Seismic Design of Steel Joist Girder Structures

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Figure 1 shows the typical framing system of a one-story steel joist girder structure. Joist girder structures are mainly intended to carry the vertical loads, with the lateral loads assigned to braced bays along the perimeter of the structure. The main function of the joists and joist girders in this structural system is to carry the roof or floor loads to the column members. The member size for the joists and joist girders under various gravity loads can be selected using the *Standard Specifications—Load Tables & Weight Tables for Steel Joists and Joist Girders* (hereafter, SJI Specifications) (SJI, 2002). The configurations of both joists and joist girders are highly optimized, with most members being angles with thicknesses specified down to $\frac{1}{32}$ of an inch. This characteristic makes these structures economical and light, especially for large column-free spaces. The columns are designed primarily for axial loads, resulting in slender, light columns. Resistance to lateral loads is generally provided by braces in the end frames, with allowable drifts under wind loads ranging from H/50 to H/200, where H is the height of the structure. Thus these frames are designed to considerably more liberal drift limits than typical buildings.

In the past, these structures were designed without regard to seismic loads. This approach is changing with the advent of performance-based design, as the need to protect the structure, its occupants, and its contents under earthquake loads becomes more important. In addition, as these structures tend to cover very large areas, the need to develop effective diaphragm action in the roof to carry the lateral loads to the perimeter also becomes a concern. In this context, there are clear advantages to exploiting frame action by turning all the frames into lateral-load resisting elements. Currently there are no explicit codes addressing the design of steel joist girder structures as moment-resisting frames, and only a technical digest published by SJI is available as a reference (SJI, 1999). Furthermore, special considerations should be taken because the prevailing seismic design philosophy for most building frames, the strong column-weak beam (SCWB) frame concept, cannot produce ductile behavior economically for these types of structures. For the SCWB approach it will be difficult to design a joist girder that will produce ductile behavior and avoid buckling of the joist members. Thus, adoption of a weak column-strong beam (WCSB) philosophy will be explored in this study as a possible solution for the seismic design of one-story steel joist girder frames. In one-story frames, either a SCWB or a WCSB can lead to satisfactory performance if the structure is properly designed and detailed.

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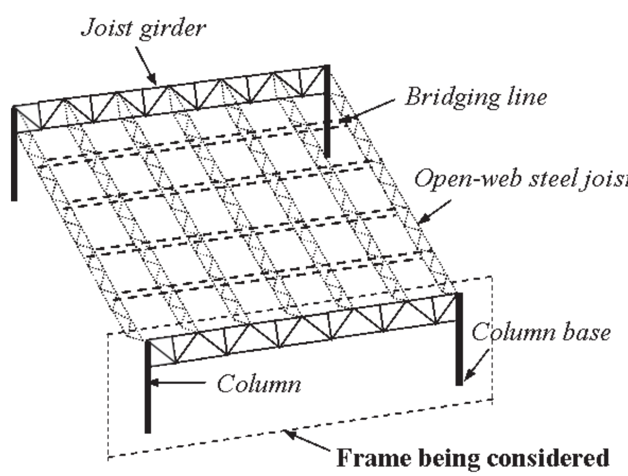

Fig. 1. One-bay, one-story steel joist girder structure.

The main objectives of this study are to develop a safe and efficient seismic design methodology for steel joist girder structures, including a set of step-by-step seismic design guidelines for use in the design office. Experimental and analytical studies aimed at verifying this procedure are described. The main steps of the design procedure are illustrated with design examples for two different seismic areas.

SEISMIC DESIGN PROCEDURE

The proposed seismic design procedure for a one-story steel joist girder structure is summarized in Figure 2. The procedure is divided into two parts. Steps 1 through 3 (see Figure 2) constitute the gravity load design and are essentially similar to what is currently done for joist girder structures without rigid connections. Steps 4 through 9 constitute the seismic design and are divided into three broad procedures: (a) the determination of the loads and the distribution of those loads to the joist girder elements and their connections (Steps 4 though 6); (b) the assessment of the performance of the system (Step 7); and (c) the detailing of the system (Steps 8 and 9). To understand the proposed procedure and the accompanying design examples, five related topics will be briefly reviewed:

1. The equivalent beam theory (EBT).

2. The conversion of forces for the joist girder design.

3. The use of plastic design to ensure the ductility of the system.

4. The design of the moment connection at the joist girder-column interface.

5. The design of the column base plate.

From the full-scale test, it was found that column instability is a major issue after achieving a WCSB mechanism. Although out-of-plane instability has a great effect on the ultimate load-carrying capacity of 3-D skeleton frames, the ability of OMF columns to withstand the limited inelasticity expected is beyond the scope of this paper. Thus, it is assumed that the frame or member is laterally braced.

The procedure will be illustrated with the design results for two 30-ft-high, three-bay frames, one designed for the Los Angeles, CA area (high seismic) and one for the Atlanta, GA area (low seismic). The design examples in this study were intended to satisfy the ordinary moment frame (hereafter, OMF) requirements in the IBC 2003 (ICC, 2003) and the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2005), hereafter referred to as the AISC Seismic Provisions. While this type of structural system is not explicitly recognized in those codes, the design process and detailing procedures developed in this project provide equal or superior performance to that of OMFs. In particular, a capacity design approach is used to ensure that a ductile mechanism controls the behavior of the structure. It should be clearly understood that the weak link in the typical joist girder system is usually the joist girder. The design and manufacture of joists or joist girders will generally result in members whose actual strength, as opposed to nominal strength, is difficult to calculate and whose behavior is controlled by buckling of the diagonal members. Thus the design procedure developed herein intends to find an upper bound to the forces introduced into the joist to allow for an elastic design of that member. As for a typical OMF, the connections will also be designed for that level of force, except in this case the column is taken as the yielding element.

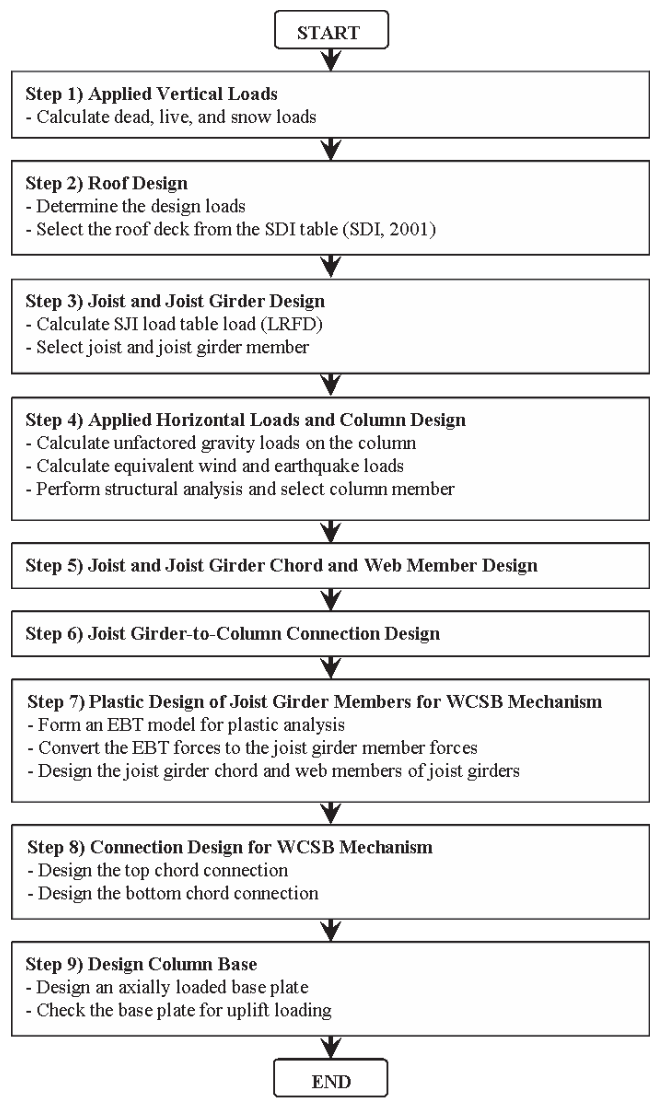

Fig. 2. Flowchart for proposed seismic design procedure.

Table 1. Design Loads

	Location	Dead Loads (per column)	Roof Live Loads	Snow Loads	Wind Loads [a]	Earthquake Load [b]
3-bay	Los Angeles, CA	14.1 kips (62.7 kN)	0.067 kips/in (11.7 kN/m)	0.000 kips/in (0.0 kN/m)	5.8 kips (25.8 kN)	32.3 kips (143.7 kN)
	Atlanta, GA	14.1 kips (62.7 kN)	0.067 kips/in (11.7 kN/m)	0.017 kips/in (3.0 kN/m)	6.5 kips (28.9 kN)	6.1 kips (27.1 kN)

[a] Considering wind directionality factor.
[b] Based on a period of 0.6 s for strength design.

A typical plan and elevation of a three-bay joist girder system is shown in Figures 3 and 4. For the two sites, a comparison of the lateral loads from Table 1 is instructive. The ratio of earthquake to wind loads ranges from roughly 0.94 in a low seismic zone (Atlanta, GA) to almost 5.6 for a high seismic zone (Los Angeles, CA). The earthquake loads were taken from IBC 2003 (ICC, 2003), and the other design loads were taken from ASCE/SEI 7 (ASCE, 2005). The seismic loads shown are based on the code-prescribed provision for strength, using a natural period for the structure as limited by the code (0.6 s) that is vastly different from that given by a more rational analysis (1.56 s). The code allows the latter period to be used in calculating the forces needed for drift calculations. The lateral forces for drift calculations using the computed period are considerably smaller (13.7 kips compared to 32.3 kips for the Los Angeles structure). Even though a "service earthquake design approach" based on a 10% probability of exceedance in 50 years may be more reasonable for these structures, a "maximum credible earthquake approach," based on a ⅔ of the 2% probability of exceedance in 50 years limit, was used for these design examples because current seismic design codes (ICC, 2003; ASCE, 2005) only address the latter.

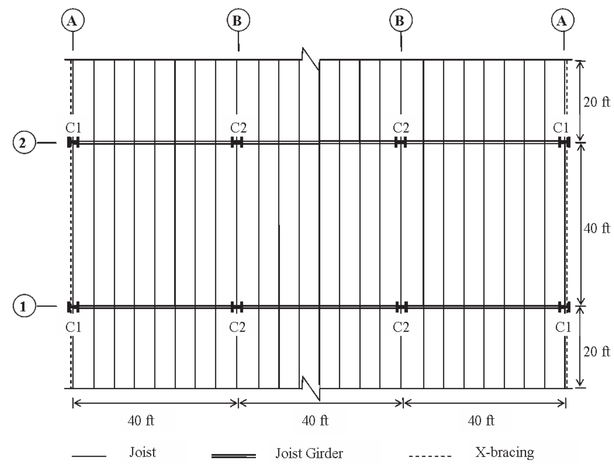

Fig. 3. Plan of a three-bay frame.

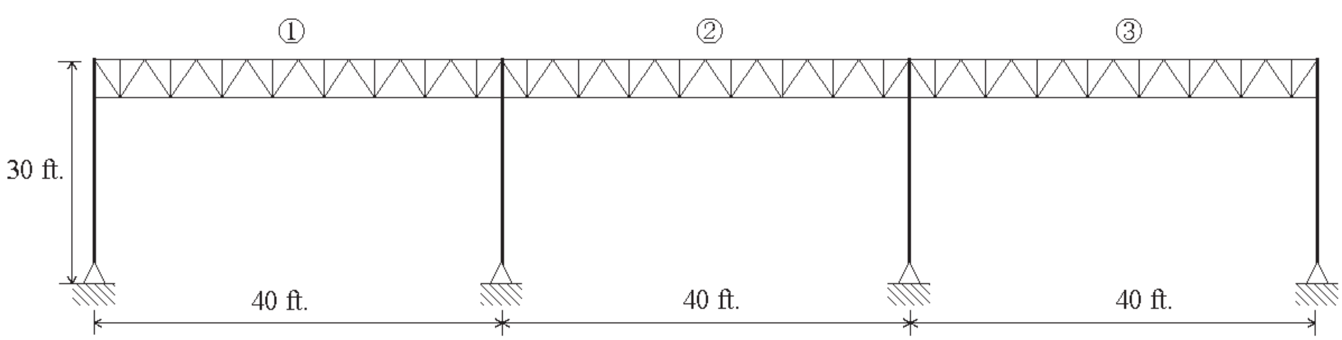

Fig. 4. Elevation of a three-bay frame.

Equivalent Beam Theory

To simplify the lateral load analysis for a joist girder structure, an equivalent beam theory (EBT) model can be used. In the EBT, the intricate joist girder is replaced with an equivalent moment of inertia, I_{eq}, beam. The equivalent moment of inertia for the joist girder is approximated by

$$I_{eq} = 0.027 \times N \times P_{npp} \times S_{jg} \times d_{jg} \tag{1}$$

where

I_{eq} = equivalent moment of inertia, in.^{4}

N = number of joist spaces

P_{npp} = panel point load designation, kips

S_{jg} = joist girder span, ft.

d_{jg} = effective joist girder depth, in. (see Figure 5)

The panel point load comes from the design of the perpendicular roof joists, while the joist girder depth is specified by the engineer. A joist girder depth equal to approximately one-tenth of the span has been found to typically provide an economical joist girder design. However, other constraints may govern the selection of the joist girder depth. Equation 1 (SJI, 2002) provides an approximate moment of inertia based on satisfying strength and deformation criteria under gravity loads. The usefulness and robustness of the EBT have been ascertained by Beckman (1996).

Using the EBT model, the end moments for the joist girder can be readily obtained using a conventional frame analysis program or hand calculations, without the need to model the joist girder members explicitly. Figure 6 shows an example of the application of the EBT model. Figure 6a shows the loading and Figure 6b shows the resulting forces for a one-story structure in Boston, MA, for which the critical load combination is $1.2D + 1.3W + 0.5S$ as given in older versions of ASCE 7. This wind load is almost exactly the same as given in the newer ASCE 7 versions, which includes a load

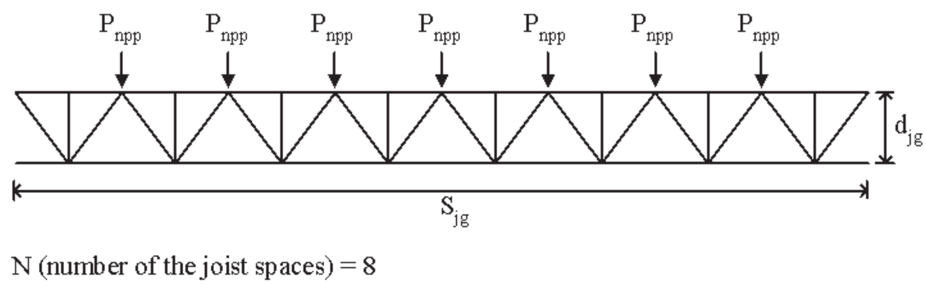

Fig. 5. Definition of dimensions for the equivalent moment of inertia (I_{eq}) *of the joist girder.*

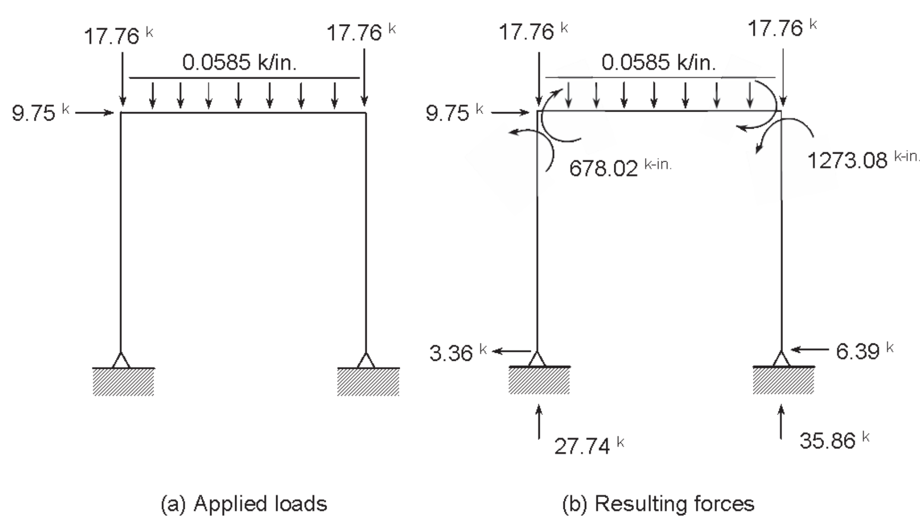

Fig. 6. Example of an EBT model for 1.2D + 1.3W + 0.5S.

Table 2. Main Structural Members					
	Location	Joist	Joist Girder	Column in. × lb/ft (mm × kg/m)	
3-bay	Los Angeles, CA	24K4	40G8N8K	Exterior	W14×68 (W360×101)
				Interior	W14×74 (W360×110)
	Atlanta, GA	24K4	40G8N8K	Exterior	W14×43 (W360×64)
				Interior	W14×61 (W360×91)

factor for wind of 1.6 but includes a directionality factor that is commonly taken as 0.85 (ASCE, 2005).

In the typical construction sequence for a steel joist girder structure in the United States, the bottom chord and stabilizer plate are welded after the dead loads have been applied to the structure. This sequence minimizes the dead load moments to the column but requires that the resulting continuity be included in the analysis (SJI, 2002). The moments, shears, and axial loads obtained from the EBT can then be used to proportion the columns. The columns are checked using the usual interaction equations in the AISC *LRFD Specification for Structural Steel Buildings* (AISC, 1999). Table 2 shows the joist, joist girders, and columns selected for the design examples.

Conversion of Forces

To determine the required size of the elements of the joist girder, special consideration should be taken when addressing the moment induced in the joist girders. The free body diagram in Figure 7 shows how the EBT member forces can be converted to the joist girder element forces. The design procedure for the joist girder is as follows:

1. Convert the EBT member forces (Figure 7a) to the joist girder element forces (Figure 7b);

2. Build a truss model with a simply supported condition for the joist girder;

3. Analyze the truss model using a linear elastic analysis considering the joist girder element force determined in step 1;

4. Select the appropriate angle sections for the joist girder members and specify appropriate welds.

Table 3 shows the angle sizes selected for the joist girder elements for the design examples. The elastic capacity of the members in Table 3 needs to be checked against the forces given by the plastic collapse assumed by the WCSB mechanism since they are assumed to remain elastic throughout the loading history.

Plastic Design with a WCSB Mechanism

For joist girder structures designed as part of an OMF system as described below, it is acceptable to use the seismic response modification coefficient, R, equal to 3.5 and deflection amplification factor, C_d, equal to 3.0 prescribed for tra-

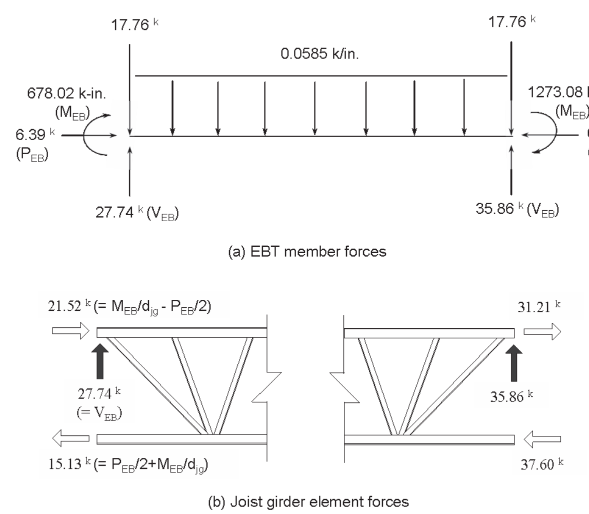

Fig. 7. Conversion of forces for 1.2D + 1.3W + 0.5S.

Table 3. Nominal Joist Girder Member Sizes				
	Location	Bay No. [a]	Top Chord in. (mm)	Bottom Chord in. (mm)
3-bay	Los Angeles, CA	1, 3	2L3×3×½ (2L76×76×12.7)	2L4×3×½ (2L102×76×12.7)
		2	2L3×3×¼ (2L76×76×6.4)	2L3×3×¼ (2L76×76×6.4)
	Atlanta, GA	1, 3	2L3×3×5⁄16 (2L76×76×7.9)	2L3×3×⅜ (2L76×76×9.5)
		2	2L3×3×¼ (2L76×76×6.4)	2L3×3×¼ (2L76×76×6.4)

[a] Refer to Figure 4.

ditional OMF systems (ICC, 2003). These are based on the assumption of limited ductility in the connections and yielding of the members without global buckling. The design of the connections and the critical joist girder element is based on a strength (or capacity) design approach. In a strength design approach, members are sized based on a reasonable approximation of the expected maximum forces. The maximum force that can be delivered by the column, $V_{u,top}$, to the joist is given by

$$V_{u,top} = \frac{\sum M_{pc}}{h} = \frac{(K)(1.1R_y M_{pc})}{h} \tag{2}$$

where

R_y = ratio of the expected yield stress to the minimum specified yield stress (taken as 1.1 for A572 Grade 50 steels from the AISC *Seismic Provisions*)

M_{pc} = nominal plastic flexural strength of the connected column

The shear from Equation 2 is intended to be an upper bound to the forces from the column yielding, with the additional factor K intending to account for the column base fixity (see Figure 8). For a nominally pinned base connection K = 1.0 (in other words, full moment at the top of the

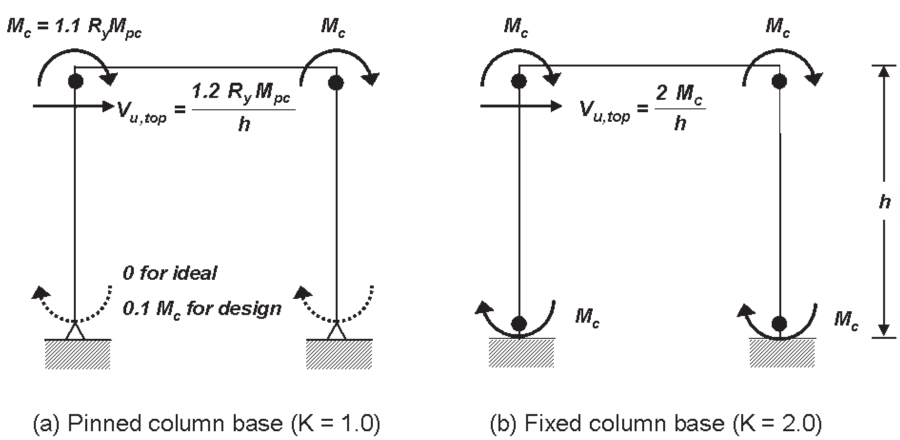

Fig. 8. Theoretical K *factors for the column plastic capacity.*

column and zero at the bottom). Such connections cannot be designed in practice, and for a typical pinned connection K should be taken as 1.1. If the column base is rigid, K = 2.0 (in other words, full moment at the top and bottom of the column). Because the plastic hinges in the columns are not expected to occur until very large deformations arise, little strain hardening is expected and thus the yield and not the tensile strength values are used in Equation 2. This assumption was verified in the full-scale tests. This lateral load is combined with the applicable gravity loads given by $1.2D + 0.5L + 0.2S$ acting on the simply supported joist girder. The yielding zone of the column should be properly braced as required by the AISC Seismic Provisions.

Joist Girder-to-Column Connection Design

For the joist girder-to-column connection (see Figure 9), one type of fully-restrained moment connection (Fisher, West, and Van de Pas, 2002) was chosen. The design procedure of the top chord connection requires separate designs for the top plate, seat, and stiffener design, and an end support check for shear transfer. The design procedure for the bottom chord connection requires the design of the stabilizer plate, the determination of the weld size, and a check for column web yielding and crippling.

Column Base Design

Two broad types of column base plates are available: thin, flexible base plates and thick, rigid base plates. A unified design method (Thornton, 1990) for base plates is used herein. This design method has been incorporated into the AISC *LRFD Manual of Steel Construction* from the 2nd Edition (AISC, 1994) onward. To guarantee a WCSB mechanism, the typical required design strength of 1.1 $R_y M_{pc}$ needs to be increased to 1.2 $R_y M_{pc}$ to account for the axial force due to column base moments, as the pinned column base can be assumed to have a flexural strength not more than 10% of the column flexural strength (Kim, Leon, and Galambos, 2007). Alternatively, a frame analysis with trussed girder modeling can be performed with partially restrained (PR) column base conditions. In this case, the stiffness and resistance of the column base can be estimated by the component method (Kim, 2003; Kim, Leon, and Galambos, 2006). Since the column base fixity can have a significant effect on the frame behavior, it is necessary to estimate the stiffness and resistance of the PR column bases with reasonable accuracy.

VERIFICATION OF A NEW DESIGN PROCEDURE

Extensive experimental and analytical studies were performed to verify the expected frame behavior, that is, to ensure the formation of a WCSB mechanism under various levels of seismic excitations. As an experimental verification, a full-scale cyclic test was performed for the one-bay frame as shown in Figure 10. The height of the test frame was 18 ft (5.5 m), and the width was 40 ft (12.2 m). The test frame was composed of two plane joist girder frames connected with open-web steel joists and a metal roof deck. More detailed information related to the full-scale cyclic test can be obtained from the companion paper (Kim et al., 2006). The intended WCSB mechanism of the test frame was successfully achieved with the formation of plastic hinges at the column as shown in Figure 11. Analytical verification was also carried out through the use of pushover analyses. Figure 12

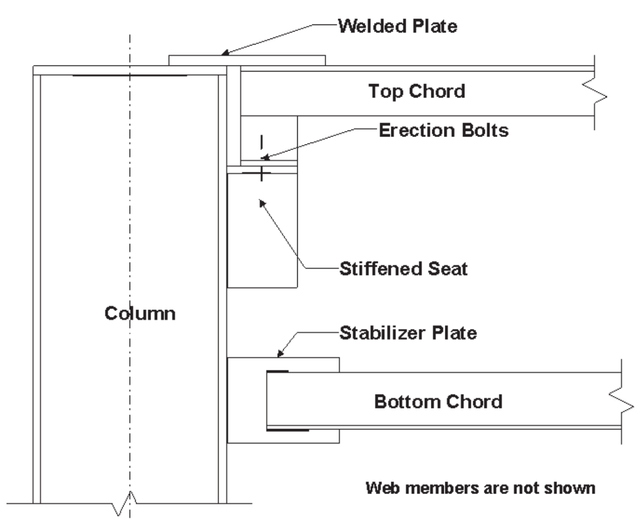

Fig. 9. Typical joist girder moment connection.

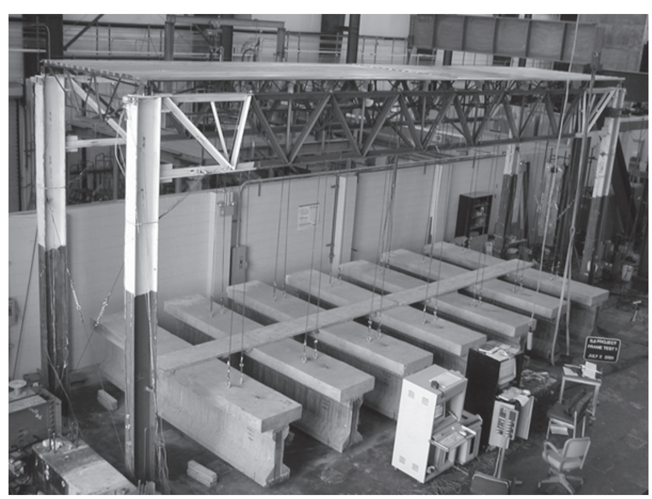

Fig. 10. Overview of test setup.

Table 4. Drift Ratios for Nonstructural Drift-Sensitive Components (FEMA, 1999)			
Drift Ratio at the Threshold of Nonstructural Damage			
Slight	Moderate	Extensive	Complete
0.004	0.008	0.025	0.050

shows the pushover curve for this structure for the case of pinned bases. Figure 13 shows a similar curve for the three-bay structure described in the design example. Both of these curves serve to illustrate the great flexibility and additional design strength of these frames. The formation of the first hinge occurs at drifts exceeding 2% and the systems achieve their ultimate strength at deformations greater than 3%. Figure 13 also illustrates a comparison between the code-prescribed forces based on a code-limited period and $R = 3.5$ and those from using an elastic design spectrum (period not limited and $R = 1$) for the three-bay example frame in Los Angeles. The elastic drift at the drift force level of 13.7 kips (corresponding to a period of 1.56 sec.) is 0.88%, which when amplified by C_d yields a total drift of 2.64% or slightly larger than the 2.5% allowable. Finally, representative values of allowable drift to limit nonstructural damage, as given in Table 4 (FEMA, 1999), are also included in the figure. It is clear that significant nonstructural damage will occur before any structural yielding unless the skin of the building and any attached contents are designed to accommodate large drifts.

DESIGN RECOMMENDATIONS

From this paper, several design recommendations can be proposed for the design of steel joist girder structures:

1. For the calculation of design loads, ASCE/SEI 7 (ASCE, 2005) should be used as a reference design code as only those load combinations were used to proportion the structures designed during these studies.

2. To obtain a ductile behavior, adoption of a weak column-strong beam philosophy can be a reasonable solution for the seismic design of one-story steel joist girder frames.

3. The joist girder-to-column connections should be designed as a fully restrained connection to achieve the WCSB mechanism.

4. To guarantee a WCSB mechanism for the frame with PR column bases, the connection of the joist girder-to-column, and the joist girder should be designed for the maximum force from the proposed design strength of $(K)(1.1R_y M_{pc})$ for both the joist girder and connection design.

RESEARCH NEEDS FOR CODE IMPROVEMENT

Two areas require further study before the requirements of the proposed design method can be made less conservative:

- *Elastic Seismic Design*: Current seismic design provisions emphasize detailing that results in large system

Fig. 11. Plastic hinge formation on the column.

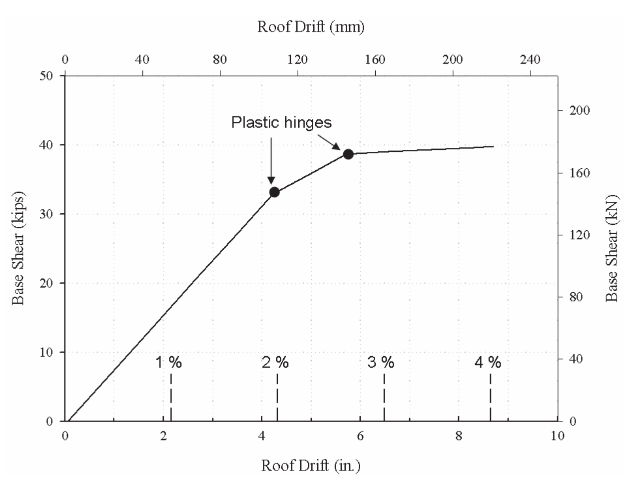

Fig. 12. Pushover curve for one-bay frame.

ductility and energy dissipation. However, in the case of single-story buildings, there may be no need to design for large ductility demands. Thus a SCWB concept may not be the best approach for flexible structures that may be able to withstand large earthquakes in the elastic range. From previous studies (Mays, 2001; Kim, 2003), all analyses show very limited inelastic action. Thus, an elastic seismic design method for steel joist girder structures should be developed, utilizing the actual period of the structure and appropriately lower R and C_d values.

- *Drift Limitations*: The allowable drift should be chosen based on the need to avoid nonstructural damage. Careful attention must be paid to the nonstructural and cladding elements as well as the primary structural system. In FEMA 356 (FEMA, 2000), the suggested drift ratio limits are 0.02 for the life safety nonstructural performance level and 0.01 for the immediate occupancy nonstructural performance level.

CONCLUSIONS

From this study, the following conclusions can be made:

1. A WCSB methodology is suggested as a safe and efficient seismic design procedure for joist girder structures whereby hinges form in the columns and the joist girder remains elastic. A set of step-by-step seismic design guidelines have been developed and illustrative design examples provided.

2. The result from a full-scale cyclic test and analyses (Kim et al., 2007) showed that the intended WCSB mechanism can be successfully achieved and that the behavior of the frame was almost entirely elastic in the joist girder.

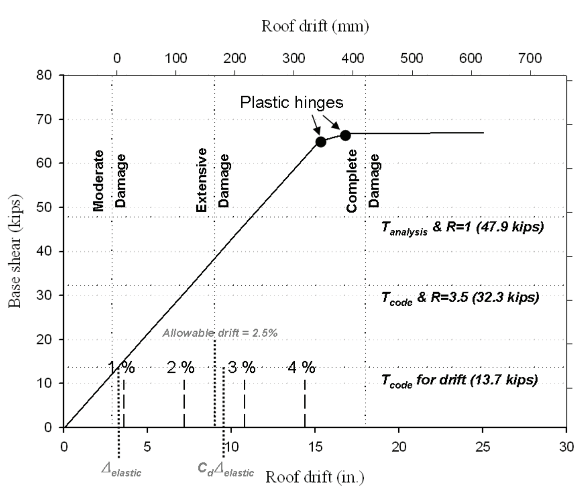

Fig. 13. Pushover curve for three-bay frame.

3. Pushover analyses indicate that the desired collapse mechanism for the one-bay and three-bay frames designed according to the proposed design procedure can be obtained.

4. To guarantee a WCSB mechanism for steel joist girder structures, the required design strength of 1.1 $R_y M_{pc}$ currently recommended by the AISC Seismic Provisions (AISC, 2005) needs to be increased to $(K)(1.1R_y M_{pc})$, with K = 1.1 for nominally pinned base connections, and K = 2.0 for fixed base connections.

ACKNOWLEDGMENTS

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NOTATION

C_d	=	displacement amplification factor
D	=	dead loads
d_{jg}	=	effective joist girder depth
I_{eq}	=	equivalent moment of inertia
K	=	factor to account for the column base fixity
L	=	live loads
M_{EB}	=	equivalent beam end moment
M_{pc}	=	nominal plastic flexural strength of the connected column
M_u	=	required flexural strength
N	=	number of joist spaces
P_{EB}	=	equivalent beam axial force
P_{npp}	=	panel point load designation
R	=	seismic response modification coefficient
R_y	=	ratio of the expected yield stress to the minimum specified yield stress
S	=	snow loads
S_{jg}	=	joist girder span
V_{EB}	=	equivalent beam end shear
$V_{u,\,top}$	=	maximum force that can be delivered by the column
W	=	wind loads

METRIC CONVERSION FACTORS

1 in.	=	25.4 mm
1 ft	=	0.3048 m
1 kip-in.	=	113000 N-mm
1 kip	=	4.448 kN
1 kip/in.	=	175 kN/m

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