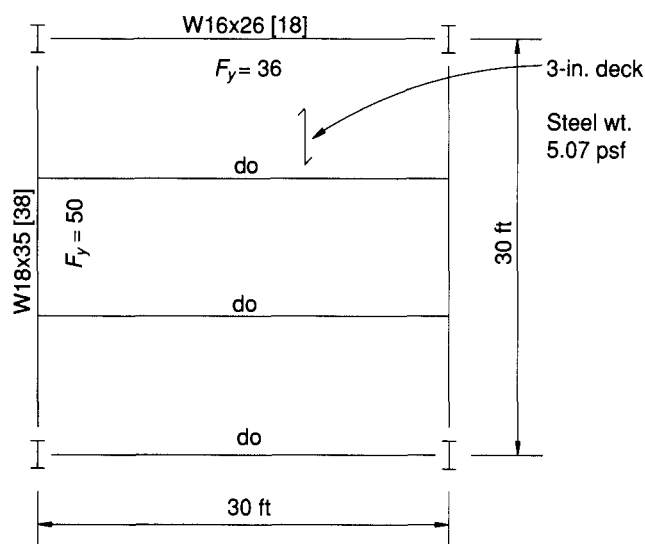
Fig. 3. 30 ft x 30 ft bay.
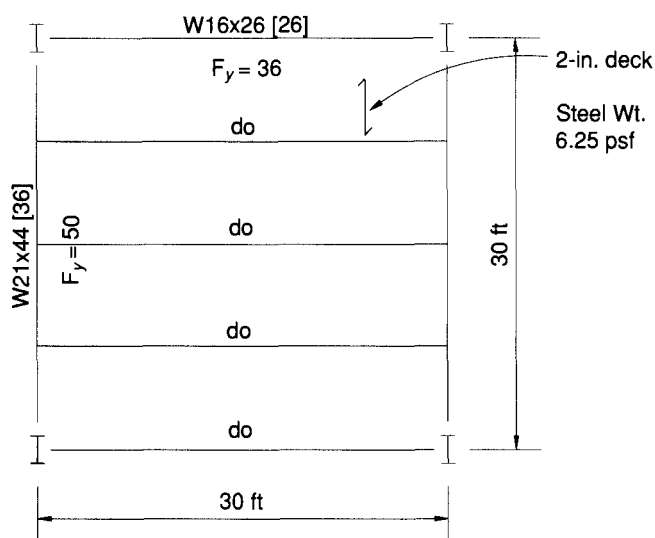
Fig. 4. Alternate 30 ft x 30 ft bay.

bay provides more flexibility in planning. Office modules of 10, 15 and 20 ft can be intermixed without column interference. The piece count is lower, that is, there are 180 square feet per steel member as compared to 125 square feet for the 25 ft bay. With the fewer pieces the job is more desireable in the eyes of a fabricator and erector. When the final markup is placed on a project, the bid price for the 30 ft square bays may well be below that for the project with the 25 ft bays. In any case, the indicated increase in cost of 3% is a small price to pay for the added flexibility.

Scheme III (Fig. 4) is also a 30 ft square bay. But there are four filler beams per bay. It is included to illustrate the added cost of decreasing member spacing and increasing piece count. The cost ratio is increased to 113% or 10% greater than the bay with three filler beams. Performance wise, there is no functional difference. Murray's (1991) required critical damping and Galambos' (1988) floor vibration ratings are virtually the same. The added cost cannot be justified on an engineering basis. (The floor vibration subject will be discussed further in the discussion on Open Web Steel Joists.)

Scheme IV (Fig. 5) is a 30 ft by 40 ft bay. The unit weight is 5.88 pounds per square foot. The steel and deck cost ratio is 109%. Note that this is less than the cost of the 30 ft square bay with the smaller filler beam spacing. This scheme may be desirable where the dimension from the service core of the building to the exterior is 40 ft. The added cost is not

Fig. 5. 30 ft x 40 ft bay.

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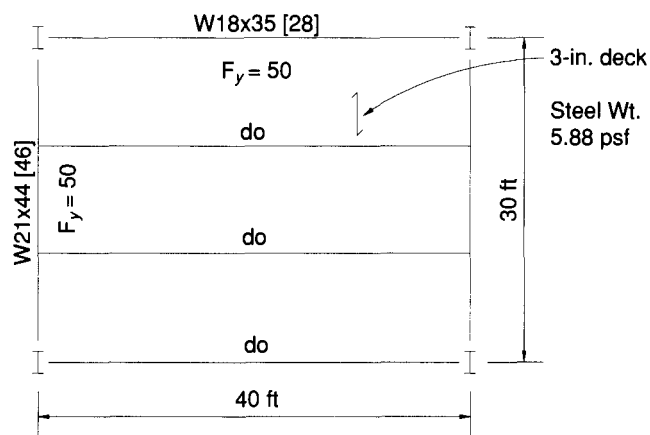| Table 5. Live Load Capacity Enhancement			
Bottom Cover PL	Design Method	LL Capacity PSF	% LL Increase
None	ASD	93	—
6 x ¾ in.	ASD	208	124%
None	LRFD	128	38%
6 x ¾ in.	LRFD	230	147%
9 x 1 in.	LRFD	310	233%
W16 × 31, F_y = 36 ksi + 32¾ in. φ studs			

extravagant if flexibility in planning is important. When the bay length reaches 45 ft or more, it is likely that alternate floor systems such as stub girders or fabricated trusses may be considered.

The selection of design live load and bay size should be considered in the preliminary design phase of any project. For the overall economic success of a project, the smallest bay size and the lowest live load probably will not produce the most economic design.

COMPOSITE FLOORS

Composite floor systems, consisting of composite metal deck with concrete fill, steel filler beams, and girders made composite by using headed stud connectors, have become a standard type of construction selected by many architects, engineers, and developers (Fig. 6). Composite floor systems are considered by many to be the highest quality type of construction. The floors are stiffer and more serviceable than open web joist systems. Adequate fire resistance ratings may
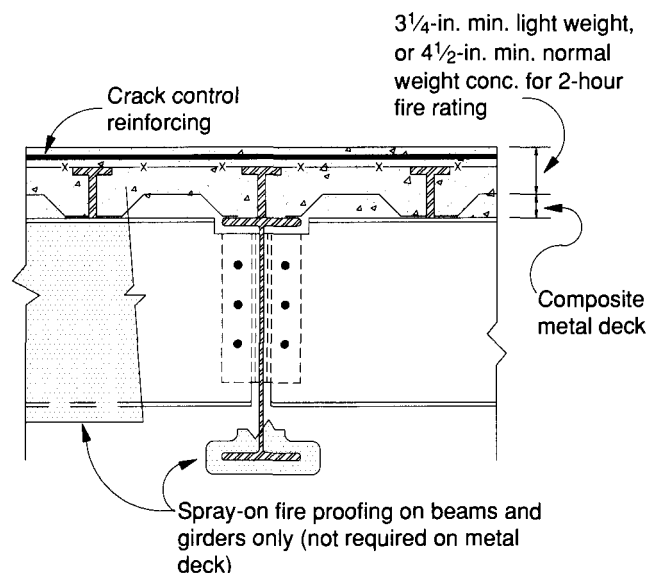
be obtained by simply providing a coat of fireproof material on the structural shape only. The combination of the concrete slab (light weight or normal weight) and composite steel deck require no additional protection when the proper slab thickness is used for a required fire rating. Furthermore, underfloor ducts for communications and electrification may be included in the system (Fig. 7). The addition of underfloor ducts adds to the cost. This added cost may be justified as the underfloor system adds significantly to the flexibility in space planning and ease of leasing.

Allowable Stress Design and
Load and Resistance Factor Design
Chapter I, Composite Construction, of the AISC's "Specification for Structural Steel Buildings" (AISC, 1989) is an allowable stress design specification (ASD) and is the standard by which composite steel beams have been designed in the USA. This design method is based upon an elastic analysis with maximum allowable stresses specified for the concrete slab compressive flange and the structural steel beam (Fig. 8). For many years the academic world has been aware that elastic design of composite sections seriously underestimates their actual strength. Still, the elastic design procedure has prevailed.

The 1986 AISC "Load and Resistance Factor Design Specification for Structural Steel Buildings" (AISC, 1986) is essentially an ultimate strength design procedure which more accurately predicts the strength of steel beams with a composite concrete compression flange. Figure 9 illustrates the two possible stress distributions for the ultimate strength design procedure. The plastic neutral axis (PNA) may be located either in the steel section or at the base of the composite concrete flange. (Note: The base of the composite con-

Fig. 6. Composite metal deck floor system section.

Fig. 7. Cellular composite steel deck system with in-floor electrical and communication distribution.

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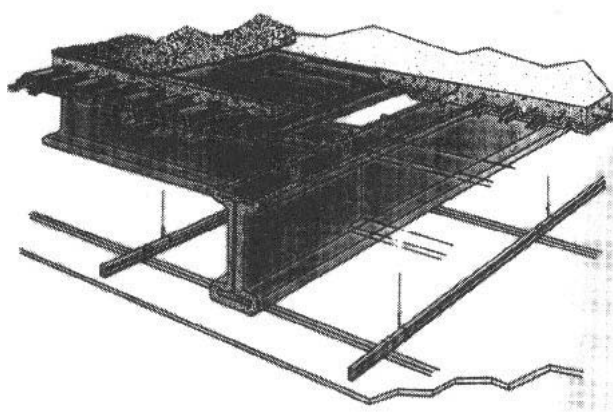crete flange is not necessarily at the bottom of the concrete slab.) The use of the ultimate design concept inherent in LRFD results in an extremely simple design procedure. With this procedure and the design aids for composite beams in the AISC Load and Resistance Factor Design Manual (AISC, 1986a), composite beam design becomes a method that requires little computation. An abbreviated design example using the design aids in the AISC LRFD Manual is given in Appendix A.
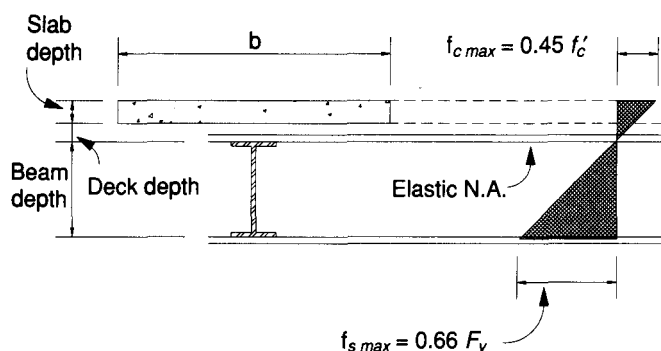
Note: The elastic neutral axis may occur in the concrete slab, between the concrete slab and steel section or in the steel section. When the neutral axis is in the concrete slab, no tension is permitted in the concrete below the neutral axis.

Fig. 8. Elastic stress distribution.
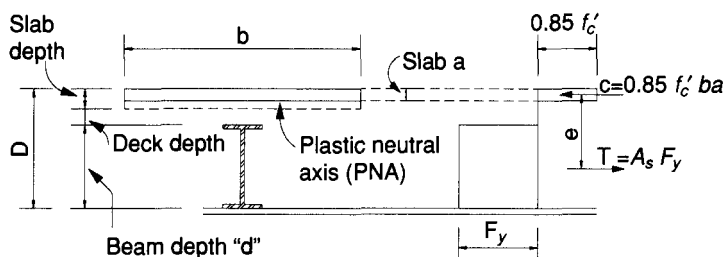
$$e = d/2 + Y2$$
$$Y2 = \text{Deck } D + \text{Slab } D - a/2$$
$$\phi M_n = \phi Ce = \phi Te$$

Note: In this case the concrete compression flange thickness "a" is less than the slab thickness.

(a) PNA in Concrete Slab
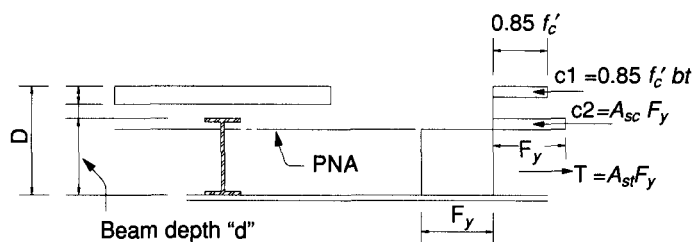
(b) PNA in Steel Beam

Fig. 9. Composite beam stress distribution.

Economy with LRFD

The use of the ultimate strength design procedure with the LRFD Specification often results in some saving of mill material. Figure 10 indicates the beam and girder sizes required for a typical bay with a 250 psf live load warehouse loading for both the LRFD and the ASD design methods. In this case, the use of LRFD results in a savings of about 20% in the cost of mill material to the fabricator or $.60 per square foot and a savings of about $.065 per square foot in the cost of headed studs. The cost of mill material and headed studs are assumed to be $560 per ton ($.28 per lb.) and $1.50 each (installed), respectively. Note that the cost of fabrication and erection does not change—the saving is simply the cost of the raw mill material and stud cost. The saving is significant and can materially reduce the cost of a project. The use of lighter weight beams will result in greater deflections which must be considered. The use of lighter weight beams will not result in a higher potential for vibration problems due to pedestrian foot traffic.

Floor Load Capacity Enhancement

From time to time it becomes necessary to increase the load capacity of an existing floor system. Sometimes, office buildings are designed for the prescribed loading of the local building code with provisions made for live load capacity increases to suit tenants' needs in specific areas. Composite floor systems offer a means by which floors may be designed for the code specified floor load and upgraded to a much higher capacity for storage or high density filing systems. With composite construction, floor load capacity enhancement is a relatively simple matter. Some designers select

Fig. 10. Typical warehouse bay plans.

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special areas for this type of live load enhancement. Columns are designed for the anticipated increase in loading. Filler beam connections are designed for the increased loading. The increase in load capacity may be achieved at a later date by simply welding a cover plate to the bottom flange of the filler beams.

To appreciably increase the capacity of non-composite beams and girders, it is necessary to add reinforcement near both the top and bottom flanges (Fig. 11). On the other hand, for compositely designed beams and girders a significant increase in load capacity can be achieved simply by adding a cover plate to the bottom flange (Fig. 12). Consider a W16x31, F_y = 36 ksi composite filler beam with thirty-two ¾-in. round headed shear studs, spanning 30'-0" and spaced at 10'-0" o.c. Calculations based on an assumption of a 100 psf live load and 60 psf dead load, the 1987 BOCA Building Code (BOCA, 1987) with its live load reduction provisions and spanning 30'-0" and spaced at 10'-0" o.c. will result in the selection of a W16x31, F_y = 36 ksi with thirty-two ¾-in. round headed studs. Table 5 indicates the live load capacities for both ASD and LRFD designs and the increase in capacity obtained by the addition of two sizes of cover plates to the bottom flange of the W16x31. The increase in live load capacity of the W16x31 with a 6-in. x ¾-in. cover plate is 147% (LRFD) and 124% (ASD) above the original design live load. A 50% addition in steel results in a 100% increase in live load capacity. The W16x31 plus a 9-in. x 1-in. cover plate is shown to indicate the upper bound moment capacity of the composite section. In this case, the thirty-two ¾-in. diameter headed studs are the limiting factor. The 9-in. x 1-in. cover plate is included only to illustrate the magnitude of capacity enhancement possible. Limitations in connection capacity or web shear strength may well be the determining factor. Computation examples for composite beam capacity enhancement are included in Appendix B.

Shored vs. Unshored Construction

Designers of composite floor systems face a difficult choice in specifying whether shored construction should or should not be used. There seems to be no evidence that either scheme is clearly superior to the other.
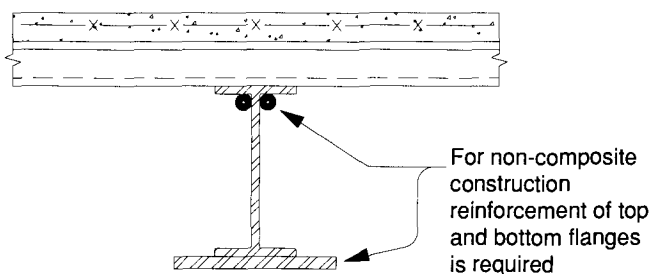
Unshored construction

The selection of an unshored system simplifies the work of the contractor. The wet concrete is simply placed on the composite metal deck after the studs and slab reinforcing are in place. But, for this condition, there are additional factors which must be considered by the design engineer. The floor beams and girders must be designed to support the wet load condition loads as non-composite sections. If the beams and girders are not cambered, the designer must consider the load due to the additional concrete required as a result of the deflection of the steel beams and girders. Ruddy suggests that even though the theoretical volume of concrete due to the ponding effect may be substantial, the actual increase in volume appears to be near 10%. For very light beams with high span/depth ratios, this figure may be unconservative. If camber is specified for the beams and girders, a different problem may be encountered. If the placement sequence of the plastic concrete is such that the system deflection is less than the specified camber, slabs thinner than that specified and headed studs with less than the required coverage may result. This could lead to floor systems with less than the required design strength. Some designers specify cambers equal to three-quarters of the theoretical wet load deflection. For moderate sized beams, cambering is reasonable if the fabricator uses a cambering machine. Some designers omit camber and design the system for a slab weight 10% to 15% greater than the theoretical weight and specify that the floor be poured flat.

Shored construction

Two advantages of shored construction are: (1) all of the deflections are based on the composite section; and (2) a strength check of the steel section alone is not required for the wet load condition. The elimination of the requirement for the wet load strength condition is significant for low live load/total load ratios. One disadvantage of shored construction is that a formation of a crack over the girders is almost certain. It is prudent that the designer specify crack control reinforcement over the girders (Fig. 6) for both shored and unshored construction. However, it is especially important for shored construction. Some designers feel that crack con-

Fig. 11. Simple beam load capacity enhancement.
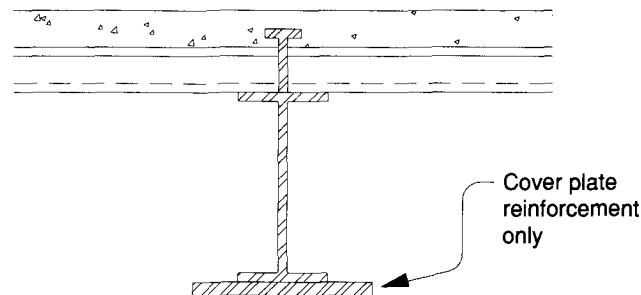
Fig. 12. Composite beam load capacity enhancement.

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trol reinforcement should also be placed in the top of the slab over filler beams. A shoring scheme which minimizes the amount of shoring required and yet controls the deflection is illustrated in Fig. 13. By placing the shores at the $\frac{1}{5}$ points of the span, the moment in the supporting composite beams and slabs are reduced and the deflection of the shored beams is minimized. The girders below the floor being poured may be shored at the load points to spread the wet concrete load to two floors during the placement of the concrete. Shored or unshored construction is also a matter of cost consideration. For any specific project, the construction manager or general contractor should be consulted, if possible.

Serviceability Considerations

Three serviceability items will be considered:

1. Floor vibration induced by foot traffic
2. Floor deflection
3. Crack control

Under certain conditions, composite floor systems can be subject to unpleasant vibrations induced by pedestrian foot traffic. These areas can be generally categorized as being large open areas without finishes and/or furnishings which will help provide damping of the system. The work of Murray and Galambos (Galambos 1988; Murray 1975, 1991; Murray, Hendrick 1977) will be helpful in gaining understanding of the subject.

Deflections in composite systems will vary with the type of construction. For unshored systems, the deflections due to the entire wet load will be time independent and will not increase with time. For shored systems, the initial wet load deflection will be subject to increase due to creep and shrinkage in the concrete compression flange. In addition, for both systems, deflections due to the weight of the finishes and average total live load will also be subject to increase due to creep and shrinkage of the concrete. One method of

"X" indicates location of shores below filler beam on floor which is being poured.

"O" indicates location of shore below girders on floor below floor which is being poured.

Warning: Filler beam shores should remain in place until concrete has reached 75% of the design strength. Loads on filler beams supporting shores should be checked for strength.

Fig. 13. Shoring plan.

accounting for creep and shrinkage is to use the modulus of elasticity of the concrete to be one half of its normal value. That is, take the modulus of elasticity $E_c = 0.5w^{1.5}(f'_c)^{0.5}$, where w is the unit weight of the concrete in pounds per cubic foot and f'_c is the concrete cylinder strength expressed in kips per square inch. Appendix A contains long-term deflection calculations for a standard filler beam W16x31 $F_y = 36$ ksi with thirty-two ¾-in. diameter headed studs spanning 30'-0" and spaced 10'-0" o.c. For this composite section, the moment of inertia for short-term loading is 1400 in.^{4} and is decreased by 15% to 1185 in.^{4} for long-term loading. For the unshored system, the wet load deflection is equal to 0.9 in. Taking the weight of the finishes to be 9 psf and a permanent average live load of 10 psf, the long-term deflection will be an additional 0.10 in. For the shored system, the long-term deflection due to the wet load finishes and an assumed permanent live load of 10 psf will be 0.37 in.

As noted earlier, it is desirable to provide crack control reinforcing in the concrete slab over the top of girders. For shored systems, when the shores are removed, it is almost certain that cracks will form over the girders. Crack control reinforcing will help distribute and limit the size of the cracks. Due to the nature of composite construction, cracking in the top of the slab over the supports is likely to occur before composite action is activated. Most composite slab systems are designed using an ultimate strength mechanism which neglects any negative bending moment at the supports. An unreinforced slab is likely to crack over the supports when real loads are applied. In addition, some slippage between the concrete and steel section may occur before the steel and concrete can act compositely. For high live load applications and/or for systems subject to moving loads such as lift trucks, it is prudent to design these slabs with top reinforcing to assure that the performance of the slab will be acceptable in the long term.

Underfloor Duct Systems

An underfloor duct system using cellular floor deck units is a system often selected by users, owners, and developers to provide virtually unlimited flexibility in the planning of office building floor space (Fig. 7). The use of this system provides a tenant access to a building's electric power and communication systems in the floor. Generally, the cellular deck units are blended with regular composite metal deck units to provide underfloor duct runs at regular intervals. Figure 14 illustrates a blend of two 3'-0" wide composite deck units and a 3'-0" wide cellular deck unit. This blend results in a 9'-0" spacing for the duct runs. Other spacings can be obtained by varying the widths of the units and/or the number of cellular and composite deck units. Cellular deck systems are not commodity items. Their capabilities, capacities, etc., vary with different manufacturers. However, comparable systems are available from different manufacturers for competitive bidding.

8
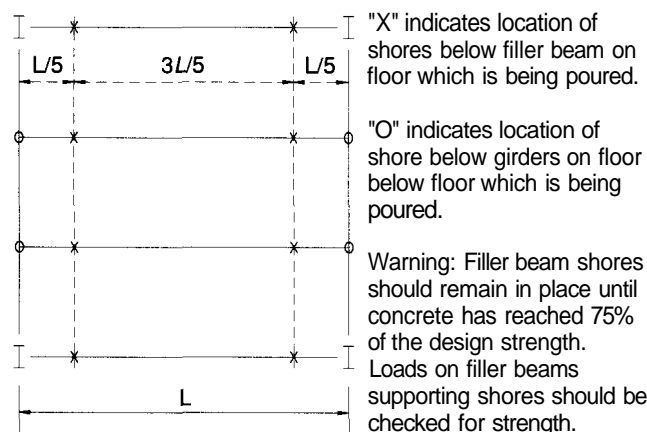
The use of underfloor duct systems with cellular deck requires additional design considerations by the architect-engineer team. It is important to orient the cells and trench headers to minimize the length of the trench headers. Figures 15a and 15b illustrate two possible solutions for a typical office building layout (100' x 180'). The plan shown in Fig. 15a results in a trench header length only two-thirds of
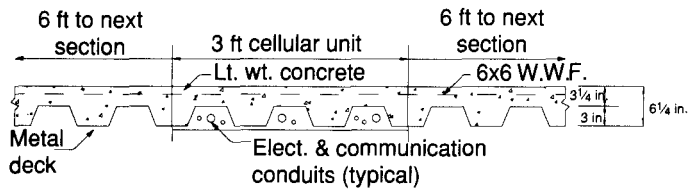
Fig. 14. Blended metal deck floor.
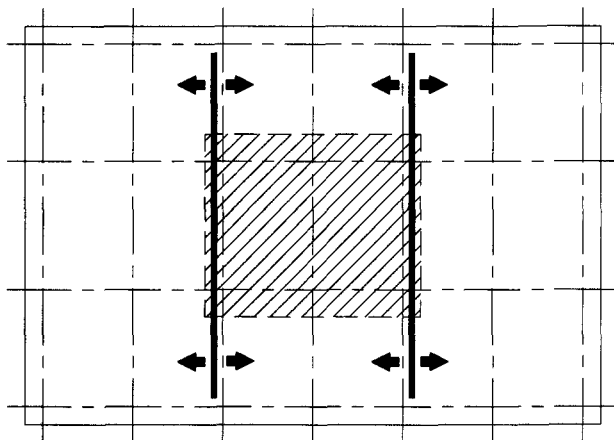
Trench header length = 200 ft
(a)
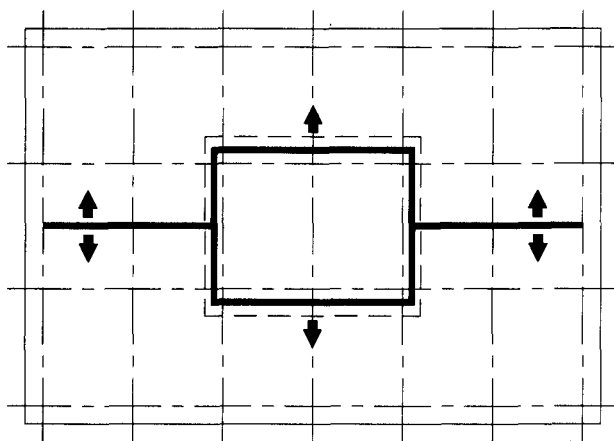
Trench header length = 300 ft
(b)

Fig. 15. Trench header and cellular deck plans.

that for the plan shown in 15b. The savings is substantial ($.15 to $.20 per square foot).

For a typical office building floor, a two-hour required fire resistance rating may be obtained by using a 3¼-in. minimum thickness lightweight concrete slab on a composite metal deck without additional fire protection. However, the cellular deck units used for ducts must be protected with an applied fireproofing material. An alternate solution to the lightweight concrete slab is the use of a 2½-*in.* minimum thick normal weight concrete slab with a sprayed-on coat of fireproofing material. The most economical selection of systems may be dependent upon the blend of composite and cellular deck units.

The introduction of cellular deck units and the accompanying trench headers require special structural considerations. The presence of the trench headers impacts upon the design of the composite floor deck units, the filler beams and the girders. The passing of the trench header over the composite deck causes the composite slab to be ineffective. Therefore, the deck must be designed to carry the gravity load without the composite slab contribution. An additional filler beam may be introduced as shown in Fig. 16. Filler beams adjacent to the trench header must be designed for the condition of the slab on one side of the beam only. If filler beams are not placed on each side of the trench duct, the deck units themselves must be designed as non-composite sections. Also, if filler beams are not located on each side of a trench header, the slab stiffness is reduced in that span and the floor may feel "soft" or "spongy." It is likely that the girder beams will have to be designed as non-composite since the trench header interrupts the composite slab and substantially reduces the physical space for studs.

Cellular deck units blended with composite deck units

Added filler beam
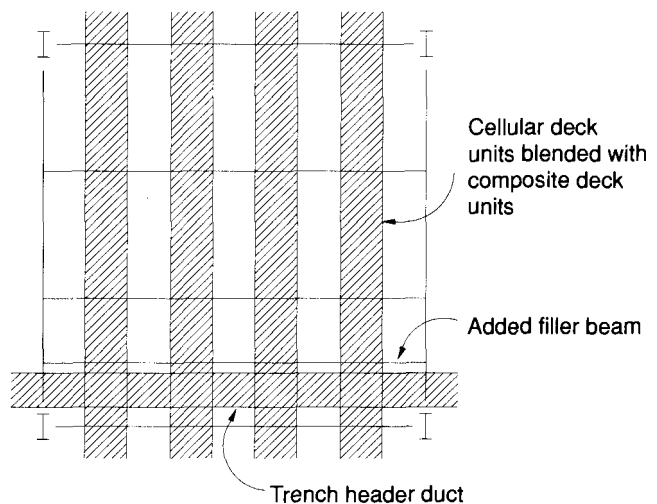
Trench header duct

Fig. 16. Added filler beam at trench header.

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OPEN WEB STEEL JOIST FLOOR SYSTEMS

Open web steel joist floor systems are used for commercial and residential projects (Fig. 17). The very large volume of floor area built annually, estimated to be in the tens of millions of square feet, suggests that the economics associated with their use overcomes any perceived serviceability shortcomings. The Standard Specifications and Code of Standard Practice are published by the Steel Joist Institute of Myrtle Beach, South Carolina (SJI 1986).

Joist Size and Spacing
The selection of the most economical joist for any given situation will generally be the deepest and lightest joist at the widest space permitted by the slab thickness. The use of joist spacings of 3'-0" or more should be considered. The decrease in the number of pieces results in heavier, more efficient sections and perhaps a reduction in the number of lines of bridging. Also, the reduction in the number of pieces to be fabricated and erected may well offset the added cost of a slightly thicker concrete slab. The performance of the floors with thicker slabs subjected to vibrations induced by pedestrian foot traffic is significantly superior (see Floor Vibrations).

Girder Beam Design
Girder beams supporting open web joist floor systems are normally designed as simply supported beams. In applications where it is desirable to use a floor-ceiling assembly to obtain a fire rating (Fig. 17), it may be economical to consider the use of continuously designed girder beams or compositely designed girder beams to minimize the required girder beam depth and weight. For example, Fig. 18 illustrates a typical office floor bay with girder beams designed as simply supported, continuous design (ASD Type 1 construction, LRFD Type FR construction), and composite girder beam design. Abbreviated calculations for the girder beams are included in Appendix A. The depth of the selected girders is limited to 18 in. to allow the ceiling fireproofing

membrane to pass under the girder uninterrupted. The tabulated live deflections are computed with reduced live loads. However, the 20 psf partition load is included, as they may be moved from time to time. The simple beam deflection (0.96 in.) is marginal and may be unacceptable to some designers. In that case, if F_y = 36 ksi mill material was to be specified, the size girder beam required would be a W18x76. The W18x76 would have a live load deflection of 0.71 in. If the continuous beam is to be selected, to be economical the difference in cost of the mill material plus the difference in cost of the connections (both shop and field) must be considered. A patented composite girder system which was described in an AISC *Engineering Journal* article may be considered and could prove to be economical (Rongoe 1984). Figure 19 illustrates the composite system.

Composite Joist Systems
Composite joist systems are widely used and very competitive in some areas. Systems vary with different manufacturers. One widely known system is produced by Canam Hambro Systems Inc. of Baltimore, Maryland. Joists are evenly spaced to accommodate 4'-0" wide plywood form material which is supported on removable clips. After a reinforced concrete slab is cast, the forming material is removed for reuse (Fig. 20).
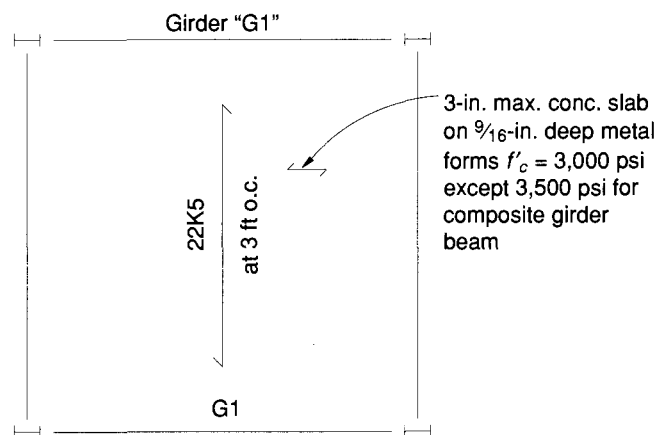
Loads

LL	=	50	x	1.6	=	80
Part	=	20	} x	1.2	=	80
DL	=	47				
Tot.	=	117	x	(1.37)	=	160 psf

Plan

Girder "G1" (LRFD design with ANSI A58.1-82 LL reduction)

Design Method	LL Deflection	Shape	F_y
Simple Span	0.96 in.	W18x60	50 ksi
Continuous (AISC Type 1)	0.38 in.±	W18x46	50 ksi
Composite	0.69 in.±	W18x35	50 ksi

Fig. 18. Typical bay girder beam comparison.

Fig. 17. Fire-rated floor ceiling assembly.

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Floor Vibration

Open web steel joist floor systems as well as some other floor systems with large open spaces lacking partitions and/or other loads can be subject to disturbing vibrations induced by pedestrian foot traffic. Span lengths less than 20'-0" or greater than 35'-0" seldom experience pedestrian traffic-induced vibration problems except for long-span floor systems with low natural frequencies. On the other hand, floors which support partitions and/or other furniture or equipment normally perform well and do not exhibit poor behavior. At this time there is no universally accepted method that can be used to evaluate this problem. The work of Murray (1975, 1981, 1991), Murray and Hendrick (1977), and Galambos (1988) include design methods which can be used to calculate floor vibration ratings.

A method of evaluating the performance of open web joist systems subject to foot traffic is contained in the Steel Joist Institute publication *Vibration of Steel Joist-Concrete Floor Slabs* by Galambos (1988). The method is especially suited for joist floors and has been used in the preparation of the floor ratings discussed herein. A number of different criteria have been proposed for use in evaluating the vibration problem. Murray (1981) has reviewed many of the proposed methods and found that the results of the different methods often conflict. Two of the methods will be discussed, one that is described in a Steel Joist Institute publication (Galambos 1988) and the method proposed by Murray in "Acceptability Criterion for Occupant Induced Floor Vibrations" (1981) and "Floor Vibrations" (1991). An ASCE report, "Structural Serviceability: A Critical Appraisal and Research Needs" (ASCE 1986), proposes an acceptance criterion which is similar to and derived from Murray's work.

The SJI publishes a booklet by Galambos, *Technical Digest No. 5, Vibration of Steel Joist Concrete Floors* (1988). This publication has recently been updated (March 1988) and includes information for use in designing floors subjected to dancing, running, and similar rhythmic activities. Floor vibration ratings are computed by the formula suggested by Wiss and Parmelee (1974):

$$R = 5.08[(fA_o)/(D^{0.217})]^{0.265}$$
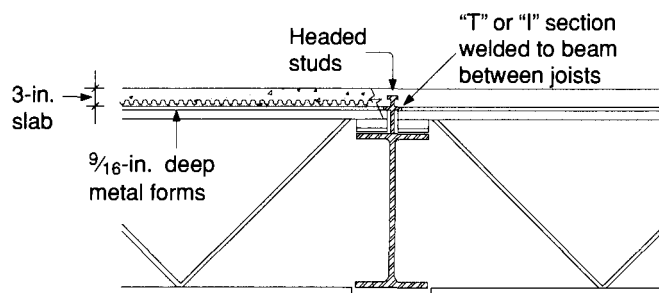
where

f = natural frequency, cps
A_o = amplitude, in.
D = % of critical damping

The SJI's suggested criteria for floor ratings is as follows:

R = 2.5 vibration is barely perceptible
R = 3.5 vibration is distinctly perceptible
R = 4.5 vibration is strongly perceptible

These values are more liberal than those suggested by the original Wiss and Parmelee paper (1974). Galambos also includes calculations for acceptance as judged by Murray's criterion as discussed below.

The work of Murray (1975, 1981) and Murray and Hendrick (1977) is thought by some engineers to be the most reliable information available on this subject. Murray's criterion for acceptance is much simpler. Murray states that a more accurate division of acceptable and unacceptable floor systems is given by

$$D = 35A_o f + 2.5$$

where

D = percent of critical damping
A_o = initial amplitude from a heel drop impact, in.
f = first natural frequency of the floor system

This method is simple to use for the designer as the result is a lower bound value for the percent of critical damping required. Murray's 1981 criterion is based on a study of the test results of 91 floor systems. Some engineers feel that Murray's criterion gives better results over a wide range of floor systems.

In some ways, the design to control vibrations is paradoxical. For instance, if a beam is made heavier to increase its stiffness, often the frequency increases at a higher rate than the amplitude decreases. As a result, the floor rating R and the percent of critical damping required D increase rather than decrease. Increasing the beam size to help reduce vibrations can result in a worse condition. Vibration problems vary inversely with the span length. As span lengths increase, the

Fig. 19. Composite beam with open web steel joists.
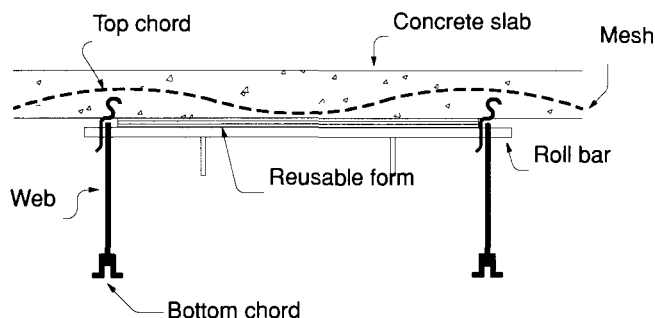
Fig. 20. Composite steel joist floor system.

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floor ratings and the required damping both decrease. Steel filler beam and joist floor systems with spans of 20 ft and less are not subject to vibration problems. Figure 21 plots both Murray's requirement for critical damping and the SJI's suggested ratings for three slab thicknesses. The SJI ratings were calculated using a critical damping ratio of 0.04. A number of joist sizes and spacings were used to calculate the ratings. It was determined that the ratings are not a function of joist size or spacing. It can be observed that the required damping and the SJI floor rating both decrease rapidly with an increase of span or slab thickness.

Similarly, as spans increase and natural frequencies decrease below 3.0 hz, vibration problems can become severe. For instance, a floor with 52DLH joists spanning 90'-0" at a 3'-0" spacing with a 3-in. concrete slab on metal forms and with a total load of 100 psf (30 ambient psf LL plus 70 psf DL) has a natural frequency of about 2.6 cps and a floor rating of 1.97 with 4% damping. Seemingly, this would be an acceptable floor system using the Wiss, Parmelee rating method. However, a repetitive loading matching this natural frequency (such as fast dancing) can lead to disastrous results. Thus, for long span floors for places of assembly where such a moving load can be expected, it may be prudent to make a vibration analysis and, when indicated, provide some positive damping. Both the SJI Digest (Galambos 1988) and the ASCE Ad Hoc Report (ASCE 1986) address this subject.
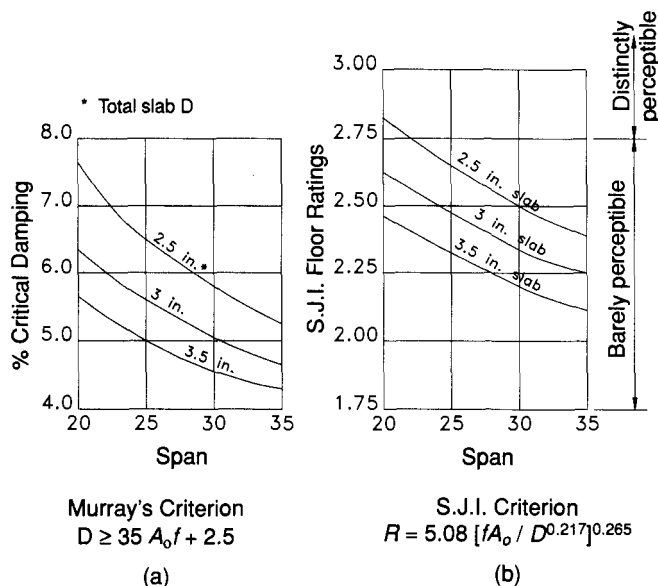
Fig. 21. Steel joist floor vibration ratings.

Murray's Criterion
$D \geq 35 A_o f + 2.5$

(a)

S.J.I. Criterion
$R = 5.08 [f A_o / D^{0.217}]^{0.265}$

(b)

WIND LOAD DESIGN

The structural design of systems to resist wind loads is one of the most interesting tasks that a structural engineer can encounter in his career. The number of possible solutions are endless. This is the area in which one's imagination and judgment can be used in the development of innovative designs and to find unique solutions to the most important and difficult problems in the design of high-rise buildings. For structural engineers, the goal for wind load design might be defined as to produce structures that perform in a satisfactory manner under the influence of wind loads and, as always, at the least possible cost. The level of performance desired may vary with the type and use of the structure. For instance, a higher level of performance would be desirable for a hospital than for a speculatively built office building. In a hospital environment, it is probably not acceptable for the occupants to feel uncomfortable due to motion induced by wind load except under extraordinary circumstances. On the other hand, the owner of a speculative office building may well accept some disturbing motion on a more or less regular basis, say five or ten years. Galambos and Ellingwood (1986) suggest that an acceptable level of performance may be to expect some occupant annoyance one time in the life of an average lease—eight years. Thus, it may seem appropriate to establish separate levels of performance for strength and serviceability for different types of structures. For a hospital, one may wish to select a 100-year storm for strength and a 50-year storm for motion. For an office building, one may wish to use the code-required wind load (usually 50 years) for strength and a 10-year storm for motion. In any case, the deflection due to wind load must be limited to an amount that the building cladding and finishes can tolerate.

Drift Limits

The selection of an appropriate drift limit for a multi-story project is a problem faced by structural engineers since the inception of skyscrapers. It is now recognized that drift control will not necessarily insure satisfactory performance with respect to human perception of motion due to wind loading. In tall buildings (buildings over 300-350 ft), acceleration due to wind loading is the parameter which must be considered in evaluating the effects of wind-induced motion. For buildings under 25 to 30 stories, drift control will probably become more important in the near future. At this time full wind tunnel studies are probably not economically justified for the less tall buildings. Two references which describe the problem are "Structural Design of Tall Steel Buildings" (CTBUH 1978) and "Human Response to Tall Buildings Wind-Induced Motion" (Reed, Hansen and Van Marke 1972).

Historically, recommended drift limits have varied widely (CTBUH 1979). Recently, a subcommittee of the ASCE Committee on Design of Steel Building Structures conducted a

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No. of Stories	Aspect Ratio	Column Stress				Web Stress Multiplier
		Floor				
		Bot.¼	2nd¼	3rd¼	Top¼	Girder and Brace
10 HT=125	5/1	10.1	8.9	7.2	7.1	1.36
	7.5/1	8.1	7.1	5.7	4.5	1.36
	10/1	6.5	5.7	4.6	3.6	1.36
20 HT=250	5/1	8.2	7.2	5.8	4.0	1.36
	7.5/1	7.4	6.8	5.4	4.2	1.36
	10/1	6.3	5.5	4.4	3.1	1.36
30 HT=375	5/1	6.8	5.9	4.7	3.1	1.36
	7.5/1	6.6	5.8	4.6	3.0	1.36
	10/1	4.6	4.0	3.2	2.1	1.36

BOCA basic building code—wind speed 70 mph. Apex of "K" bracing up.

poll of structural designers to determine the state of the art for wind load design (ASCE 1988). The report of the Task Committee includes not only the results of the poll but, in addition, its comments on and interpretation of the results. The results of the poll are at best ambiguous. However, in answer to the question, "Which drift limit would you use?" for a total of 34 different building types and exposures, the predominant answer was 0.0025 for wind service load (deflection/height). However, the committee did not make any recommendations for drift limits.

In addition to drift control for human occupancy concerns, the effect of drift on cladding elements must be considered. Cladding connections to the building frame should be designed to accommodate the wracking deflection to which it will be subjected. The deflection characteristics of braced and unbraced frames is briefly discussed in the Combination Frames section.

Braced Frames
Braced frames are often the most economical method of resisting wind loads in multi-story buildings. However, the use of bracing bents alone can result in very large uplift forces even in moderately low high-rise buildings (10-15 stories). This may not be a problem if deep foundations which can resist uplift are used. The use of bracing frames combined with other systems such as hat or belt trusses can be very efficient as shown in Fig. 30 (see Combination Bracing Systems).

The design of pin-connected K-braced frames (Fig. 22) with optimum sizes is easily performed using a method suggested by Baker (1987) of the Chicago office of Skidmore, Owings & Merrill. Similar methods have been used by other engineers.

Using a classical work method, Baker has suggested that:

$$A_i = (P_i n_i / \beta)^{0.5}$$

where

A_i = optimum area of bracing frame member
P_i = bar force in member i due to external load
n_i = bar force in member i due to virtual load
β = unknown parameter

The procedure to find the optimum areas is as follows:

1. Calculate bar forces in members due to external loads.
2. Calculate bar stresses in members due to a virtual unit load placed in the location and in the direction at a point where the deflection is to be optimized.
3. Compute member areas (A_i) using the value for β equal to one.
4. Compute the deflection at the point of the virtual load using the areas (A_i) computed with β equal to one.
5. Modify the member areas (A_i) by multiplying them by
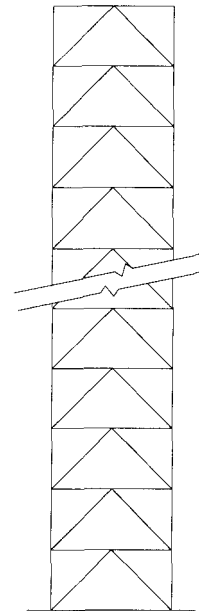
Fig. 22. K-braced bent.

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the ratio of the target deflection to the calculated deflection computed with β equal to one.

An example is shown in Appendix A. The design example set up in a tabular form with consecutive columns of calculations. A target Deflection Index (DI) of 0.0025 was selected. For simplicity, a uniform wind load of 10 kips per floor was used. The tabular columns are numbered and named with a mnemonic which can be used as a variable in a computer program. The algorithm used to compute the truss deflection is similar to that which is illustrated in "Plastic Design of Braced Multistory Steel Frames" (AISI, AISC 1968). It should be noted that the final areas are optimized for deflection only. All members must be checked for strength for all loading conditions. Undoubtedly, optimized areas for members in the upper stories will be less than those required for strength. This will result in final deflections being smaller than the target DI. Although of limited value, this same method can be used to find optimum areas for deflection limits for any pin-connected truss system.

It is important to keep the apex of the bracing members pointed in an upward direction. The deflection of a frame with an aspect ratio (building height to width) of 7.5/1 for a 10-story building, using the average stresses tabulated in Table 1, results in a maximum deflection of 3.8 in., a deflec-

tion index (DI) of 0.00253. The same frame with the apex of the braces pointed in a downward direction results in a maximum deflection of 5.11 in., DI = 0.00341 (Fig. 23). This is an increase in deflection of 35% due merely to the change in orientation of the bracing members. With the apex of the braces up, the story drift due to chord drift (column strain) in that story is eliminated. Figure 24 illustrates the deflection of a single story in a frame due to column strain. In Fig. 24a the apex of the braces point up. The force in the columns due to the story shear in that story is zero. All of the story shear load is taken by the brace members. There is no column strain, story rotation, or deflection due to column strain. In Fig. 24b the braces point down. As a result,

(a)

$$e_c = P_c h / A_c E$$
$$\alpha = e_c / .5L$$
$\Delta_1 = \alpha h$ = story Δ due to col. strain
$\Delta_2 = \alpha x$ = story Δ above floor due to col. strain

(b)

Fig. 24. K-brace frame deflection due to column strain.

Identical wind frames:
Apex up – Δ = 3.8 in.
Apex down – Δ = 5.11 in.

Fig. 23. Wind frame deflection comparison.

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the columns are subjected to axial loads. These axial loads produce column strain, story rotation, and story deflection. At any floor level above, deflection at the higher elevation is increased due to the story rotation of the lower floor which is equal to the distance above the lower floor multiplied by the lower story rotation. This phenomenon is less pronounced in higher frames.

Unbraced Frames

A method for the direct design of unbraced frames is illustrated. The method is applicable to a wide range of frames, providing that the members are arranged and proportioned as stipulated. This method can use either ASD (AISC 1978) or LRFD (AISC 1986) design specifications. The LRFD specification is better suited for the method. The following material is based upon use of the LRFD specification.

There are two stipulated design requirements. First, the number of load resistance frames should be minimized. Figure 25 represents a framing plan for a typical office building which will be used for a design example that follows. Spandrel moment resisting frames have been selected to resist east-west wind loads. This reduces the number of rigid frame connections for economy. Spandrel framing permits the selection of deeper and more efficient girders. Second, select the ratio of column to girder stiffnesses to be less than or equal to 1.5, that is:

$$(I_c/L_c)/(I_g/L_g) \leq 1.5$$

where

I_c = column moment of inertia (major axis)
L_c = column length
I_g = girder moment of inertia (major axis)
L_g = girder length

The "K" bracing frames shown in Fig. 25 are designed to provide for the lateral loads in the north-south direction. The minimum ratio of r_x/r_y is greater than 1.5 for all commonly used rolled column shapes. If the ratio of $(I_c/L_c)/(I_g/L_g)$ is kept below 1.5, the resulting effective length factor K from the alignment chart in the Steel Manual (AISC 1989) will also be less than 1.5. In the design example the required stabilizing forces in the north-south direction will be provided by the "K" braced bents. The north-south wind frames K_y will equal one. The required stabilizing forces could also be provided by a properly designed moment resisting frame. There is no need to compute the effective length factor K, since for unbraced frames when using the P-Delta method contained in the LRFD Specification (Formula Hl-5) as the value of P_{ex} is calculated for the effective length factor K_x equal to one.

The proposed method can be used with both the ASD and LRFD specifications. The use of ASD is not as economical for two reasons. In ASD, F'_{ex} is calculated using KL/r_x with the factor $K > 1.0$ as specified by Sect. 1.8 of the ASD spec-

ification, and the moment magnifier $C_m/(1-f_a/F'_{ex})$ is applied to both gravity (non-sway) and wind (sway) moments. On the other hand, for LRFD there are separate moment magnifiers for non-sway and sway moments. In non-symmetrical frames there may be significant sway moments due to gravity loads which must be considered in the design. And, finally, using LRFD, the governing load factors for unbraced frames will normally be:

$$1.2D + 1.6L + 0.5S$$
$$1.2D + 1.3W + 0.5L + 0.5S$$

The reduction of the load factor for live load from 1.6 to 0.5 when combined with wind is significant. Many columns for which the critical loading would have been wind plus gravity, using ASD, will be selected with the gravity only loading (no wind) being critical, using LRFD.

The following design procedure and design example will be made in accordance with the requirements of the LRFD specification.

The key step in the procedure is the computation of minimum member moments of inertia to control drift to a specified target deflection index (DI). From the traditional story deflection formula:

$$\delta = \frac{H}{(12E/(L_c)^2)(\Sigma_1^n(I_c/L_c)/(1+2((I_c/L_c)/(\Sigma I_b/L_b))))}$$

Fig. 25. Typical office building plan.

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the following can be derived:

$$K = 2S/n \qquad (1)$$

$$K_b = SK_c/(nK_c - S) \qquad (2)$$

$$K_c = S/(n - S/K_b) \qquad (3)$$

where

S = $\Sigma HL_c^2/(12E\delta)$ = stiffness factor

K = required average I/L values for beams and columns, in.[4]

K_b = required average beam I/L values for known values of average column I/L values, in.[4]

K_c = required average column I/L values for known values of average girder I/L values, in.[4]
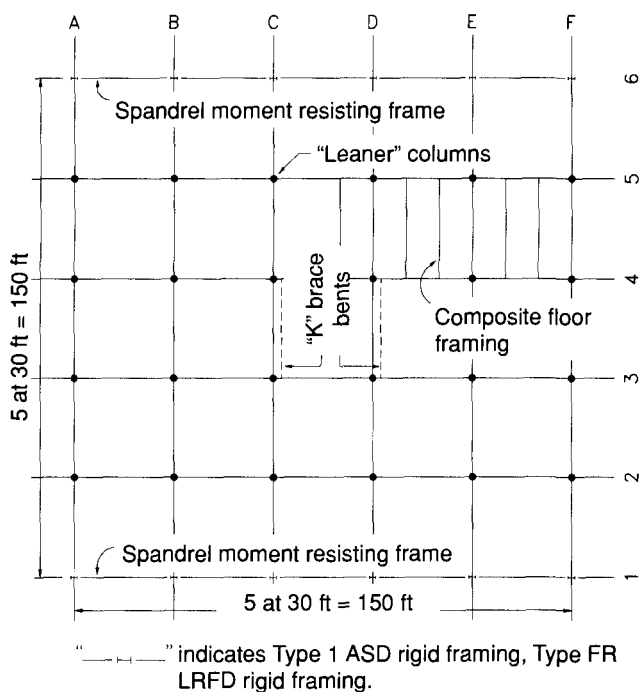
n = number of bays

L_c = story height, in.

L_b = beam length, in.

E = modulus of elasticity, ksi

δ = story deflection, in.

ΣH = total tributary story wind shear, kips

Equations 1, 2, or 3 can be used to determine the average stiffness of all frame members to control drift to a specified

The subscripts *A* and *B* refer to the joints at the two ends of the column section being considered. *G* is defined as

$$G = \frac{\sum \dfrac{I_c}{L_c}}{\sum \dfrac{I_g}{L_g}}$$

in which Σ indicates a summation of all members rigidly connected to that joint and lying in the plane in which buckling of the column is being considered. I_c is the moment of inertia and L_c the unsupported length of a column section, and I_g is the moment of inertia and L_g the unsupported length of a girder or other restraining member. I_c and I_g are taken about axes perpendicular to the plane of buckling being considered.

For column ends supported by but not rigidly connected to a footing or foundation, *G* is theoretically infinity, but unless actually designed as a true friction free pin, may be taken as "10" for practical designs. If the column end is rigidly attached to a properly designed footing, *G* may be taken as 1.0. Smaller values may be used if justified by analysis.

Fig. 26. Alignment chart for effective length of columns in continuous frames.

deflection selected by the designer. This is referred to as the target DI.

A step-by-step procedure is as follows:

1. Compute wind girder size for gravity loads and the accompanying column moments for a range of column stiffnesses.
2. Compute factored column loads for three loading conditions,
   a. Dead load only
   b. Gravity load (DL + LL)
   c. Gravity load + wind load (or dead load + wind load, if applicable)
3. Compute wind loads.
4. Compute moments of inertia to limit drift to target DI.
5. Compute P-Delta moment and load magnifiers using the target design DI for each level of girders and columns (Factor B2 in Sect. H1.2.2 of the LRFD Specification; see Design Example).
6. Compute preliminary wind moments for columns and girders. A simple portal analysis will generally suffice.
7. Select preliminary girder sizes for gravity, gravity plus wind, or minimum moments of inertia requirements.
8. Select preliminary column sizes for gravity, gravity plus wind, or minimum moments of inertia requirements.
9. Perform computer frame analysis. (DI should not exceed target DI.)
10. Check trial section member strength.

Step 6 may be omitted if members are selected to control drift only. This is the method used in the design example.

The moment and deflection magnifiers computed in Step 5 are based on the design target DI selected by the designer. As long as the final DI does not exceed the target DI, there is no need to recalculate the magnifiers. It should be noted that the magnifier is also applicable to axial loads and deflections. In the final design, some members may require adjustment due to strength requirements. If column sizes are increased, required girder stiffnesses may be calculated using Eq. 2. However, care should be taken to keep the ratio of the column stiffnesses to the girder stiffnesses $(I_c/L_c)/(I_g/L_g)$ below 1.5 to avoid the need to calculate an effective length factor for the columns (Fig. 26). Assuming the stiffnesses *(I/L* values) of the girders and columns are nearly equal, if the ratio of $(d_c^2/L_c)/(d_g^2/L_g) > 1.0$, it is more economical to add material to the columns. Conversely, if the ratio is less than one, it is more economical to add material to the girders. The procedure outlined above may be modified to suit a particular designer's resources, skills, and experience. For instance, an alternative procedure which may suit some designers would be to design the columns for strength requirements, then calculate the required girder stiffnesses using Eq. 2. If this procedure is used, as noted above, care must be taken to keep the ratio of the column to girder stiffnesses below 1.5 or the column effective length factors will
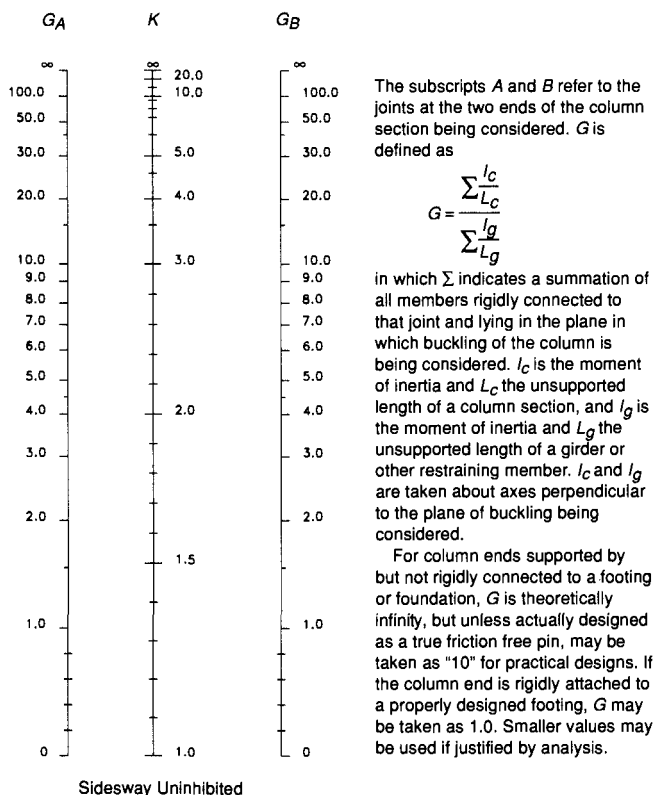
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need to be calculated to determine the nominal compressive strengths.

The design procedure is illustrated for a 10-story office building. The framing plan is shown in Fig. 25. In Fig. 25 the wind load resisting elements are the spandrel moment resisting frames on column lines one and six which resist the east-west wind loads and the "K" braced bents which resist the north-south wind loads. The LRFD design method provides for frame stability by the use of a P-Delta moment magnifier (factor B2 in Sect. H1.2.2 of the LRFD Specification). Stabilizing forces must be provided for the columns not participating in the moment resisting frames as shown in Fig. 27. Normally, the forces can easily be transmitted from the supported member to the supporting member by diaphragm action through the floor slab. These columns (nicknamed "leaner columns") are designed for the effective length factor $K = 1.0$. In the upper stories, columns and/or girder sizes are likely to be those required for gravity loading. The design procedure for the spandrel moment resisting frames is shown in Appendix B. The computations have been minimized to conserve space. Comments for each sheet follow:

Sheet 1: Design load data and typical girder and filler beam design using LRFD Manual.

Sheet 2: Gravity load girder moment design. Two cycle moment distribution is for illustration only. Design procedure is optional.

Sheet 3 and Sheet 4: Column load tabulation for corner column and typical spandrel column.

Sheet 5: Service wind load calculation.
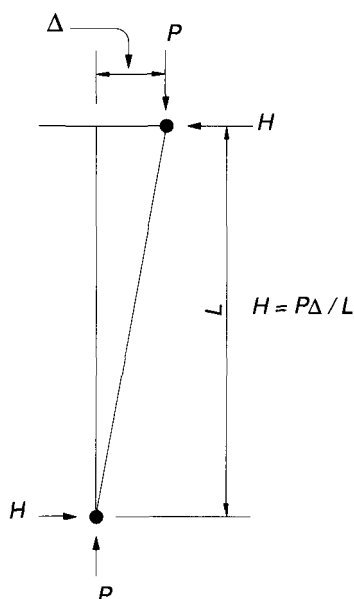
Fig. 27. Stability force for "leaner" column.

Sheet 6 and Sheet 7: Calculation of stiffnesses I/L required to limit drift to target DI of 0.0025 and calculation of moment magnifiers $B2$ at each column lift and each floor level. Note that vertical load must be factored. The wind load may or may not be factored so long as the accompanying DI has a matching load factor.

Sheet 8: Selection of column and girder sections to control drift. The maximum column effective length factor K is about 1.4.

Sheet 9: Wind frame elevation used with computer analysis to compute wind moments and axial loads.

Sheet 10: Summary of frame deflections due to wind loads and final girder design summary. Girder bottom flange bracing requirements must also be checked.

Sheet 11: Final column design load summary.

Sheet 12 and Sheet 13: Final design of two columns, one corner column, and one typical spandrel column.

A P-Delta procedure may be developed for use with the ASD Specification. Section C2.2 of the Specification permits the use of a rational method to determine the design parameters. The design procedure for ASD will be similar to that for LRFD with the following exceptions. First, the design is made using service loads except that a load factor of 1.3 should be applied to the sum of the vertical loads when calculating the P-Delta moment magnifier. Both sway and non-sway moments must be magnified by the $C_m/(1-f_a/F'_{ex})$ moment magnifier which occurs in the combined stress interaction Eq. 1.6-1a. And finally, the effective length factor K_x must be calculated in order to compute F'_{ex}.

Special Wind Frames

The introduction of powerful digital computers and powerful computer programs has revolutionized the design of lateral load resisting systems. Now the widespread availability of powerful microcomputers and frame analysis programs provides a means for the small consulting firm to use advanced design concepts in moderate size projects. For example, the structural consultant can now consider the use of facade bracing, tubular design hat and/or belt trusses combined with bracing frames and combinations of moment resisting and bracing frames.

An understanding of the deflection characteristics of braced and unbraced (moment resisting) frames is helpful in the design of linked braced and unbraced frames as well as braced frames that are combined with hat and belt trusses. Figures 28a and 28b illustrate the overall deflection shapes of a braced frame and moment resisting frame (member curvature not shown). The braced frame deflects in the manner of a cantilever beam—that is, the slope of the frame increases with the height. For members participating in chord action in braced frames, the wracking deflection to which cladding is subjected is equal to the shear deflection due to the strain in the bracing members. For members not par-

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ticipating in chord action, such as spandrel members in core braced buildings, the wracking deflection is equal to the total story deflection. The story to story deflections also increase appreciably with the height. On the other hand, for moment resisting frames, the story deflections are more or less constant from top to bottom, except at near the top the story deflections tend to decrease. Combining the two systems results in a very satisfactory solution. To illustrate the fact, a combined frame was proportioned for an 18-story building with a plan similar to that shown in Fig. 25. In this example, a spandrel moment resisting frame was linked with a K-braced frame as shown in Fig. 29. Both the moment resisting frame and K-braced frame were proportioned to resist one-half of the wind load for a DI of 0.0025. Figure 30 shows a plot of the moment resisting frame alone, the K-braced frame alone, and the combined frame, all subjected to the full wind load. The combined frame has a maximum deflection at the top of 6.25 in. (DI = 0.0023) for the full wind load. The maximum deflection at the top for one-half wind

Fig. 30. Combined frame deflections.

(a)   (b)
K-braced   Moment resisting
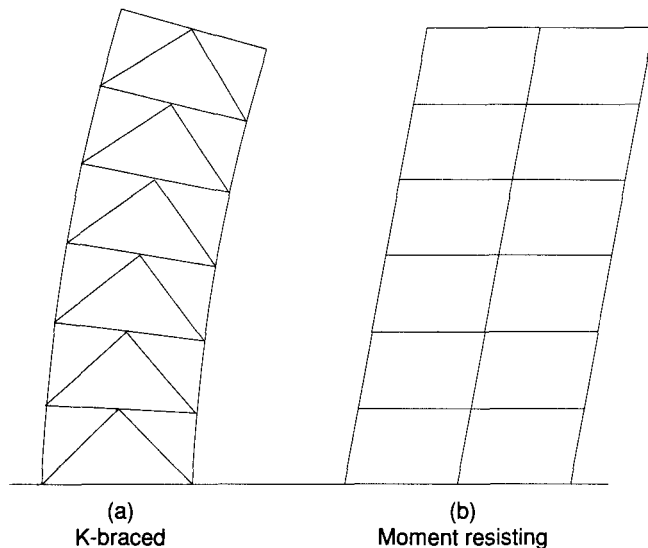
Fig. 28. Deflection characteristics.
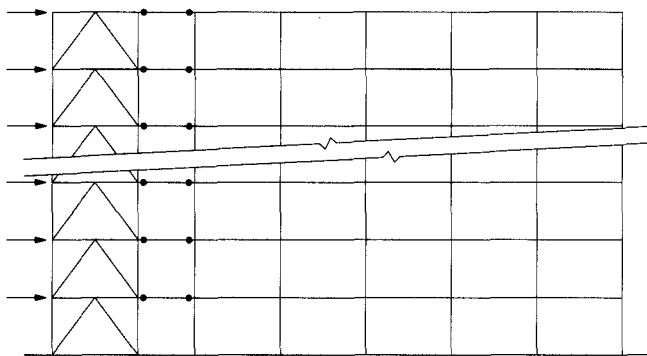
Fig. 29. Linked frame computer model.
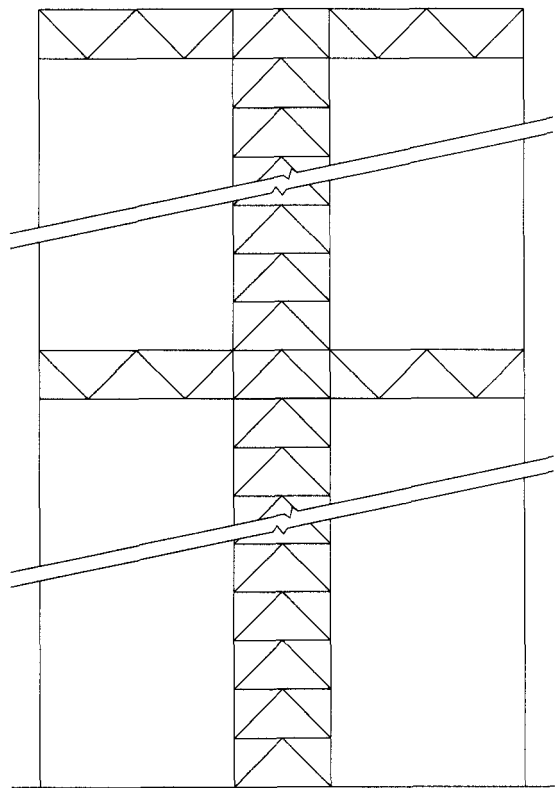
Fig. 31. K-braced frame with hat and belt trusses.

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load would be 6.48 in. and 6.31 in. for the moment resisting frame and the K-braced frame respectively. Thus the combination frame has slightly less deflection than would be expected (6.25 in. vs. 6.39 in.). The maximum story DIs are 0.0023, 0.00267, and 0.00314 respectively for the combined frame with full wind load and the moment resisting and braced frames with one-half of the wind load. The maximum girder moment in the moment resisting frame of 204 kip-ft at the second level for the one-half wind loading is reduced to 158 kip-ft at the sixth level for the combined frames. The maximum uplift in the braced bent of 661 kips for the braced bent is increased to 734 kips for the combined frames. As a result of the different deflected shapes, the K-bracing takes a relatively larger proportion of the total story wind shear near the bottom and proportionally less near the top. In Fig. 30 the curve for the combined frames takes a slight "S" shape.

The use of hat and/or belt trusses is a design method which can significantly increase the efficiency of bracing frames (Fig. 31). As an example, Fig. 32 illustrates a very slender bracing frame with an aspect ratio of 12.4. With the introduction of the hat truss members at the elevator penthouse level, the maximum deflection is reduced from 4.26 (DI = 0.0030) to 2.66 (DI = 0.0019). The purpose of the hat and belt trusses is to simply limit the rotation of the bracing bent as shown in Fig. 33. Hat trusses can be very helpful in reducing deflection and high uplift forces in relatively low buildings (10 to 15 stories). Hat and belt trusses are effective in reducing the rotation of a bracing bent as shown in Fig. 33. The design of hat and belt trusses is described by McNabb and Muvdi (1977) and Taranath (1974).

Facade bracing can be used to create a tubular structure

Fig. 33. Deflected shape bent with hat truss.

Fig. 32. Elevator core linked frame.
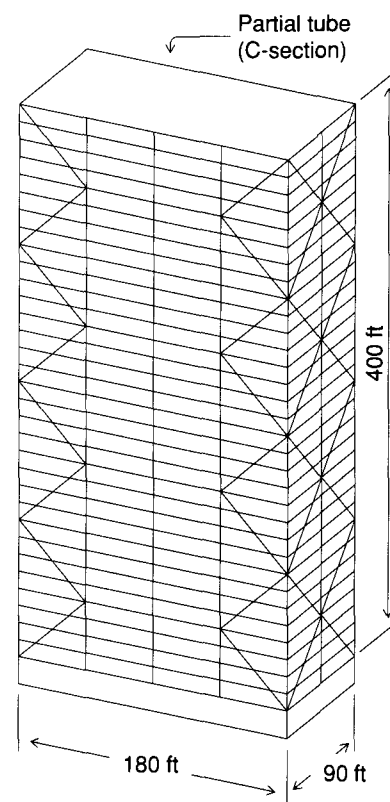
Fig. 34. Facade bracing.

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which can be an extremely efficient method of providing lateral load resistance in a building frame. The premier example may be the John Hancock Building located in Chicago, Illinois. The design is essentially the same as that for a conventional K-braced frame except that the effective depth of the vertical truss element will be the width of the bracing system. Colaco (1974) has described a method that uses facade bracing to create partial tubular structures in mid-rise buildings (Fig. 34).
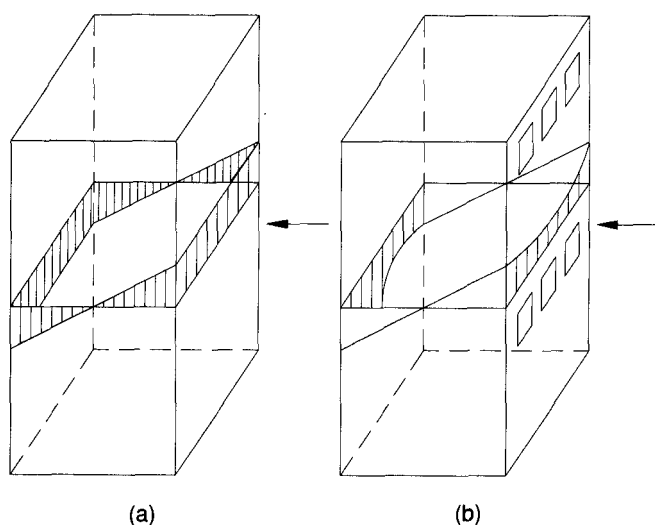
Fig. 35. Tubular frame.
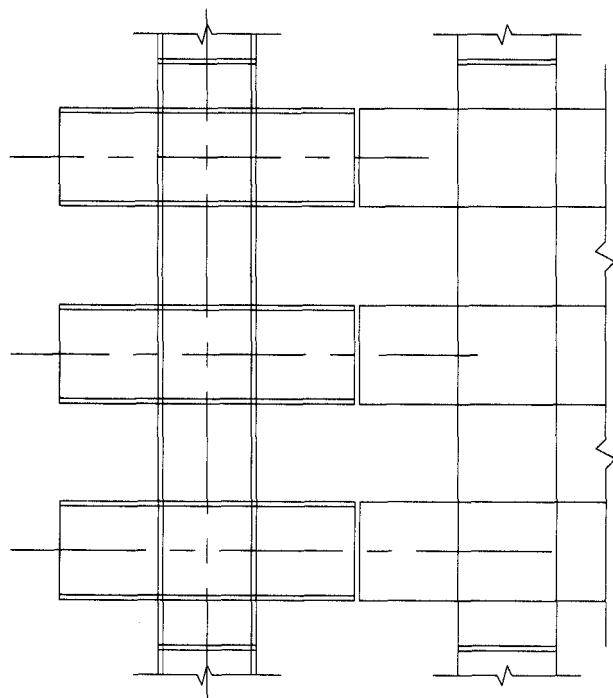
Fig. 36. Tree column.

The concept of tubular design is illustrated in Fig. 35a and 35b. Figure 35a depicts the stress distribution that would exist in a tube with solid walls. In a tubular building frame, the walls are punctured with holes for windows. As a result of the opening, shear lag associated with the bending in beams and columns on the windward and leeward sides reduces the efficiency of the tube (Fig. 35b). In tall buildings, often the exterior walls are framed with tree columns (Fig. 36). Tree columns often consist of H-shaped columns with H-shaped beam stubs fabricated from steel plates or deep rolled wide flange shapes. Tree columns have shear connections made midway between columns. Tube structures are not likely to be economical for the design of buildings less than 25 or 30 stories in height. Nair (1986) has described a modified tube concept. The method seeks to concentrate the column axial loads in the corners of the structure by creating a "soft sided tube."

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APPENDICES

LRFD COMPOSITE BEAM DESIGN

L = 30 ft   Spacing = 10 ft   b = (30/8)2 = 7.5 ft f_c = 3.5 ksi F_y = 36.0 ksi   3¼-in. slab + 3-in. deck

Loads

LL	100	(1.6) =	160
DL	60	(1.2) =	72
Total	160	(1.45) =	232 psf

Live Load Reduction (1987 BOCA, sect. 1115)

A_i = (10)(30)(2) = 600 sq. ft
L = $L_o(0.25 + 15/\sqrt{A_i})$ = 100(0.25 + 15/√600) = 86 psf
Reduction = 100 − 86 = 14 psf
W = 10 [232 − 14(1.6)] / 1000 = 2.1 kip/ft
M_u = 2.1(30)²/8 = 236.3 kip-ft

(See AISC LRFD Manual for nomenclature.)

Preliminary Beam Section

Beam weight = $\left[\dfrac{M_u(12)}{(d/2 + Y_{con} - a/2)\phi F_y} \right]$

For M_u = 236.3 kip-ft, Y_{con} = 3 + 3.25 = 6.25. Assume a = 1½ in.

Preliminary Beam Selections

Section Nom. Depth	$\dfrac{M_u(12)(3.4)}{\phi F_y}$	$d/2$	$Y_{con} - a/2$	Wt.
14	315.1	1	5.5	25.2
16	315.1	8	5.5	23.3
18	315.1	9	5.5	21.7

Try W16x26, F_y = 36, $Y2$ = 5.5, M_u (Req'd) = 236.5

From pg. 4-23 LRFD Manual (see next page), for ΣQ_n = 242 kip, ϕM_n = 248 kip-ft

$$a = \frac{242}{(.85)(3.5)(7.5 \times 12)} = 0.90 \text{ in.}$$

$$Y2 = 3 + 3.25 - 0.90/2 = 5.8 \text{ in.}$$

Selection is satisfactory.

For ¾-in. dia. headed stud with f_c' = 3.5 ksi

Conc. wt. = 115 pcf from LRFD Manual, pg. 4-7.
Q_n = 19.8 kips/stud
No. of studs req'd = (242/19.8)2 = 26

Use: W16x26, F_y = 36 ksi, with twenty-six ¾-in. dia. headed studs.

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$F_y = 36$ ksi

COMPOSITE DESIGN
COMPOSITE BEAM SELECTION TABLE
W Shapes

$\phi = 0.85$ $\qquad$ $\phi_b = 0.90$

Shape	$\phi_b M_p$	PNA[c]	$Y1^a$	ΣQ_n	ϕM_n (kip-ft) $Y2^b$ (in.)										
	Kip-ft		In.	Kips	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W 18x40	212	TFL	0.00	425	329	345	360	375	390	405	420	435	450	465	480
		2	0.13	368	321	334	347	360	373	386	399	412	425	438	451
		3	0.26	311	312	323	334	345	356	367	378	389	400	411	423
		4	0.39	254	303	312	321	330	339	348	357	366	375	384	393
		BFL	0.53	197	293	300	307	314	321	328	335	342	349	356	363
		6	2.26	152	282	288	293	298	304	309	315	320	325	331	336
		7	4.27	106	265	269	273	277	280	284	288	292	295	299	303
W 18x35	180	TFL	0.00	371	285	298	311	324	338	351	364	377	390	403	416
		2	0.11	325	278	290	301	313	324	336	347	359	370	382	393
		3	0.21	279	271	281	291	301	311	321	331	340	350	360	370
		4	0.32	233	264	272	280	289	297	305	313	322	330	338	346
		BFL	0.43	187	256	263	269	276	283	289	296	303	309	316	323
		6	2.37	140	245	250	255	260	265	270	275	280	285	290	295
		7	4.56	92.7	227	230	233	237	240	243	246	250	253	256	260
W 16x36	173	TFL	0.00	382	268	282	295	309	322	336	349	363	377	390	404
		2	0.11	328	261	272	284	295	307	319	330	342	353	365	377
		3	0.22	273	252	262	272	281	291	301	310	320	330	339	349
		4	0.32	219	244	251	259	267	275	282	290	298	306	314	321
		BFL	0.43	165	234	240	246	252	258	264	270	275	281	287	293
		6	1.79	130	227	232	236	241	245	250	255	259	264	268	273
		7	3.44	95.4	215	219	222	226	229	232	236	239	243	246	249
W 16x31	146	TFL	0.00	328	231	243	254	266	278	289	301	313	324	336	347
		2	0.11	285	225	235	245	255	265	275	285	295	305	315	326
		3	0.22	241	218	227	235	244	252	261	269	278	286	295	303
		4	0.33	197	211	218	225	232	239	246	253	260	267	274	281
		BFL	0.44	153	204	209	214	220	225	231	236	242	247	253	258
		6	2.00	118	196	200	204	208	212	217	221	225	229	233	237
		7	3.79	82.1	183	186	189	192	195	198	201	204	207	209	212
W 16x26	119	TFL	0.00	276	193	203	212	222	232	242	252	261	271	281	291
		2	0.09	242	188	196	205	214	222	231	239	248	257	265	274
		3	0.17	208	183	190	197	205	212	220	227	234	242	249	256
		4	0.26	174	177	184	190	196	202	208	214	220	227	233	239
		BFL	0.35	140	172	177	182	187	192	197	202	206	211	216	221
		6	2.04	104	164	168	171	175	179	182	186	190	194	197	201
		7	4.01	69.1	151	154	156	159	161	164	166	169	171	173	176

[a]Y1 = distance from top of the steel beam to plastic neutral axis.
[b]Y2 = distance from top of the steel beam to concrete flange force.
[c]See Fig. 4.3 for PNA locations.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

COMPOSITE BEAM LOAD CAPACITY ENHANCEMENT

Problem: Change occupancy from 100 psf LL office loading to 175 psf min. LL for automated filing system.

Original design—AISC *Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings,* Nov. 1, 1978, ASD.

Original Loads

	ASD	L.F.	LRFD
LL	100	(1.6)	160
3¼-in. slab + 3-in. deck	46		
Misc	4		
Ceil.	5	(1.2)	72
Steel	5		
	160 psf	(1.45)	232 psf

ASD Design

$L = 30$ ft   Spacing $= 10$ ft $L = 100 (0.25 + 15/\sqrt{600}) = 86$ psf $W = (160 - 14) \, 10/1000 = 1.46$ kip/ft

Beam selection: W16x31, $F_y = 36$ ksi, with thirty-two ¾-in. dia. studs.

Check design using LRFD.

$W_u = (86 \times 1.6 + 60 \times 1.2) \, (10) \, / \, 1000 = 2.10$ kip/ft
Light weight concrete $= 115$ pcf $Q_n = 19.8$ kips/stud
$\Sigma Q_n = (32/2) \, (19.8) = 317$ kips $A_s F_y = 328.3 > 317$ **O.K.**

$$a = \frac{317}{0.85 \, (3.5) \, (90)} = 1.18 \text{ in.} \qquad Y2 = 6.25 - \frac{1.18}{2} = 5.66$$

From pg. 4-23 LRFD Manual, for W16x31, $F_y = 36$ ksi, with thirty-two ¾-in. dia. studs, $\Sigma Q_n = 317$ by interpolation.

From interpolation of Table in LRFD Manual page 4-23:

$\phi M_n = 312$ kip-ft
DL moment $= 0.72(30)^2/8 = 81$ kip-ft
LL factored moment available $= 312 - 81 = 237$ kip-ft
LL working load moment (LF = 1) $= 231 / 1.6 = 144$ kip-ft
LL (ksf) $= 8M / (L^2 \times \text{spacing}) = 8 \, (144) \, / \, [(30)^2 10] = 0.128$ ksf, or 128 psf

Upper Bound Moment Capacity of Section Controlled by Shear Stud Capacity

Note: To find upper-bound moment capacity, add a cover plate with $A F_y = \Sigma Q_n$

Use 9x1 cover plate

Assume A=8.8 in.^{2}
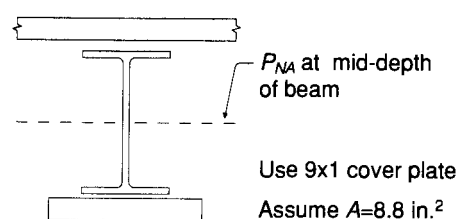
$\Sigma Q_n = 16 \, (19.8) = 316.8$ kip-ft
Equiv. area $= 316.8 \, / \, 36 = 8.8$ in.^{2}, Use 9-in. ×1-in. cover plate
 Assume plate width $= w = 8.8$ in.
Plate thickness $= t = 1$ in.

$\phi M_n = \Sigma Q_n e + M_{p \, beam}$
$e = t \, / \, 2 + d + Y2$
$\phi M_n = 85 \, (M_p) + .9 \, (\Sigma Q_n) \, (t/2 + d + Y2) \, / \, 12$
$\phi M_n = 146 + .85 \, (316.8) \, (.5 + 15.88 + 5.66) \, / \, 12 = 640$ kip-ft

This value is too high for practical use. Connection and web shear design will control.

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Live load moment capacity $= (640 - 81) / 1.6 = 349$

$$LL = \frac{349\ (8)}{(30)^2\ (10)}\ (1000) = 310\ \text{psf}$$

ASD Analysis
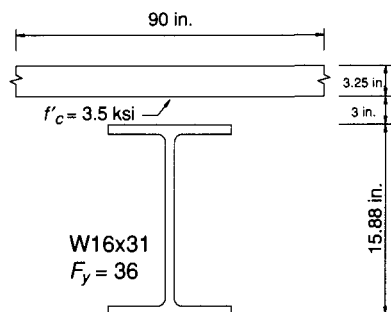
W16x31 $F_y = 36$ ksi

$n = 9$
$f_c' = 3.5$ ksi

Find NA (ΣM at bottom) $A_{steel} = 9.12$ in.^{2}, $A_{conc} = 90\ (3.25)\ /\ 9 = 32.5$ in.^{2}

$9.12\ (15.88\ /\ 2)$		72.4
$32.5\ (15.88 + 3 + 3.25\ /\ 2) =$		666.4
		738.8

$$\bar{y} = (738.8\ /\ 41.62)$$
$$= 17.75\ \text{in.}$$

Compute I_x

W16	$375 + 9.12\ (17.75 - 15.88\ /\ 2)^2 = 375 + 878$		$= 1253$
Conc.	$90\ /\ 9\ (3.25)^3\ /\ 12 + 32.5\ (15.88 + 3.0 + 3.25\ /\ 2 - 17.75)^2 =$		276
		$I_x =$	1529

$$S_{bot} = \frac{1529}{17.75} = 86.1 \qquad S_{top} = \frac{1529}{4.38} = 349$$

Moment cap. $= \dfrac{S\ F_s}{12} = \dfrac{86.1\ (24)}{12} = 172.2$ kip-ft

DL moment $= .6\ (30)^2\ /\ 8 \qquad = 67.5$ kip-ft
LL moment cap. $= \qquad\qquad 104.7$ kip-ft

$$f_c = \frac{172.2\ (12)}{349}\ /\ 9 = .66\ \text{ksi}\quad \textbf{O.K.}$$

ASD Analysis

6x¾-in. cover plate,
$F_y = 36$
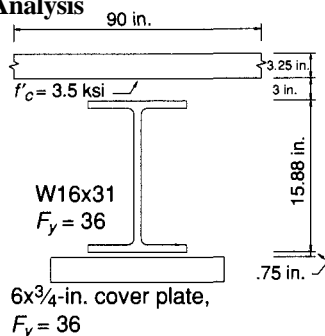
W16x31 + 6 x ¾-in. cover plate $F_y = 36$ ksi

$n = 9$

Find NA

Plate 6 (.75)	4.50	(.75 / 2)	1.7
W16	9.12	(.75 + 15.88 / 2)	79.3
Conc.	32.50	(.75 + 15.88 + 3 + 3.25 / 2) =	690.8
	46.12		771.8

$$\bar{y} = (771.8\ /\ 46.12)$$
$$= 16.73\ \text{in.}$$

Compute I_x

6×¾-in. plate	$6\ (.75)^3\ /\ 12 + 4.5\ (16.73 - .75\ /\ 2)^2 =$	1204
W16	$375 + 9.12\ (.75 + 15.88\ /\ 2 - 16.73)^2 =$	965
Conc.	$90\ /\ 9\ (3.25)^3\ /\ 12 + 32.5\ (.75 + 15.88 + 3 + 3.25\ /\ 2 - 16.73)^2 =$	694
		2863

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$S_{bot} = 2863 \, / \, 16.73 = 171$

$S_{top} = 2863 \, / \, (.75 + 15.88 + 6.25 - 16.73) = 466$

Moment cap. $= 171 \, (24 \, / \, 12) = 342$ kip-ft

$f_c = [342 \, (12) \, / \, 466] \, / \, 9 = .98$ ksi **O.K.**

$V_h = A_s F_y \, / \, 2 = (4.5 + 9.12) \, / \, 2 \, (36) = 245$ kip

$V_h' = V_{studs} = 16 \, (10.75) = 172$ kip

$$S_{eff} = S_s + \sqrt{\frac{V_h'}{V_h}} \, (S_{tr} - S_s)$$

$$= 47.2 + \sqrt{\frac{172}{245}} \, (171 - 47.2) = 151$$

Effective moment cap. $= 151 \, (24) \, / \, 12 = 302$ kip-ft

DL moment $= 0.6 \, (30)^2 \, / \, 8 \qquad = \underline{\quad 67.5 \quad}$

Available LL moment cap. $\qquad\qquad 234.5$ kip-ft

$$LL = \frac{234.5 \, (8)}{(30)^2} \, (1000 \, / \, 10) = 208 \text{ psf}$$

LRFD Analysis

W16x31 + 6 x ¾-in. cover plate $F_y = 36$ ksi, with thirty-two ¾-in. dia. studs
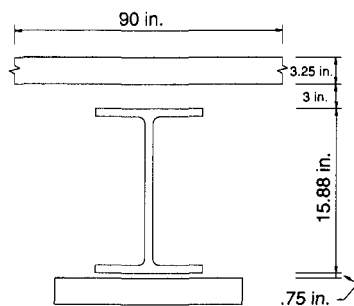
$E \quad = (115)^{1.5} \sqrt{3.5} = 2307$ ksi $n = \cancel{12.57} \quad 13$

$Q_n \quad = 19.8$ kip

$\Sigma Q_n = \begin{array}{ll} 16 \, (19.8) & = 317 \text{ kip Use} \\ 36 \, (9.12) & = 328 \end{array}$

$a \quad = \dfrac{317}{.85 \, (3.5) \, 90} = 1.18$ in.

$\Sigma T + C = \Sigma Q_n + A F_y = 317 + (9.12 + 4.5) \, (36) = 807.3$

$T = C = (\Sigma T + C) \, / \, 2 = 403.7$ kip

$\Sigma A_s = 9.12 + 4.5 = 13.62$

$A_{s \, comp.} = (C - \Sigma Q_n) \, / \, F_y = (403.7 - 317) \, / \, 36 = 2.41$ in.^{2}

$A_{s \, tension} = (\Sigma A_s - A_{s \, comp.}) = 13.62 - 2.41 = 11.21$ in.^{2}

$A_{flange} = 5.525 \, (.44) = 2.43$ in.^{2}

$A_{web} = 9.12 - 2 \, (2.43) = 4.26$ in.^{2}

$t_w = 4.26 \, / \, [15.88 - 2 \, (.44)] = .284$ in.

Assume PNA is above bottom of top flange.

$Y1 = -[TF + (A_{flange} - A_{comp}) \, / \, BF] = -[.44 - (2.43 - 2.41) \, / \, 5.525]$

$\qquad = -.44 + .004 = .44$ in.

Take PNA at bottom of top flange.

COMPOSITE BEAM LOAD CAPACITY ENHANCEMENT (cont.)

Find centroid of steel force:

Plate	4.5	(.375)	=	1.69
Bottom Flange	2.43	(.75 + .22)	=	2.36
Web	4.26	(.75 + 15.88 / 2)	=	37.02
	11.19			41.07

$$\bar{y}_1 = \frac{41.07}{11.19} = 3.67 \text{ in.}$$

Centroid of tension force is 3.67 above bottom of cover plate.

LRFD Analysis

W16x31 + 6-in x ¾-in. cover plate F_y = 36 ksi with thirty-two ¾-in. dia. studs

Find centroid of compressive force (in terms of compressive force).

W16 (top flange)	87.48	(.75 + 15.88 − .44 / 2)	=	1436
Concrete (ΣQ_n)	317	(.75 + 15.88 + 6.25 − 1.18 / 2)	=	7066
	404.5			8502

$$\bar{y}_2 = \frac{8502}{404.5} = 21.02 \text{ in.}$$

$e = \bar{y}_2 - \bar{y}_1 = 21.02 - 3.67 = 17.35$ in.

$\phi M_n = \Sigma T \, e \, / \, 12 = .85 \,(404)\,(17.35) \, / \, 12 = 496$ kip-ft

$M_{u \, DL} = .72 \,(30)^2 \, / \, 8 \qquad = \underline{81.0}$ kip-ft

$M_{u \, LL} \qquad\qquad\qquad\qquad = 415$

$LL_{fact.} = [415 \,(8) \, / \, (30)^2] \, 1000 \, / \, 10 = 369$ psf

$LL_{work} = 369 \, / \, 1.6 = 230$ psf

COMPOSITE BEAM LONG TERM DEFLECTION

Typical Filler Beam

W16x31 F_y = 36 ksi, with thirty-two ¾-in. dia. studs
L = 30 ft Spacing = 10 ft

Loads

LL	100
3¼-in. slab + 3-in. deck	46
Misc.	4
Ceiling	5
Steel	5
	100 psf

$E_c = w^{1.5}\sqrt{f_c'} = (115)^{1.5} \sqrt{3.5} = 2{,}307$ ksi $n = 13$
½E_c = 1,154 $n = 25$

90 in.

3.25 in.

3 in.
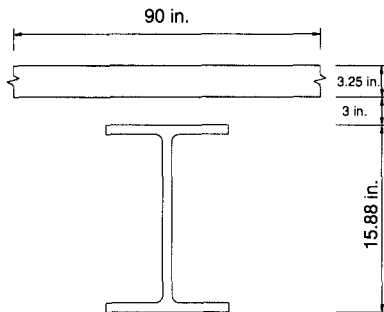
15.88 in.

Find moment of inertia for short term loading (i.e., full value of E_c)

A_c = 3.25 (90) / 13 = 22.5 in.²
ΣM

A_c	22.5	(15.88 + 3 + 3.25 / 2) =	461.4
W16	9.12	(15.88 / 2) =	72.4
	31.12		533.8

$$\bar{y} = \frac{533.8}{31.62} = 16.88 \text{ in.}$$

Compute I_x

W16	375 + 9.12 (16.88 − 15.88 / 2)²	=	1,104
A_c	22.5 (15.88 + 3 + 3.25 / 2 − 16.88)² =		296
		I_x =	1,400

Find moment of inertia for long term loading (i.e., E_c = .5$w^{1.5}$ $\sqrt{f_c'}$ = 1154 $n = 25$

A_c = 3.25 (90) / 25 = 11.7 in.²
ΣM

A_c	11.7	(15.88 + 3 + 3.25 / 2) =	239.9
W16	9.12	(15.88 / 2) =	72.4
	20.82		312.3

$$\bar{y} = \frac{312.3}{20.82} = 15.0 \text{ in.}$$

Compute I_x

A_c	11.7 (15.88 + 3 + 3.25 / 2 − 15.0)² =		355
W16	375 + 9.12 (15.0 − 15.88 / 2)²	=	830
			1185

Deflection Summary

Assume 10 psf ambient LL for long term deflection
        76 psf LL for short term deflection
        M_{LL} = 85.5 kip-ft M_{DL} + 10 psf = 78.8 kip-ft

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COMPOSITE BEAM LONG TERM DEFLECTION (cont.)

Shored Construction

$$\Delta_{LL} = \frac{ML^2}{161\ I} = \frac{85.5\ (30)^2}{161\ (1400)} = .34 \text{ in.}$$

$$\Delta_{DL\ long\ term} = \frac{78.8\ (30)^2}{161\ (1185)} = \underline{.37} \text{ in.}$$

$$\text{Total Defl.} = .71 \text{ in.}$$

Unshored Construction

(Assume camber overcomes dead load deflection.)

Long term load causing deflection 9 psf DL + 10 psf LL = 19 psf M = 21.4 kip-ft

$$\Delta_{LL} = .34$$

$$\Delta_{DL\ +\ 10\ psf\ LL} = \frac{21.4\ (30)^2}{161\ (1185)} = \underline{.10}$$

$$\text{Total Defl.} = .44 \text{ in.}$$

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STEEL JOIST TYPICAL BAY

Loads

LL	50 x 1.6 = 80		Joist $L = 30$ ft
Part.	20		Use 22K5 @ 3 ft
3-in. slab	38		
Joist	4 x 1.2 = 80		Girder
Ceiling	3		
Misc.	2		

$$\text{Reduced LL} = 50 \left(.25 + \frac{15}{\sqrt{1800}} \right) = 30$$

117 x 1.37 = 160

$w_{SL} = (30 + 67)(30) + 50 = 2{,}960 \text{ lb/ft}$

$w_u = (30 \times 1.6 + 67 \times 1.2)(30) + 60 = 3{,}910 \text{ lb/ft}$

$w_u / w_{SL} = 1.32$

Simple Span

$M_{SL} = 2.96 (30)^2 / 8 = 333 \text{ kip}$

$M_u = 440 \text{ kip-ft}$ $Z_{AF_y = 36} = \dfrac{440 (12)}{.9 (36)} = 163$ W18×76, W24×68

$Z_{F_y = 50} = \dfrac{440 (12)}{.9 (50)} = 177$ W18×60, W24×55

Use maximum beam depth = 18 in.

W18×60 $\Delta_{LL} = \dfrac{w_{LL}}{w_{TL}} \left(\dfrac{ML^2}{161 \, I} \right) = \dfrac{1.5}{2.96} \left(\dfrac{333 (30)^2}{161 \times 984} \right) = .96 \text{ in.}$

W18×76 $\Delta_{LL} = .96 \times \dfrac{984}{1330} = .71 \text{ in.}$

Continuous Design (assume max. moment $= wL^2 / 11.3$)

$M_u = \dfrac{3.91 (30)^2}{11.3} = 311 \text{ kip-ft}$ $Z_{F_y = 36} = \dfrac{311 (12)}{.9 (36)} = 115$ W18×60, W24×55

$Z_{F_y = 50} = \dfrac{311 (12)}{.9 (50)} = 83$ W18×46, W21×44

Conservatively take $\Delta = \dfrac{ML^2}{(3.5) \, 161 \, I}$ where $M = \dfrac{wL^2}{8}$

$\Delta \, (\text{W18×60}) = \dfrac{1.5}{2.96} \left(\dfrac{333 (30)^2}{(3.5) (161) (994)} \right) = .27 \text{ in.}$

$\Delta \, (\text{W18×46}) = \dfrac{1.5}{2.96} \left(\dfrac{333 (30)^2}{(3.5) (161) (712)} \right) = .38 \text{ in.}$

Composite Beam Design

Use LRFD Composite Beam Tables, pages 4-36 and 4-49.

Take $Y2 = 2.5 + 3 - 1 = 4.5$ in., use $f'_c = 3.5$ ksi.

For $M_u = 440$ kip-ft $M_{SL} = 333$ kip-ft

Try W18×35 $F_y = 50$ $\phi M_n = 466$ kip-ft $\Sigma Q_n = 387$ $Y2 = 4.5$ in. $\phi M_p = 249$ kip-ft

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STEEL JOIST TYPICAL BAY (cont.)

b = (30 / 8) 2 = 7.5 ft = 90 in.

$T_{req'd}$ = 387 / [(.85) (3.5) (90)] = 1.45 in. $Y2$ = 5.5 − .72 = 4.78 in.   **O.K.**

n = (387 / 23.6) 2 = ~~32.8~~   thirty-four ¾-in. dia. headed studs

$I_{lower\ bound}$ = 1370

$$\Delta_{LL} = \frac{1.5}{2.96} \left(\frac{333\ (30)^2}{161\ (1370)} \right) = .69 \text{ in. } \textbf{O.K.}$$

Check wet load strength. Assume const. LL 0.3 kip/ft*

Wet load = [.3 (1.6) + .042 (30) 1.2 + .035 (1.2)] = 2.03 kip/ft

M_u = 2.03 (30)^2 / 8 = 228 kip-ft < 249 kip-ft

Use W18x35 F_y = 50 ksi, thirty-four ¾-in. dia. studs.

* Some provision should be made for construction live load. In this case 9K per bay has been assumed.

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K-FRAME BRACING OPTIMIZATION

$P = 5^K \quad (10^K / FL)$

Target DI = .0025 rad.
(DI = deflection at x / total height)

Brace length (BRACEL)

$= \sqrt{12.5^2 + 12.5^2} = 17.68 \text{ ft} = 212.1 \text{ in.}$

Column length = 12.5 ft = 150 in.

Bay length (L) = 25 ft = 300 in.

Frame Elevation
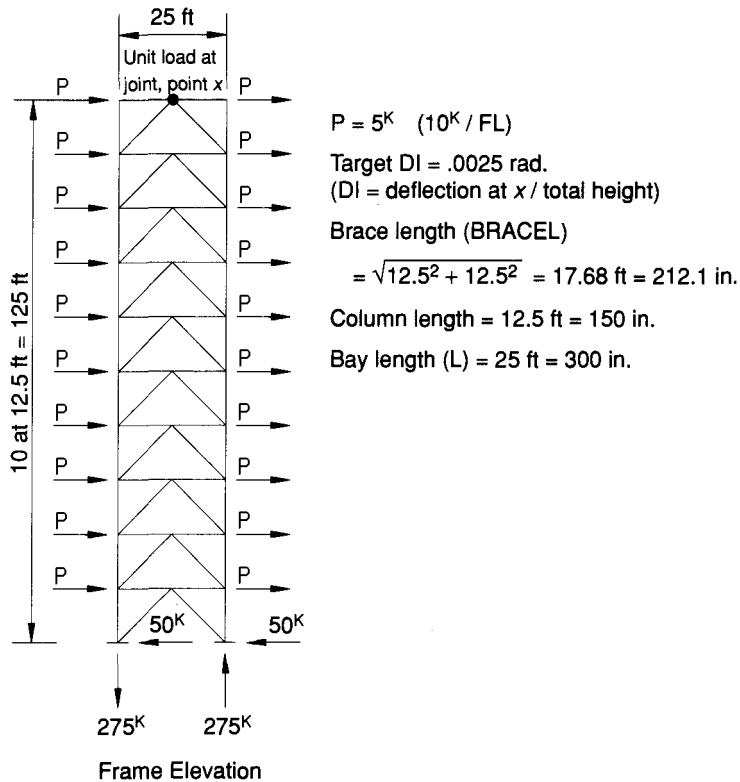
Col. No.	1	2	3	4	5	6	7	8	9
Item	WINDF	STYSHR	STYMOM	COLP	GIRDP	BRACEP	COLP1	GIRDP1	BRACP1
FL(I)	0	0	0						
10	10	10	125	0	5	7.07	0	0	.707
9	10	20	250	5	10	14.14	.5	.5	.707
8	10	30	375	15	15	21.21	1.0	.5	.707
7	10	40	500	30	20	28.28	1.5	.5	.707
6	10	50	625	50	25	35.36	2.0	.5	.707
5	10	60	750	75	30	42.43	2.5	.5	.707
4	10	70	875	105	35	49.50	3.0	.5	.707
3	10	80	1,000	140	40	56.57	3.5	.5	.707
2	10	90	1,125	180	45	63.64	4.0	.5	.707
1	10	100	1,250	225	50	70.71	4.5	.5	.707

**Table 1.
Member Loads**

1 — WINDF: Story wind force (kip)
2 — STYSHR: Story wind shear = WINDF (I) + STYSHR (I + 1)
3 — STYMOM: Story moment = STYSHR (I) x HT (I)
4 — COLP: Col. axial load = COLP (I + 1) + STYMOM (I + 1) / L
5 — GIRDP: Girder axial load = STYSHR (I) / 2
6 — BRACEP: Brace axial load = STYSHR (I) / 2 x [brace L / (L / 2)]
7 — COLP1: Col. virtual load = COLP (I + 1) + (1) x HT (I + 1) / L
8 — GIRDP1: Girder virtual load = ½ = .5
9 — BRACP1: Brace virtual load = .5 x [brace L / (L / 2)] = .707

HT (I) = Story height
L = Bay length (c.t.c. columns)
brace L = Brace length

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K-FRAME BRACING OPTIMIZATION (cont.)

Table 2.
Member Areas and Strains
λ = 1.0

Col. No.	10	11	12	13	14	15
Item	COLAR	GIRDAR	BRACAR	COLD	GIRDD	BRACED
FL (I)						
10	0.0	0.0	2.24	0.0	.0231	0.0
9	1.58	2.24	3.16	.0164	.0327	.0231
8	3.87	2.74	3.87	.0200	.0401	.0283
7	6.71	3.16	4.47	.0213	.0463	.0327
6	10.0	3.54	5.00	.0259	.0572	.0366
5	13.69	3.87	5.48	.0283	.0567	.0401
4	17.75	4.18	5.92	.0306	.0612	.0433
3	22.14	4.47	6.32	.0327	.0654	.0463
2	26.83	4.74	6.71	.0347	.0694	.0491
1	31.82	5.00	7.07	.0366	.0732	.0572

10 — COLAR: Col. area $= \sqrt{(P_i n_i)/\lambda} \approx \sqrt{\text{(col4)} \times \text{(col7)}}$
11 — GIRDAR: Girder area $\approx \sqrt{(P_i n_i)/\lambda} \approx \sqrt{\text{(col5)} \times \text{(col8)}}$
12 — BRACAR: Brace area $\approx \sqrt{(P_i n_i)/\lambda} \approx \sqrt{\text{(col6)} \times \text{(col9)}}$
13 — COLD: Column strain = [COLP x HT (I)] / (COLAR x E)
            = [(col4) x HT (I)] / [(col10) x E]
14 — GIRDD: Girder strain = (GIRP x L / 2) / (GIRDAR x E)
            = [(col5) x L / 2] / [(col11) x E]
15 — BRACED: Brace strain = (BRACEP x BRACEL) / (BRACAR x E)
            = [(col6) x BRACEL] / [(col12) x E]

  E = Young's modulus = 29.0 ksi.

Table 3.
RHOs and Deflections
λ = 1.0

Col. No.	16	17	18	19	20	21	22
Item	COLRHO	SUMROC	GIRRHO	BRARHO	SUMRHO	FLDEL	TOTDEL
FL (I)							
10	0.0	.00166	0.0	.00022	.00188	.282	2.479*
9	.00011	.00155	.00015	.00031	.00201	.302	2.197
8	.00014	.00141	.00019	.00038	.00198	.297	1.895
7	.00015	.00126	.00022	.00044	.00192	.288	1.598
6	.00017	.00109	.00024	.00049	.00182	.273	1.310
5	.00019	.00090	.00027	.00053	.00170	.255	1.037
4	.00021	.00069	.00029	.00058	.00156	.234	.782
3	.00022	.00047	.00031	.00062	.00140	.210	.548
2	.00023	.00024	.00033	.00065	.00122	.183	.338
1	.00024	0.0	.00034	.00069	.00103	.155	.155

16 — COLRHO: Col. floor RHO = 2 x COLD / L = 2 x (col13) / L
17 — SUMROC: Sum of col. RHO — 1st story: SUMROC = (col17) = 0
      Above: SUMROC (I) = SUMROC (I - 1) + COLRHO (I)
               = (col17) + (col16)
18 — GIRRHO: Girder fl. RHO = GIRDD / HT = (col14) / HT
19 — BRARHO: Brace fl. RHO = BRACED x (2 x brace L) / (L x HT)
               = (col15) x (2 x brace L) / (L x HT)
20 — SUMRHO: Sum of RHOs @ FL = SUMROC + GIRRHO + BRARHO
               = (col17) + (col18) + (col19)
21 — FLDEL: Floor deflection = SUMRHO x HT = (col20) x HT
22 — TOTDEL: Total floor deflection = FLDEL (I) + TOTDEL (I - 1)
               = col21 (I) + col22 (I - 1)

* Max Δ, λ = 1.0.

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K-FRAME BRACING OPTIMIZATION (cont.)

Col. No.	23	24	25	26	27	28	29
Item	COLAR	GIRDAR	BRACAR	C.R.	ACOL	AGIRD	ABRAC
FL (I)							
10	0.0	0.0	2.24	.6	0.0	0.0	1.48
9	1.58	2.24	3.16	—	1.04	1.48	2.09
8	3.87	2.74	3.87	—	2.55	1.81	2.55
7	6.71	3.16	4.47	—	4.43	2.09	2.95
6	10.0	3.54	5.00	—	6.60	2.34	3.30
5	13.69	3.87	5.48	—	9.04	2.55	3.62
4	17.75	4.18	5.92	—	11.72	2.76	3.91
3	22.14	4.47	6.32	—	14.61	2.95	4.17
2	26.83	4.74	6.71	—	17.71	3.13	4.43
1	31.82	5.00	7.07	.66	21.00	3.30	4.67

Table 4.
Optimum Areas

23, 24, 25 — Repeat of columns 10, 11, 12
26 — Correction factor
27 — ACOL: Optimum column area = (col23) x (col26)
28 — AGIRD: Optimum girder area = (col24) x (col26)
29 — ABRAC: Optimum brace area = (col25) x (col26)

Correction factor (CR)
  Target D.I. = 0.0025 radians
  Calculated D.I. with $\lambda = 1.0 = \Delta_{top}$ / tot. height
                 $= 2.48 / (10 \times 12.5 \times 12) = .00165$
  C.R. = Calculated D.I. / Target D.I.
       $= .00165 / .0025 = .66$

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UNBRACED FRAME DESIGN

Loads

$$LL = 100 \times 1.6 = 160$$
$$DL = \underline{65} \times 1.2 = \underline{78}$$
$$Total \quad 165 \times 1.44 = 238$$

Typical filler beam:* $L = 30$ in.   Spacing $= 10$ ft $A_i = 600$ ft^2

Reduced LL $= 100 (.25 + 15 / \sqrt{600}) = 86$

$w = 10 [.086 (1.6) + .065 (1.2)] = 2.16$ kip/ft

$M_u = 2.16 (30)^2 / 8 = 243$ kip-ft

From the LRFD Manual (pg. 4-23) try W16x26 $F_y = 36$ $Y2 = 5.5$ in. $\phi M_n = 248$ kip-ft $\Sigma Q_n = 242$

$a = 242 / [.85 (3.5) (90)] = .90$

$Y2 = 6.25 - .90 / 2 = 5.8 > 5.5$ in.

$n = (242 / 19.8) 2 = \cancel{24.4}$ twenty-six ¾-in. dia. studs

Use W16x26 $F_y = 36$ with twenty-six ¾-in. dia. studs.

Typical girder: $L = 30$ in.   Spacing $= 30$ ft $A_i = 1800$ ft^2

Reduced LL $= 100 (.25 + 15 / \sqrt{1800}) = 60$

$P_u = 300 [1.6 (.06) + 1.2 (.065)] = 52.2$ kip

$M_u = 52.2 (10) = 522$ kip-ft

From the LRFD Manual (pg. 4-36) try W18x40 $F_y = 50$ $Y2 = 5$ in. $\phi M_n = 526$ kip-ft $\Sigma Q_n = 432$

$a = 432 / [.85 (3.5) (90)] = 1.61$

$Y2 = 6.25 - 1.61 / 2 = 5.45 > 5$ in.

$n = (432 / 19.8) 2 + 2 = \cancel{45.6}$ forty-six ¾-in. dia. studs

Use W18x40 $F_y = 50$ ksi with forty-six ¾-in. dia. studs.

* Note: Wet load strength and deflection calculations omitted for brevity.

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UNBRACED FRAME DESIGN (cont.)

W14x43 K = 32.4 M = 87ᴵᴷ | W14x48 K = 38.8 M = 55ᴵᴷ | K = 38.8 M = 48ᴵᴷ

L = 30
W24x55 K = 450   W24x55 K = 450

V	42.3		42.3		
AM/L	3.0				
V_u	39.3	x = 10			
	.41		.27	.27	.27	.27
FEMDL	-112		-112	-112	-112	-112
FEMTL	-273		-273	-273	-273	-273
C.O.	-22		-56	-22	-22	-22
ADD	-295		-329	-295	-295	-295
DIST	+121		+9	-9		
MOM	-174 + 200		-320	-304	-295	-295
Z			119	113		109
SECT	W24x55 A – 36			W24x55 A – 36		W24x55 A – 36

MAXIMUM POSITIVE MOMENT

FEM	-273	-273	-112	-112	-273
C.O.	-22	-56	+22	+22	-22
ADD	-295	-329	-90	-90	-295
DIST	+121	+65	-65	-55	+55
MOM	-174	-264	-155	-145	-240

$$+M = 39.3(10) - \frac{.38(10)^2}{2} - 174 = 200^{\,IK}$$

GRAVITY FOR WIND LOAD
LF = .5L + 1.2D

Col. M = 50ᴵᴷ | Col. M = 21ᴵᴷ | Col. M = 16ᴵᴷ

	.41		.27	.27	.27	.27
	-112		-112	-112	-112	-112
-112	-163		-163	-163	-163	-163
+7	-7		-33	-7	-7	-7
-105	-170		-196	-170	-170	-170
+43	70		4	4		
62	-100		-192	-174	-170	-170

Gravity Girder Moment for Wind Load

Spandrel Beam Loads

A_i = 16 × 30 = 480 ft²
LL = 100 (.25 + 15 / √480) = .094

Wall Load = 25 psf

$$P_u = (16 \times 10)\,(.094)\,1.6 = 24.1$$
$$ (16 \times 10)\,(.065)\,1.2 = \underline{12.5}$$
$$ 36.6$$

W_a = (.025) (12.5) (1.2) = .38

$$\text{FEMTL} = \frac{36.6\,(30)}{4.5} + \frac{.38\,(30)^2}{12} = 244 + 29 = 273$$

$$\text{FEMDL} = \frac{12.5\,(30)}{4.5} + 29 = 112$$

$$Z = \frac{273\left(\dfrac{12}{12}\right)12}{.9\,(36)} = 110$$

Try W24x55 A36

For Wind-Gravity Loading

P_u = 24.1 (.5 / 1.6) + 12.5 = 20.0
FEMTL = (20 / 36.6) 244 + 29 = 163ᴵᴷ
FEMDL = 112ᴵᴷ

Design Loads

LL = 100 × 1.6 = 160
DL = 65 × 1.2 = 78
165 × 1.44 = 238

UNBRACED FRAME DESIGN (cont.)

Typical Corner

Working from bottom to top:

Number of Stories	TL (psf)	LL (psf)
9	165.0	100.0
1	60.0	30.0

Total = 10 Stories

Number of Stories	Bay: Length (ft)	Width (ft)
10	16.00	16.00

Number of Stories	Added DL (kips)
10	9.00

Type of LL Reduction: 1984 BOCA Code

Corner Column*

Floor	Load	Red	Tot P	DL/TL	Fact P $1.6L+1.2D$	$0.5L+1.2D$
Roof	24	0	24	.70	31	24
9	51	7	68	.62	92	64
8	51	14	105	.65	141	100
7	51	15	141	.67	188	136
6	51	16	176	.68	234	172
5	51	16	211	.69	279	207
4	51	17	246	.69	325	242
3	51	17	280	.70	370	*211*
2	51	17	314	.71	413	312
1	51	17	348	.71	459	347

* Loads shown in kips

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UNBRACED FRAME DESIGN (cont.)

Typical Spandrel

Working from bottom to top:

Number of Stories	TL (psf)	LL (psf)
9	165.0	100.0
1	60.0	30.0

Total = 10 Stories

Number of Stories	Bay: Length (ft)	Width (ft)
10	16.00	30.00

Number of Stories	Added DL (kips)
10	9.00

Type of LL Reduction: 1984 BOCA Code

Typical Spandrel Column*

Floor	Load	Red	Tot P	DL/TL	Fact P $1.6L+1.2D$	$0.5L+1.2D$
Roof	38	0	37	.63	50	35
9	88	20	106	.60	144	97
8	88	29	165	.63	222	155
7	88	31	222	.65	298	212
6	88	32	279	.66	373	269
5	88	32	335	.67	447	325
4	88	30	394	.67	524	382
3	88	29	453	.67	603	440
2	88	29	512	.67	681	498
1	88	29	572	.67	761	556

* Loads shown in kips

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UNBRACED FRAME DESIGN (cont.)

Basic Wind Speed: 70   Exposure: B
Wind Distribution Coefficients
  Windward .8   Leeward .5
Bent: Spandrel Wind Frame

Parapet:	Ht	Width	Area	Penthouse:	Ht	Width	Area
	3.13	76.00	238		.00	.00	0

Number of Stories: 10

No. of Stories	Story Ht	Panel Width	Area
10	12.50	76.00	950

Parapet H = 5.2 kips   Penthouse H = 0 kips

Floor	Floor H	Sum H	Vel Press	H Top	H Bot	Fl Elev
Roof	19.9	19.9	17	9.5	9.3	125.00
10	18.5	38.5	15	9.3	9.3	112.50
9	18.5	57.0	15	9.3	9.3	100.00
8	18.5	75.5	15	9.3	9.3	87.50
7	17.0	92.5	15	7.7	7.4	75.00
6	14.8	107.3	12	7.4	7.4	62.50
5	14.8	122.1	12	7.4	7.3	50.00
4	14.0	136.1	11	6.7	6.5	37.50
3	12.6	148.8	10	6.2	6.2	25.00
2	12.3	161.1	10	6.2	6.2	12.50

Total Wind Shear = 167.3

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UNBRACED FRAME DESIGN (cont.)

Stiffness values *(I/L)* output are to be multiplied by beam or column length (ft) in order to get the required member moment of inertia (in.^{4}).

Point of contraflexure is assumed to be at mid-height of all columns

E = 29,000 ksi
Number of Stories: 10
Number of Rows of Columns: 6
Story DI (Deflection/Story Ht.) = .0025

Floor	Story Ht	Wind Load	Sum of Wind Load	Vert Load	Sum of Vert Load
10	12.50	19.90	19.90	1639	1639
9	12.50	18.50	38.40	1350	2989
8	12.50	18.50	56.90	1235	4224
7	12.50	18.50	75.40	1293	5517
6	12.50	17.00	92.40	1293	6810
5	12.50	14.80	107.20	1293	8103
4	12.50	14.80	122.00	1293	9396
3	12.50	14.00	136.00	1293	10689
2	12.50	12.60	148.60	1293	11982
1	12.50	12.30	160.90	1293	13275

UNBRACED FRAME DESIGN (cont.)

Floor	Factor	Stiffness *(I/L)*	Moment Magnifier
10	3.43	16.47	1.259
		32.94	1.259
9	6.62	24.12	1.250
		31.78	1.242
8	9.81	39.43	1.235
		47.09	1.228
7	13.00	54.74	1.226
		62.40	1.224
6	15.93	69.43	1.225
		76.47	1.226
5	18.48	82.59	1.229
		88.72	1.233
4	21.03	94.84	1.236
		100.97	1.238
3	23.45	106.76	1.241
		112.55	1.245
2	25.62	117.77	1.249
		122.98	1.252
1	27.74	128.07	1.256
		133.16	1.260

Factor $= Q \times$ (Story Ht^2) / (12 $\times E \times \Delta$)
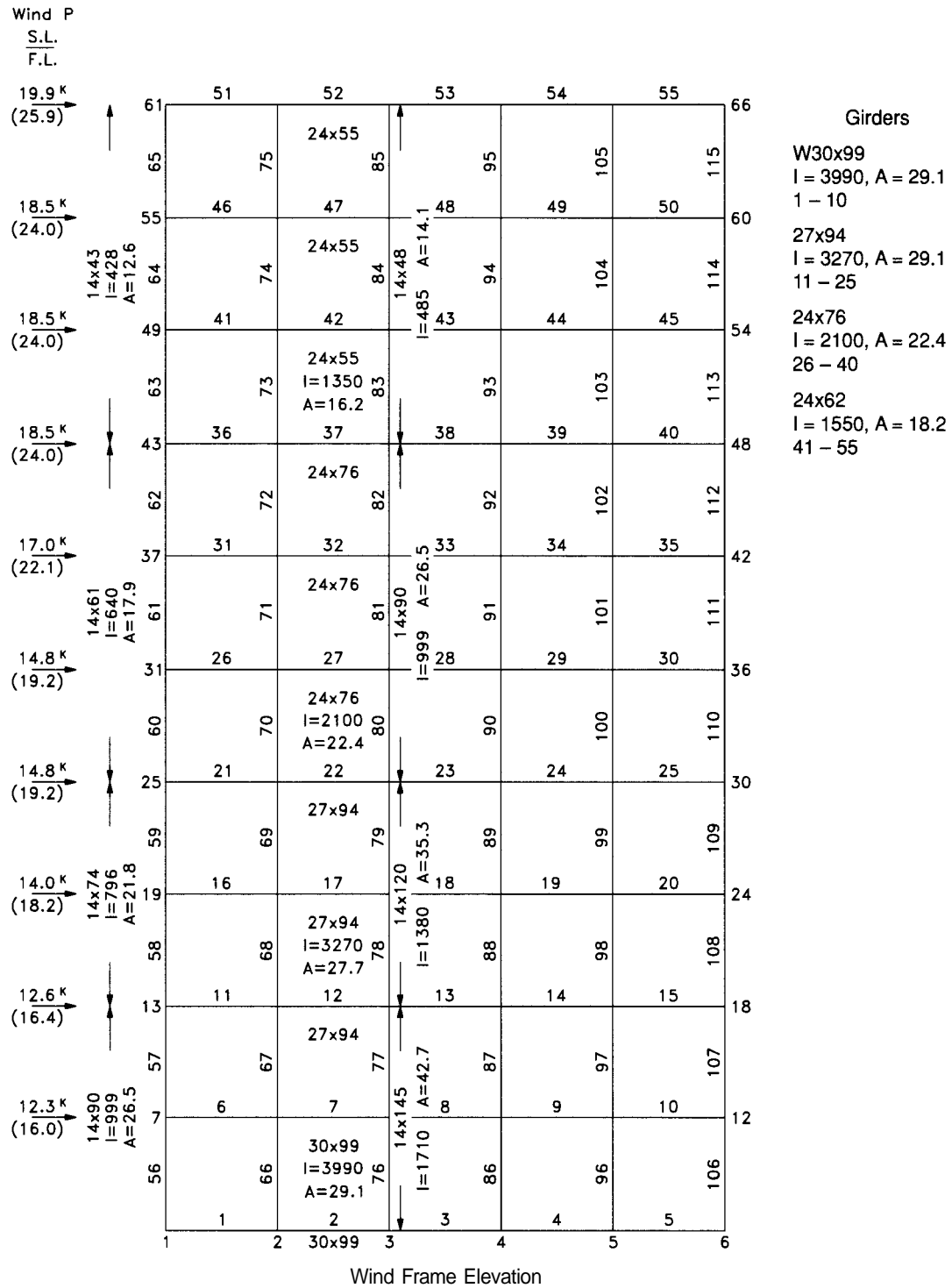Stiffness $= [2 \times$ Factor / (No Bays)] $\times$ 12
Magnifier $= 1$ / [1 $-$ (Sum P / Sum H) $\times$ DI]

UNBRACED FRAME DESIGN (cont.)

Column Trial Sections

Floor	Stiff.	Typ. Col. Avg.	Typ. Col. I	Typ. Col. Sect.	Corner Col. I	Corner Col. Sect.	Girder Stiff.	Girder I	Girder Sect.
Roof							16.47	494	—
9	32.94			14x48		14x43	24.12	724	24x55
8	31.78	37.27	466		280		39.43	1180	24x55
7	47.09						54.74	1642	24x76
6	62.40			14x90		14x61	69.43	2083	24x76
5	76.47	75.86	948		569		82.59	2478	24x76
4	88.72						94.84	2935	27x94
3	100.97	106.8	1335	14x120	801	14x74	106.76	3203	27x94
2	112.55						117.77	3533	27x94
1	122.98	128.1	1601	14x145	961	14x90	128.07	3842	30x99
	133.16								

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Wind Frame Elevation

Girders

W30x99
I = 3990, A = 29.1
1 – 10

27x94
I = 3270, A = 29.1
11 – 25

24x76
I = 2100, A = 22.4
26 – 40

24x62
I = 1550, A = 18.2
41 – 55

Wind P
S.L.
F.L.

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UNBRACED FRAME DESIGN (cont.)

Service Wind Load Deflections

Floor	Joint No.	Elev.	Δ	DI
10	61	125.0	3.63	0.0024
9	55	112.5	3.19	0.0024
8	49	100.0	2.89	0.0024
7	43	87.5	2.52	0.0024
6	37	75.0	2.20	0.0024
5	31	62.5	1.81	0.0024
4	25	50.0	1.41	0.0024
3	19	37.5	1.06	0.0024
2	13	25.0	0.68	0.0023
1	7	12.5	0.32	0.0021

Final Girder Design

Fl.	Prelim. Sect.	Mom. M_u	Mag.	Max. Mom.	Z	$\dfrac{M(12)}{\phi_b Z}$	Final Sect.
8	24×55	G 319 G 202 W 85	1.24	319 ~~313~~	134	31.7	W24×55 $F_y = 36$
5	24×76	G 321 G 204 W 161	1.23	~~321~~ 402	200	26.8	W24×76 $F_y = 36$
2	27×94	G 323 G 206 W 231	1.25	~~323~~ 495	278	23.7	W27×94 $F_y = 36$
1	30×99	G 326 G 207 W 251	1.26	~~326~~ 533	312	22.8	W30×99 $F_y = 36$

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UNBRACED FRAME DESIGN (cont.)

Final Design Column Loads

Mem. No. Sect.	Grav. Load		Wind + Gravity Loading						
	P_u	M_u	$P_{u\,grav}$	$P_{u\,wind}$	$M_{u\,grav}$	$M_{u\,wind}$	Mag.	$P_{u\,max}$	$M_{u\,max}$
56 W14×90	459	93	347	95	57	152	1.26	467	249
58 W14×74	370	90	277	63	56	115	1.25	356	200
60 W14×61	279	76	207	36	47	97	1.23	251	166
63 W14×43	141	90	100	10	57	58	1.23	112	123
76 W14×145	761	59	556		24	276	1.26	556	372
78 W14×120	603	66	440		27	224	1.25	440	307
80 W14×90	447	60	325		25	180	1.23	325	246
83 W14×48	222	65	155		27	89	1.23	155	136

Note: The first four columns (from the top) are corner columns and axial loads are increased under the influence of wind.

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UNBRACED FRAME DESIGN (cont.)

Final Column Design

Corner Column — 3rd Lift

$L = 12.5$ $K_x < 1.5$
$B_2 = 1.23$ (from page ??)

Gravity loads: $P_u = 279$ kip
$M_{ux\ grav} = 76$ kip-ft

Wind loads: $P_u = 207 + 36\ (1.23) = 251$ kip
$M_{ux\ grav} = 47$ kip-ft
$M_{ux\ wind} = 97$

Try W14x61 $L = 12.5$ ft $F_y = 36$ $r_x = 5.98$ $r_y = 2.45$ $K_x < 1.5$ $K_y = 1.0$

$L / r_x = 12.5\ (12) / 5.98 = 25.08$
$P_{ex} / A_G = 457.95$ (from pg. 6-131 LRFD Manual)
$P_{ex} = 17.9\ (457.95) = 8,197$
$\phi_c P_n = 457 - (457 - 428)\ (.25) = 450$ kip (from pg. 2-21)
$M_{nx} = 275 - (12.5 - 10.2)\ 2.30 = 266$ kip-ft (from pg. 3-15)

For gravity load

$$c_m = .6 - .4\ (26\ /\ 76) = .46 \qquad B_1 = \frac{.48}{1 - 279\ /\ 8,197} = \cancel{.50} > 1.0$$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}}\right) = \frac{279}{450} + \frac{8}{9}\left(\frac{1.0\ (76)}{266}\right) = .62 + .25 = .87 < 1.0$$

For wind load

$$c_m = .6 - .4\ (31\ /\ 50) = .35 \qquad B_1 = \frac{.35}{1 - 251\ /\ 8,197} = \cancel{.36} < 1.0$$

$$M_{ux} = B_1\ M_{u\ grav} + B_2\ M_{u\ wind} = 1\ (47) + 1.23\ (97) = 166$$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}}\right) = \frac{251}{450} + \frac{8}{9}\left(\frac{166}{266}\right) = .56 + .55 = 1.07 \qquad \textbf{n.g.}\quad \text{too high}$$

Use W14x74 $F_y = 36$ ksi or W14x61 $F_y = 50$ ksi.

Final Column Design

Spandrel Column — 1st Lift

$L = 12.5$ ft $K_x < 1.5$ $K_y = 1.0$ $B_2 = 1.25$

Under the influence of gravity load column can experience single curvature bending.
Therefore M_1/M_2 is negative (-1.0). Take $c_m = 1.0$.

Gravity loads: P_u $= 603$ kip
$M_{ux\ wind} = 66$ kip-ft

Wind loads: P_u $= 440$ kip
$M_{ux\ grav} = 27$ kip-ft
$M_{ux\ wind} = 224$ kip-ft

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UNBRACED FRAME DESIGN (cont.)

Try W14x120 $F_y = 36$ $r_x = 6.24$ $r_y = 3.74$ $K_x < 1.5$ $K_y = 1.0$

$$L / r_x = 12.5 (12) / 6.24 = 24.0$$
$$KL / r_y = 12.5 (12) / 3.74 = 40.1$$
$$P_{ex} = 496.91 (35.3) = 17{,}541 \text{ kip}$$
$$\phi_c P_n = 992 \text{ kip}$$
$$M_{nx} = 502 \text{ kip-ft}$$

For gravity load

$$c_m = 1.0 \qquad B_1 = \frac{1}{1 - 603 / 17{,}541} = 1.04$$

$$\frac{603}{992} + \frac{8}{9}\left(\frac{(1.04)\ 66}{502}\right) = .61 + .12 = .73 < 1.0$$

For wind load

$$c_m = 1.0 \qquad B_1 = \frac{1}{1 - 440 / 17{,}541} = 1.03$$

$$\frac{440}{992} + \frac{8}{9}\left(\frac{(1.03)\ 27 + (1.25)\ 224}{502}\right) = .44 + .61 = 1.05$$

Try W14x120 $F_y = 50$ ksi or W14x132 $F_y = 36$ ksi.

Note: If a W14x132 column is selected, the drift will be reduced. The mill material costs are close; 120 (.28) = $33.60 / ft vs. 132 (.25) = $33.00 / ft.

Try W14x132 $F_y = 36$ ksi.

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