The Restraint Girder System—Local Web Buckling Behavior and Design Considerations

NEIL WEXLER, P.E. and FENG BAO LIN, Ph.D., P.E.

ABSTRACT

The Restraint Girder System (RGS) was previously presented as a new design option for composite girders.[14] RGS is a composite steel and concrete girder, made continuous with rigid or semi-rigid connections. The RGS design provides beneficial cost, depth and deflection results, and is easy to construct. Various parts and components of the Restraint Girder System were previously studied by the writer, Van Dalen, Leon, Johnson, Kemp and others. Local web buckling was studied by Kemp. This paper presents an analysis for web buckling considerations when US steel shapes are designed with RGS, provides a definitive classification for all US shapes and provides recommendations to be included in a future RGS addition to the Code. This paper presents the mathematical analysis for web buckling considerations, indicates how U. S. steel sections with RGS are rated for local web buckling and provides recommended options for ductility.

PAST RESEARCH

Extensive knowledge is available on steel girders to column moment connections (non-composite); also, extensive knowledge is available on composite girders with simple end connections. Rigid and semi-rigid connections used together with composite girders represents a new approach and little knowledge exists on such design and behavior. Also, semi-rigid composite connections are new; little information exists on the design and behavior of entire girder systems using composite connections (RGS). Tests on semi-rigid composite connections were done by Van Dalen, Leon and others. "Ductility" was studied by Kemp and Johnson. Semi-rigid composite connections were studied by the writer, Leon and others.

U. S. sections vary in slenderness from European or Canadian counterparts and their local web buckling failure must be properly taken into account by designers of RGS. Kemp[11] pointed to contradictions which exist between experimental observations and current codes and suggested that the current codes may be unsafe; then he proposed a theoretical explanation. This paper is a second in a series on RGS by the writer; its purpose is to provide a method for local web buckling evaluation of U. S. steel sections designed with RGS using current U. S. Codes.

Neil Wexler, P.E., P.C. is president, Consulting Structural Engineers, New York.

Feng Bao Lin is professor of Civil Engineering at Polytechnique University, Brooklyn, NY.

INTRODUCTION

The most common modes of failure which inhibit the development strength of a semi-rigid composite connection are:

1. Local flange buckling at the support.
2. Local web buckling at the support.
3. Lateral torsional buckling of the steel section in the negative-moment region.

Load and Resistance Factor Design Specification for Structural Steel Buildings (LRFD),[1] classify steel sections as compact, non-compact and slender. The variable which is specified for qualifying sections include the slenderness (width to thickness ratio) of its compression elements. Local buckling code provisions include research by White (1956), Lay and Galambos (1965, 1967). Changes were made for LRFD purposes as described by Yura, et al (1978). Compact sections are capable for developing a *fully plastic* stress distribution and they possess a rotational capacity of approximately 3 before the onset of local buckling. Noncompact sections can develop the yield stress in compression elements before local buckling occurs, but will not resist inelastic local buckling at the strain levels required for a fully plastic stress distribution. Slender compression elements buckle elastically before the yield stress is achieved.

The dividing line between compact and noncompact sections is the limiting width-thickness ratio λ_p and λ_{ps} For a section to be compact, all of its compression elements must have width-thickness ratios smaller than the limiting λ_p.

Values of λ_p are given in Table C-B5.1 of reference 1. For webs, these values are as follows:

For $P_u / \phi_b P_y \leq 0.125$:

$$\lambda_p = \frac{640}{\sqrt{F_y}}\left(1 - \frac{2.75P_u}{\phi_b P_y}\right) \quad (1)$$

For $P_u / \phi_b P_y > 0.125$:

$$\lambda_p = \frac{191}{\sqrt{F_y}}\left(2.33 - \frac{P_u}{\phi_b P_y}\right) \geq \frac{253}{\sqrt{F_y}} \quad (2)$$

A greater inelastic rotation capacity provided by the limiting values λ_p given in Table C-B5.1 may be required for some structures in areas of high seismicity. It has been suggested that in order to develop a ductility of from 3 to 5 in a structural member, ductility factors for elements would have to lie in the range of 5 to 15. Thus, in this case it is prudent to provide for an inelastic rotation of 7 to 9 times the elastic rotation (Chopra and Newmark, 1980). In order to provide for this rotation capacity, the limits λ_{ps} for local web buckling would be as follows (Galambos, 1976):

For $P_u / \phi_b P_y \leq 0.125$:

$$\lambda_{ps} = \frac{520}{\sqrt{F_y}}\left(1 - \frac{1.54P_u}{\phi_b P_y}\right) \quad (3)$$

For $P_u / \phi_b P_y > 0.125$, Equation 2 applies.

BEHAVIOR AND DESIGN

In non-composite girders subject to pure bending actions, the web is half in tension and half in compression. The addition of tension concrete reinforcement results in a shift of the neutral axis. This is equivalent to the application of coincidental axial forces to the previously pure bending actions. The result is an increase of the web depth which is in compression. This increases the slenderness ratio of the web in compression, resulting in a potential reclassification of the section as non-compact or slender.

The design criteria for the Restraint Girder System with Semi-Rigid Composite Connections is to select concrete reinforcement such that the cross-section is "under reinforced" as in concrete design. Then, connection components are designed and the section is checked for local and lateral buckling.

Local web buckling is evaluated in this paper by first developing an expression for axial strains as a function of the connection geometry (y_3, d) and the reinforcement ratio (γ_1, γ_2).[8] Then, by varying the amounts of concrete reinforcement, values of λ_p and λ_{ps} are calculated for different beam depths. The procedure above is repeated for different concrete floor thicknesses (y_3), (Tables 1 through 9). Values of λ_p and λ_{ps} thus obtained are compared with values of h_c / t_w listed in the front pages of the AISC Manual.

For this work the following assumptions are made:

1. The strength of members shall be based on satisfying the applicable conditions of equilibrium and compatibility of strains.
2. Plane elements remain plane.
3. Strain is assumed directly proportional to the distance from the neutral axis.
4. Tensile strength of concrete is neglected.

Consider a composite semi-rigid connection. By triangular similarity, the following relationships are derived:

$$\frac{\varepsilon_s}{\gamma_1 \varepsilon_y} = \frac{c}{y_3 + c}$$

$$\frac{\varepsilon_s}{\gamma_2 \varepsilon_Y} = \frac{c}{d - c}$$

$$\varepsilon_s = \frac{\gamma_1 \varepsilon_y c}{y_3 + c} = \frac{c\gamma_2 \varepsilon_y}{d - c}$$

$$\frac{\gamma_1}{y_3 + c} = \frac{\gamma_2}{d - c}$$

$$\gamma_1 d - \gamma_1 c = \gamma_2 y_3 + \gamma_2 c$$

$$\gamma_1 d - \gamma_2 y_3 = (\gamma_1 + \gamma_2)c$$

$$\therefore c = \frac{\gamma_1 d - \gamma_2 y_3}{\gamma_1 + \gamma_2} \quad (4)$$

$$\varepsilon_s = \frac{\gamma_1 \varepsilon_y c}{y_3 + c} = \frac{\gamma_1 \varepsilon_y}{\frac{y_3}{c} + 1}$$

Substituting c from Equation (4) to obtain:

$$\varepsilon_s = \frac{\gamma_1 \varepsilon_y}{y_3 / (\gamma_1 d - \gamma_2 y_3 / \gamma_1 + \gamma_2) + 1}$$

$$= \frac{\gamma_1 \varepsilon_y}{y_3(\gamma_1 + \gamma_2) + \gamma_1 d - \gamma_2 y_3 / \gamma_1 d - \gamma_2 y_3}$$

$$= \frac{\gamma_1 \varepsilon_y(\gamma_1 d - \gamma_2 y_3)}{\gamma_1 y_3 + \gamma_2 y_3 + \gamma_1 d - \gamma_2 y_3}$$

$$= \frac{\gamma_1 \varepsilon_y(\gamma_1 d - \gamma_2 y_3)}{\gamma_1 y_3 + \gamma_1 d}$$

$$\therefore \varepsilon_s = \varepsilon_y \frac{\gamma_1 d - \gamma_2 y_3}{d + y_3} \quad (5)$$

Consider steel section alone

$$\varepsilon_1 + \varepsilon_2 = \gamma_2 \varepsilon_y$$

$$\varepsilon_1 - \varepsilon_2 = \varepsilon_s$$

Adding the above relationships, and substituting Equation 5, we obtain:

$$2\varepsilon_1 = \gamma_2\varepsilon_y + \varepsilon_s = \gamma_2\varepsilon_y + \varepsilon_y\frac{\gamma_1 d - \gamma_2 y_3}{d + y_3} = \varepsilon_y\left[\gamma_2 + \frac{\gamma_1 d - \gamma_2 y_3}{d + y_3}\right]$$

$$= \varepsilon_y\left[\frac{\gamma_2 d + \gamma_2 y_3 + \gamma_1 d - \gamma_2 y_3}{d + y_3}\right] = \varepsilon_y\frac{d(\gamma_1 + \gamma_2)}{d + y_3}$$

$$\therefore\ \varepsilon_1 = \varepsilon_y\frac{d(\gamma_1 + \gamma_2)}{2(d + y)} \qquad (6)$$

Equation 6 represents the bending component of strain.

$$\varepsilon_2 = \varepsilon_y\left[\frac{2\gamma_2 d + 2\gamma_2 y_3 - d\gamma_1 - d\gamma_2}{2(d + y_3)}\right]$$

$$= \varepsilon_y\left[\frac{\gamma_2 d + 2\gamma_2 y_3 - d\gamma_1}{2(d + y_3)}\right]$$

$$\therefore\ \varepsilon_2 = \varepsilon_y\left[\frac{d(\gamma_2 - \gamma_1) + 2\gamma_2 y_3}{2(d + y_3)}\right] \qquad (7)$$

Equation 7 represents the axial component of strain.

When the axial component of strain is zero, then the section is in pure bending, such as in the case of a non-composite connection ($y_3 = 0$; $\gamma_1 = \gamma_2 = 1.0$).

$$\frac{P_u}{\phi_b P_y} = \frac{\varepsilon_2 E_s A_s}{\phi_b \varepsilon_y E_s A_s} = \frac{\varepsilon_2}{\phi_b \varepsilon_y}$$

Substitute ε_2 from Equation 7 to get:

$$\frac{P_u}{\phi_b P_y} = \frac{\gamma_2(2y_3 + d) - \gamma_1 d}{2\phi_b(d + y_3)} \qquad (8)$$

Equation 8 provides the relationship between axial strains and the connection geometry (y_3, d) as well as concrete-ratio reinforcement (γ_1, γ_2). It is therefore useful when studying the effects of concrete-ratio reinforcement on the web buckling criteria, for a given connection and concrete slab geometry. Values of λ_p and λ_{ps} are obtained by substituting (8) into (1), (2) and (3). The results thus obtained are then tabulated for various geometries and reinforcement ratios in Tables 1 through 9.

CONCLUSIONS

The values of λ_p and λ_{ps} listed in Tables 1 through 9 represent the limiting width-thickness ratios between compact and non-compact sections, when designed with RGS. By comparing the h_c/t_w ratios listed in the front tables of the AISC Manual with the values of λ_p and λ_{ps} in Tables 1 through 9, sections which become non-compact become evident. Such sections are indicated thus * in the tables. For a section to become non-compact, h_c/t_w is greater than λ. For design purposes therefore, it is best to select sections with h_c/t_w less than λ_p and preferably less than λ_{ps}, or else web reinforcement is required. It is interesting to note that only W12×14, W14×22 and W16×26 became non-compact with RGS, and that only if $\gamma_2 = 1.0$ (or close to 1.0). If $\gamma_2 = 0.8$, all U.S. sections remain compact. For this reason, a value of $\phi = 0.8$ should be incorporated with the selection of the reinforcement to ensure that sections remain compact.

RECOMMENDATIONS

In lieu of such observations, it is reasonable to impose on RGS design a requirement for ductility to ensure that local web buckling does not cause premature failure of U.S. steel sections. Such a requirement is best included in a future LRFD specification in the form of a resistance factor ϕ, such that

$$A_{RF} \leq \frac{\phi z F_y}{d_s F_{yR}}$$

where

$$\phi = 0.80$$

This requirement limits the amount of concrete reinforcement to ensure that it yields first, and that web buckling does not inhibit the development strength of the composite semi-rigid connections. With this requirement, U.S. steel sections remain compact. RGS design can then be accomplished as described by the writer previously.[14]

APPENDIX I. REFERENCES

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APPENDIX II. NOTATION

The following symbols are used in this paper:

c = compression depth of steel sections
d = overall depth of steel sections
$\gamma_1; \gamma_2$ = ratio of actual to yield strain
ε_1 = bending component of strain
ε_2 = axial component of strain
ε_s = strain magnitude of steel top flange
ε_y = yield strain
h_c / t_w = web slenderness
λ_p = web slenderness limit for compact sections (rotation capacity of at least 3)
λ_{ps} = same as λ_p, but rotation capacity is at least 7
P_y = axial yield strength
P_u = axial force
ϕ_b = resistance factor
LRFD = Load and Resistance Factor Design
RGS = Restraint Girder System

Table 1.									
Y_3 = 3 in.						(γ_1 = 1 ϕ = 0.85 F_y = 50)			
Beam	γ_2 = 1			γ_2 = 0.8			γ_2 = 0.6		
	$P_u/\phi_b P_y$	λ_P	λ_{Ps}	$P_u/\phi_b P_y$	λ_P	λ_{Ps}	$P_u/\phi_b P_y$	λ_P	λ_{Ps}
W10	0.271	55.61	55.61	0.127	59.50	59.50	–0.018		
W12	0.235	56.68	56.68	0.094	67.90	62.95			
W14	0.208	57.31	57.31	0.069	73.41	65.75			
W16	0.186	57.91	57.91	0.050	78.11	67.88			
W18	0.168	58.39	58.39	0.034	82.10	69.72			
W21	0.147	58.96	58.96	0.015	86.80	71.85			
W24	0.131	59.39	59.39	0.000	90.52	73.55			

Table 2.									
Y_3 = 3.5 in.						(γ_1 = 1 ϕ = 0.85 F_y = 50)			
Beam	γ_2 = 1			γ_2 = 0.8			γ_2 = 0.6		
	$P_u/\phi_b P_y$	λ_P	λ_{Ps}	$P_u/\phi_b P_y$	λ_P	λ_{Ps}	$P_u/\phi_b P_y$	λ_P	λ_{Ps}
W10	0.305	54.69	54.69	0.157	58.69	58.69	0.000	90.5	
W12	0.266	55.74	55.74	0.121	60.46	59.86	–0.023		
W14	0.235	56.58	56.58	0.094	67.16	62.95			
W16	0.211	57.23	57.23	0.072	72.59	65.45			
W18	0.192	57.74	57.74	0.055	76.85	67.37			
W21	0.168	58.39	58.39	0.034	82.10	69.72			
W24	0.150	58.88	58.88	0.017	86.35	71.63			

Table 3.									
Y_3 = 3.75 in.						(γ_1 = 1 ϕ = 0.85 F_y = 50)			
Beam	γ_2 = 1			γ_2 = 0.8			γ_2 = 0.6		
	$P_u/\phi_b P_y$	λ_P	λ_{Ps}	$P_u/\phi_b P_y$	λ_P	λ_{Ps}	$P_u/\phi_b P_y$	λ_P	λ_{Ps}
W10	0.321	54.26	54.26	0.171	58.31	58.31	0.022	85.08	71.12
W12	0.280	55.37	55.37	0.134	59.31	59.31	–0.011		
W14	0.249	56.20	56.20	0.106	64.17	61.56			
W16	0.223	56.91	56.91	0.083	69.88	64.20			
W18	0.203	57.45	57.45	0.065	77.40	66.19			
W21	0.178	58.12	58.12	0.043	79.83	68.69			
W24	0.159	58.63	58.63	0.025	84.36	70.75			

Table 4.									
Y_3 = 4.25 in.							($\gamma_1 = 1$ $\phi = 0.85$ $F_y = 50$)		
	$\gamma_2 = 1$			$\gamma_2 = 0.8$			$\gamma_2 = 0.6$		
Beam	$P_u / \phi_b P_y$	λ_P	λ_{Ps}	$P_u / \phi_b P_y$	λ_P	λ_{Ps}	$P_u / \phi_b P_y$	λ_P	λ_{Ps}
W10	0.351	53.45	53.45	0.198	57.58	57.58	0.045	79.38	68.47
W12	0.308	54.61	54.61	0.159	58.63	58.63	0.011	87.80	72.37
W14	0.274	55.53	55.53	0.129	59.44	59.44	–0.016		
W16	0.247	56.26*	56.26*	0.105	64.45	61.70			
W18	0.225	56.85	56.85	0.085	69.42	63.98			
W21	0.198	57.58	57.58	0.061	75.40	66.70			
W24	0.177	58.15	58.15	0.042	80.11	68.84			

Table 5.									
Y_3 = 4.5 in.							($\gamma_1 = 1$ $\phi = 0.85$ $F_y = 50$)		
	$\gamma_2 = 1$			$\gamma_2 = 0.8$			$\gamma_2 = 0.6$		
Beam	$P_u / \phi_b P_y$	λ_P	λ_{Ps}	$P_u / \phi_b P_y$	λ_P	λ_{Ps}	$P_u / \phi_b P_y$	λ_P	λ_{Ps}
W10	0.365	53.07	53.07	0.211	57.23	57.23	0.057	76.39	67.15
W12	0.321	54.26	54.26	0.171	58.31	58.31	0.021	85.36	71.19
W14	0.286	55.20	55.20	0.140	59.15	59.15	0.000		
W16	0.258	55.96*	55.96*	0.115	61.91	60.53	–0.029		
W18	0.235	56.58	56.58	0.094	67.16	62.95			
W21	0.208	57.31	57.31	0.069	73.41	65.75			
W24	0.186	57.91	57.91	0.050	78.11	67.88			

Table 6.									
Y_3 = 5 in.							($\gamma_1 = 1$ $\phi = 0.85$ $F_y = 50$)		
	$\gamma_2 = 1$			$\gamma_2 = 0.8$			$\gamma_2 = 0.6$		
Beam	$P_u / \phi_b P_y$	λ_P	λ_{Ps}	$P_u / \phi_b P_y$	λ_P	λ_{Ps}	$P_u / \phi_b P_y$	λ_P	λ_{Ps}
W10	0.392	52.34	52.34	0.235	56.58	56.58	0.078	71.14	64.72
W12	0.346	53.58*	53.58*	0.194	57.69	57.69	0.042	80.11	68.84
W14	0.309	54.58	54.58	0.161	58.58	58.58	0.012	87.53	72.22
W16	0.280	55.37*	55.37*	0.134	59.31	59.31	–0.011		
W18	0.256	56.01	56.01	0.113	62.45	60.75			
W21	0.226	56.82	56.82	0.086	69.15	63.84			
W24	0.203	57.45	57.45	0.065	74.40	66.19			

Table 7.									
Y_3 = 5.25 in.							($\gamma_1 = 1$ $\phi = 0.85$ $F_y = 50$)		
	$\gamma_2 = 1$			$\gamma_2 = 0.8$			$\gamma_2 = 0.6$		
Beam	$P_u/\phi_b P_y$	λ_P	λ_{Ps}	$P_u/\phi_b P_y$	λ_P	λ_{Ps}	$P_u/\phi_b P_y$	λ_P	λ_{Ps}
W10	0.405	51.99	51.99	0.247	56.26	56.26	0.089	68.43	63.47
W12	0.358	53.26*	53.26*	0.205	57.39	57.39	0.051	77.84	67.81
W14	0.321	54.26	54.26	0.171	58.31	58.31	0.021	85.36	71.19
W16	0.291	55.07*	55.07*	0.144	59.66	59.66	0.000		
W18	0.266	55.74	55.74	0.121	60.46	59.86	−0.022		
W21	0.235	56.58	56.58	0.094	67.16	62.95			
W24	0.211	57.23	57.23	0.072	72.59	65.45			

Table 8.									
Y_3 = 5.5 in.							($\gamma_1 = 1$ $\phi = 0.85$ $F_y = 50$)		
	$\gamma_2 = 1$			$\gamma_2 = 0.8$			$\gamma_2 = 0.6$		
Beam	$P_u/\phi_b P_y$	λ_P	λ_{Ps}	$P_u/\phi_b P_y$	λ_P	λ_{Ps}	$P_u/\phi_b P_y$	λ_P	λ_{Ps}
W10	0.417	51.67	51.67	0.258	55.96	55.96	0.099	65.89	62.37
W12	0.370	52.93*	52.93*	0.215	57.12	57.12	0.060	75.58	66.78
W14	0.332	53.96	53.96	0.181	58.04	58.04	0.030	83.09	70.90
W16	0.301	54.80*	54.80*	0.153	58.80	58.80	0.000		
W18	0.275	55.50	55.50	0.140	59.15	59.15	−0.015		
W21	0.244	56.34	56.34	0.102	65.17	62.00			
W24	0.219	57.01	57.01	0.080	70.60	64.50			

Table 9.									
Y_3 = 6.5 in.							($\gamma_1 = 1$ $\phi = 0.85$ $F_y = 50$)		
	$\gamma_2 = 1$			$\gamma_2 = 0.8$			$\gamma_2 = 0.6$		
Beam	$P_u/\phi_b P_y$	λ_P	λ_{Ps}	$P_u/\phi_b P_y$	λ_P	λ_{Ps}	$P_u/\phi_b P_y$	λ_P	λ_{Ps}
W10	0.463	50.42	50.42	0.299	54.85	54.85	0.135	59.28	59.28
W12	0.413	51.77*	51.77*	0.254	56.07	56.07	0.095	66.89	62.81
W14	0.373	52.85*	52.85*	0.218	57.04	57.04	0.063	74.86	66.41
W16	0.339	53.77*	53.77*	0.188	57.85	57.85	0.036	81.55	69.50
W18	0.312	54.50	54.50	0.163	58.53	58.53	0.014	87.08	72.00
W21	0.278	55.42	55.42	0.132	59.36	59.36	−0.012		
W24	0.251	56.15	56.15	0.108	60.01	61.34			