Composite Girders with Partial Restraints: A New Approach

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ABSTRACT

Most designs for buildings with steel frames are based on girders with simple connections. To eliminate the problems associated with traditional construction (such as deep and heavy girders and large deflections during construction) structural engineers have been searching for a new design system for a long time. The stub girder system is one example of such efforts. However, the stub girder system proved to be uneconomical for most common buildings. A new and different approach to composite steel/concrete designs was undertaken by the writer resulting in light building frames and cost savings. This new design system is called partial Restraint Girder System ("RGS") (Figure 1). (A composite section is obtained in buildings with metal deck and concrete floors by welding steel studs to the top flange.) With RGS two types of restraint are possible: the first makes use of moment connections to columns; the second includes concrete reinforcement.

In buildings utilizing composite girders, deflections were controlled by either shoring, camber, or further increase in girder size. RGS has arisen as a viable and cost effective alternative.

In traditional designs, the engineer determined the building's moment diagram from a moment distribution or stress analysis. In the RGS method, the Structural Engineer can control the maximum and minimum values of moment on the moment diagram (the governing design values) from the outset to fit his design, by establishing the amount of restraint.

Although composite girders with partial restraints improve the moment resistance of composite girders significantly, such design is commonly ignored and the codes of practice give no guidance as to procedures that might take advantage of the improved properties.

PAST RESEARCH

Extensive knowledge is available on non-composite girder-to-column moment connections. Reference 3 provides a good summary with design examples of various non-composite girder-to-column moment connections. Composite girders with girder-to-column moment connections is a new topic and less covered in past research.

Karl Van Dalen (Reference 4), Ammerman, and Leon (Reference 6) and others, realized the significance of composite girders with negative concrete reinforcement (composite connections). They tested specimens to determine strength, stiffness, and ductility. Reference 6 provides a good summary of past research on composite girders with negative reinforcement (semi-rigid connections).

Also, Ammerman recognized the significance of semi-rigid composite connections (Reference 9) and suggested a method for the design of frames incorporating such connections. By considering the construction phase, RGS improves on the previous work. This paper describes RGS, provides the mathematical formulae which describe the system and provides examples of buildings designed and built with RGS.

INTRODUCTION

The traditional design for buildings with steel frames is based on composite girders with simple connections. The disadvantage of this traditional design is that the entire moment requirement is at one portion of the girder resulting in large size girders. Also, girders have large mid-span deflections during construction when the concrete is wet. In order to eliminate these disadvantages, the designer specified camber or temporary shoring. However, since both of these methods are costly and difficult to implement, contractors often preferred to do without them and instead, increased girder sizes even further. With partial restraints, girder sizes can be decreased and the deflections reduced.

Two different restraint types are possible, as follows:

a. Girder-to-column moment connection.
b. Negative concrete reinforcement.

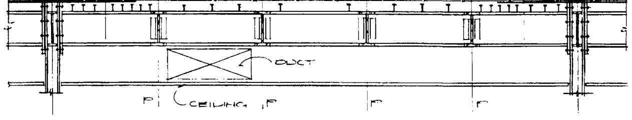

Fig. 1. Composite girder with partial restraint.

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The two different types of restraint are best utilized based on the following rule of thumb: when deflections during construction are large, and/or the girder sizes are governed by construction loads, girder-to-column connections are preferred; when deflections do not govern, and the girder size is governed by superimposed loads, negative concrete reinforcement bars are preferred; however excellent results are achieved when a combination of both restraint types is used.

Just like a composite girder with simple connections, the design of a Restraint Girder System is also done in two phases—construction phase, when the concrete is wet and the final phase, after the concrete hardens.

CONSTRUCTION PHASE

At this phase, the steel girder alone supports all the loads. Some steel girders with simple connections have significant mid-span deflections at this phase. Introducing end moment connections results in reduced mid-span deflections. For example, for a beam with fixed connections, Figure 2 shows that mid-span deflections can be reduced by as much as 58 percent if only one end is fixed and by 80 percent if both ends are fixed. These are very significant reductions. Considering that the end moment connections also have the added benefit of reducing the mid-span moment, it becomes quite clear what a powerful design tool has been created.

To provide the rigidity required for this phase, the end moment connection must be designed as a rigid connection (AISC Type 1 construction) for this stage (Figure 3). It provides just enough strength and rigidity to hold the original angles between the members unchanged. During this phase all connection components are stressed elastically. The connection is strong enough to hold the original angles between members unchanged, reducing the center moment and mid-span deflections.

FINAL PHASE

At this phase, the steel girder acts compositely with the concrete. The girder now is both strong and rigid.

Once the concrete hardens, superimposed loads such as partitions, mechanical, ceiling, and live loads are applied. At this time, the moment at the girder end wants to increase, however since the connection has reached its elastic capacity, it will deform plastically. This now corresponds to AISC Type 3 construction; the connection now becomes semi-rigid. All excess moment "shaken-off" by the semi-rigid moment connection is now transferred to the middle section of the girder. Since this middle section is composite with the concrete, it is both strong and rigid. Therefore, any deflections associated with the final phase are small.

The design described above results in smaller girders and reduced costs. Where before A36 steel was used, governed by deflection considerations, A572 steel now often becomes more economical.

The sensitivity of the RGS system to deflections when the concrete is wet and unevenly placed ought to be investigated by the design engineer for each individual project. In some cases the RGS system should be specified with recommenda-

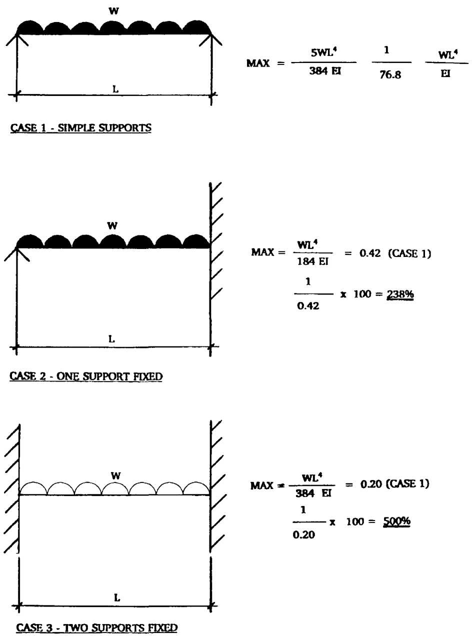
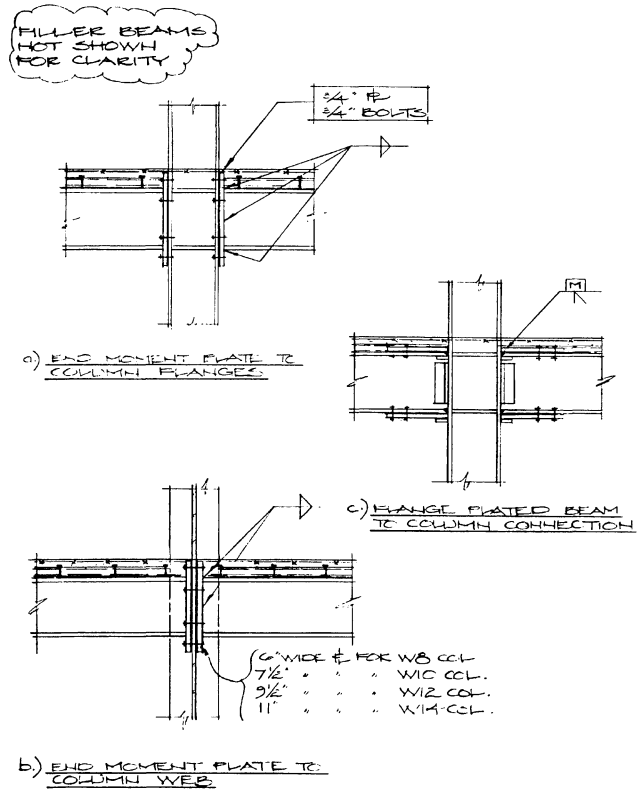


Figure 2



a.) END MOMENT PLATE TO COLUMN FLANGES

c.) FLANGE PLATED BEAM TO COLUMN CONNECTION

b.) END MOMENT PLATE TO COLUMN WEB

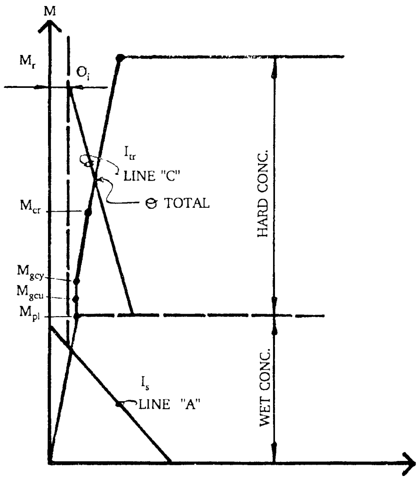
Figure 3

tions for the concrete pour sequence and the acceptable locations of construction joints.

END-MOMENT CONNECTIONS

A cost efficient end-moment connection is an end-plate connection (for moderate size moments) (Figure 3a and 3b). It performs well as a rigid connection during the construction phase, and a semi-rigid connection at the final phase. An end plate is a particularly good choice because not only does it deliver forces to the column but it also reinforces the column by spreading compression forces over larger areas, just like a bearing plate, thus reducing the need for compression column stiffeners. It is especially economical when full penetration welds are not required. However, other connections can also be used. Figure 3d shows a girder to column moment connection with angles. Angles are also a good choice because the bottom flange is reinforced against local buckling by the horizontal leg of the angle. Reference 3 and others provide design guidelines for the design of such connections.

Increasing the connection size beyond that which provides full fixity during the construction phase is usually not necessary and proves to be uneconomical. Therefore, for best economy, the end moment connection need not be over-designed. Details A and B in Figure 3 show a relatively inexpensive moment connection. The connection shown in detail C is more expensive.



M pl -	End moment for plate loads from a design moment diagram, when the only loads applied are plate loads.
M gcu -	Girder to column moment connection - calculated design strength.
M gcy -	Girder to column moment connection - yield strength.
M cr -	Moment value at which the concrete cracks. Stress is transferred to rebars.
M r -	Moment value at which rebars yield.

Note: G/C moment connection may or may not actually yield. For RGS, unless shown otherwise by analysis, the connection should be assumed to yield and therefore it should be designed with ductility in mind.

Fig. 4. Girder-to-column connection + reinforcement.

One way to evaluate a girder with moment connections is by making use of the connection moment rotation curve (Figure 4). A composite girder with partial restraint behaves just like a steel girder with full restraint when the concrete is wet (line A). After the concrete hardens, and additional loads are superimposed, the connection provides additional restraint until yielding; then the girder behaves just like a simple supported composite girder. Figure 4 shows a moment rotation curve with concrete reinforcement added at the joint. The connection curves shown are diagrammatic and in reality yielding may occur sequentially.

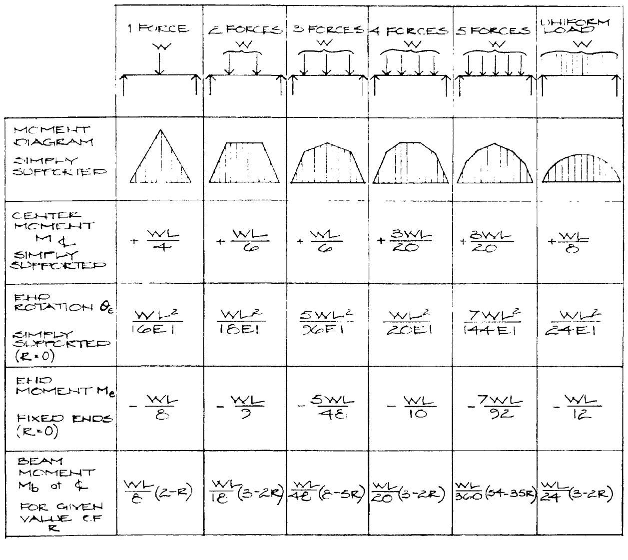
In order to determine the various points on the moment rotation curve, Figure 5 is reproduced herein from Reference 3.

ADDITIONAL RESTRAINT

Research done by Professors Karl Van Dalen and Hernan Godoy at Queen's University, Kingston, Ontario (Reference 4) revealed that additional moment strength can be achieved at the beam-column connection if only 0.46 percent of the concrete slab area is provided as slab reinforcement. This additional strength is at least equal to the ultimate moment capacity of the composite beam and is not influenced by the type of connection between the steel elements. The rotational capacity of the composite beam—column connection is also at least equal to that of a conventional, non-composite rigid steel connection.

The AISC specifications for Structural Steel for Buildings (Reference 2) allows the calculations of the negative design moment strength based on the plastic stress distribution of the composite section, provided that the following are met:

a. Shear connectors are located in the negative moment region.



	1 FORCE	2 FORCES	3 FORCES	4 FORCES	5 FORCES	UNIFORM LOAD
	W	W	W	W	W	W
MOMENT DIAGRAM SIMPLY SUPPORTED						
CENTER MOMENT M ℄ SIMPLY SUPPORTED	$+\frac{WL}{4}$	$+\frac{WL}{6}$	$+\frac{WL}{6}$	$+\frac{3WL}{20}$	$+\frac{3WL}{20}$	$+\frac{WL}{8}$
END ROTATION θ_e SIMPLY SUPPORTED (R=0)	$\frac{WL^2}{16EI}$	$\frac{WL^2}{18EI}$	$\frac{5WL^2}{96EI}$	$\frac{WL^2}{20EI}$	$\frac{7WL^2}{144EI}$	$\frac{WL^2}{24EI}$
END MOMENT M_e FIXED ENDS (R=0)	$-\frac{WL}{8}$	$-\frac{WL}{9}$	$-\frac{5WL}{48}$	$-\frac{WL}{10}$	$-\frac{7WL}{92}$	$-\frac{WL}{12}$
BEAM MOMENT M_b at ℄ FOR GIVEN VALUE OF R	$\frac{WL}{8}(2-R)$	$\frac{WL}{18}(3-2R)$	$\frac{WL}{48}(8-5R)$	$\frac{WL}{20}(3-2R)$	$\frac{WL}{360}(54-35R)$	$\frac{WL}{24}(3-2R)$

Fig. 5. Moments and end rotation for various load/beam conditions.

b. The slab reinforcement is adequately developed.
c. Steel beam is compact and braced.

The designer can use this additional strength to reduce the girder size further. Only additional studs and negative concrete reinforcement are needed. However, in order to ensure a uniform cracking pattern in the slab in the vicinity of the column, Karl Van Dalen (Reference 4) recommends that at least twice the minimum area of steel reinforcement be extended on each side of the column centerline (Figure 6).

UNBALANCED LOADS

For a long time engineers assumed that unbalanced loads might overstress non-composite steel girders designed with partial restraint and therefore avoided the use of such restraint in steel buildings. RGS however is not very sensitive to unbalanced loads for two reasons:

a. Traditionally, structures have been analyzed on the basis of "frame action"—meaning beams with joints which are allowed to rotate when subjected to unbalanced moments. For building structures with concrete floors an additional horizontal restraint exists. This restraint is provided by the concrete diaphragm and is usually ignored by engineers. Unbalanced loads create a joint rotation; any joint rotation is associated with horizontal and vertical translations. However, in certain buildings with girders connected to concrete floor diaphragms horizontal translations are restrained. Therefore unbalanced loads in buildings with concrete floors are generally less able to generate joint rotation.
b. If the load is increased the reinforcement might yield. Tests indicate that reinforced concrete has some ductility to transfer of moments from one section to another after first yielding of reinforcement. When tensile reinforcement reaches yield at one section it will continue to yield while the section rotates. For a continuous member, the load will increase until all sections reach yield or until the concrete reached ultimate strain. As a result for most concrete structures moment redistribution is possible and accepted by the Building Codes.

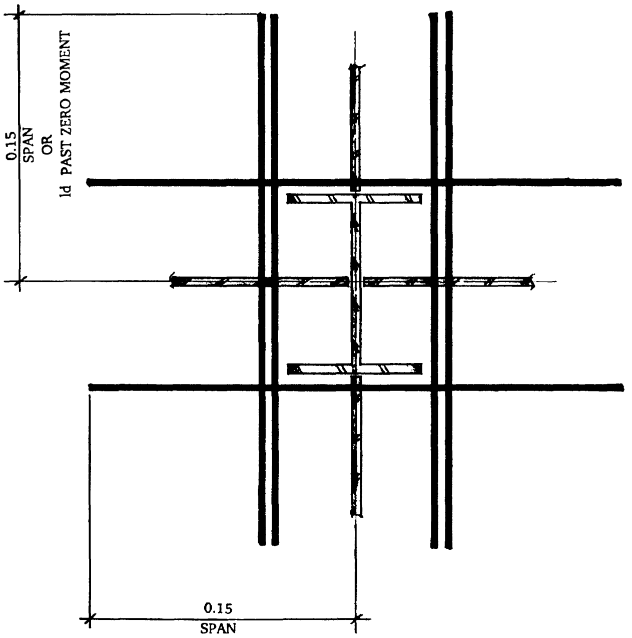


Fig. 6. *RGS—Concrete reinforcement.*

Because of the above reasons, for most common buildings using RGS, adequate strength is assured with unbalanced loads present if adequate concrete reinforcement is provided.

DUCTILITY

Ductility is associated with the ability of the joint to rotate after yielding. Joint rotation can be prevented by premature local or overall buckling of the bottom flange and buckling of the web. The use of under-reinforced sections and class 1 steel shapes assures adequate post-yielding rotations.

COMPOSITE STUDS

Stud-design criteria is similar to composite girders without restraint, with the exception that if top reinforcement is used for restraint, then additional studs are required between the point of maximum negative moment and point of zero moment. The number of such studs shall be selected to develop the negative moment (Figure 7).

ALTERNATIVES

Alternatives to composite girders with partial restraint include the stub girder system, haunch girders, and simple composite girders. These alternatives however, are expen-

COMPOSITE STUDS

A. POSITIVE MOMENT

$$N = \frac{F_y A_s}{2q} \qquad \text{EQ'N. 7-1}$$

$$N' = N \left[\frac{M - M_s}{M_{cap} - M_s} \right]^2 \qquad \text{EQ'N. 7-2}$$

B. NEGATIVE MOMENT

$$N = \frac{F_{yr} A_r}{2q} \qquad \text{EQ'N. 7-3}$$

WHERE: $d_s = h_b - t_{fl}$

$$d_{eff} = h_b - t_{fl} + d_{md} + d\ conc - t\ cov - \frac{\phi\ bar}{2}$$

q - STUD CAPACITY
M_s - MOMENT CAPACITY FOR STEEL SECTION ALONE
M_{cap} - MOMENT CAPACITY FOR COMPOSITE SECTION
M - MOMENT FROM DIAGRAM

Fig. 7. *Composite beam-stud requirements.*

sive, complicated, deep, and might require duct openings, camber, shoring.

ADVANTAGES

The advantages of composite girders with partial restraint are many. First, the design usually results in shallow and small girders with no duct penetrations. The lighter weight girders also have small mid-span deflections. Camber or shoring are not required. Wind loads can be incorporated as part of the 33 percent increase in allowable stresses at no additional costs. Medium to large spans can be accommodated. Long term creep deflections associated with shored construction are eliminated. The engineering analysis and design is simple and suitable for hand calculations or computer use. All engineering principles involved are based on the AISC specifications and in accordance with standard practice.

It is important to point out what is new about RGS. Girder-to-column moment connections, rigid or semi-rigid, are not new; negative concrete reinforcement with composite construction is not new for wind loads, but it is new for gravity loads when used to reduce the girder size. (Some reports indicate that negative reinforcement with composite construction has been used with the stub girder system). The use of rigid and semi-rigid construction in the same frame at the same location is new. The use of such construction in conjunction with negative concrete reinforcement is also new. The traditional use of moment connections alone or the traditional use of negative reinforcement alone cannot provide the benefits which are created by the RGS system. For this reason, RGS has arisen as a viable new alternative system for building construction. The final result, which is a small girder system with controlled deflections, and without shoring or camber, is new.

EXAMPLES

1. *Capitol Square Office Building—Columbus, Ohio*
(Figure 9)

This is a 28-story office tower with a triangular floor plan. The bay size is 30×30. The floor construction consists of 2-in. deep metal deck and 3.25-in. light weight concrete. The filler beams are W14×22 at 10 feet on center. The girders are composite, restrained, hunched girders. Both hunches and restraints were used in order to develop the maximum possible negative moment at the columns and reduce the mid-span moment. The straight portion of the girders was W14×30. Hunches were fabricated from 16-in.-deep sections, cut diagonally. Restraint was obtained with end plates. This continuous girder system was also used to provide additional lateral load resistance.

Contractor: Turner Construction Company, Columbus, Ohio.
Steel Fabricator: Ohio Steel Fabricators

2. *Minolta Office Building—Ramsey N.J.*
(Figure 8)

A 4-story office building with a steel frame on a concrete foundation. The building has a horseshoe footprint and 120,000 sq. ft of space. The bay size is 25×30 with girders framing the long direction. 3.25 inches of light-weight concrete were poured over 2-in. composite metal deck. 14-in. deep composite filler beams are spaced at 10 feet on center and span the 25-ft dimension. 14-in. composite girders with partial restraint at columns support all gravity and wind loads. Simply supported girders, 50 ksi steel, would have been W14×53 with a 1-in. camber and 57 studs. Using partial restraint, only W14×43 girders were used with no camber and 24 studs. Many mechanical units and a continuous roof screen created "heavy congestion" on an open web joist-framed roof. Despite the roof congestion, the total steel weight was under 7.5 lbs. per sq. ft. No floor deflections or vibrations were reported. The contractor and steel fabricator reported easy fabrication and construction details.

Steel fabricator: Mulach Steel, Pennsylvania

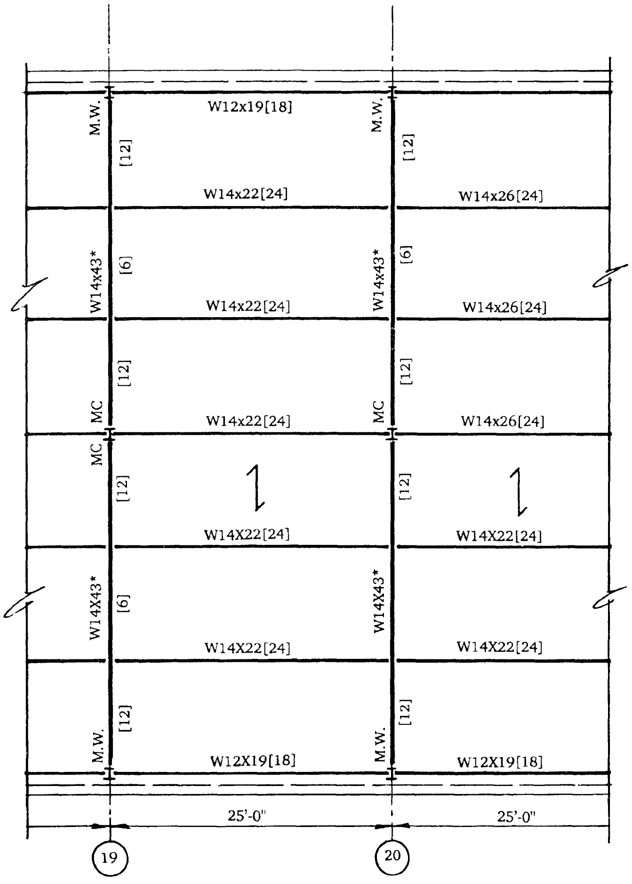


Fig. 8. Part plan—Minolta Office Building.

3. *Parking/Retail Building—New York, N.Y.* (Figure 10)

This is a 45,000 sq. ft building with retail on the ground floor, parking in the basement, and a playground/community area on the roof. The bay size varies; filler beams are 18 to 22 feet long and the girders 17 to 35.5 feet long.

A traditional design with simply supported composite girders would have resulted in W18×76 or W24×62 cambered girders. Using Partially Restraint Girders resulted in W16×45 ($F_y = 50$) for the first exterior girder and W16×36 ($F_y = 50$) for interior girders without camber. The end-moment connection is an end plate. The shallower girders were necessary for a lower overall building height. Figure 11 shows a comparison

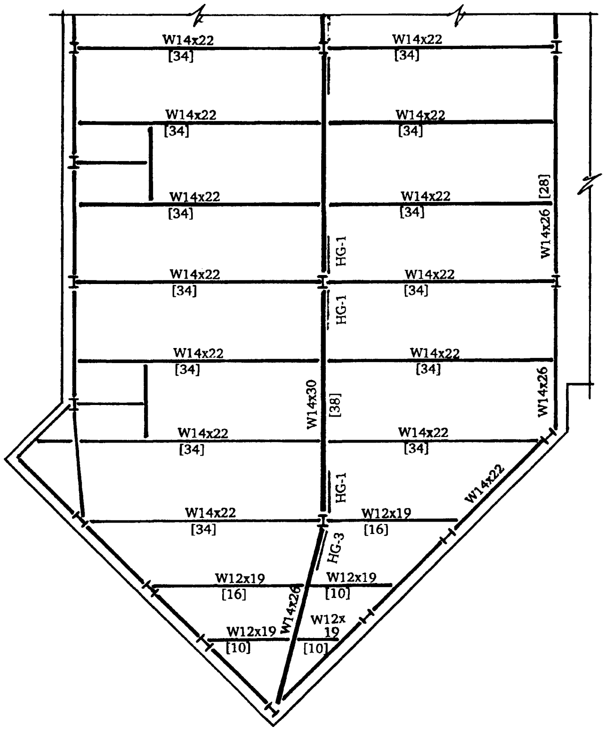


Fig. 9. Part plan—Capitol Square Office Building.

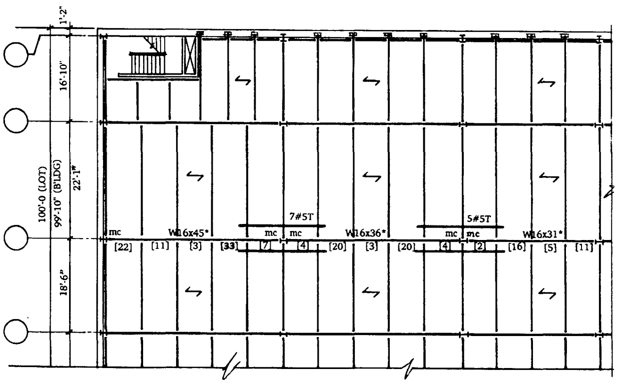


Fig. 10. Part plan—Parking/Retail Building.

Span	Section	M+ (Ft-k)	M- (Ft-k)	Neg. Studs no G/C	Neg. Studs yes G/C	Pos. Studs	Pl. Defl.	Neg. Reinf.	
1	W24x62	490	-	-	-	52	0.70*	-	A
2	W24x62	490	-	-	-	52	0.70*	-	
3	W21x44	490	-	-	-	30	0.42	-	
1	W16x57 **	452	120	-	-	56	0.67*	5 #5	B
2	W16x45 **	368	120	-	-	64	0.34	5 #5	
3	W16x31 **	255	120	-	-	24	0.39	-	
1	W16x45 **	367	244	-	7x2	66	0.58	7 #5	C
2	W16x36 **	300	190	-	4x2	40	0.46	5 #5	
3	W16x31 **	255	160	-	2x2	32	0.39		
1	W16x45 **	367	244	13x2	-	66	1.87*	10#6	D
2	w16x36 **	300	190	11x2	-	40	2 44*	11#5	
3	W16x31 **	255	160	9x2	-	32	0.95*		

(1) * Camber is recommended

(2) ** Fy=50 ksi

A - Traditional Design
B - G/C Moment Connection
C - G/C Moment Connection + Negative Reinforcement - <u>Least Cost</u>
D - Negative Reinforcement

Fig. 11. Summary.

CASE A - G/C MOMENT CONNECTION ONLY

$$A_{RM} = 0.46\% \ Ac \qquad \text{EQ'N. 6-1}$$

CASE B - CONC REINFORCEMENT (WITH OR WITHOUT G/C MOMENT CONNECTION)

$$A_{RM} = 0.80\% \ Ac \qquad \text{EQ'N. 6-2}$$

$$A_{RR} = \frac{K\,[M_U/\emptyset - M_N - [M_{GCi}(1-X/L) + M_{GCj}(X/L)]}{(X/L)F_{YR}(d_{EFF\cdot})} \qquad \text{EQ'N. 6-3}$$

$$A_{RF} =< \frac{(ZF_Y - M_{GC})K}{dsF_{YR}} \qquad \text{EQ'N. 6-4}$$

A_R SHALL BE THE LARGEST OF A_{RM} <u>OR</u> A_{RR} BUT IT NEED BE NO LARGER THAN A_{RF}.

$$M_{Ri} = A_R\,F_{YR}\,(d_{EFF\cdot}) < M_{i\ SUP} \qquad \text{EQ'N. 6-5}$$

$$M_{RTx} = M^T{}_{Ri} + M^T{}_{GCi} + M^T{}_{Rj} + M^T{}_{GCj} \qquad \text{EQ'N. 6-6}$$

$$M_{Ux} \leq \emptyset\,[M_{Nx} + M_{RTx}] \qquad \text{EQ'N. 6-7}$$

Ac - AREA OF CONCRETE

M_N - MAXIMUM MOMENT CAPACITY BASED ON PLASTIC TRANSFORMED SECTION

M_{GC} - GIRDER TO COLUMN CONNECTION MOMENT CAPACITY, SUBSCRIPTS i, j INDICATE GIRDER END i & j. $M_{GCi} < M_{GCj}$

M_R - MOMENT CAPACITY (NEGATIVE) PROVIDED BY CONCRETE REINFORCEMENT

M_{RT} - TOTAL AVAILABLE RESTRAINT

X - DISTANCE FROM END i TO LOCATION OF MAXIMUM POSITIVE MOMENT

Misup - MAXIMUM NEGATIVE MOMENT DUE TO SUPERIMPOSED DL + LL AT END i ASSUMING CONTINUOUS GIRDERS

T - SUPERSCRIPT-TRANSPOSED LOCATION

K - DUCTILITY FACTOR

Figure 12

of different design schemes using RGS alternatives. Scheme C which includes a girder-to-column moment connection and negative concrete reinforcement is the best design alternative. It requires no camber, yet the result is small girder sizes, economical connections, and light concrete reinforcement.

ANALYTICAL EVALUATIONS

Figure 12 shows formulas which govern the design of RGS using LRFD. Equations 6-1 and 6-2 are based on Van Dalen (Reference 4). While Van Dalen does not address the case of a girder-to-column moment connection alone, an increased amount of minimum reinforcement is recommended by the writer to control cracking and for unbalanced loads.

Equation 6-3 is based upon partial-partial restraint when the amount of concrete negative reinforcement required, based on loads, is less than the maximum that can be provided.

Equation 6-4 represents the maximum amount of concrete negative reinforcement that can be provided and still ensure its yielding.

A ductility factor K is provided to ensure that the concrete reinforcement will yield first. This factor can vary depending upon certain factors such as the steel girder size, the type of girder-to-column moment connection and/or the type of bottom flange restraint.

The number of composite studs should be selected based on the formulas shown in Figure 7.

Figure 13 shows tables prepared by the author for a quick design of RGS in a engineering office.

ADDITIONAL RESEARCH

It is recommended that resources be allocated for research into the following and other topics for better prediction of the behavior of the RGS system in building structures:

1. Requirements for column stiffeners.
2. Behavior under reversed loading. Wind and seismic loading.
3. Short and long term effects of shrinkage, creep, relaxation.
4. Non-linear behavior of steel connections and concrete reinforcement.
5. Pattern loading conditions.
6. Column unbraced length.

1-1/2" MD + 2-1/2" ST. COCN. (Fy=50 Ksi)

b=90"
Ar min=1.80 sq.in.

Section	ϕM_N	ϕM_{RF}	ϕM_N + K ϕM_R	A_{RF}
W12 x 19	185	110	267	1.7
W12 x 22	212	130	309	2.1
W12 x 26	247	167	372	2.6
W12 x 30	282	193	426	3.0
W12 x 35	232	229	494	3.5
W12 x 40	362	260	557	4.2
W12 x 45				
W12 x 50				
W14 x 22	235	144	343	2.0
W14 x 26	275	175	406	2.5
W14 x 30	313	211	471	3.0
W14 x 34	350	238	528	3.4
W14 x 38	386	270	588	3.8
W14 x 43	430	306	659	4.4
W14 x 48	472	349	733	5.0
W14 x 53	512	388	803	5.5
W16 x 26	302	188	443	2.4
W16 x 31	354	230	526	2.9
W16 x 36	406	273	610	3.5
W16 x 40	455	311	678	3.9
W16 x 45	494	352	758	4.4
W18 x 35	427	269	629	3.0
W18 x 40	482	317	719	3.6
W18 x 46	545	367	820	4.2
W18 x 50	585	409	891	4.7

Figure 13a

2" MD + 3-1/4" LT.WT. CONC (Fy=50Ksi)

b= 90"
Ar min = 2.34 sq. in.

Section	ϕM_N	ϕM_{RF}	ϕM_N+ K ϕM_{RF}	A_{RF}
W12 x 16	176	95	247	1.4
W12 x 19	208	116	295	1.7
W12 x 22	239	140	344	2.0
W12 x 26	279	178	412	2.6
W12 x 30	318	206	472	3.0
W12 x 35	365	245	548	3.6
W12 x 40	411	276	618	4.0
W12 x 45	455	310	687	4.6
W12 x 50	497	347	757	5.1
W14 x 22	262	153	376	2.0
W14 x 26	306	185	444	2.4
W14 x 30	349	218	512	2.8
W14 x 34	391	252	580	3.3
W14 x 38	432	284	645	3.7
W14 x 43	482	321	722	4.2
W14 x 48	530	362	801	4.7
W14 x 53	570	402	871	5.2
W16 x 26	333	198	481	2.3
W16 x 31	392	242	573	2.8
W16 x 36	449	286	663	3.3
W16 x 40	536	326	780	3.8
W16 x 45	548	369	824	4.3
W16 x 50	601	412	910	4.8
W16 x 57	665	470	1017	5.5
W18 x 35	475	291	693	3.1
W18 x 40	536	343	793	3.6
W18 x 46	595	397	892	4.2

Figure 13b

7. Shored versus unshored construction.
8. Local buckling.

CONCLUSIONS

A new girder system was presented. The system resolves the problems associated with traditional steel construction. Restraint Girder System ("RGS") provides a powerful tool for reducing girder sizes and mid-span deflections. End restraint is achieved with end-moment connections and/or concrete top reinforcement. The design is especially economical when the end-moment connection is detailed with economy in mind. The shape of the moment diagram can be controlled by the designer, resulting in cost savings. The new design method can be used with significant savings for most steel buildings with metal deck concrete and composite action. The resulting analysis and design method will most likely become a favorite for steel design into the next century, creating a new class of building structures. In the future engineers might find it practical to apply RGS knowledge to other building structural components such as filler beams and also to bridges.

3" MD + 3-1/4" LT.WT (Fy=50 Ksi)

b = 90"
Ar min = 2.34 sq.in.

Section	ϕM_N	ϕM_{RF}	$\phi M_N + K \phi M_R$	A_{RF}
W12 x 16	195	100	270	1.4
W12 x 19	230	123	322	1.7
W12 x 22	264	146	373	2.0
W12 x 26	309	185	447	2.6
W12 x 30	352	214	512	3.0
W12 x 35	405	255	596	3.5
W12 x 40	457	286	671	4.0
W12 x 45	506	322	747	4.5
W12 x 50	554	361	824	5.0
W14 x 22	264	158	382	2.0
W14 x 26	336	192	480	2.4
W14 x 30	384	226	553	2.8
W14 x 34	430	261	625	3.2
W14 x 38	476	294	696	3.6
W14 x 43	531	332	780	4.1
W14 x 48	585	374	865	4.7
W14 x 53	637	416	949	5.1
W16 x 26	363	204	516	2.3
W16 x 31	428	249	614	2.8
W16 x 36	491	296	713	3.3
W16 x 40	540	337	792	3.8
W16 x 45	600	380	885	4.3
W16 x 50	658	425	976	4.8
W16 x 57	730	485	1093	5.5
W18 x 35	509	299	733	3.0
W18 x 40	575	353	840	3.6
W18 x 45	640	408	946	4.2
W18 x 50	703	455	1044	4.7

Figure 13c

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