

DISCUSSION

Flexible Moment Connections for Unbraced Frames Subjected to Lateral Forces—A Return to Simplicity

Paper by LOUIS F. GESCHWINDNER and ROBERT O. DISQUE
(Second Quarter, 2005)

Discussion by Christopher M. Foley and John A. Schaad

The authors are to be commended for presenting a very well documented discussion and application example of the flexible moment connection (FMC) approach for low-rise steel building frames. The authors provide the reader with an excellent history of the approach and theoretical basis for its development in addition to providing a clear example illustrating application. This discussion aims to give further support to the authors' approach through providing the results of a series of nonlinear static analyses of the partially restrained frame designed in the original manuscript. The discussion results will further justify the flexible moment connection approach for low-rise frames in zones of low to moderate seismicity. Furthermore, the inelastic analysis results provided will give support to use of the FMC methodology for planar frames with pin-connected bases.

The authors present the design and manual analysis of a two-story, four-bay, partially restrained frame using portal-method-type assumptions based upon a priori knowledge related to the behavior of the framework under gravity and lateral loading. This frame is restated for completeness of discussion in Figure 1. The yield stress of all members in the frame was taken as 50 ksi, and the connection elements were assumed to have a yield stress of 36 ksi. A connection design was not given in the original manuscript, so the discussers utilized the work of Kishi and Chen (1998) to develop nonlinear moment rotation models for subsequent

distributed plasticity analysis. These three-parameter power models for the connections used in this discussion are given in Figure 2. The initial stiffness and moment capacities for the connections are also given in the figure. As described in the original paper, these connections are quite stiff (at least with reference to their initial stiffness). However, the moment-rotation curves degrade quite rapidly as a result of the shape parameter n . Thus, it is likely that the initial stiffness is a nonconservative option for linear connection models. A stiffness magnitude generated with a beam-line approach is likely more appropriate, but may not be necessary.

To evaluate the connection demands and ultimate load factors corresponding to the LRFD factored gravity and lateral load combinations, the discussers carried out an inelastic analysis of the framework shown in Figure 1 with the nonlinear connection models shown in Figure 2. Two factored loading combinations were considered:

$$\gamma (1.2D + 1.6L) \quad (1)$$

$$\gamma (1.2D + 0.5L + 1.3W) \quad (2)$$

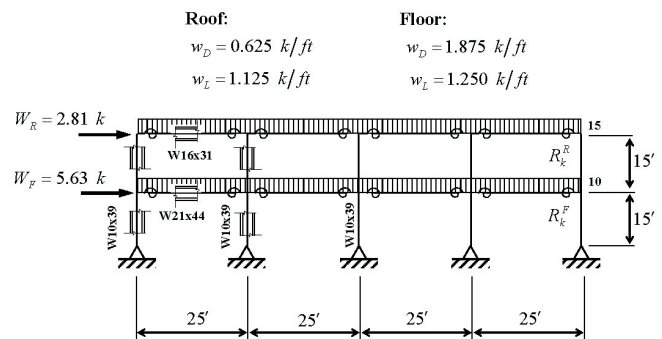


Fig. 1. Base frame topology with member sizes.

Christopher M. Foley is an associate professor; department of civil and environmental engineering, Marquette University, Milwaukee, WI.

John A. Schaad is formerly a graduate research assistant, Marquette University, Milwaukee, WI.

The inelastic analysis utilized distributed plasticity modeling (Foley and Vinnakota, 1999), which included the common linear residual stress distribution across the flanges and constant residual tension stress through the web. Twenty finite elements were used to model column members, and 30 finite elements were used to model beam members. The goals for these analyses were to illustrate the ultimate load factors (γ_u) that the frames, as designed, could attain and to evaluate the connection demands and member yielding at designer target levels ($\gamma = 1.0$).

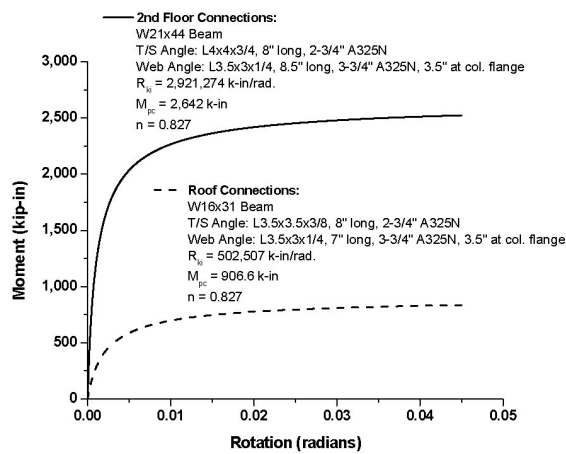


Fig. 2. Three-parameter power model representations of the roof and floor connections used in the inelastic frame analysis.

The load deformation response for the two factored load combinations shown above is provided in Figures 3 and 4 (nodes 10 and 15 are shown in Figure 1). As indicated, the ultimate load factors for the factored load combinations are well above the target levels $\gamma = 1.0$ of that would be used to assess the strength of the frame. Figure 3 illustrates that the gravity load response is highly nonlinear up to the strength limit state. The distributed plasticity analysis indicated yielding does occur in the members of this frame at $\gamma = 1.0$ with the load combination given by Equation 1 applied. However,

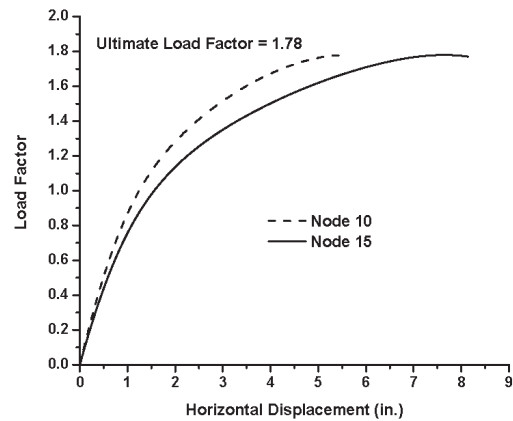


Fig. 4. Load-deformation response (upper right-most node in model) for factored gravity loading combination: $\gamma_u (1.0D + 0.5L + 1.3W)$.

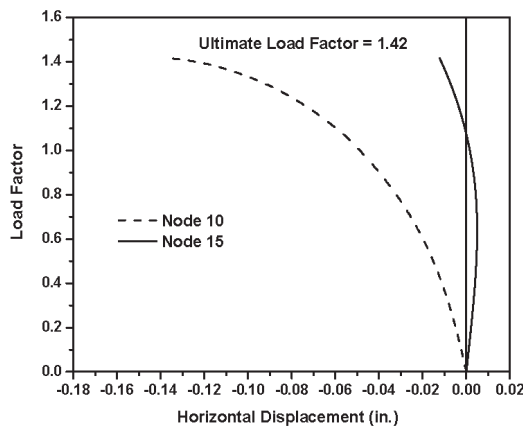


Fig. 3. Load-deformation response (upper right-most node in model) for factored gravity loading combination: $\gamma_u (1.2D + 1.6L)$.

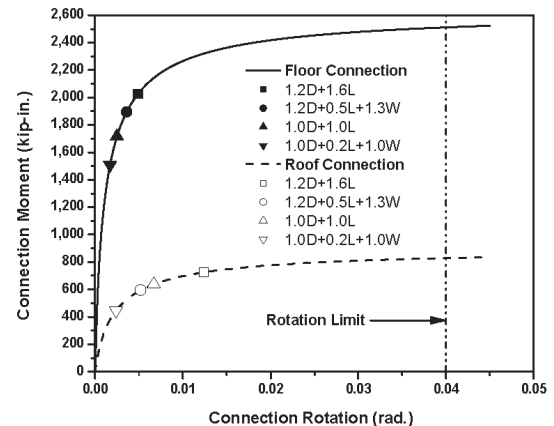


Fig. 5. Connection demands with factored load combinations and service loading combinations applied ($\gamma = 1.00$ in all cases).

the magnitude of this yielding is less than 0.5% of the cross-sectional area. The nonlinear response up to $\gamma = 1.0$ is therefore due to connection and geometric nonlinearity. When the factored load combination described by Equation 2 is applied incrementally up to $\gamma = 1.0$, the lateral load behavior shown in Figure 4 was accompanied by no yielding in any members, and the nonlinear response was again due to connection and geometric nonlinearity. The ultimate load factor for the lateral load combination computed using distributed plasticity analysis again surpasses the target level of $\gamma = 1.0$. The FMC approach assumes that “yielding” is isolated in the connections at load levels corresponding to $\gamma = 1.0$. This behavior is indeed seen in the inelastic analysis results, and yielding of the members does not begin to occur until γ_{ult} is approached.

The connection demands at factored and service load levels are given in Figure 5. The factored load combinations at strength design load levels ($\gamma = 1.0$ for all combinations) resulted in moderate demands placed on the connections. An often-used rotation limit of 0.04 radian is also provided in the figure. No connections are in any danger of violating this rotation limit. The roof connections rotate significantly into the nonlinear regime under service gravity loads ($1.0D + 1.0L$), which would imply some permanent deformation in the connections upon unloading. The connection demands under service lateral loads ($1.0D + 0.2L + 1.0W$) also extend into the nonlinear regime. This is expected using the flexible moment connection methodology, and shakedown to the initial linear branch is expected with subsequent reloadings. The connections perform very much within the assumptions of the FMC methodology, and no rotation limits at factored load levels are exceeded.

Serviceability (deflection) was checked using a pin-based analytical model with fully restrained (FR) and partially restrained (PR: linear and nonlinear) connections. SAP2000 (CSI, 2004) was used to analyze the pinned base frame with linear connections. The stiffness of the connections at each end of the beams was set to the connection initial stiffness shown in Figure 2. The interstory drift at the first floor level was 0.67 in., which exceeds an $H/300$ interstory drift limit. The overall building drift was 0.78 in. Therefore, overall

drift would meet an $H/400$ limit, but the pinned bases in the model result in traditional interstory drifts being violated. Interstory drift with nonlinear connections was 0.83 in. at the first story and 0.17 in. at the second story for service load combination: $1.0D + 0.2L + 1.0W$. The British multiplier of 1.5 (Salter, Couchman, and Anderson, 1999) used with linear frame analysis results and FR connections yield conservative estimates of interstory drift: 1.00 in. and 0.17 in. at the first and second stories, respectively. If more realistic base connections (for example, unpinned) were utilized, an interstory drift limit at the first story level of $H/400$ could likely be satisfied. Vertical deformations in the roof and floor beams using nonlinear three-parameter power model connections were 0.87 in. and 0.56 in., respectively, at service gravity load levels corresponding to the load combination: $1.0D + 1.0L$. These both meet $L/360$ vertical deflection limits 1.0 commonly used (roof is arguably close).

It is hoped that these additional data provide enhanced support for designers to embrace the simplicity afforded by the FMC approach and the safe, serviceable, and economical designs that can result from its application. However, one should always be mindful of the fundamental assumptions upon which the FMC approach is based.

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