Multistory Rigid Frames with Composite Girders Under Gravity and Lateral Forces

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Much work has been done in the area of composite design using the strength of the concrete floor slab and steel girders acting together by means of shear connectors. The major portion of the work done to date has pertained to beams or girders, simply supported or continuous, acted upon by vertical loads. Little information is found in the literature on the design and use of composite girders in multistory rigid frames, when horizontal loads govern the design of the girders.[5,7,11]

The use of composite design of girders in multistory rigid frames subjected to severe horizontal loads can lead to substantial material savings.[9] Using recent research involving the response of composite sections in positive moment regions adjacent to columns, stiffness factors can be developed for use in analysis and subsequent design.

This paper compares preliminary designs, with and without composite girders, of a 12-story, two-bay, asymmetrical rigid frame subjected to wind loading. The effects of governing design parameters, such as varying story height, longitudinal steel in the negative moment region, and the length of composite partial slab width adjacent to columns, are discussed and a recommended procedure for designing with composite girders is presented.

STIFFNESS CONSIDERATIONS

Recent research[5,6,7] shows that four different regions of effective bending stiffness in unbraced multistory frames subjected to both vertical and horizontal loading must be considered. These regions are illustrated in Fig. 1. Region 1 is that segment of the girder subjected to positive bending moment, where the full effective slab width may be utilized in resisting the flexural moment. This region is bounded by the inflection point of the moment curve and the end of Region 2.

Region 2 is also shown between Region 1 and the leeward side of an interior column, or between Region 1 and an exterior windward column, where a reduced effective slab width must be used. The effective length of Region 2 is taken from the center line of the column to a point at which a 45° line intersects the effective slab width bound by Region 1, as shown in Fig. 2. (In the composite frame discussed in this paper, the largest girder flange width, b_f, was 9 in.; the length of Region 2 was computed as 36.5 in.)

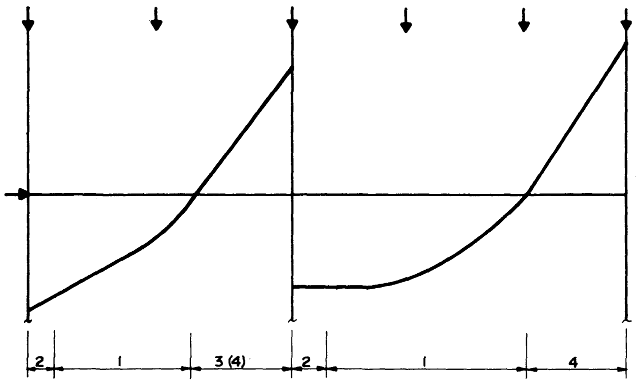

Fig. 1. Girder moment regions

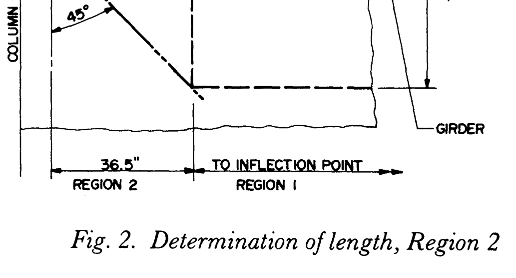

Fig. 2. Determination of length, Region 2

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The stiffness of the composite girders in Region 2 actually is close to that based on the full effective slab width used in Region 1. However, the effective slab width assumed for Region 2 is conservatively taken as the width of the column flange.

Region 3 is the region of negative moment occurring at an interior column. In the basic composite frame analysis that follows, no distinction is made between Regions 3 and 4, and both the analysis and design are based on the bare steel girder properties between the point of inflection and the column. It is the addition of negative slab steel to the girder moment of inertia, provided there is continuity in the slab, that distinguishes Region 3 from Region 4. Region 4, therefore, is a negative moment region occurring between Region 1 and the leeward exterior column.

In the basic composite analysis, Regions 3 and 4 are assumed to furnish the same stiffness, i.e., the floor is assumed noncontinuous at the interior column, and the floor steel is not included in the moment of inertia of the bare girder section. Thus, in this case, which will be called the basic composite frame, only Regions 1, 2, and 4 are considered, Region 3 at the interior column being conservatively assumed to act like Region 4.

DESIGN OF THE COMPOSITE FRAME

Figure 3 shows the frame to be analyzed. Joint and member designations are shown in Fig. 4.

The ICES STRUDL-II Computer Program[8] was used in the analysis phase of this work. Any other multistory analysis computer program could be used; however, it must be capable of producing internal forces at intermediate locations along the girder length and accept girders with variable moments of inertia.

The applied vertical loading is shown in Fig. 5. A floor design live load of 50 psf is assumed for all floors. The axial column live load is reduced in accordance with Sect. 1203 of the Standard Building Code.[10] Wind loads are taken in accordance with the Standard Building Code, using wind pressures specified in Table 1205.1 for a wind speed of 110 mph, and a shape factor of 1.3. The equivalent lateral loads obtained for each story level are given in Fig. 3. These loads, when multiplied by the 20-ft frame spacing used in this study, give the total lateral load at each joint.

Internal forces are developed separately for the following loading cases: dead load, wind load, dead load plus live load, 3/4 dead load plus 3/4 wind load, and 3/4 live load plus 3/4 wind load. Wind load must be applied both from the left and then from the right in all these load combinations. Maximum column design axial loads and moments as well as maximum girder moments are obtained for the load combinations.

In order to establish a base of reference, the frame is first analyzed as a bare frame, using the same load combinations. The internal forces from this basic bare frame analysis are then used to obtain input properties for the composite frame. After the composite analysis run, it is necessary to check for movement of inflection points to

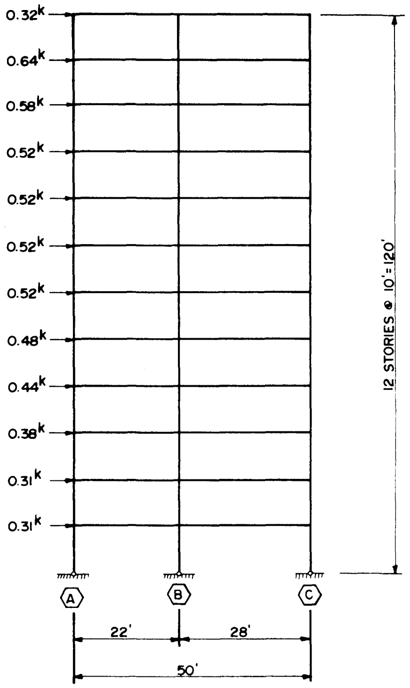

Fig. 3. Frame elevation with lateral loads

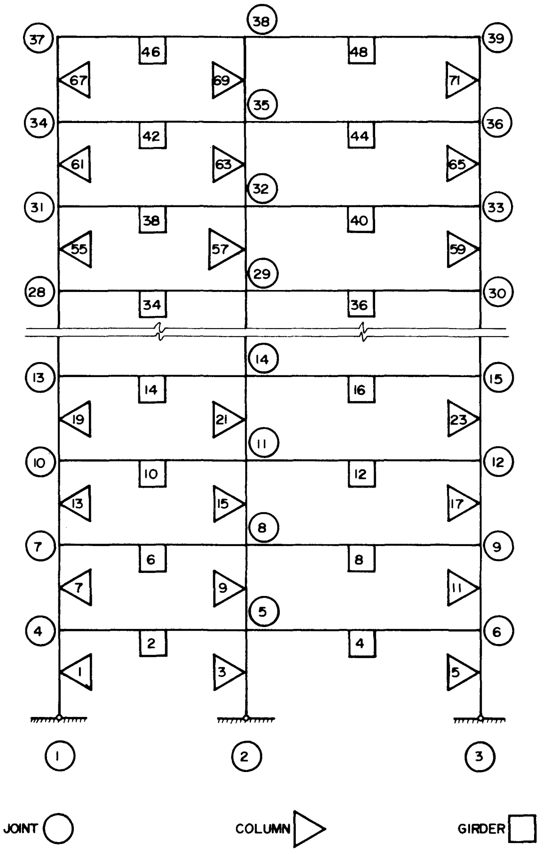

Fig. 4. Joint and member designation

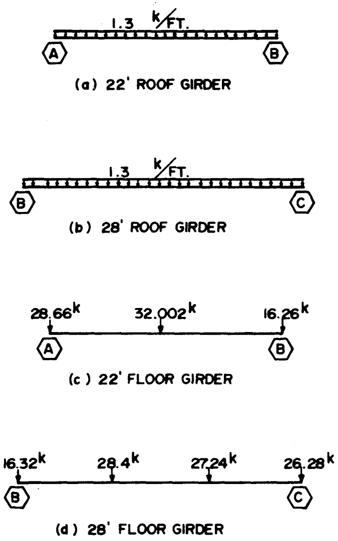

Fig. 5. Typical girder loads

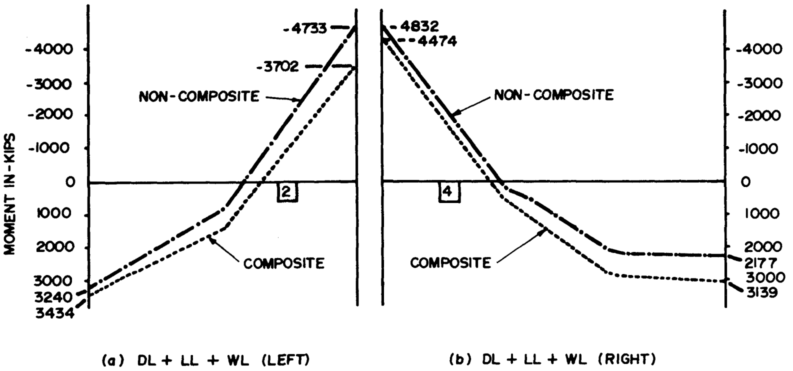

Fig. 6. Girders 2 and 4, design moments, noncomposite and composite frames

verify that they do not appreciably change their position from those of the bare frame. Next, all columns and beams are checked against internal forces for adequacy or a possible further reduction in size. Final internal forces for the noncomposite as well as the composite frame obtained from the analysis are shown in Table 1. References 1, 2, and 12 were used in selecting column sizes for the bare and the basic composite frame, using $K_xL_x/K_yL_y = 1.0$ (see Appendix) for both frames. Thus, this initial design of the noncomposite and composite frame does provide a valid method of estimating savings in frame weight. Table 2 lists final column sizes for the bare and the composite frame.

Girder sizes for both the bare and the composite frame are selected on the basis of the maximum design moment, positive or negative, of the load combinations. It is assumed that the floor beams framing into the girders provide sufficient lateral support to the girders. Internal forces were obtained at close intervals, and Ref. 3 was used in selecting the final girder size. All girders are initially designed on the basis of the maximum negative moment; then, when the maximum positive moment is found to be greater than the maximum negative moment, the composite section is checked against the positive moment. Compression flange bracing is not a concern in Regions 1 and 2, where the composite section is active. Furthermore, no bracing is required in Regions 3 and 4, so long as the inflection point falls within the L_c or L_u limits. Should the distance from the columns to the inflection point be greater than L_u, a

Table 1. Noncomposite and Composite Design Forces

Column No.	Noncomposite Axial* (kips)	Composite Axial* (kips)	Noncomposite Moment (kip-in.)	Composite Moment (kip-in.)	22-Ft Girder No.	Noncomposite Moment (kip-in.)	Composite Moment (kip-in.)	28-Ft Girder No.	Noncomposite Moment (kip-in.)	Composite Moment (kip-in.)
1 & 7	433	418	−2796	−2507	2	−4733	−3702	4	−4832	−4533
3 & 9	522	548	−4901	−4866	6	−3896	−2474	8	−3540	−3547
5 & 11	482	500	+3208	+3086	10	−2952	−2166	12	−3530	−3440
13 & 19	328	321	−1687	−1154	14	−2630	−1990	16	−3566	−3154
15 & 21	447	461	−2547	−2336	18	−2336	−1872	20	−3503	−3116
17 & 23	385	396	+1697	+1743	22	−2243	−1550	24	−3145	−3047
25 & 31	244	240	−1111	−995	26	−1724	−1371	28	−3036	−2743
27 & 33	360	367	−2174	−2045	30	−1608	−1298	32	−2794	−2421
29 & 35	295	301	+1759	+1549	34	−1468	−1200	36	−2431	−2102
37 & 43	173	173	−1025	−736	38	−1200	−969	40	−2265	−1906
39 & 45	280	279	−1659	−1517	42	−1279	−935	44	−2040	−1629
41 & 47	215	218	+1521	+1394	46	−733	−569	48	−1501	−842
49 & 55	109	111	−734	−631						
51 & 57	193	191	−1230	−1144						
53 & 59	138	138	+1263	+1122						
61 & 67	43	44	−348	−295						
63 & 69	88	86	−730	−749						
65 & 71	55	56	+929	+802						

* Reduced per Ref. 9.

Table 2. Member Selection, Columns

Column No.	Noncomposite Frame		Composite Frame		Comp. Frame with 0.2 (−) Steel	
	Shape	Wt. (lb)	Shape	Wt. (lb)	Shape	Wt. (lb)
1 & 7	W14X142	2840	W14X136	2720	W14X136	2720
3 & 9	W14X219	4380	W14X219	4380	W14X219	4380
5 & 11	W14X167	3340	W14X167	3340	W14X158	3160
13 & 19	W14X103	2060	W14X84	1680	W14X84	1680
15 & 21	W14X142	2840	W14X136	2720	W14X142	2840
17 & 23	W14X111	2220	W14X111	2220	W14X111	2220
25 & 31	W14X74	1480	W14X74	1480	W14X68	1360
27 & 33	W14X119	2380	W14X119	2380	W14X119	2380
29 & 35	W14X95	1900	W14X95	1900	W14X95	1900
37 & 43	W14X61	1220	W14X61	1220	W14X61	1220
39 & 45	W14X95	1900	W14X87	1740	W14X87	1740
41 & 47	W14X78	1560	W14X74	1480	W14X74	1480
49 & 55	W14X43	860	W14X43	860	W14X43	860
51 & 57	W14X68	1360	W14X68	1360	W14X68	1360
53 & 59	W14X61	1220	W14X61	1220	W14X61	1220
*61 & 67	W 8X28	560	W 8X24	480	W 8X24	480
63 & 69	W14X43	860	W14X43	860	W14X43	860
65 & 71	W14X43	860	W12X40	800	W12X40	800
Total		33,840		32,840		32,660

* A W12 or W14 section might be desirable from a fabrication standpoint.

brace to the compression flange is necessary. Table 3 lists final girder sizes for the noncomposite and the basic composite frame. Positive moments are not listed in these tables, but they are generally increased due to the large increase in the moment of inertia in Region 2 over the noncomposite girder moment of inertia in that same region. In Fig. 6, design moments are plotted for Girders 2 and 4, for both the noncomposite and the composite frame design. It should be noted that it is seldom necessary to check a girder size for positive bending where the girder is composite, since the negative moment usually exceeds the positive moment, and therefore governs the selection. Table 3 shows that the total reduction in frame weight was of the order of 8.7%.

The number of shear connector studs required is based on the smaller shear force obtained from one of the following two formulas:[2]

$$V_h = 0.85 f'_c A_c / 2 \qquad \text{AISC Formula (1.11-3)}$$

or

$$V_h = A_s F_y / 2 \qquad \text{AISC Formula (1.11-4)}$$

Partial composite action could be investigated here, and this might possibly lead to some savings in connectors. However, this would change the moment of inertia used in the region of positive moment and consequently would affect the stiffness of the girders, thus requiring a reanalysis of the structure. Clearly, an optimum design exists between full and partial composite girder design, which will depend on the frame and the cost relationship of the connectors versus the increase in frame weight. This discussion will deal only with the development of those advantages gained by using full composite design.

Table 3. Member Selection, Girders

22-Ft Girder	Noncomposite Frame		Composite Frame		Comp. frame with 0.2 (−) Steel	
	Shape	Wt. (lb)	Shape	Wt. (lb)	Shape	Wt. (lb)
2	W27X94	2068	W24X76	1672	W24X68	1496
6	W27X84	1848	W21X62	1364	W24X55	1210
10	W21X68	1496	W18X55	1210	W21X49	1078
14	W21X62	1364	W21X49	1078	W21X44	968
18	W21X55	1210	W18X45	990	W21X44	968
22	W21X55	1210	W21X44	968	W18X40	880
26	W18X45	990	W16X40	880	W18X35	770
30	W18X45	990	W16X40	880	W18X35	770
34	W18X40	880	W16X36	792	W18X35	770
38	W18X35	770	W14X30	660	W16X26	572
42	W18X35	770	W14X30	660	W16X26	572
46	W14X26	572	W12X22	484	W12X19	418
Total		14,168		11,638		10,472
28-Ft Girder						
4	W27X84	2352	W24X84	2352	W24X84	2352
8	W24X68	1904	W24X68	1904	W24X68	1904
12	W24X68	1904	W24X68	1904	W24X68	1904
16	W24X68	1904	W21X68	1904	W24X61	1708
20	W24X68	1904	W21X68	1904	W24X61	1708
24	W24X68	1904	W21X68	1904	W24X61	1708
28	W24X68	1904	W24X61	1708	W24X55	1540
32	W21X62	1736	W21X55	1540	W24X55	1540
36	W21X55	1540	W21X49	1372	W21X49	1372
40	W21X55	1540	W21X44	968	W21X44	968
44	W21X55	1540	W18X40	880	W18X40	880
48	W16X45	1260	W14X26	572	W16X26	572
Total		21,392		18,912		18,156
Grand Total (Incl. Table 2)		69,400		63,390		61,288

The generally accepted method for determining the number of shear connectors is to use AISC Formulas (1.11-3) and (1.11-4) and, dividing the total horizontal shear, V_h, by the shear connector value, q, to find the number of shear connectors that must be equally spaced between the point of maximum positive moment and the point of zero moment on the girder. Since the horizontal load may reverse its direction, positive moments may at some time occur anywhere in the girders. Thus, it seems logical to space the shear connectors equally over the entire length of the girders, and to base their number on the length of the maximum positive moment region obtained from the various load combinations with wind acting from either direction. When positive moment occurs at an interior column, a concrete wedge is formed in front of the column face.[6] In order to be assured of the anchorage of this wedge to the beam flange, connectors are placed 2 in. from the end point and on 4-in. centers for the first three connectors. Studs, ¾-in. in diameter and 3 in. long, resulting in 1 in. of cover, are selected. Obviously, the shear connectors are an added expense and must be included in the frame cost for a valid comparison with the basic frame. A typical connector layout is shown in Fig. 7. In the event that vertical loads control the girder design, with inflection points near each end, the studs are positioned between these two inflection points.

Since in Allowable Stress Design an increase of ⅓ in design stress is permitted if wind or seismic forces are used in the design, a check of the load combinations not involving these forces is necessary. It must be remembered that this increase is only permitted when the girders are temporarily shored during construction.[2] The design of the upper-story girders is found to be governed by vertical loads, but lower-story girders are generally controlled by some combination involving the horizontal wind load. However, primary factors affecting the design forces in the girders are the loads and the frame geometry. After composite girders are selected, as described previously, these girder properties have to be checked against a final vertical load analysis.

Drift is an important factor in multistory frame design. Although it is difficult to accurately calculate drift for a frame in a completed structure, tolerance levels have been suggested for rigid frames.[4,11] Table 4 lists the maximum drift, in inches, obtained for joints 37 and 39, with lateral loads acting to the left against the exterior wall of the 28-ft bay. Drift for the last and next-to-last frame member selections are shown. The weight of the noncomposite girder frame was decreased and that of the composite girder frame increased in the final computer run. This resulted in almost identical drifts for the two frames. Drift is directly related to moment of inertia and only incidently to weight. In several girders the composite moment of inertia was slightly reduced by using a shallower depth but heavier steel section, to avoid the need for lateral bracing in a region of negative moment and, at the same time, to hold the increase in frame weight to a minimum. Lateral bracing may sometimes be more economical than changing a section; however, in this example selections were made on the basis of composite girders not being braced in the region of negative moment. In essence, if drift becomes a significant factor in frame design, deeper girders offer better control of this factor than do shallow ones, but their use must be weighed against the possibility of the need to provide lateral bracing. The drift ratio for the noncomposite girder frame is 0.0027, compared to 0.0028 for the composite frame.

Table 4. Wind Drift, Joints 37 and 39

	Joint	Noncomposite Frame	Composite Frame
Preliminary computer run	37 39	3.6 in. 3.6 in.	4.1 in. 4.1 in.
Final computer run	37 39	3.9 in. 3.9 in.	4.0 in. 4.0 in.

STUDY OF DESIGN VARIABLES

First, the effectiveness of the slab in Region 2 was studied. The length of Region 2 is based on the conservative assumption that the effective slab width increases linearly from the column flange width to the full effective slab width, as shown in Fig. 4. For girder properties used in the stiffness analysis, the full effective slab width beginning at the column flange could probably be utilized.[6] In evaluating the resistance of the section to moment, the effective slab width is kept flush with the column flange width over the entire length of Region 2. Therefore, it was deemed desirable to study the effect of a change in length of Region

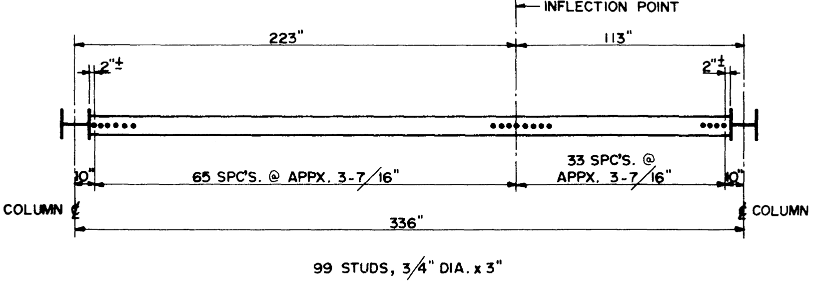

Fig. 7. Shear connector spacing, Girder 4

Table 5. Effect of Changes in Length of Region 2 on Column Forces and Moments

Columns	Axial Force (kips)			Moment (kip-ft)			% Change in Moment (36.5 to 60 in.)
	Region 2: 0 in.	Region 2: 36.5 in.	Region 2: 60 in.	Region 2: 0 in.	Region 2: 36.5 in.	Region 2: 60 in.	
1 & 7	264	267	267	211	204	203	Reduced
3 & 9	655	650	650	367	369	369	—
5 & 11	574	577	577	253	257	259	1
13 & 19	408	409	409	33	33	33	—
15 & 21	532	529	529	156	157	157	—
17 & 23	457	459	459	141	145	147	1
25 & 31	320	321	321	32	31	31	—
27 & 33	417	416	416	128	129	130	1
29 & 35	350	350	350	126	129	131	2
37 & 43	232	232	233	33	31	31	—
39 & 45	305	305	305	102	100	99	Reduced
41 & 47	245	246	246	114	116	118	2
49 & 55	144	144	144	29	28	28	—
51 & 57	269	269	269	36	33	33	—
53 & 59	146	146	146	93	93	94	1
61 & 67	56	56	56	16	19	19	—
63 & 69	114	114	114	29	30	30	—
65 & 71	55	56	56	65	66	67	2

2. Computer analysis runs for a length of zero inches (full effective slab width all the way to the column) and for a length of 60 in. were made, while keeping all other variables constant. The resulting internal forces are shown in Tables 5 and 6. All increases in internal forces were less than 2%, and it may be concluded that a change in the assumed length of Region 2 from 36.5 to 60 in. will not appreciably affect the final member sizes. More research is needed in this area to permit a greater effective slab width, a shorter length of Region 2, or perhaps both.

The AISC Specification[2] allows the use of longitudinal reinforcing steel acting compositely with the steel girder in the negative moment region of continuous composite members. In this work, one computer analysis run was made using 0.20 sq in. per ft of effective slab width in Region 3. The slab was assumed to be continuous at the interior column, and the effective slab width was reduced by the width of the column flange. In a second analysis run, the reinforcing steel area was taken as 0.40 sq in. per ft of effective slab width. Typical geometry and effective section properties are shown in Fig. 8 for girders **2**, **8**, and **12**. Each steel girder was reduced by one shape weight for this analysis, and new moments of inertia were calculated which included reinforcing steel in the region of negative moment. New internal forces are obtained from these analysis runs which are based on the new trial section properties. Member selection, as given in Tables 2 and 3, shows that the columns were generally not affected enough by the new girder properties to change their sizes from those of the basic composite frame. However, six girders changed: 22-ft girders **2**, **14**, **38**, **42**, **46**, and 28-ft girder **16**. The forces shown for comparison are the result of the wind load acting from the left and are not necessarily those used in the design selection.

Table 6. Effect of Changes in Length of Region 2 on Girder Moments

22-Ft Girder	Moment (kip-in.)			% Change in Moment (36.5 to 60 in.)
	Region 2: 0 in.	Region 2: 36.5 in.	Region 2: 60 in:	
2	−3604	−3702	−3773	2
6	−2421	−2474	−2515	2
10	−2135	−2166	−2200	2
14	−1960	−1957	−1991	2
18	−1854	−1882	−1914	2
22	−1504	−1525	−1546	1
26	−1307	−1341	−1360	1
30	−1276	−1298	−1313	1
34	−1234	−1201	−1213	1
38	−965	−969	−970	—
42	−954	−924	−925	—
46	−567	−569	−570	—
28-Ft Girder				
4	−4419	−4533	−4588	1
8	−3395	−3497	−3545	1
12	−3351	−3441	−3490	1
16	−3036	−3153	−3201	2
20	−2914	−2989	−3033	1
24	−2858	−2916	−2952	1
28	−2665	−2684	−2703	1
32	−2402	−2394	−2404	—
36	−2104	−2102	−2107	—
40	−1766	−1772	−1776	—
44	−1635	−1629	−1630	—
48	−846	−842	−842	—

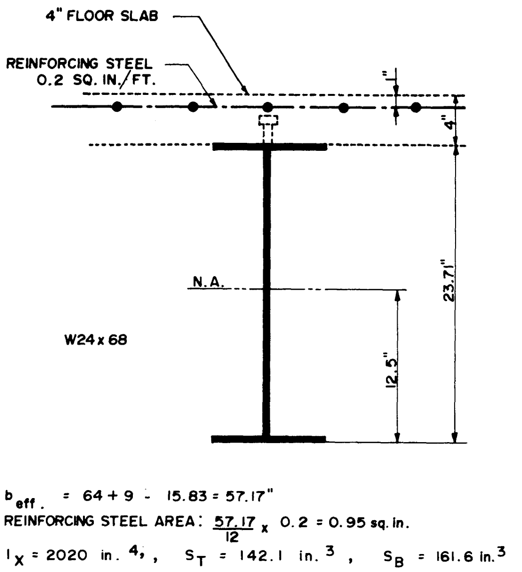

Fig. 8. Properties with negative steel, Girders 2, 8 and 12

The following observations can be made by referring to Tables 7 and 8. First, the moments at the interior column for the lower eight stories tended to increase slightly with the inclusion of reinforcing steel to the moment of inertia in the region of negative moment. The change in the girder moments was more significant; the moments of the 22-ft girders increased slightly, whereas those in the 28-ft girders decreased. Tables 2 and 3 indicate a savings in frame weight of approximately 3.3%, realized by using 0.2 sq in. per ft of reinforcing steel in negative moment regions adjacent to the interior column. Although definite conclusions cannot be drawn from the analysis of only one frame, it is obvious that the savings in frame weight between the noncomposite and the basic composite frame were much more significant than those realized between the basic composite frame and the composite frame having a nominal amount of negative moment steel included in the girder section properties. It would also appear that additional reinforcing steel placed in the slab solely for the purpose of reducing frame weight will yield only a small decrease in weight, the increase in moment of inertia generally being not sufficient to decrease the girder sizes of the frame. It is possible, however, that the use of negative moment steel could have a more pronounced effect in a frame having a number of interior bays. The effect of reinforcing steel on frame drift was even less significant. For example, the drift for joint 39 decreased from 4.066 to 3.961 in. for 2.0 sq in. per ft of negative steel, representing a decrease of 2.6%.

RECOMMENDED PROCEDURE FOR DESIGN OF FRAME WITH COMPOSITE GIRDERS

After securing the necessary data, the design of the rigid frame can be accomplished as follows:

Step 1:

Analyze the non-composite frame with tentatively selected column and girder sizes for all combinations of loading which must be considered. This analysis will yield moment

Table 7. Effect of Negative Steel on Column Forces and Moments

	Axial Force (kips)			Moment (kip-in.)		
Column	0.0 sq in./ft	0.2 sq in./ft	0.4 sq in./ft	0.0 sq in./ft	0.2 sq in./ft	0.4 sq in./ft
1	267	262	258	2460	2457	2455
3	650	659	666	4427	4451	4473
5	577	573	570	3086	3063	3044
13	409	407	405	391	385	380
15	529	536	541	1882	1904	1922
17	459	456	454	1743	1721	1702
25	321	319	318	370	372	372
27	416	421	424	1553	1581	1603
29	350	348	346	1549	1526	1509
37	232	231	230	373	373	372
39	305	308	311	1202	1215	1227
41	246	244	243	1394	1375	1361
49	144	143	142	338	336	335
51	269	271	272	397	388	384
53	146	146	145	1122	1108	1098
61	56	56	56	224	216	214
63	114	115	115	355	360	351
65	56	55	55	803	785	775

Table 8. Effects of Negative Steel on Girder Moments

22-Ft Girder	Moment (kip-in.)		
	0.0 sq in./ft	0.2 sq in./ft	0.4 sq in./ft
2	−3702	−3863	−4003
6	−2474	−2607	−2717
10	−2166	−2293	−2396
14	−1957	−2098	−2208
18	−1882	−2011	−2111
22	−1525	−1635	−1721
26	−1340	−1458	−1544
30	−1298	−1404	−1482
34	−1201	−1287	−1350
38	−969	−1053	−1116
42	−925	−1003	−1043
46	−569	−616	−632
28-Ft Girder			
4	−4533	−4472	−4422
8	−3497	−3446	−3403
12	−3441	−3391	−3351
16	−3154	−3106	−3068
20	−2989	−2945	−2910
24	−2916	−2873	−2840
28	−2684	−2641	−2609
32	−2394	−2355	−2327
36	−2102	−2066	−2042
40	−1772	−1738	−1714
44	−1629	−1716	−1739
48	−842	−902	−913

inflection points and internal forces which enable a final girder and column size selection for the first basic composite analysis.

Step 2:
Select tentative girder sizes using the maximum negative moment values found from the noncomposite frame analysis. Assume composite section properties in areas between points of inflection and the end of Region 2, or the other point of inflection if vertical loads govern. Then, determine the length of Region 2 and all section properties for this region. Next, reanalyze this basic composite frame using the appropriate girder properties in Regions 1, 2, and 4.

Step 3:
Verify the final column and girder sizes used in Step 2 and reanalyze the frame, if necessary. Interpretation of the last analysis results and the use of appropriate design aids should make this a simple task. The final column design must be in agreement with the AISC Specification.[2]

Step 4:
Proportion the shear connectors for the composite girders, basing their spacing on the least value of V_h given in Ref. 2, and distribute them uniformly over the portion of the girder which is subject to positive moment. Additional shear connectors are placed in that portion of the girder which is subject to negative moment, using the same spacing. The final girder design must be in complete agreement with the AISC Specification.

Step 5:
Prudence dictates that the frame be checked using vertical loading only. This analysis should be made using girder moment of inertia properties which correspond to positive and negative moment regions obtained from a previous vertical load analysis.

Step 6:
Although this is not a part of the technical analysis and design procedure, the designer should make a cost estimate of the shear connectors required against the savings in frame material weight. This will provide a basis for deciding whether to use a noncomposite or composite rigid frame design.

Temporary shoring must be used unless the designer does not wish to make use of the allowable stress increase permitted in the AISC Specification. The composite frame design presented herein was based on the use of temporary shoring, and a 25% reduction in the load combinations which included wind. Also, secondary moments introduced by deflection and axial forces in the columns may have to be investigated in taller frames. Finally, it is important that the concrete slab be in full contact with the column face and that the shear connectors have at least 1 in. of cover to assure the performance as predicted in Ref. 6, upon which Region 2 has been based. In addition, some longitudinal slab steel at exterior columns should be used to help resist the slab tension in Region 2.

SUMMARY AND CONCLUSIONS

A comparison was made between the preliminary designs of a noncomposite 12-story, two-bay, assymetrical rigid frame and a similar rigid frame having composite girders. Girders were divided into different regions, and forces were determined at all locations of interest. Both of these rigid frames were analyzed using the STRUDL-II Computer Program. The composite rigid frame was analyzed in a similar manner for a noncomposite frame, with composite section properties of the girder being used in areas of positive moment produced by the governing load combination. Where the positive moment region was adjacent to a column, the effective slab width was reduced over a certain distance from the column face, and correspondingly reduced composite section properties were used in this region. The total frame weight and the required number of shear connectors were calculated, and frame weight and drift were compared to those of the noncomposite rigid frame.

The effect of changing the partially effective slab length at columns was studied in further analyses using three different lengths. Also, the use of reinforcing steel as part of the girder section properties was considered in other analyses in regions of negative moment.

Through the use of composite girders in the rigid frame of this study, a decrease in frame weight of 8.7% was realized. It can be concluded that the savings in frame weight achieved through composite frame design is significant and should be compared with the cost of furnishing and applying shear connectors. The necessary use of temporary shoring must also be considered. Further savings in frame weight of the order of 3% was achieved through the inclusion of the negative slab reinforcement at the interior columns, leading to an overall frame weight reduction of 11.7%. It is anticipated that in multibay, multistory rigid frames, even greater savings in material will be realized when making use of the composite behavior of the girders. Figure 9 illustrates the savings in frame weight.

NOMENCLATURE

A_c = Actual area of effective concrete flange used in composite design
A_s = Area of steel girder section
F_b = Bending stress permitted in the absence of axial force
F_y = Steel yield strength
I_x = Moment of inertia about major axis of girder, in.^{4}
K_x = Effective length factor for buckling about x-axis
K_y = Effective length factor for buckling about y-axis
L_c = Maximum unbraced length of the compression flange for which the allowable bending stress, F_b, may be taken as $0.66F_y$ (Ref. 1)
L_u = Maximum unbraced length of the compression flange for which the allowable bending stress, F_b, may be taken as $0.6F_y$ (Ref. 1)

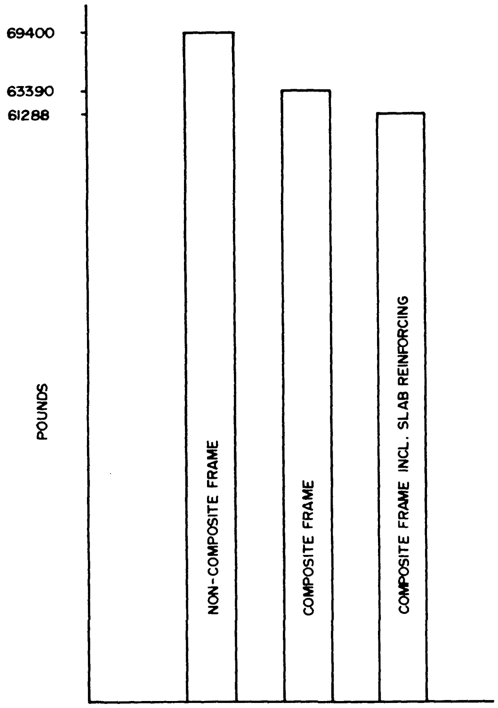

Fig. 9. Comparison of frame weights

L_x = Actual unbraced length for buckling about the x-axis
L_y = Actual unbraced length for buckling about the y-axis
S_b = Section modulus of transformed section, bottom of steel, in.^{3}
S_t = Section modulus of transformed section, top of slab, in.^{3}
V_h = Total horizontal shear to be resisted by shear connectors under full composite action
b_{eff} = Effective width of composite slab, in.
f'_c = Concrete stress, ksi
h = Height of story or building
q = Allowable horizontal shear force resisted by one shear connector
Δ = Drift, horizontal movement caused by loads and allowed by joint rotation and frame member distortion

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APPENDIX

Reference 13 lists the following four conditions which must be met if a frame is to be designed without considering frame instability, i.e., if the effective length of columns is taken to be one ($K = 1$) and the $P\Delta$ effect is to be neglected. They are as follows:

1. All columns are to be proportioned to satisfy the interaction formulas (Ref. 2) with K equal to unity in the calculations of F_a and F'_e. The coefficient C_m is computed from the formula

$$C_m = 0.6 - 0.4\,M_1/M_2 \geq 0.4$$

2. The maximum column axial load ratios f_a/F_a and $f_a/0.6F_y$ are not to exceed 0.75.
3. The maximum in-plane column slenderness ratio L/r is not to exceed 35.
4. The bare-frame first order story-drift index, Δ/H, at working load, shall be controlled so that

$$\Delta/H < (1/7)\Sigma V/\Sigma P$$

where H = story drift
Δ = drift of story due to V
ΣV = total story shear due to lateral loads
ΣP = total gravity load on story

Recent work conducted at Lehigh University by Cheong-Siat-Moy and Lu would change these conditions somewhat. Condition No. 1 remains unchanged. Condition No. 2 remains as shown unless lateral (wind or seismic) loads are involved, when it should be changed to $f_a/1.33F_a < 0.75$ or $f_a/0.8F_y < 0.75$. Condition No. 3 remains unchanged. Condition No. 4 becomes more stringent, with the 1/7 coefficient changing to 1/9 in order to keep the ratio of second order story drift (sway) to first order story sway less than 1.12. In addition, the Lehigh work would limit the ratio of the beam fixed-end moment, M_g, to the wind moment, M_w, to a minimum of 0.25, i.e., $M_g/M_w > 0.25$. The fixed-end moment for non-prismatic beams is not easily calculated and, in the case considered here, the ratio of the beam end-moment due to gravity loads M'_g can be substituted for M_g, which changes this condition to $M'_g/M_w > 0.2$.

The purpose of this paper is to illustrate that a significant reduction in frame weight is possible when girders are designed to act integrally with the floor slab in a rigid frame, even when lateral loads are significant, and that such a design is practical using modern computer methods. An effective column length equal to the actual column length, $K = 1$, was justified in a check of the previously noted conditions. All frame conditions paralleled those of the models used in the development of the previous conditions, with the exception of the non-prismatic (composite) girders, and the ability of the ICES STRUDL-II program to accept segmented member properties made this difference insignificant. It should be noted, though it was not illustrated in this paper, that the $P\Delta$ effect can be very easily incorporated in the final design, since sway may be obtained at every joint in the first order analysis. Also a significant decrease in the bottom story sway can be realized by fixing the column bases. Indeed, using as comprehensive a program as STRUDL-II, it would be feasible to eliminate the noncomposite frame preliminary analysis if a composite frame assumed to be 10% lighter produced a greater savings through material weight cost than the estimated cost of shear connectors and shoring.