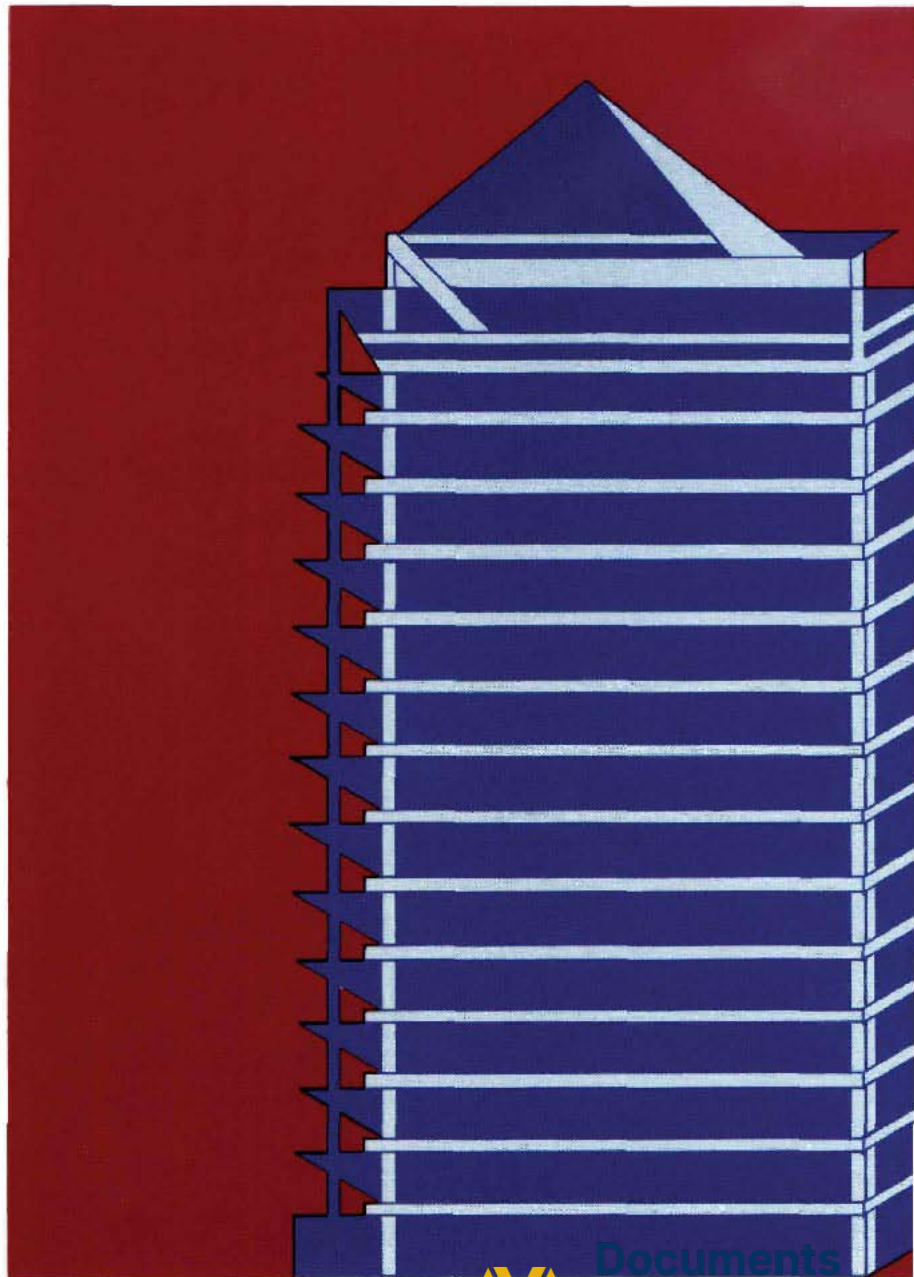




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Low-and Medium-Rise Steel Buildings



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Low- and Medium-Rise Steel Buildings

Design Guide for Low- and Medium-Rise Steel Buildings

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TABLE OF CONTENTS

BASIC DESIGN RULES FOR ECONOMY	1	Girder Beam Design	10
LIVE LOAD AND BAY SIZE SELECTION	1	Composite Joist Systems	10
Live Load Selection	2	Floor Vibration	11
Bay Size Selection.....	2	WIND LOAD DESIGN	12
COMPOSITE FLOORS	5	Drift Limits	12
Allowable Stress (ASD) and Load		"K" Bracing Frame	13
Resistance Factor Design (LRFD)	5	Unbraced Frame Design	15
Economy with LRFD	6	Special Wind Frames	17
Floor Load Capacity Enhancement.....	6	APPENDICES	23
Shored vs. Unshored Construction.....	7	LRFD Composite Beam Design.....	23
Serviceability Considerations	8	Composite Beam Load Capacity Enhancement	25
Underfloor Duct Systems	8	Composite Beam Long Term Deflection	29
OPEN WEB JOIST FLOOR SYSTEMS	10	Steel Joist Typical Bay	31
Joist Size and Spacing	10	K-Frame Bracing Optimization	33
		Unbraced Frame Design	36

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.

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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 non-destructive 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

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).

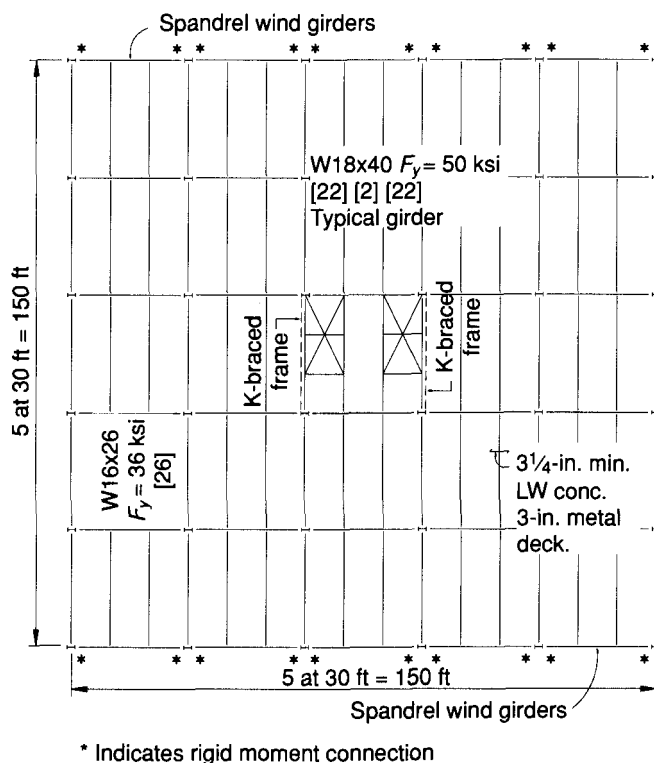


Fig. 1. Typical office building floor plan.

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

Table 2. Framing Cost Comparison—ASD			
Section	Loading		
	50 + 20	80 + 20	100
Filler BM	$W16 \times 26 F_y = 36$	$W16 \times 26 F_y = 50$	$W16 \times 26 F_y = 50$
Studs	26	26	24
Cost	\$702	\$772	\$763
Girder	$W21 \times 44 F_y = 50$	$W21 \times 50 F_y = 50$	$W21 \times 44 F_y = 50$
Studs	44	54	56
Cost	\$436	\$501	\$454
Ave. col. wt/story	1027	1079	1027
Cost	\$288	\$302	\$288
Total cost	\$1426	\$1575	\$1495
Relative cost	1.0	1.10	1.06
Premium	—	\$.17	\$.09

Table 3. Framing Cost Comparison—LRFD			
Section	Loading		
	50 + 20	80 + 20	100
Filler BM	$W16 \times 26 F_y = 36$	$W16 \times 26 F_y = 36$	$W16 \times 26 F_y = 36$
Studs	12	22	26
Cost	\$639	\$684	\$702
Girder	$W18 \times 35 F_y = 50$	$W18 \times 40 F_y = 50$	$W18 \times 35 F_y = 50$
Studs	48	62	54
Cost	\$366	\$429	\$375
Ave. col. wt/story	913	1036	965
Cost	\$256	\$290	\$270
Total cost	\$1261	\$1403	\$1347
Relative cost	\$.88	\$.98	\$.94
Premium*	(\$.18)	(\$.03)	(\$.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-

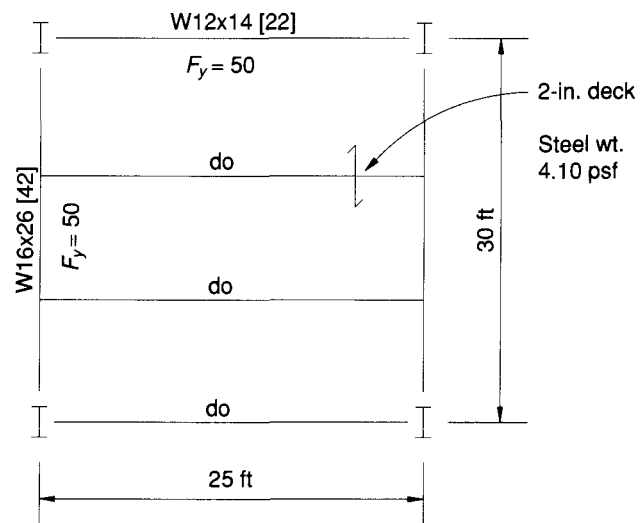


Fig. 2. 25 ft x 25 ft bay.

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

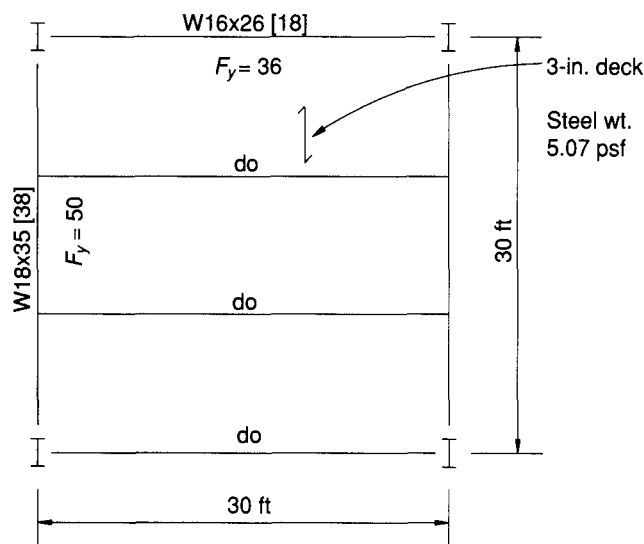


Fig. 3. 30 ft x 30 ft bay.

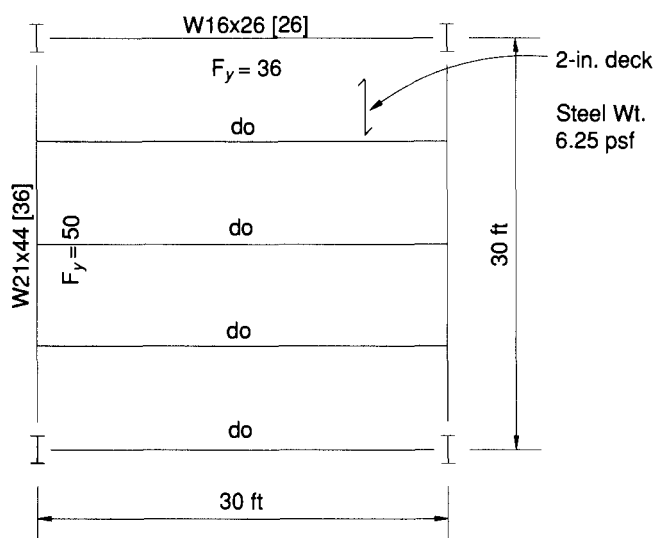


Fig. 4. Alternate 30 ft x 30 ft bay.

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 desirable 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

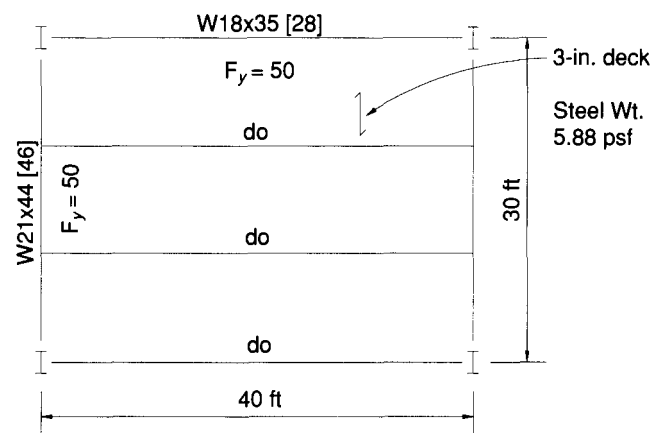


Fig. 5. 30 ft x 40 ft bay.

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%
W16x31, $F_y = 36$ 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

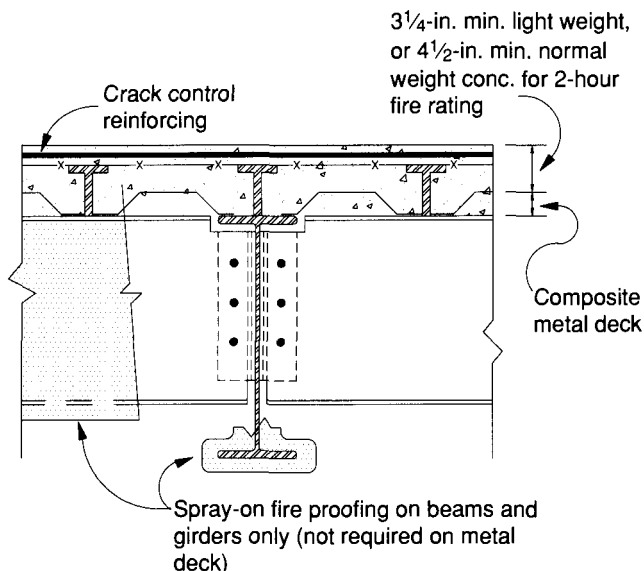


Fig. 6. Composite metal deck floor system section.

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-

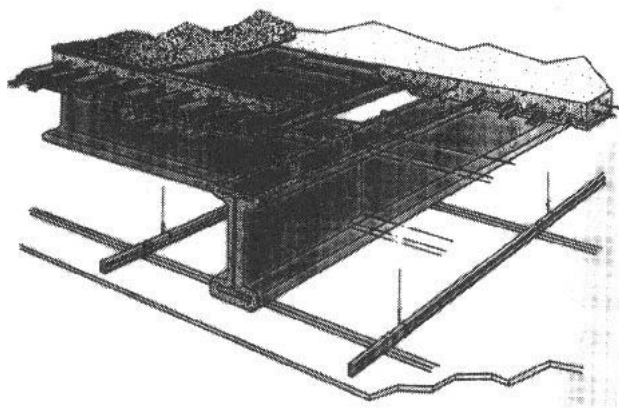
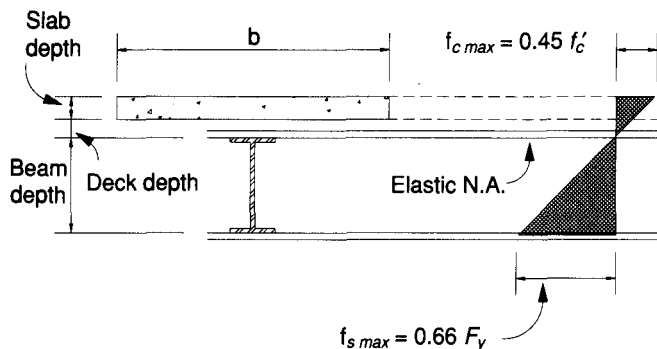


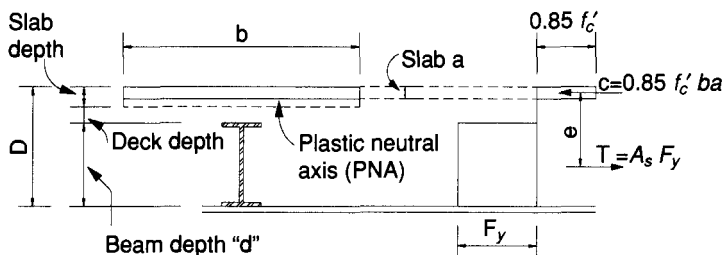
Fig. 7. Cellular composite steel deck system with in-floor electrical and communication distribution.

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.



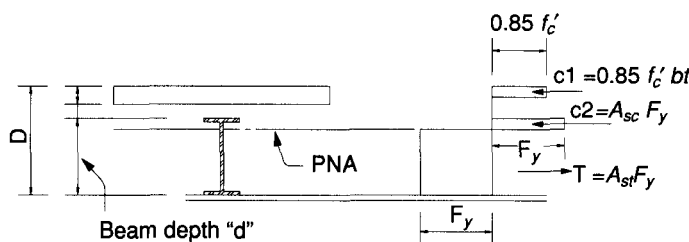
$$e = d/2 + Y_2$$

$$Y_2 = \text{Deck } D + \text{Slab } D - a/2$$

$$\phi M_n = \phi C e = \phi T e$$

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

30 ft x 40 ft Bay Live load = 250 psf			
	W24x55 $F_y = 50$	W24x68 $F_y = 50$	
W30x108 $F_y = 50$	[46]	[64]	
	do	do	
	[2]	do	do
	do	do	
W33x141 $F_y = 50$	[46]	[69]	
	do	do	
	[2]	do	do
	do	do	

Mill material wt. = 9,840 lbs.
286 $\frac{3}{4}$ -in. dia. studs

Costs:
Mill mat. 9,840 (.28) = 2755
Studs 286 (1.50) = 429
\$3,184

LRFD Savings
(3976 - 3184) / (30 x 40)
= \$.66 per sq. ft

(a) LRFD

Mill material wt. = 12,390 lbs.
338 $\frac{3}{4}$ -in. dia. studs

Costs:
Mill mat. 12,390 (.28) = 3,469
Studs 338 (1.50) = 507
\$3,976

(b) ASD

Fig. 10. Typical warehouse bay plans.

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 $\frac{3}{4}$ -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 $\frac{3}{4}$ -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 $\frac{3}{4}$ -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 $\frac{3}{4}$ -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.

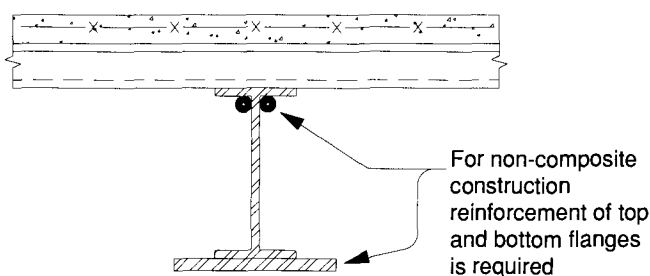


Fig. 11. Simple beam load capacity enhancement.

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-

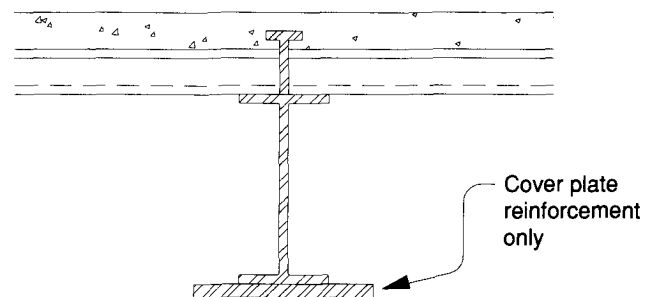


Fig. 12. Composite beam load capacity enhancement.

control 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 $1/5$ 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

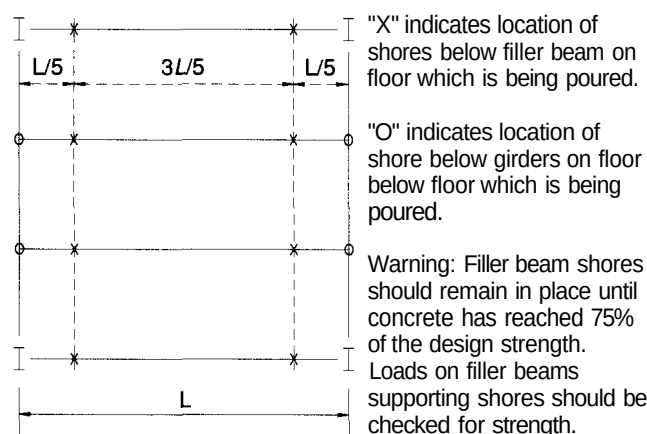


Fig. 13. Shoring plan.

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 $3/4$ -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.

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

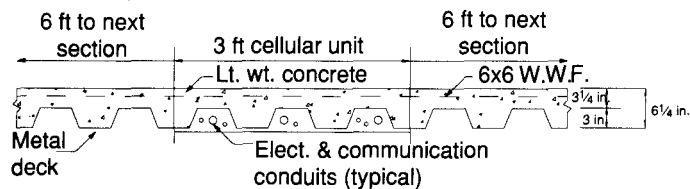
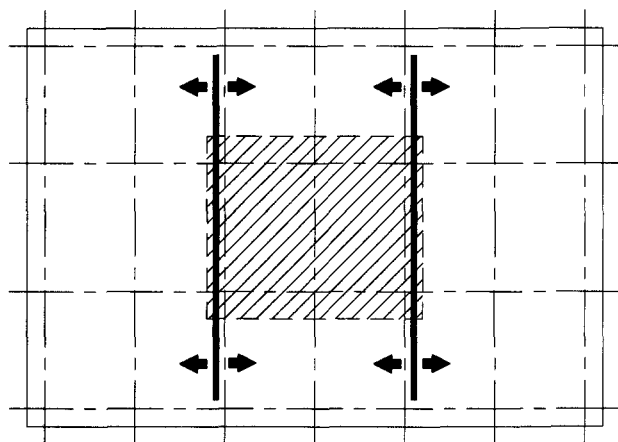
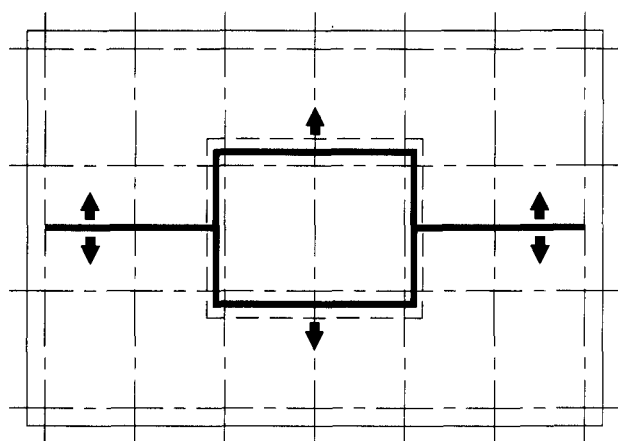


Fig. 14. Blended metal deck floor.



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 3 1/4-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 2 1/2-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.

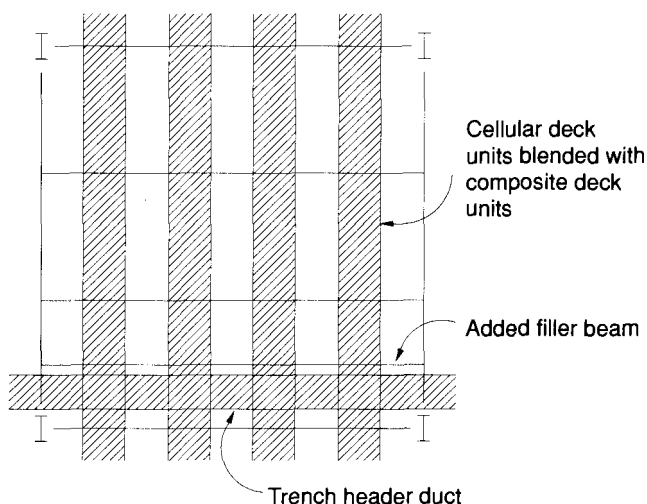


Fig. 16. Added filler beam at trench header.

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

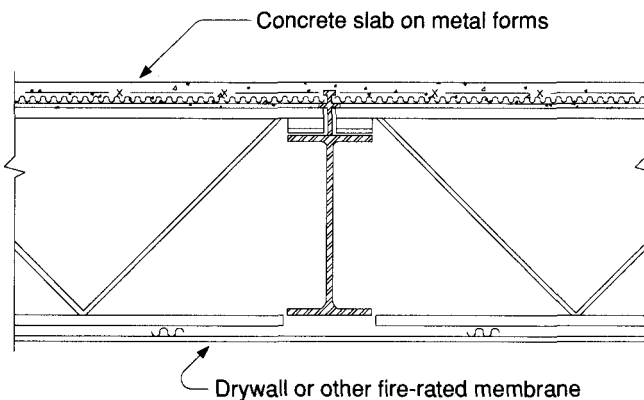
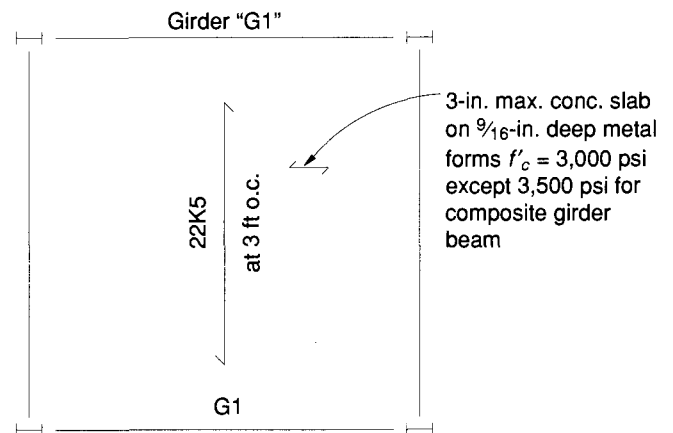


Fig. 17. Fire-rated floor ceiling assembly.

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

$$\begin{aligned} \text{LL} &= 50 \times 1.6 = 80 \\ \text{Part} &= 20 \\ \text{DL} &= 47 \end{aligned} \quad \left. \begin{array}{l} \\ \\ \end{array} \right\} \times 1.2 = 80$$

$$\text{Tot.} = 117 \times (1.37) = 160 \text{ psf}$$

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.

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):

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

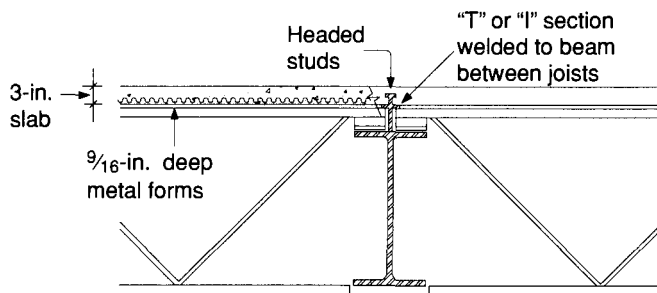


Fig. 19. Composite beam with open web steel joists.

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

$$D = 35A_of + 2.5$$

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

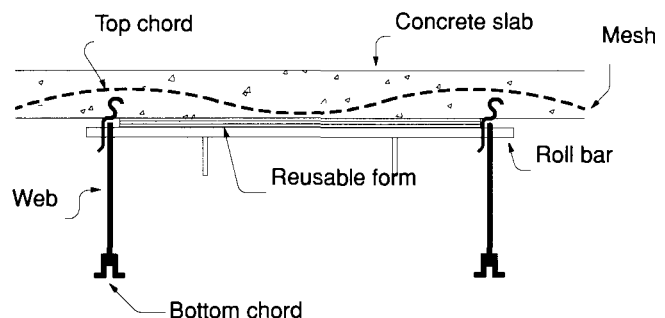


Fig. 20. Composite steel joist floor system.