A design-oriented model for the collapse resistance of composite floors subjected to column loss

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ABSTRACT

A design-oriented model is proposed for computing the load-resisting capacity of composite steel–concrete floors subjected to interior column loss. The model is based on the premise that floor collapse is resisted through the development of membrane action in the slab elements and catenary forces in the steel beams. A number of simplifying assumptions are made in the model pertaining to the deformed shape of the system, development of failure-resisting mechanisms, and overall system behavior. The proposed model is incremental in nature and tracks the evolution of damage in a floor system loaded up to failure. The model is shown to be able to capture the effect of influential variables on collapse resistance. Key limitations on the model are highlighted and discussed.

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1. Introduction

Steel moment-resisting frames are a common type of building system on the US West Coast. As shown in Fig. 1, these buildings are generally comprised of perimeter moment frames (enclosed within dotted boxes in Fig. 1) and a gravity system (the remainder of the structural system outside the dotted boxes in Fig. 1). The latter is typically comprised of composite floors, where steel beams are composite with a concrete deck, and gravity columns to which the floor beams are attached through shear connections. The primary purpose of the moment frames is to transmit lateral loads, such as wind and seismic forces. The gravity system ‘leans on’, i.e. derives its stability from, the moment system, and serves to transmit gravity loads to the foundations. This type of arrangement minimizes the number of expensive-to-construct moment-resisting connections in the structural system, reducing the cost and contributing to the popularity of the system.

Well-constructed steel moment frame buildings have been shown to perform well under seismic loading. However, the key disadvantage, from a collapse resistance perspective, is that the floor system may be especially vulnerable to column loss, e.g. due to blast or vehicle collision, say in a parking garage. Moreover, since they are not designed for large flexural moments, gravity columns are relatively small and therefore more susceptible to damage than their counterparts in moment-resisting bays.

The few efforts that have been devoted to investigating the robustness of typical composite floors document divergent information about the collapse resistance of such systems. Based on an experimental study, Astaneh-Asl et al. [1] concluded that their composite floor system could absorb removal of a middle perimeter column without collapse. Using an analytical model, Foley et al. [2] suggested that, in the event of a center column loss, a composite floor system with bolted clip angle connections could be expected to carry the dead and service live loads, but without the full dynamic load effects. Sadek et al. [3] used a computational model to show that their concrete deck–steel beam composite floor prototype could not successfully absorb the loss of a central column. They attributed the differences in the conclusions of these three studies to differences in geometry, connection types, and modeling assumptions employed.

Using a modified version of the model developed by Sadek et al. [3], Alashker et al. [4] investigated the key parameters influencing composite floor robustness and concluded that the steel deck was the most influential component contributing to collapse resistance. They also showed that increasing the connection strength by adding more bolts was not necessarily beneficial in increasing the overall collapse strength. The objective of this paper is to synthesize the information from Sadek et al. [3] and Alashker et al. [4] into a design-oriented model that could be conveniently used to compute the collapse capacity of composite floors. The proposed model is based upon established models for computing the collapse resistance of reinforced concrete (RC) slabs.
2. Collapse resistance of floor systems: previous research

Early studies pertaining to the collapse resistance of floor systems focused primarily on RC floors. The results of these studies suggest that the membrane forces that inevitably develop in the plane of a loaded RC slab play a key role in its collapse resistance. At low levels of loading, flexural cracking in the slab causes the neutral axis in critical slab sections to rise, forcing the edges of the slabs to expand slightly outwards. Compressive membrane forces will develop in the slab if its edges are restrained against lateral movement. At higher deformation levels, the compressive membrane forces switch to tensile as the slab gradually moves away from flexural action as the primary source of load resistance. If the edges are unrestrained, the changes in slab geometry create a tendency for the slab edges to retreat inwards, leading to the formation of a compressively loaded ring at the outside edges of the slab, as shown in Fig. 2. Regardless of the restraint condition at the slab edges, the change in slab shape at high deformation levels gives rise to what are commonly known as tensile membrane forces, sometimes known as catenary forces. These membrane forces are primarily carried by the slab's steel reinforcement and are anchored to the compression ring or edge restraints.


While very few studies have focused on the room-temperature response of composite floors – as discussed in the introduction – the majority of published work on composite floor collapse resistance have actually addressed the response at elevated temperatures. Many of the contemporary studies in this area are based upon, or inspired by, the 1995 Cardington tests, which experimentally investigated the behavior of a full-scale eight-story steel building in fire. Some recent notable studies include Huang et al. [10,11], Elghazouli and Izzuddin [12] and Bailey et al. [13,14].

3. Finite element modeling of a prototype floor system

Consider the shaded region in Fig. 1, which represents four adjacent floor panels in the gravity-bay portion of a prototype steel framed building designed by the National Institute of Standards and Technology [15]. Sadek et al. [3] and Alashker et al. [4] used detailed finite element (FE) models to investigate the collapse resistance of this floor system. A primary assumption made in both studies is that the four adjacent panels in a typical floor are behaving in isolation from the rest of the building, as shown in Fig. 3. By exploiting symmetry, only a quarter of the model was simulated in both studies, as depicted in Fig. 4.

The floor system in Fig. 1 consists of a 76.2 mm deep metal deck with an 82.5 mm lightweight concrete (17.3 kN/m²) topping with nominal compressive strength of 20.7 MPa. The metal deck is 0.9 mm thick with yield strength of 248 MPa. The concrete slab is reinforced using a wire mesh 152 mm by 152 mm, W1.4 steel, with yield stress of 448 MPa. Single-plate shear tab connections, comprised of ASTM A36 steel ($f_y = 248.2$ MPa), are used to connect the steel beams to the columns via three 22.2 mm diameter A325 high-strength bolts. The shear tabs are 9.5 mm thick and the shear connections are designed according to the AISC LRFD Specifications (AISC-Specs 2005). ASTM A992 structural steel ($f_y = 344.8$ MPa) is used in the beams and columns.

The floor system was modeled as shown in Fig. 4 in LS-DYNA; see Hallquist [16]. The concrete slab was modeled using eight-node brick elements. The concrete behavior was modeled using a smeared representation employing a three-dimensional, three-invariant, non-associative, concrete plasticity model. The welded wire fabric mesh in the slab was modeled using truss elements, which have a uniaxial, bilinear stress–strain relationship. The mesh bars were assumed to be fully bonded to the concrete slab. The steel deck was modeled using shell elements, and interpenetration between steel deck elements and adjacent concrete slab elements was not permitted. The inelastic response of the steel deck and beams was modeled using a $f_y$ plasticity model with kinematic
hardening. Elements were eroded when a pre-specified strain-to-failure was reached, signifying fracture of the steel. The shear tab connection was represented using one single row of shell elements with a thickness equal to that of the beam web. The shear studs connecting the beam top flanges to the concrete slab through the metal deck were modeled using beam elements that were embedded in the concrete slab and fully bonded to the concrete elements. The stud elements were attached to the beams using finite-sized connection elements that were deleted when the shear studs reached their full load-carrying capacity. The models were thoroughly validated through comparisons of simulation data to a variety of test results. Further details about the finite element model (FEM) and validation studies can be found in Sadek et al. [3] and Alashker et al. [4].

Sadek et al. [3] investigated the resistance of the floor region in Fig. 3 to loss of the central column by pushing down on the central column stub and computing the resistance of the system to the applied action. Since the remainder of the floor system was not loaded as the column stub was pushed down, all beams remained essentially straight and the formed plastic hinge lines at the main beam locations, but remained planar in between these lines. The resistance of the floor was generated by the membrane action that developed in the floor as well as the tensile ‘catenary’ forces that developed in the beams. The floor membrane forces were anchored to the compression ring that developed at the floor edges. The system exhausted its ability to support a load when the connections between the main beams and column stub failed.

Alashker et al. [4] conducted their investigation by pushing down with a load applied uniformly to the entire floor. This was shown to lead to a significantly different load–deflection response than the center-column-loaded cases in Sadek et al. [3]. In addition to forming plastic hinge lines along the main beams, the floor no longer planar, but took on a curved shape in response to the applied uniform loading. The beams deflected as well, but their deformation level was moderate compared to the overall deformation of the system. As in the previous case, the resistance of the floor came from membrane action in the floor as well as tensile forces in the beams. In contrast to the previous case, however, the system was able to absorb loss of the beam-to-center-column connections and keep carrying significant loads beyond. Failure progressed next to the B1–B3 connection [see Figs. 3 and 4], followed by fracture of the steel deck and reinforcing steel mesh, after which the system collapsed.

4. Proposed model

A model is proposed for computing the load capacity of a composite floor system with shear tabs, such as that shown in Figs. 1 and 3, subjected to the loss of a central column. Following are discussions of the various aspects of the model.

4.1. General simplifying assumptions

Since the intent is to develop a design-oriented model suitable for office use, the following simplifying assumptions are made for convenience. The aggregate effect of these assumptions on the model’s predictive capabilities is investigated later on in the paper by comparing model results to simulation data obtained using the computational model developed in Sadek et al. [3] and Alashker et al. [4].

1. The configuration of the affected floor panels is $2 \times 2$ as shown in Fig. 3. Panels outside this region are considered to have no influence on collapse resistance.
2. The floor slab components, including the steel deck, steel reinforcement and concrete slab, are attached together and deform in a compatible manner. Once the floor slab reaches its yield membrane force, it deforms in a ductile, perfectly plastic manner up till failure.
3. The flexural deformations in the steel beams are negligibly small compared to the overall system deformation. In other words, the steel beams remain straight within the deformed system as shown in Fig. 5.
4. The steel reinforcement and steel deck are fully yielded throughout the system such that the local deformed shape of the floor is as shown in Fig. 5(b).
5. The flexural capacity of the floor slab is negligibly small and its capacity to resist loading stems from the membrane forces that develop within the slab. Similarly, the flexural capacity of the shear tab connections is assumed to be negligible and the axial ‘catenary’ force in the beams stems from the axial capacity of the connections.
6. The axial deformations applied to a steel beam (due to catenary action) are accommodated within the shear tab connections at either end of the beam and equally split between both connections.
7. The deformation levels are small enough such that the sine or tangent of an angle equals the angle itself in radians.
8. The horizontal component of the membrane forces in the floor slab is adequately supported through compression ring action that develops at the periphery of the slab system. Likewise, the horizontal component of the catenary forces in the beams is adequately supported through frame action in the beam and column system.

There are other specific assumptions built into the model, e.g. pertaining to the angle of rotation at the edges, etc., that are outlined later on in the paper.

### 4.2. Deformed shape of the floor system

Since the steel beams are assumed to remain straight, the geometric configuration of the steel grillage is uniquely defined by the deflection of the center column, \( \delta \), in Fig. 5. The angles of rotation of beams B1 and B2, respectively, are

\[
\begin{align*}
\theta_1 &= \frac{2\delta}{L_y} \\
\theta_2 &= \theta_1 \frac{L_x}{L_y}
\end{align*}
\]

where \( L_x \) and \( L_y \) are the lengths of the floor region along the \( x \)-axis and the \( y \)-axis, respectively. As previously mentioned, a key assumption in the model is that the steel reinforcement and steel deck are yielded throughout the floor slab. As a result, the floor slab supports the applied uniform loading by deforming into a curved shape, as shown in Fig. 5. Applying membrane action theory as done in Hawkins and Mitchell [6] and Mitchell and Cook [7], the vertical deflection, angle of rotation and membrane forces in the slab can be computed.

Consider a floor slab strip far enough away from the influence of the end beams so that it is subjected to pure one-way action. As depicted in Fig. 6, the strip, which is of span length \( L_y/4 \) for the particular layout considered, takes on a parabolic shape in response to loading. Applying vertical force equilibrium, the following expression can be written:

\[
2T_1 L_0 \theta' = u \frac{L_y}{4} L_x,
\]

where \( T_1 \) is the tension membrane force per unit width in the \( y \)-direction, \( u \) is the uniform distributed loading acting on the floor system, and \( \theta' \) is the slope at either end of the strip (Fig. 6) with respect to the plane of the slab (Fig. 5(b)). Eq. (3) can be arranged such that \( \theta' \) is expressed in terms of \( u \).

As shown in Fig. 5(b), the total angle of rotation of the slab at any point along the \( x \)-edge (\( \theta_{t.x} \)) is assumed equal to the sum of the rotation angle due to the geometric configuration of the steel beams (\( \theta_{1,x} \), which is linearly interpolated from \( \theta_1 \)), and that due to uniform loading (\( \theta' \)).

\[
\theta_{t.x} = \theta_{1,x} + \theta'.
\]

Strictly speaking, Eq. (4) is only accurate at strips away from the edges, i.e. where Eq. (3) is valid. In addition, Eq. (4) violates compatibility at the beam locations where the slab must conform to the straight shape taken by the beam. However, as shown later on, the effect of these discrepancies is small.

Along the \( y \)-edge, the total angle of rotation of the slab (\( \theta_{t.y} \)) is obtained from the geometry of the steel grillage (\( \theta_{2,y} \), which is linearly interpolated from \( \theta_2 \)).

\[
\theta_{t.y} = \theta_{2,y}.
\]

Unlike Eq. (4), Eq. (5) ignores the contribution from the curved shape of the slab. However, this is justified because (a) the major membrane forces in a composite slab will develop along the direction of the flutes, assumed to be aligned in the short direction, and therefore inaccuracies in Eq. (5) will have a minor influence on the accuracy of the solution, and (b) the slope of the slab curve, corresponding to \( \theta' \) in the short direction, will be small in the long direction. Fig. 7(a) compares the angle of rotation computed from Eq. (4) versus that computed from the numerical model of the prototype floor system in Fig. 1, while Fig. 7(b) shows a similar comparison for Eq. (5). Clearly, some differences exist between the proposed model and the simulation results, but the overall comparison is, nevertheless, deemed reasonable given the extent of the assumptions employed.

Fig. 8 shows a comparison between the deformed geometry assumed in the proposed method and that computed from the simulation model in Alashker et al. [4] for the prototype system in Fig. 1. As shown in Fig. 5, the floor is divided into six strips aligned in the \( y \)-direction, each 1.524 m wide, and the plots in Fig. 8 are made for the middle line of each strip. The comparisons are conducted for \( \delta = 1100 \) mm, the point at which the prototype system starts to collapse. Again, the model matches in certain areas and diverges...
4.3. Load capacity of floor system

The capacity of the floor to resist applied loading at a given deformation level is comprised of two components, namely the resistance of the floor slab and the resistance of the beam grillage. Foley et al. [2] proposed a similar summation approach in their model for estimating the collapse resistance of composite slabs. However, they did not account for the geometric constraints imposed by the steel beams on the deck and did not consider connection failure, factors that the proposed model addresses.

The capacity of the floor slab at displacement $\delta$, $V_{slab}$, is computed by integrating the vertical component of the membrane force in the floor slab along the periphery of the floor, as described by Eq. (6). This integral equation, which is a function of $w$ because it includes terms from Eq. (4), can be evaluated in a discrete manner by slicing the floor into strips and summing up their effects.

$$V_{slab}(\delta, w) = \int_T T_x \theta_x \mathrm{d}x + \int_T T_y \theta_y \mathrm{d}y,$$

where $T_x$ and $T_y$ are the tension membrane forces per unit width in the x-direction and the y-direction, respectively. The membrane forces at yield are defined as follows:

$$T_x = f_{yy}A_{yy} + \Psi f_{yd}A_D,$$

$$T_y = f_{yy}A_{yx},$$

where $f_{yy}$ is the yield stress of the mesh reinforcement, $A_{yy}$ is the area of mesh reinforcement per unit width aligned in the y-direction, $A_D$ is the mesh reinforcement area per unit width aligned in the x-direction, $A_{yx}$ is the area of mesh reinforcement per unit width aligned in the x-direction, $f_{yd}$ is the yield stress of the deck, and $A_D$ is the deck steel area per unit width. The factor $\Psi$ represents the effective area of the deck that yields.

The first part of Eq. (7) represents the tensile force in the reinforcing bars, while the second part represents the tensile force due to deck yielding. Since the deck is attached to the steel beams by shear studs, only a portion of the deck effectively yields at peak load, hence the $\Psi$ factor in Eq. (7). This factor is found to range between 0.4 and 0.5 in the prototype case studied herein, with a reasonable average of $\Psi = 0.45$. Eq. (8) does not have a second part like Eq. (7) because the steel deck is considered to have negligible resistance perpendicular to its flutes.

The floor capacity attributed to the beams at displacement $\delta$, $V_{beam}$, is limited by the axial capacity of the beam shear connections (Fig. 9b). Once a connection fails, it is assumed that the affected beam is no longer able to contribute to floor resistance. There is evidence in Alashker et al. [4] that, even if a connection at one end fails, the beam can still contribute to resistance if the connection at the other end is still intact; however, this reserve capacity is ignored for the sake of simplicity. The vertical component of all the ‘catenary’ forces in beams with intact connections represents the beam contribution to resistance, i.e.

$$V_{beam}(\delta) = \sum_i T_{beam,i} \theta_i,$$

where $i$ is the beam number. Note that $V_{beam}$ is a function of $\delta$ only, and not $w$, because it does not contain $\theta'$ as the slab equations do. In other words, $V_{beam}$ is uniquely defined by the geometry of the grillage which is, in turn, a direct function of $\delta$ only.
The beam capacity, $T_{beam,i}$, is a function of the deformation level, number of bolts, bolt capacity and bolt failure mode. The axial deformation in a shear tab connection is computed from the stretch that occurs along the beam axis as the system is loaded, which is split equally between connections at both beam ends. For convenience, the deformation of an individual bolt is assumed to be equal to the axial deformation of the connection. The appropriateness of this assumption is discussed later on in the paper. The axial resistance of the connection at the applied deformation level is therefore the sum of the resistances of individual bolts, which can be computed from any rational method, e.g. as in Sadek et al. [3], or from experimental results, if available. A connection is considered to have failed once its bolts are no longer able to support the load.

From Eqs. (6) and (9), the total floor capacity, which is expressed in terms of $\delta$ and $w$, is therefore

$$V_{floor}(\delta, w) = V_{slab}(\delta, w) + V_{beam}(w).$$

Vertical equilibrium necessitates that

$$V_{floor} = wL_xL_y.$$  \hspace{1cm} (11)

For a given $\delta$, Eqs. (10) and (11) can be solved together to compute the applied loading, $w$.

### 4.4. Failure criteria

Two failure criteria are employed in the model. The first is a local one, and it pertains to the beams. As previously mentioned, the resistance of a beam is conservatively assumed to vanish as soon as one of its connections fails. Once a beam is determined to have failed at a given deformation level, the contribution of that beam is removed from the analysis, as described next. The second criterion is a global one. It pertains to the overall system, which is assumed to fail when the vertical displacement at the centre of the floor reaches 10% of the shortest span length of the floor. This criterion is similar to that proposed by Park [5] and is intended to signal when the floor slab is no longer able to carry load due to fracture of its steel components.

### 4.5. Analysis procedure

The following analysis procedure is well suited for programming in a spreadsheet or for hand calculation. It progresses as follows.

1. Divide the floor into strips along both directions. Experience has shown that 4–6 is a reasonable number.
2. Determine the yield capacities of the floor slab, $T_x$ and $T_y$, from Eqs. (7) and (8), respectively.
3. Select an initial deformation level, $\delta$, for the removed center column. A good number to start with would be short span divided by 20.
4. Apply Eqs. (1) and (2) to determine the geometry of the beam grillage.
5. Apply Eq. (3) to get $\theta'$ in terms of $w$.
6. For each strip, apply Eq. (4) or (6) to get the angle of rotation associated with the strip.
7. Compute the deformation level in each connection and determine if the connection has exceeded its failure condition. If so, the beam attached to this connection no longer contributes to load resistance. Repeat for all connections.
8. Apply Eq. (6) through (9) to get the capacity of the floor, including in the calculation only beams that have not failed.
9. Equate (10) and (11) and solve for $w$.
10. Increment $\delta$ and go to step 4. A reasonable increment is short span over 100.
11. Stop the calculation when $\delta$ reaches 10% of the shorter span.

### 5. Model accuracy

The computational results in Alashker et al. [4] are used to investigate the accuracy of the proposed model. In the calculations in Alashker et al. [4], the bolt force versus deformation model in Sadek et al. [3] was employed, although, in general, any appropriate bolt force–deformation model could have been utilized. For simplicity, only the elastic and hardening portions of the model, and not the softening segment, were employed, i.e. the bolts were assumed to shed all load once the peak capacity was achieved.

The base simulation case is that of the prototype system shown in Figs. 1 and 3. Fig. 9 shows details of the shear tab connection. Although no such designation in made in Alashker et al. [4], the following naming scheme is used for easy reference in this paper. The base simulation case is designated B3–D1–R1, where the numerical part of the first segment of the name is the number of bolts, in this case 3. The numerical part of the second segment is the ratio of the deck thickness to the base case, where values of 0.5 and 2 are investigated. The numerical value of the third part is the ratio of steel reinforcement to the base case, where values of 0 (no reinforcement) and 2 are investigated. For example, B5–D2–R2 implies a simulation case with 5 bolts in the shear tab, double the deck thickness, and double the amount of steel reinforcement.

The results of the finite element analyses in Alashker et al. [4] and the corresponding computations using the proposed model are summarized in Figs. 10–13, and peak capacities are listed in

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**Fig. 9.** Development of ‘catenary’ force in steel beams.
Table 1: The major observation from the figures is that the proposed model deviates from the FEM results in Alashker et al. [4] at small deformation levels, but approaches the FEM results as the deformation level increases. This is expected, given the simplifying assumptions made, which focus on resistance mechanisms that only develop at large deformation levels. It is evident from the results that the proposed model does a reasonable job of capturing the effect of the variables on peak load capacity. In addition, the proposed model is conservative in general; the average prediction for peak capacity is 89% of the simulation result. The standard deviation for peak capacity is also rather small (8.8%), indicating that the model is capable of adequately treating the variables in question.

Fig. 10 shows a comparison between the force versus deformation responses computed from the finite element model (FEM) and the proposed design model for the base case. Clearly, the peak capacity is reasonably well captured: 972 kN from the design model versus 1100 kN from the FEM. The first two events in the sequence of failure are also captured at the correct displacement level, i.e. failure of the B1–column connection followed by the B2–column connection. The B1–B3 connection failure observed in the FEM is, however, not captured by the proposed model. In spite of the discrepancies noted, the success of the model is considered reasonable, considering the simplifications employed in the proposed model.

6. Model limitations

The proposed model has a number of limitations. First, the proposed model is tailored to the structural configuration shown in Fig. 1. However, the format in which it is presented is general enough to accommodate most common beam layouts. In addition,
the model assumes shear tab connections because they are widely used. Different connection types may present other challenges not considered in this paper.

It is assumed that the beams remain straight after loading. As shown in Fig. 8, this assumption leads to some differences between the assumed shape in the proposed model and the shape computed from the simulation model. This discrepancy could be reduced by taking into account the deformation of the beams. However, the benefit in terms of accuracy will come at the cost of added complexity in the model. As discussed in the previous section, the model is already reasonably accurate, so the added complexity is difficult to justify for the conditions considered. The decision to include beam deformation depends on the judgment of the user, but it is generally conservative to ignore such secondary effects.

It is further assumed that the axial stretch accommodated in a shear tab connection causes equal deformation demands in all connection bolts. This is clearly an approximation, since connection rotation causes the bolt deformation demands to vary along the depth of the connection. However, this simplification can be refined, at the cost of added effort, by computing individual bolt deformations as a combination of the axial stretch that occurs due to floor deformation in addition to the deformation attributed to connection rotation. While this was not observed to be necessary for the prototype floors considered, as evinced by the good comparisons between the model and simulation results, it may be required for connections in which the connection depth is significantly larger than that considered herein.

Simulation results for the prototype system show that binding, i.e. where the beam flange bears upon the column flange, may occur early on in the loading regime. However, its effect does not persist because of the stretching action to which the connections are subjected under increasing loads. Nevertheless, special attention must be paid to computing the connection axial strength in the presence of binding action if the beams are deep, since such cases may lead to large increases in bolt deformation demands and precipitate premature connection failure.

The failure condition, at which the capacity is computed, i.e. at 10% of the shorter span, is an empirical one. As previously noted, this criterion has been used by others, but it should be refined as new information becomes available. In addition, the model, in its current form, was developed for computing static, pushdown capacity and is not directly applicable to conditions in which dynamic effects are critical. Finally, the proposed model is calibrated to a computational model, albeit one that was thoroughly validated in Sadek et al. [3] and Alashker et al. [4]. Experimental research will provide added credibility to the model and is recommended as future work.

7. Summary and conclusions

A design-oriented model was proposed for computing the collapse resistance of steel–concrete floor systems subjected to interior column loss. The model borrows from existing RC slab models that rely on the development of membrane action to resist collapse loads and is based on a number of additional assumptions. Key assumptions include idealizations pertaining to the deformed shape of the system, development of failure-resisting mechanisms, and overall system behavior. The model was presented as a step-by-step procedure implemented within a framework that tracks the evolution of damage in a floor system loaded up to failure. The model was proposed in a format that could be easily adjusted to take into account various beam configurations and accounted for deck thickness and strength, reinforcement area and strength, as well as connection strength. Extensive comparisons with numerical simulation results show that the model was able to reasonably capture the effect of influential variables on collapse resistance in spite of the simplifying assumptions made. Key limitations on the model were presented and experimental research was recommended to provide conclusive evidence of the accuracy of the proposed model.

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