QUANTIFYING INHERENT ROBUSTNESS IN STRUCTURAL STEEL FRAMING SYSTEMS

Christopher M. Foley[1], Kristine Martin[2] and Carl Schneeman[3]

[1]*Marquette University, Haggerty Hall 267, 1515 W. Wisconsin Ave., Milwaukee, WI 53233, USA, E-mail: chris.foley@marquette.edu*
[2]*Marquette University, Currently: Framatone ANP, Inc., Naperville, IL 60563, USA*
[3]*Marquette University, Currently: Walker Parking Consultants, Minneapolis, MN 55416, USA*

Introduction

It is well known that Vierendeel action in multi-story steel frames can be a source of inherent robustness and will provide a significant measure of general structural integrity in structural steel moment resisting framing systems. In situations where there is insufficient number of stories above a compromised column, or when simply-connected floor framing is assumed as in the case of most modern steel framing systems, the robustness inherent in the system remains to be fully understood. Grierson *et al.* (2005) points out those progressive collapse scenarios in steel structures that begin with upper-story columns becoming ineffective. The collapse sequence associated with these scenarios is a propagation of failures down the structural system to the ground level through debris loading accumulation.

Many analytical efforts to date only consider components found in the structural steel skeleton. Furthermore, the analytical models assumed that pin-connected beams and girders existed at interior columns and these analytical models did not support analysis considering ineffective interior columns, interior girders, exterior girders, or in-fill beams. If robustness in the structural steel framing system is to be quantified, the analyses must go beyond the simple removal of columns around the perimeter of the framework.

The objectives of this paper are to provide a brief overview of the methodologies that have been proposed and validated via experimental testing for quantifying the catenary and membrane mechanisms in concrete floor framing systems; and outline a new methodology for quantifying the membrane and catenary capacity in structural steel floor framing systems along with high-level provisions for ensuring structural integrity through the preservation of catenary and membrane action.

Membrane Action in Concrete Floor Systems

Researchers in the field of reinforced concrete have had a long history of attempting to understand the tensile behavior of structural concrete slab systems and proposing methodologies for quantifying the beneficial effects of catenary action and membrane action. Much of the research conducted in this regard has made its way into ACI 318 provisions for general structural integrity (Hawkins and Mitchell 1979; Mitchell and Cook 1984). Researchers studying the response of structural steel systems to fire have also begun in earnest to understand and capitalize on the inherent robustness present in steel framing systems that is contributed by the concrete deck (Allam *et al.* 2000; Bailey *et al.* 2000; Huang *et al.* 2003a; Huang *et al.* 2003b).

It has been long recognized that flat plate concrete floor systems have the potential to suffer from disproportionate collapse from a rather simplistic event: punching shear failure at interior and exterior columns (Hawkins and Mitchell 1979; Mitchell and Cook 1984). There was a series of systematic efforts carried out to develop design procedures that could limit the probability of a punching shear failure leading to progressive collapse. The first of such efforts was that conducted by Hawkins and Mitchell (1979). Mitchell and Cook (1984) enhanced this methodology to include procedures that allow quantifying the role of catenary action in the behavior of the floor system and its partnering with membrane action to mitigate progressive collapse in concrete floor system.

M. Pandey et al. (eds), Advances in Engineering Structures, Mechanics & Construction, 239–254.

When a concrete floor plate is loaded to the point of inelastic behavior, there is a tendency for the bottom fibers (assuming loading is from the top) to lengthen. This lengthening, however, is restrained by the concrete slab at the perimeter of the panel being loaded. Of course, steel beams in the systems considered in the present study will provide restraint to this outward movement. In the purely theoretical sense, the concrete slab will have a load versus vertical deflection response that exhibits snap through prior to the formation of membrane tension in the system. This *hanging net* effect cannot take place without significant vertical deformation in the floor system. In the hanging configuration, all sections through the floor plate are subjected to tensile forces and it is imperative that properly developed tension reinforcement exists in the slab and vertical support at the panel edges be maintained.

The beauty of the work of Hawkins and Mitchell (1979) is that the expressions for computing the membrane capacity of concrete floor panels are rather simplistic and include a significant amount of *engineering feel*. The fundamental assumption of the proposed methodology is that the deformed membrane between supports follows a circular shape. This makes the mathematics tractable and errors are minor when compared to the more correct catenary parabola. The basic slab system and membrane forces considered are schematically shown in Figure 1.

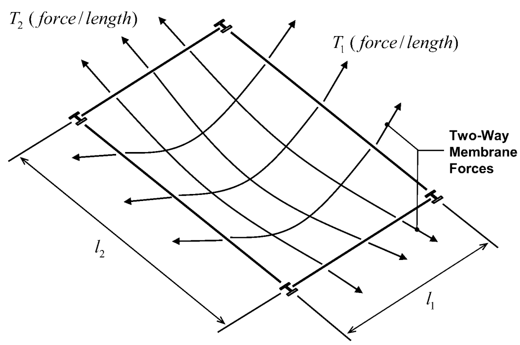


Figure 1 Two-Way Membrane Action in Reinforced Concrete Slab.

Two slab span directions are assumed: the first is defined as the short direction, l_1; and the second is termed the long direction, l_2. The reinforcement area on a per unit length basis in the short and long directions are A_{s1} and A_{s2}, respectively. The normal strains in the fibers of the membrane are assumed to be uniform over the membrane thickness and are functions of its curvature. Uniformly distributed loading over the surface of the membrane is assumed and positive loading is taken to be downward. Membrane tension forces (edge tensions) per unit length parallel to the short and long directions are T_1 and T_2, respectively. These forces are assumed to be in the direction tangent to the deformed membrane's mid-surface at the edges.

A typical structural mechanics solution procedure (*e.g.* imposition of vertical equilibrium, ensuring compatibility of deformations, and adherence to constitutive laws for the material) is employed to develop a relationship for the capacity of the tensile membrane that is a function of the edge tension, strain in the membrane (and therefore, vertical deflection) and the panel dimensions. When the panel dimensions differ (*i.e.* they are rectangular) the membrane capacity of the panel based upon the tensile reinforcement capacity at the edges can be written as (Hawkins and Mitchell 1979),

$$w_{edge} = \frac{2T_1 \sin\left(\sqrt{6\varepsilon_x}\right)}{l_1} + \frac{2T_2 \sin\left(\frac{l_1}{l_2}\sqrt{6\varepsilon_x}\right)}{l_2} \tag{1}$$

where: ε_x is the tensile strain in the membrane fibers parallel to the short direction, which is the dominant membrane direction. If the slab panel is square, there is no dominant direction. As l_2/l_1 increases, the slab panel begins to behave as a single direction membrane (*i.e.* a catenary).

Concrete slab systems quite often have different reinforcement patterns at the edges than that found in the *middle strip* areas within the panel span. As a result, if the mid-span reinforcement controls the tensile capacity of the membrane, the vertical load carrying capacity is (Hawkins and Mitchell 1979),

$$w_{pos} = 2\sqrt{6\varepsilon_x} \cdot \left[\frac{T_1'}{l_1} + T_2' \cdot \frac{l_1}{l_2^2} \right] \tag{2}$$

where: T_1' and T_2' are the tensile membrane forces per unit length within the mid-span (positive moment) regions of the panel parallel to the short- and long-directions, respectively.

The strain in the direction parallel to the short and long dimensions of the panel is related to one another as a result of the assumed circular shape of the membrane. If one knows the strain in the direction parallel to the short dimension, the strain in the direction parallel to the long dimension is computed using (Hawkins and Mitchell 1979),

$$\varepsilon_1 = \varepsilon_2 \cdot \left(\frac{l_2}{l_1} \right)^2 \tag{3}$$

Therefore, once the strains in the two directions are computed (short direction assumed, then long direction computed), the constitutive laws for the reinforcement can be used to determine the state of stress and then the tensile membrane forces on a per foot basis follow.

Once the strain in the direction parallel to the short direction is known, the maximum deflection within the panel can be computed using (Mitchell and Cook 1984),

$$\delta = \frac{3 l_1 \varepsilon_1}{2 \sin\left(\sqrt{6\varepsilon_1}\right)} \tag{4}$$

The vertical deflection is important when assessing the capacity of the membrane. Assuming end anchorage is present, the membrane is capable of carrying more loading in a highly deflected configuration for a fixed tensile force capacity. Therefore, if a large amount of loading is present and there is a fixed tensile capacity for the reinforcement in the membrane (assuming no rupturing of the reinforcement), then there is a tendency for the membrane to continue to deflect vertically to generate greater vertical components in the catenary forces. Therefore, the vertical deflection given by equation (4) can be used to determine if a slab panel will become debris loading for a panel below, or will impede modes of egress from the structure.

Mitchell and Cook (1984) provide an enhanced description of the post-failure response of concrete slab structures that is pertinent to situations considered in the present study. The response of a slab structure after initial failure depends upon the amount and details of the steel reinforcement, the vertical support conditions and the horizontal restraint conditions at the panel edges (Mitchell and Cook 1984). When the slab panel has vertical support surrounding its edges (*e.g.* steel beams at the perimeter of the panel), the slab is capable of providing its own in-plane compression ring restraint conditions at the perimeter. This compression ring helps to resist the horizontal component of the maximum tensile forces. If the edges of the panel are allowed to deform vertically, then this compression ring cannot form.

When "stiff" beams are present at the perimeter of the slab panel, the membrane action in the slab panel facilitates the slab system hanging off the perimeter beams. When an interior slab panel is considered, the adjacent regions of the floor system will help to restrain the edges of the overloaded panel. Edge or corner panels can develop the necessary compression ring behavior if the edges are supported by beams that have significant flexural stiffness when compared to the slab itself.

Membrane Action in Composite Deck Structural Steel Systems

Although structural steel floor framing systems are significantly different in many ways than that of a two-way flat plate or flat slab cast-in-place concrete system, there are enough similarities to justify using the theory and expressions developed by Hawkins and Mitchell (1979) and Mitchell and Cook (1984) in assessing the robustness of structural steel framing systems. It is felt that membrane and catenary action are indeed possible within the structural steel framing systems commonly found in buildings. More importantly, it is felt that this catenary and membrane behavior, to a large extent, is inherent in the systems typically constructed. The tension reinforcement present in these systems will need to be quantified and their anchorage discussed prior to detailed examination of ineffective supporting member scenarios.

In composite steel-concrete floor systems, there is typically welded-wire mesh and light gauge steel deck that can be utilized as tension reinforcement within the slab system should membrane and/or catenary action be needed. However, one must understand the usefulness of these components as reinforcing mechanisms in the slab system before one can count on this reinforcement as being inherent sources of membrane and catenary reinforcement for the floor system. The light-gauge steel deck is essentially a unidirectional spanning entity. In the direction parallel to the flutes in the deck, the steel deck is highly likely to be a very useful form of tension reinforcement for facilitating catenary action. However, in the direction orthogonal to the flutes, the steel deck likely has puddle welds or TEK screws that are unlikely to preserve tensile forces within the deck in this orthogonal direction. Furthermore, the fluted nature of the deck results in a tension force that has two distinct elevations at the floor deck soffit. This makes relying on the steel deck providing tensile membrane or catenary reinforcement in two directions very difficult. Therefore, the present analysis assumes that the steel deck provides one-way reinforcement within the floor framing system. It should be noted that if the steel deck panels are not continuous over the supporting beam, a force-transfer mechanism is questionable without edge beams providing vertical support.

The welded-wire fabric present in the floor system is also a source of membrane and catenary tension reinforcement. This steel fabric generally has a slightly elevated yield stress when compared to the usual mild-steel reinforcement. Furthermore, the spacing of the wires in the mesh can change with direction. This reinforcement will be assumed as sufficient to develop catenary and membrane forces if it is considered continuous through the panel perimeter and appropriately lapped.

In the steel building system considered in this study, a panel is defined as having two in-fill beams and two girders bounding a panel of concrete slab. In most cases, the perimeter of the slab panel will have puddle welds or even steel studs connecting the steel deck to the perimeter beams/girders. Furthermore, these perimeter members will have significantly greater flexural stiffness when compared that of the slab. As a result, the slab system can be assumed to develop compression ring anchorage if the perimeter beams remain in tact during a compromising event.

Ineffective Interior Column

Typical (economical) steel gravity floor framing systems implement simply-connected beams and girders. This system, theoretically, has no inherent robustness because if an interior column is lost, there are no moment resisting connections to help span the compromised column and a theoretical mechanism immediately forms. However, the scenario generated by an interior column being rendered ineffective results in activation of two-way membrane action in the composite-steel concrete floor framing system and two-way flexure/catenary grillage action in the structural steel framing. There is a synergy between these two component systems that has only recently been studied in relation to fire (Allam *et al.* 2000; Bailey *et al.* 2000; Burgess *et al.* 2001; Cai *et al.* 2002; Huang *et al.* 2000a; Huang *et al.* 2000b; Huang *et al.* 2003a; Huang *et al.* 2003b). In addition, the beams and girders do indeed have connections at their ends that support not only tension forces (Owens and Moore 1992), but bending moments as well (Astaneh-Asl *et al.* 1989b; Liu and Astaneh-Asl 2000a; Liu and Astaneh-Asl 2000b; Owens and Moore 1992; Wales and Rossow 1983).

In the present analysis, a deformation compatibility approach is used in conjunction with two separate static analyses: the first considering two-way membrane action in the slab; and the second considering two-way-grillage catenary/flexure action in the steel framing. These two analysis components are described in the schematics in Figures 2 and 3.

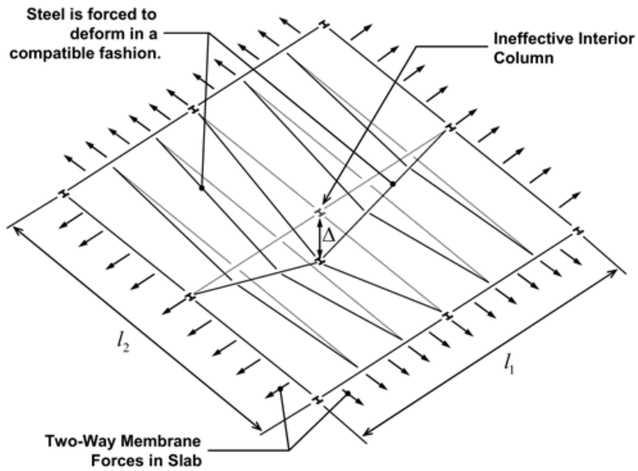


Figure 2 Two-Way Membrane Action Resulting from Ineffective Interior Column.

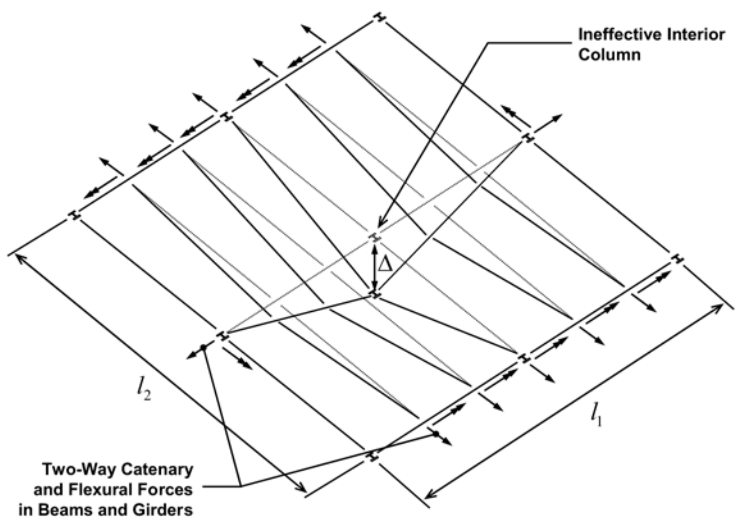


Figure 3 Two-Way Catenary/Flexure Action Resulting from Ineffective Interior Column.

As the interior column is rendered ineffective, the slab and grillage of steel members are forced to deform in a compatible manner and they both resist vertical deformation to the extent that their strength allows. The two-way membrane behavior in the slab is assumed to follow the theory described previously. Two way grillage (catenary/flexure) behavior in the steel framing can be computed using nonlinear structural analysis. These two theories can be used together to evaluate the robustness present in the typical interior 30-ft by 30-ft simple structural steel framing bay.

The framing connections that are assumed are considered flexible and are most commonly fabricated as double web angles (and sometimes referred to as web cleats). In order to assess the capabilities of double angle connections in facilitating the 3D grillage behavior, the web cleat moment capacity, tension capacity, and shear capacity needed to be determined. This process can be started by looking at the web cleat connection as being composed of bolt elements as shown in Figure 4.

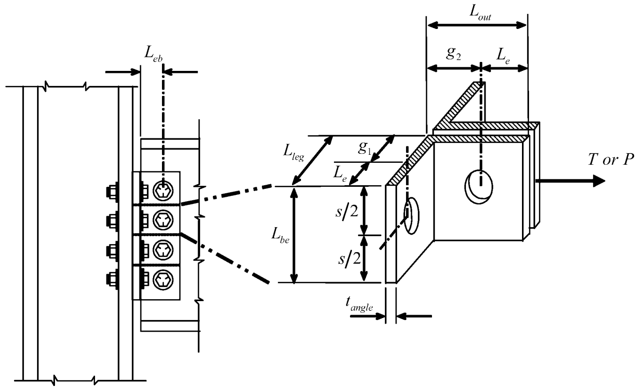


Figure 4 Web-Cleat to Bolt Element Transformation.

Researchers have been studying methodologies for determining pure-moment and tension capacities of bolted angle connections for quite some time (Astaneh-Asl *et al.* 1989a; Astaneh-Asl *et al.* 2002; Astaneh-Asl *et al.* 1989b; DeStefano and Astaneh-Asl 1991; DeStefano *et al.* 1991; DeStefano *et al.* 1994; Liu 2003; Liu and Astaneh-Asl 2000a; Liu and Astaneh-Asl 2000b; Shen and Astaneh-Asl 1999; Shen and Astaneh-Asl 2000; Wales and Rossow 1983).

The present study uses the approach of Shen and Astaneh-Asl (2000) and Liu and Astaneh-Asl (2000b) to develop nonlinear tension and compression behavior for bolt elements. These bolt elements can then be assembled to form web cleats whereupon moment-rotation behavior or tension/compression response of the connection can be developed. A trilinear tension-deformation response for the bolt element is derived using the procedure suggested by Liu and Astaneh-Asl (2000a); Liu and Astaneh-Asl (2000b); Shen and Astaneh-Asl (1999) and Shen and Astaneh-Asl (2000) with slight modifications. The compression and tension response models are shown in Figure 5.

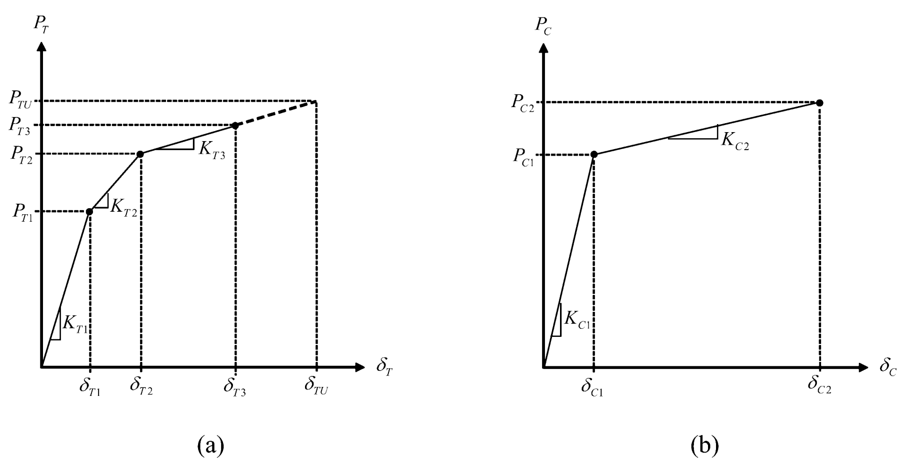


Figure 5 Double Angle Bolt Element Response: (a) Tension and (b) Compression.

Three characteristic points on the tension response are generated using procedures recommended by Shen and Astaneh-Asl (2000) with slight modification. Point (P_{T1}, δ_{T1}) is defined using the yield moment in the legs of the angle. The initial stiffness, K_{T1}, is essentially the linear elastic stiffness of the bolt element considering bending of the legs perpendicular to the beam web and the axial extension of the leg parallel to the beam web. Point (P_{T2}, δ_{T2}) corresponds to the plastic mechanism capacity of the angle legs perpendicular to the beam web. The post-yield mechanism stiffness is defined as K_{T2}. The final point on the tension-deformation response is (P_{TU}, δ_{TU}). This point corresponds to the ultimate loading for the bolt element exclusive of bolt tension rupture or bolt shear rupture. It is defined through consideration of the angle legs perpendicular to the beam web forming catenary tension between the bolts in the support and the legs parallel to the beam web. The tension in the catenary at this ultimate loading is taken to be the loading corresponding to fracture on the net area through the angle leg perpendicular to the beam web. The final stiffness in the response is defined as K_{T3}.

The catenary tension force may or may not be able to form as a result of limits states being exceeded and therefore, a third point (P_{T3}, δ_{T3}) is defined. The loading, P_{T3}, is defined through consideration of the following bolt-element limit states;

- catenary tension fracture in the angle legs perpendicular to the beam web;
- tear-out bearing failure of the bolts in the beam web;
- tear-out bearing failure of the bolts in the angles;
- tension fracture of the bolts including prying action (Thornton 1985);
- tension fracture of the bolts excluding prying (superfluous);
- shear fracture of the bolts.

The yield point on the bolt element compression-deformation response (P_{C1}, δ_{C1}) is defined by considering three strength limit states;

- yield in the angle legs parallel to the beam web;
- yielding in the beam web;
- shear fracture of the bolts.

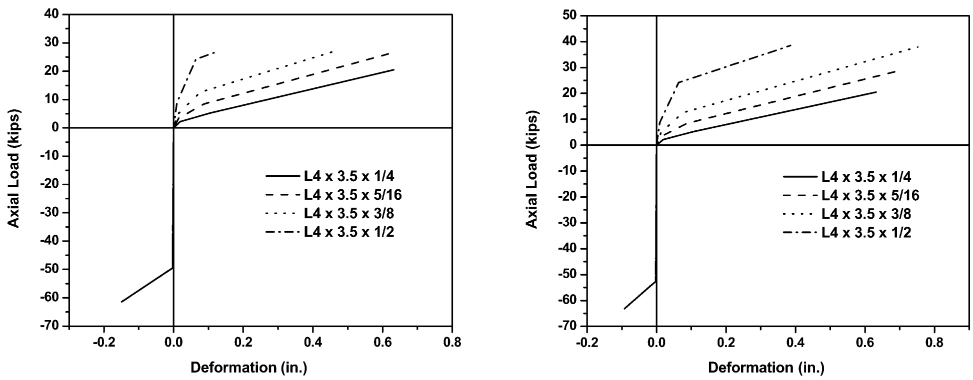
The ultimate loading capacity of the bolt element in compression is defined through consideration of the following strength limit states;

- crushing in the angle legs denoted by the ultimate tension stress being reached in the angle legs parallel to the beam web (conservative);
- crushing in the beam web denoted by the ultimate tension stress in the beam web being reached (again, conservative);
- 20% increase above the ultimate bolt shear stress magnitude.

The initial stiffness, K_{C1}, is defined using the smaller two stiffness magnitudes. The first is based upon web yielding and the second is based upon angle leg yielding. Post-yield stiffness is defined rather arbitrarily using a 0.5% multiplier to account for moderate strain hardening in the material on the way to crushing.

It should be noted that the behavior of the supporting element (*e.g.* a column flange, a column web, a girder flange) is omitted. This is likely very important, but the complexity incurred through consideration of this behavior would render the analysis proposed intractable. Expected yield and ultimate tensile stresses for the materials are used as recommended in the GSA guidelines (GSA 2003). Further details of the formulation and example computations can be found in Foley *et al.* (2006).

The tension and compression response for the bolt elements are shown in Figure 6 for the W18x35 and W21x68 wide flange shapes, respectively. These wide-flange shapes are consistent with the 3-story SAC-FEMA Boston building assumed as the analysis prototype (FEMA 2000b; Foley *et al.* 2006).



(a) (b)

Figure 6 Bolt Element Tension and Compression Response for L4x3.5 Double Angles: (a) W18x35; and (b) W21x68.

The tension-deformation response varies considerably with beam shape and angle thickness. This is a byproduct of the varying limit states considered in the computations. For example, when thin angles are considered, the catenary tension action is allowed to form and rupture of the angle legs is the controlling limit state. However, as the angles get thicker, other limit states control the behavior. This is indicated by the "capping" of the tension forces in the 5/16, 3/8, and 1/2-inch angle thickness in the W18x35 beam shape and the 3/8 and 1/2-inch angle thickness with the W21x68 girder shape. The compression-deformation response is consistent indicating that the limit states controlling strength are consistent as well.

The bolt element ultimate strengths can be used to contribute to the determination of the tension capacity of the double angle connections through simple summation of the bolt element tension strengths in any given connection. The tensile capacity of the double-angle connection is determined through consideration of two additional limit states beyond those assumed for the bolt elements (Foley *et al* 2006):

- shear rupture of the bolts;
- tension fracture of the bolts including prying;
- block shear rupture in the angle legs parallel to the beam web;
- block shear rupture in the beam web;
- bearing tear-out failure in the angle legs parallel to the beam web;
- bearing tear-out failure in the beam web;
- catenary tension rupture in the angle legs perpendicular to the beam web.

The pure moment capacity of the double web-angle connection is determined using the bolt element tension- and compression-deformation response parameters described previously. The pure moment condition is defined by the deformation compatibility and internal equilibrium. The process for determining the pure moment capacity of the connection begins with defining the tension and compression response for each bolt element in the connection. A controlling state of deformation in the extreme tension angle, or extreme compression angle is assumed. These deformations are taken from the appropriate angle force-deformation curves. The connection rotation angle is then varied until the summation of all forces determined using the bolt element response curves sum to zero. This corresponds to the pure moment capacity of the connection. It should be noted that this process is iterative and the compression or tension deformation limit states may control the behavior. Details of the procedure can be found in (Foley *et al.* 2006).

The shear strength of the double angle connection given the beam shape chosen can be determined using the AISC Manual (AISC 2001). It should be noted that unfactored strengths were utilized and therefore, all manual-obtained strengths were divided by 0.75. The shear strengths for the double angles and beam shapes considered assume: $L_{ev} = 1.5''$; $L_{eh} = 1.5''$; $\phi = 1.0$; and 3/4" A325N bolts in STD holes. No consideration of expected strengths of the material in defining the shear strength was given.

The beams in the grillage are assumed to be W18x35's and the girders are W21x68's. From the AISC-LRFDM (AISC 2001), the W18 sections can support 3-5 rows of bolts, while the W21 sections can support 4-6 bolt rows with traditional spacing and end distances. Therefore, only these numbers of bolt rows were considered. Double-angle connections alone have a tensile capacity that ranges from 0.1-0.30 of the squash load of the cross-section (Foley *et al.* 2006). These are fairly significant tensile capacities (if taken as cumulative over all beam and girder members within the 3D system. The loading capacities are consistent with those found in testing by (Owens and Moore 1992). The moment capacities are very low, however (Foley *et al.* 2006). They range from 0.05 – 0.20 of the plastic moment capacity of the beam cross-section. This is consistent with the strength portion of the definition of a flexible connection (AISC 2005).

Bilinear moment-rotation response and axial load-extension response curves can be generated for the double angle connections using the bolt element response shown in Figure 5. Compression response characteristics are only used for defining moment rotation response. The connections in the grillage are not expected to go into compression in the ineffective column scenario considered. Tension secant stiffness for the bolt element, k_{BE}, is defined using point (δ_{T2}, P_{T2}) on the tension-deformation response. The tensile capacity of each bolt element in the double-angle connection then contributes to the tensile and moment capacity of the connection. The bilinear tension-deformation response of the bolt element is then characterized by the secant stiffness and the bolt element tensile capacity, P_{T3}.

The rotational and axial stiffness of the web-cleat connections are estimated using the magnitudes of the bolt element secant stiffness. In the case of axial tension, the axial stiffness of the double angle connection is simply the sum of the stiffness of each bolt element in the web cleat,

$$K_{\delta} = \sum_{i=1}^{n_b} k_{BE,i} \tag{5}$$

In general, if the bolt element stiffness, k_{BE}, is known and there is n_b bolt elements in the web cleat connection, the rotational stiffness can be computed as (Foley *et al.* 2006),

$$K_{\theta} = \sum_{i=1}^{n_b - 1} i^2 \cdot \left(k_{BE} \cdot s^2\right) \tag{6}$$

where s is the pitch of the bolt elements (taken as a constant value of 3 inches).

The axial stiffness and flexural stiffness of the web cleat connections can be defined as a function of the axial rigidity and flexural rigidity of the connected member. This is mathematically defined as,

$$K_{\delta} = \alpha_{\delta} \cdot \frac{AE}{L} \tag{7}$$

$$K_{\theta} = \alpha_{\theta} \cdot \frac{EI}{L} \tag{8}$$

The rotational stiffness of the web-cleat connections are well below the stiffness limit corresponding to flexible connections (AISC 2005) given by $\alpha_{\theta} = 2$. The majority of the rotational stiffness multipliers are in the range; $0.05 \le \alpha_{\theta} \le 1.50$ (Foley *et al.* 2006). One exception is the 5 bolt arrangement in a W18x35 beam member. The axial stiffness multiplier for the majority of the connection arrangements lies in the range $0.10 \le \alpha_{\theta} \le 1.8$ With the 5-bolt connection in the W18x35 member giving $\alpha_{\delta} = 2.3$.

The analysis begins by computing the capacity of the concrete-steel composite slab system acting as a two-way membrane using equations (1) through (4). The steel deck is assumed to be 2VLI22 (Vulcraft 2005) and 40% of the cross-sectional area is assumed to be effective as tensile reinforcement (Foley *et al.*

2006). Welded-wire-mesh is assumed in the concrete deck: 6x6-W1.4xW1.4 (shrinkage and temperature reinforcement). When the interior column looses effectiveness, the concrete slab panel is 60-feet by 60-feet. The membrane capacity of the concrete slab-steel deck system is approximately 50-psf at 26.2 inches of vertical deflection at the center of the panel (Foley *et al.* 2006). This magnitude of vertical deflection corresponds to an approximate rotational demand of 0.073 radians, which is well below the limit of 0.21 radians (GSA 2003). It should also be noted that the rotation computed here is a total rotation (elastic plus plastic components). Therefore, the magnitude computed is conservative. The tension force in the steel deck running perpendicular to the in fill beams is approximately 566 lbs/in (Foley *et al.* 2006) along one edge of the panel.

The capacity of the steel grillage is then computed. A structural model for the steel floor framing system was developed for use in MASTAN2 (Zieman and McGuire 2000). A schematic of the analytical model is shown in Figure 7.

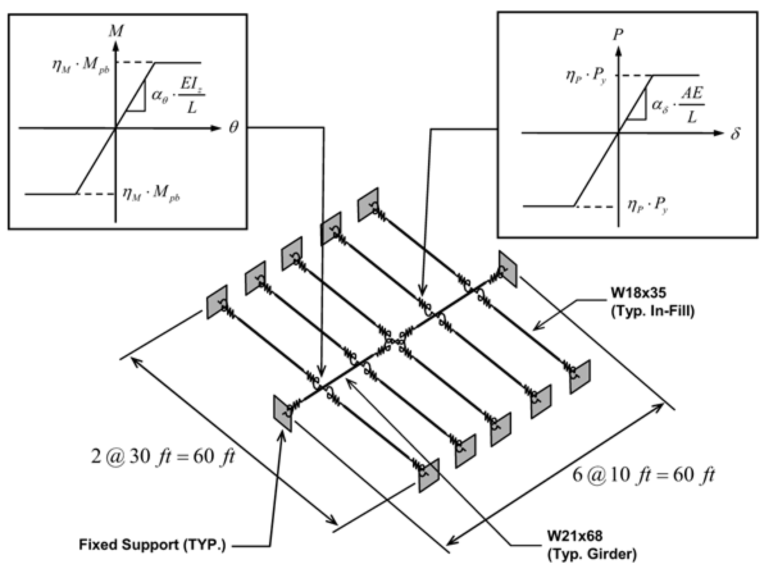


Figure 7 Steel Grillage Model Schematic (System 1) Illustrating Axial and Moment Connection Modeling for Nonlinear Analysis.

All members are modeled using multiple elements: in-fill beams are modeled using 10 elements and girders are modeled using 9 elements. The in-fill beams were modeled using 4 analytical segments. Two segments (*i.e.* 1/2 of the beam length) were centered on the beam mid-span. The end 1/4 lengths of beam were subdivided into 4 additional segments to facilitate connection modeling. Therefore, all in-fill beams contain end segments that are 1/16th of their span. The end segments in the girders (at column supports and interior column location) were broken down into 4 segments yielding end connection segments of 1/12th the girder span.

The end connections were modeled in the analytical segments of the beams and girders located immediately adjacent the fixed supports, the supporting girders, and the interior column. The connection rotational stiffness, K_θ, was input using the built-in capability. The connection moment capacity was interjected into the analytical model by adjusting the beam or girder's plastic moment capacity to $\eta_M \cdot Z_x$. The axial loading characteristics were included in a slightly different manner. MASTAN2 does not allow axial spring characteristics to be directly modeled. The cross-sectional areas of the beam or girder in the end connection segments were defined to be $\eta_P \cdot A_g$. This reduction in cross-sectional area also created implied linear spring stiffness in this isolated region of the beam equal to $\alpha_\delta \cdot AE/L = \eta_P \cdot AE/L$.

The method of modeling connections creates a "stub member" that has an axial capacity and a moment capacity that is the same as the connection intended. Three systems with varying connection characteristics were considered (Foley *et al.* 2006):

System 1	System 2	System 3
$\eta_M = 0.10 \quad \alpha_\theta = 0.50$	$\eta_M = 0.50 \quad \alpha_\theta = 5.0$	$\eta_M = 0.30 \quad \alpha_\theta = 2.0$
$\eta_P = 0.20 \quad \alpha_\delta = 0.20$	$\eta_P = 0.30 \quad \alpha_\delta = 0.30$	$\eta_P = 0.30 \quad \alpha_\delta = 0.30$

System 1 has strength and stiffness characteristics typical of web-cleat connections used in structural steel floor framing systems. System 2 has strength and stiffness characteristics typical of partially restrained beam-to-girder connections (Rex and Easterling 2002) and there is long-standing use of partially restrained girder-to-column connections. The axial strength and stiffness were increased slightly from that of System 1. A third system was considered. This system had a better balance between axial capacity and moment capacity than system 2. The axial strength and stiffness for the connections in system 3 were left the same as those in system 2. The bending strength and stiffness of the connections were reduced to a level slightly above that in System 1 and below that in system 2. The moment and axial strength characteristics are consistent with web cleat connections that are relatively thick (compared to the typical thickness used) and a number of bolt rows that fills up the beam and girder web (Foley *et al.* 2006).

MASTAN2 then uses these pieces of information to create an interaction (yield) surface of the form shown in Figure 8 for three systems considered.

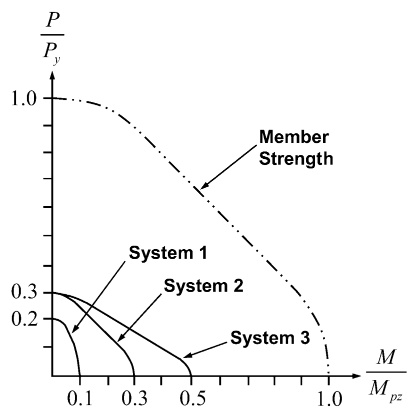


Figure 8 Member and Connection Interaction Surfaces for Connected Member and Three Grillage Systems (connection characteristics vary).

It should be noted that minor-axis bending is assumed to have a connection capacity that is equal to the minor axis moment capacity of the members and the connection stiffness in the minor-axis direction is infinite relative to the flexural rigidity of the connected beam (*i.e.* the connection is fully-restrained).

Each floor system is evaluated independently under the assumption that it carries its own loading. The slab system was determined previously to be capable of supporting approximately 50 psf through membrane action. The total unfactored live loading used for design of the system is: 80-psf dead loading; and 50 psf office occupancy live loading. The total point in time live loading that can be assumed present at the time a column is rendered ineffective can be computed as (GSA 2003);

$$q_{p.i.t.} = 1.0(80\ psf) + 0.25(50\ psf) = 93\ psf$$

The steel grillage will then be required to carry the following superimposed loading (with a deformation that is compatible with the slab membrane);

$$q_{grillage} = \beta_{dynam} \cdot (1.0D + 0.25L) - 50 = \beta_{dynam} \cdot (93psf) - 50psf$$

At pseudo-static loading levels $(\beta_{dynam} = 2.0)$ prescribed in the GSA Guidelines (GSA 2003), the grillage will need to support a uniformly distributed loading of 136 psf. However, this assumes that the supporting column is "vaporized". Furthermore, former studies (Liu *et al.* 2005; Marchand and Alfawakhiri 2004; Powell 2005) and relatively recent research (Foley *et al.* 2006) have shown that the multiplier commonly used to simulate dynamic loading can vary considerably. If the supporting column is not "vaporized", but simply compromised (*i.e.* it still has a fraction of its initial load capacity), then one might argue that the point-in-time loading alone needs to be carried $(\beta_{dynam} = 1.0)$ without dynamic multiplication. Therefore, in this case, the grillage must support 43 psf superimposed loading.

The MASTAN2 model shown in Figure 7 was analyzed using 2nd order inelastic analysis and a reference superimposed loading on the steel grillage of 108 psf. The load deformation response for the three systems is shown in Figure 9.

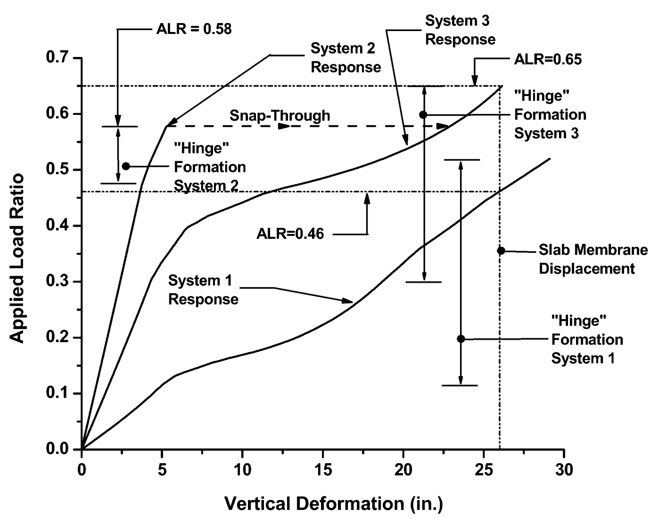


Figure 9 Load Deformation Response of Three Grillage Systems Considered.

The load deformation response of system 1 indicates that there is a very early transition from flexural behavior to catenary behavior in the grillage. The connection strengths and stiffness result in the cross-sections at the ends of the members reach the yield surfaces very early in the response and the large displacements result in catenary tension in the grillage forming. This transition is exhibited by the shallow yield plateau-like response and subsequent stiffening behavior. The applied load ratio that results in deformations compatible with the membrane displacement computed earlier (26 inches) is 0.46. This indicates that the capacity of the system (both slab and grillage) is;

$$q_{cap} = 0.46(108) + 50 \approx 100\ psf$$

Therefore, System 1 can definitely support the point in time live loading and there is some reserve for dynamic amplification: $\beta_{dynam} = 100/93 = 1.08$. If one were to assume that the system could continue to deflect without membrane reinforcement in the slab rupturing, or the anchorage of this reinforcement being compromised (*e.g.* deflection to approximately 30 inches), the membrane capacity would increase and the catenary capacity of the grillage could increase. This increase is shown in Figure 9 at $ALR = 0.52$. This would result in the system capacity moving upward to,

$$q_{cap} = 0.52(108) + 50 \approx 106\ psf$$

and the dynamic multiplier would naturally increase as well to $\beta_{dynam} = 106/93 = 1.14$. One should note that shrinkage and temperature welded wire fabric reinforcement was assumed as well as 22-gauge steel deck. Greater capacities can likely be attained if thicker deck is used and mild-steel reinforcement rather than welded wire mesh (Foley *et al.* 2006).

At 26 inches of vertical displacement, the total rotation over the beam and girder span of 30 feet was computed previously as approximately 0.07 radians. This is very close to the plastic rotational limit of 0.06 radians recommended for web-angle connections (FEMA 2000a). However, the present rotational demand is "total" and the plastic demand will likely align itself close to this limit. Therefore, the rotational demands at the level of loading considered are not likely to cause rupture of the connections.

The same reference loading was applied to the steel grillage of system 2. The load deformation response of the grillage system 2 is also shown in Figure 9. It is interesting to note that the catenary (stiffening) response is not present. The reason for this is that a plastic mechanism (flexural) forms at an applied load ratio of 0.58 with vertical deformation slightly less than 5 inches. This amount of vertical deformation is not sufficient to *activate* the geometric stiffness for the members in the floor system. In other words, analytically, catenary action is not allowed to form and the system numerically "fails". It is understood that there will be a *conversion* to catenary action once the mechanism forms, but the structural analysis is not able to consider this transformation because the tangent stiffness matrix of the system is singular at the instant this group of beam mechanisms forms.

The number of hinges that form in System 2 and the loading range over which they form is much less than that of System 1. One would like to have a system where there is a significant number of hinges forming so that full advantage of the structural indeterminacy and load redistribution is taken. When the hinges form over very short loading ranges, there is less redundancy and toughness in the *system*. The significantly smaller deformation in System 2 at the formation of the collapse mechanism would indicate that the grillage will form a bending moment collapse mechanism first with subsequent reliance on back-up capacity catenary action after significant vertical deformation.

Experimental rotations attained by Rex and Easterling (2002) for the partially-restrained beam-to-girder connections were reported to be on the order of 0.05 radians. If one were to rely on catenary action after the flexural mechanisms occurs, the vertical deformations in the system would likely rapidly increase to those found in the first system (approximately 26 inches). As a result, even though the flexural mechanism forms early at 5 inches of deformation there will need to be an additional 21 inches of deformation in the grillage needed to activate catenary action. As a result, the rotational demands on these connections are likely to be on the order of 0.07 radians. It is unclear if the PR beam-to-girder connection can support his level of rotational demand without fracture.

The axial stiffness and strength of the connections in system 3 are consistent with those of system 2 and therefore, it is expected that the catenary behavior will be the same in the two systems once it is activated. The load deformation response of system 3 is shown in Figure 9. After the formation of the flexural collapse mechanism in system 2, it is likely that the steel grillage will need to abruptly accumulate an additional 20 inches of deflection in order to reach the catenary tension stiffening that comes from the contribution of geometric stiffness. This behavior is analogous to snap-through behavior in arches and is schematically indicated in Figure 9. It should be noted that the response of system 3 indicates that system 2 will indeed be able to reach the same load carrying capacity of system 3, but it is not economically advantages to provide additional bending moment capacity and stiffness when there is no enhancement in load carrying capacity. Furthermore, dynamic snap-through behavior may have adverse ramifications with regard to system integrity and toughness.

System 3 is capable of supporting an applied load ratio of 0.65 at 26 inches of vertical deformation, which is compatible with the deformations needed for the slab system membrane to support 50 psf. This reveals that a relatively economical (simple-framing) system can support the following superimposed floor loading;

$$q_{cap} = 0.65(108) + 50 \approx 120\ psf$$

This magnitude of loading reveals that this system can allow for a dynamic amplification factor of $\beta_{dynam} = 120/93 = 1.30$.

Concluding Remarks

An introduction to methodologies available for computing the membrane tension capacity of concrete-steel composite slab systems was provided. A hypothetical event whereby an interior column in a typical structural steel framing system is rendered ineffective was outlined and a methodology for computing the load carrying capacity of the steel floor framing system after this event was described. Inelastic static structural analysis was conducted using bilinear connection characteristics typical of simple framing connections in steel systems. The analysis conducted included bilinear connection modeling for axial load-deformation response and moment-rotation response within the structural steel framing system.

A static nonlinear analysis of the typical 30-ft by 30-ft framing system that included nonlinear connection behavior consistent with that of web-cleat connections was conducted. The analysis indicated that while it is doubtful that the typical structural steel framing system could support the GSA-level dynamic loading estimates, one can say with certainty that the typical structural steel framing system can ***resist*** progressive collapse in the event an internal column is rendered ineffective. This statement is supported by the fact that the point-in-time loading of $1.0D + 0.25L$ can be supported through catenary and flexural action in the structural steel framing and membrane action in the composite concrete-steel deck system with moderate levels of dynamic amplification reserve.

The analysis conducted suggests that it is better to have smaller moment capacity and flexural stiffness for connections distributed throughout the floor framing system (as is typically found in structural steel interior framing arrangements). When the moment capacity is low, there is a smooth transition between the formation of the flexural mechanism and the catenary tension behavior that is essentially secondary after the initial collapse. If the moment capacity is too large, there will likely be snap-through-type behavior whereupon a significant magnitude of vertical displacement will rapidly take place prior to the formation of catenary action.

In general, a balance between flexure and catenary action in the steel grillage can be attained when the following axial and moment characteristics are met in regard to the connections at the ends of the beams and girders in the structural steel system;

$$M_{conn} \le 0.30 M_{pb} \text{ and } K_\theta \le 2\frac{EI}{L} \qquad P_{conn} \le 0.3P_y \text{ and } K_\delta \le 0.3\frac{AE}{L}$$

It is interesting to note that this behavior is nearly approached for typical structural steel framing systems. Axial capacity, axial stiffness, and rotational stiffness characteristics needed are easily attained by providing web-cleat connections (double angle connections) that "fill up" the web of the connected beam or girder. The moment capacity recommendations may be able to be attained if better modeling of the connection limit states is performed. Foley *et al.* (2006) found that 1/2-inch thick web connection angles that fill up the web of the connected member can approach $0.2M_{pb}$.

The results of the study suggests that typical structural steel framing systems have significant levels of inherent robustness and general structural integrity without providing any special design effort. Furthermore, if slight increases in connection angle thickness and the number of bolt rows used in these connections are provided, the toughness of the system in response to abnormal events can be significantly enhanced.

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