USE OF POST-TENSIONING CABLES ON BUILDINGS SUBJECT TO PROGRESSIVE COLLAPSE

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ABSTRACT

Requirements to comply with progressive collapse guidelines in Federal and Military construction projects result in specific design and constructability demands for the main resisting framing of a building. While the cast-in-place reinforced concrete and steel industries have provided a number of practical solutions, the precast concrete industry has focused on load-bearing panel construction and façade and cladding components, thus limiting the application of precast to main framing systems. This paper discusses the use of high strength steel cables or threaded rods, similar to those used in post-tensioning but installed without a significant prestress, to provide a ductile connection in precast beam joints, thus allowing for a continuous beam behavior in the event their support column is removed, as required by progressive collapse guidelines. A preliminary analysis shows that this solution is potentially effective for low to mid rise buildings with typical office spans, requiring only minimal aesthetic and cost-impacting modifications to a typical precast design for ordinary office buildings.

Keywords: Progressive collapse, post-tensioning, main framing, ductile behavior, connection, catenary.

INTRODUCTION

After the U.S. General Services Administration (GSA)[1] and the U.S. Department of Defense (DoD)[2] issued design and analysis guidelines for buildings subject to potential progressive collapse, a number of authors developed solutions for different structural types and configurations to comply with these guidelines. Later on, the Interagency Security Committee (ISC)[3] issued a set of security criteria for Federal facilities which confirms the requirement to use the GSA guidelines. While the GSA and UFC guidelines differ in some details, essentially both require structural framing members and the structure as a whole to provide a certain level of strength after the notional removal of a column or load-bearing panel, allowing the designer to choose from a number of methods related to the desired building size, type, and/or required Level of Protection. Therefore, current progressive collapse requirements are not threat-specific, but instead are intended to prevent, mitigate, or reduce the potential for progressive collapse in the event of a major incident threatening the stability of the building.

Generally speaking, most of the already developed mitigation solutions apply to steel and reinforced cast in place concrete structures. A likely reason is, since progressive collapse requirements emphasize the need for connection resiliency, moment reversal, and redundancy, construction types which provide those features per se, or can be more readily adapted to do so, are the most obvious candidates. Standard precast framing construction, commonly based on simply supported beams with open or non-structural sealed joints and columns with bearing-only corbels having minimal connectors to the beams, does not appear to provide the required features. On the other hand, precast components are successfully used as cladding or “skin” components in buildings subject to progressive collapse threats, and load bearing precast panels have performed very well in extreme events such as the attack to the Khobar Towers.

The purpose of this paper is to demonstrate that a properly designed, detailed, and built precast frame can comply with the requirements of progressive collapse guidelines. While some modifications to a standard gravity-only or minimal lateral demand design are necessary, this study illustrates how ordinary precast construction can be modified to meet progressive collapse requirements and is intended to demonstrate that aesthetic and additional cost impacts can be minimized to make the solution competitive.

This study focuses on the response of a typical row of exterior precast spandrel beams, on which one intermediate external support column has been notionally removed as required by the guidelines. The study’s basic concepts are essentially applicable to the removal of a corner or an interior column too, but additional details and structural checks need to be developed for such cases. Progressive collapse guidelines contain specific structural demands for other components such as vertical ties (columns) and horizontal in-plane ties (slabs), but since adequate solutions for these requirements are well documented and applicable to typical precast construction, they are not discussed here.

PROGRESSIVE COLLAPSE GUIDELINES

While a detailed review of the GSA and UFC references is outside the scope of this paper, a reduced summary is provided for better understanding of the requirements.

Progressive collapse guidelines require building structures to be analyzed for the notional removal of one ground floor vertical support member at a time. For example, in a building supported by columns, analysis must include the response after the removal of one first floor exterior column over the long side of the building, one over the short side, a corner column and one interior column, as shown in Figure 1 below. Requirements may change for buildings with uneven spans, unusual layouts, or different floor arrangements.

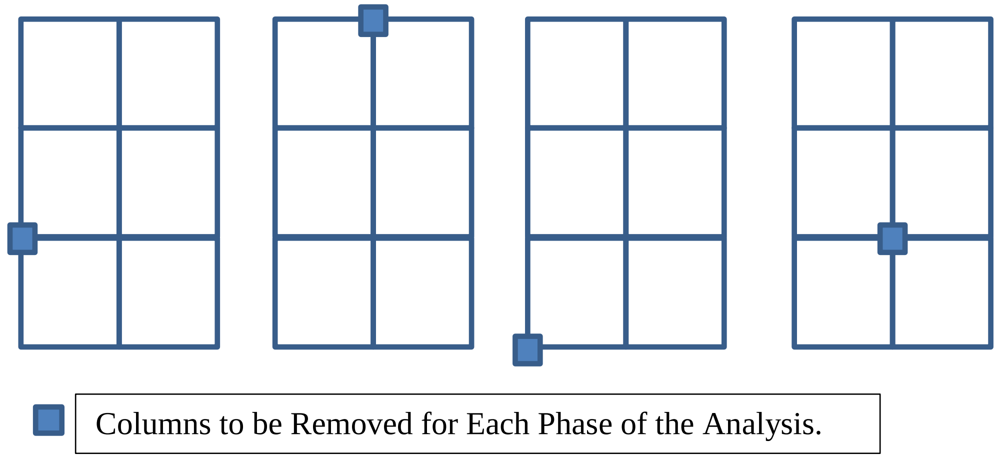

Figure 1 – Notional Column Removal for Typical Floor Plan

The removal of one supporting column requires that the beams supported by it must be able to withstand a double span condition. The guidelines allow the designer to model this event in a number of different ways, from Linear Elastic Static to Non Linear Plastic Dynamic. While some specific buildings requiring a higher Level of Protection as defined in the UFC guidelines or higher than ten stories according to the GSA guidelines need to be analyzed with more sophisticated methods, most low to moderate occupancy, low to mid rise buildings can be analyzed using Linear Elastic Static techniques.

This paper focuses on the basic feasibility of a type of solution and therefore only uses basic analysis: more advanced studies such as High Fidelity Finite Elements Analysis, as well as full scale testing, should be considered as the solution is further evaluated. The following analysis is also based on the GSA guidelines, but note that the UFC document is conceptually similar and most of the findings on this paper also apply to structures designed under it.

The GSA guidelines “recommend” the following load combination when designing for progressive collapse with Static Analysis techniques (not specifying whether it is Linear or Elastic-Plastic):

$$\text{Check Load} = 2(\text{DL} + 0.25\text{LL}) \tag{1}$$

Note that the incidence of live loads over the analysis is minimized by the use of a 0.25 load factor.

The basic acceptance criteria for Linear Elastic Static Analysis according to GSA is based on Demand Capacity Ratios (DCR) defined as:

$$\text{DCR} = \text{Qud} / \text{Qce} \tag{2}$$

Where Qud = Acting force on component / connection (moment, shear, axial or combination)
Qce = Ultimate, unfactored capacity of the component / connection.

Maximum DCR values are listed in the GSA document for different structural types and materials, but they are always greater than one. Note that the higher the DCR, the less demanding is the structural performance: for example, a DCR of 3 (applicable to low slenderness steel beams in flexural response) implies that a beam with an ultimate capacity of 1/3 of the theoretical demand would be acceptable.

For reinforced concrete structures, GSA requires a DCR $\leq$ 2 for typical structure configurations and DCR $\leq$ 1.5 for atypical configurations. Since this analysis focuses on regular grid and repetitive floor plans, a maximum DCR of 2 is used. Although no special consideration is given to precast structures, the “Design Guidance” section for reinforced concrete emphasizes the need for redundancy, proper detailing to provide structural continuity and ductility, and the ability to resist load reversals. While all these features can be achieved in precast design, they are not often present in conventional precast frames.

Note that the individual inability of a component or connection to meet the minimum DCR demand does not automatically imply the structure as a whole fails to meet progressive collapse requirements: if the structure has the ability to redistribute applied forces, for example, in multiple continuous beams after a plastic hinge forms at the maximum moment location, a step by step analysis may be performed and may result in an adequate performance.

LINEAR STATIC DESIGN APPROACHES

The GSA document leaves the particular analysis approach to the designer. For the typical case of a row of beams which lose a support column, the response of the structure will be a combination of flexural action, in which the beam to beam connection must provide some degree of continuity and the beams themselves must be designed to withstand a double span condition, and catenary action, in which vertical deflection of the connection above the failed column will create axial (tensile) forces in the beams, which are resisted by the components and connections, designed either for flexural action only or specifically for the catenary action forces. The UFC document describes these two methods as “Direct Design – Alternate Path” and “Indirect Design – Tie Force”, respectively. See Figure 2.

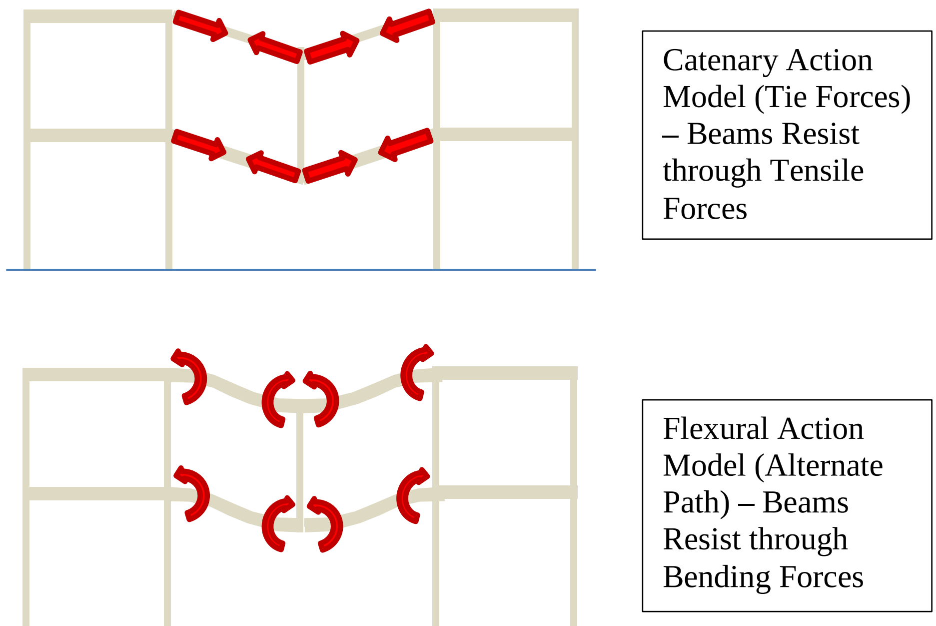

Figure 2 – Linear Static Design Approaches

The progressive collapse design guidelines do not elaborate on the interaction of these two response modes or which one would be prevalent in a specific structure, leaving the choice to the designer as long as the building type and use are compatible with the design method. In addition, the incidence of catenary action on the structural response increases with the deformation of the beams, but the UFC guidelines allow the designer to assume an end rotation of 0.2 rad (11.3 deg), resulting in a maximum deflection of (0.2 x L / 2) = 0.1 L, where L is the total span after the column is removed (twice the typical span of the structure). The guidelines only require the designer to show that the members work adequately for this notional displacement, without having to calculate what the actual predicted deflection will be.

PRECAST SOLUTIONS FOR PROGRESSIVE COLLAPSE

To some extent, the birth of the concept of progressive collapse was the result of a precast structure failure: the Ronan Point Building, a 22-story apartment complex built in London in 1968, where the explosion of a gas kitchen in the 18^{th} floor caused the failure of an entire corner of the building. While there is no accurate, widely accepted definition of what constitutes “progressive collapse”,[4] an event originating in a small area which extends to a large portion of the structure qualifies as “disproportionate collapse”, which is a condition of progressive collapse. Only a few years after this incident, the cast in place reinforced concrete Skyline Tower in Virginia failed in a similar way, although the event was originated by an early shoring removal.

After the Ronan Point incident, the structural engineering community published a number of studies and publications on the subject. While the mechanics of the models were not always similar to what contemporary progressive collapse criteria requires, most of the recommendations of that time still apply today. For example, NBSIR 75-715[5] proposes to consider failure at any one story level, not just ground level, and focuses on housing construction (closely following the Ronan Point incident), but its recommendations for structural continuity in floor to wall joints and load bearing panel construction are applicable to modern precast construction.

After the initial spike in interest due to the Ronan Point and Skyline Towers failures, the number of publications and standards related to progressive collapse began to fade. The interest in progressive collapse was renewed after a series of high profile terrorist attacks in various parts of the world. It is worth noting that of the many buildings attacked in various ways over the last two decades, arguably one of the best performances or least damaging outcomes belongs to a precast structure. The Khobar Towers (an US Air Force housing complex in Saudi Arabia), which did not collapse after being attacked with the equivalent of 20,000 lbs. of TNT, sustained fatalities, but these were mostly related to glass shards and flying debris. The Khobar Towers used cables to provide continuity between load bearing precast panels, a solution closely related to what is proposed in this paper.

In comparison, the attack to the Murrah Federal Building in Oklahoma created a relatively moderate initial damage, but resulted in a large structural collapse and loss of lives. Figure 3 shows the façade of the Khobar Towers after the attack.

Figure 3 – Building 131, Khobar Towers, Saudi Arabia.

In contemporary progressive collapse prevention design, load bearing precast panel construction has been well analyzed and described[6]. The use of vertical ties, either as cast in place dowels spliced to conventional rebar or continuous cables anchored at each floor level, is relatively easy to implement and does not present any particular difficulty for precast construction. Note that the use of vertical cables to mitigate progressive collapse has been proposed for cast in place concrete structures, too[7]. Similarly, horizontal ties for diaphragm action can be materialized with additional reinforcement in a cast in place concrete floor or in a thin poured layer on top of hollow core precast slabs. Since load bearing panels can behave as deep beams once a supporting panel underneath is removed, the development of alternate path solutions is restricted to designing adequate panel to panel connections, for which an entire floor height is available and therefore is not difficult to implement.

However, the efficiency of load bearing precast panels is limited by the need for openings. As long as openings do not cover more than a small percentage of the panel surface, their behavior can be considered equivalent to that of a solid panel, providing for local strengthening or reinforcement around openings when necessary. However, when continuous glazed surfaces are required as in many typical office buildings, load bearing panels are no longer a valid solution. In such cases, steel or cast in place reinforced concrete framing (beams and columns) are the most common solutions used today.

Cast in place concrete has the advantage of being continuous by nature; therefore, a proper detailing and design of reinforcement allows for adequate designs with limited additional costs. One drawback of concrete beams is, since they are primarily designed for flexural response using concrete's compressive strength, they have limited catenary action tensile capacity unless specifically designed for it. On the other hand, steel structures need to be connected and detailed with special care, since many moment connection designs adequate for small displacements (and thus for conventional loads) are not adequate when large displacements are expected, as in the case of a support column being removed. However, steel beams designed for flexural response only have an inherent high tensile capacity, so they are also adequate for catenary action as long as their connections are properly designed.

CASE STUDY - PRECAST FRAMING CHARACTERISTICS

Figure 4 shows a typical office building façade using standard precast framing components. Story height is assumed to be 15 feet, with a typical span of 30 feet in both directions and a regular column grid. These spans comply with the maximum loaded area of 1,800 sf over a single support component as defined in the GSA guidelines. Floors consist of 4-inch thick concrete on steel deck and are supported on secondary floor beams which load the façade spandrel beams and interior beams. A continuous strip of windows is assumed to extend from column to column, thus not allowing for the developing of any load bearing wall action. Only two stories and a roof parapet are depicted in the drawing, but that is not a design requirement. The solutions proposed here are intended to be applicable to any building up to ten stories high, as described in the GSA guidelines.

The spandrel beams are conventional precast double tees, and their dimensions as part of an exposed façade are often controlled by architectural / aesthetic requirements; in this case they have been assumed to be 5 feet deep. The columns are of precast construction, assumed to be 18 inches × 18 inches, and are fitted with corbels supporting the spandrel beams. However, since progressive collapse guidelines (in this case GSA) state that the correct way to model the removal of a vertical support is to remove the vertical element only and not the "connection/joint or horizontal elements that are attached to the vertical element at the floor level", it is not sufficiently clear whether the corbel should be removed or not. Even if this is the case, adequate load transfer connections can be provided within or above the spandrel beam depth, thus complying with the strictest interpretation of the requirements.

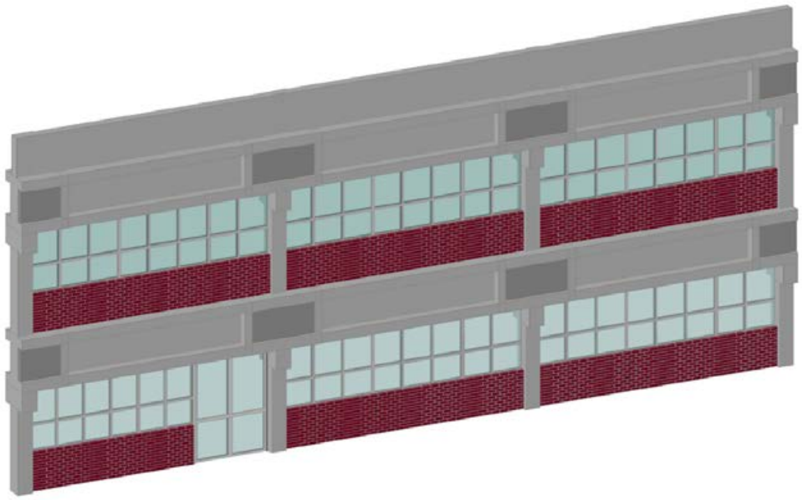

Figure 4 – Typical Precast Framing Office Building Façade

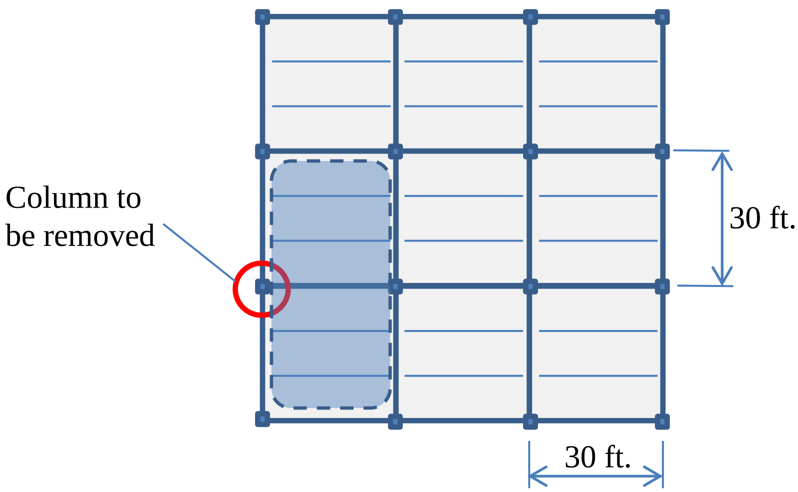

Figure 5 – Typical Precast Framing Office Building Floor Plan

The typical spandrel beam was first predesigned for a single span condition, as it would be for a conventional building not subject to progressive collapse check requirements. The design load was based on ASCE 7[8] assuming gravity loads only, and therefore the governing combination is:

$$W = 1.2\ DL + 1.6\ LL \qquad (3)$$

For the assumed building materials, sizes and dimensions, and an assumed average live load of 75 psf (the average of 50 psf office loads and 100 psf corridors / assembly areas / computer rooms), factored live loads represent 42% of the total factored load. Therefore, a change in use of the building or architectural layout may have a significant impact over the total design load. On the other hand, for progressive collapse checks, the check load combination W = 2 (DL + 0.25LL) results in a live load incidence of only 10% over the total design load. Therefore, live loads have only a small influence over progressive collapse checks.

The required flexural reinforcement ratio for gravity loads' single span condition would be in the order of 0.3%. This is barely the minimum recommended for crack width control, which is consistent with the large dimension of a typical spandrel beam. However, a precast beam will have additional longitudinal reinforcement for constructability (ties connections, crack control) which can collaborate in extreme loading cases. The only additional requirement is that all of these bars must either be continuous over the entire span or be lap spliced to develop full tensile capacity, which is not required in conventional design. The additional cost should be minimal if any, since normal spans (in this case 30 feet) allow for the use of single piece longitudinal bars. Figure 6 shows a cross section of the assumed spandrel beam, with the contribution of the "non-flexural" reinforcement to tensile catenary action almost doubling overall tensile capacity.

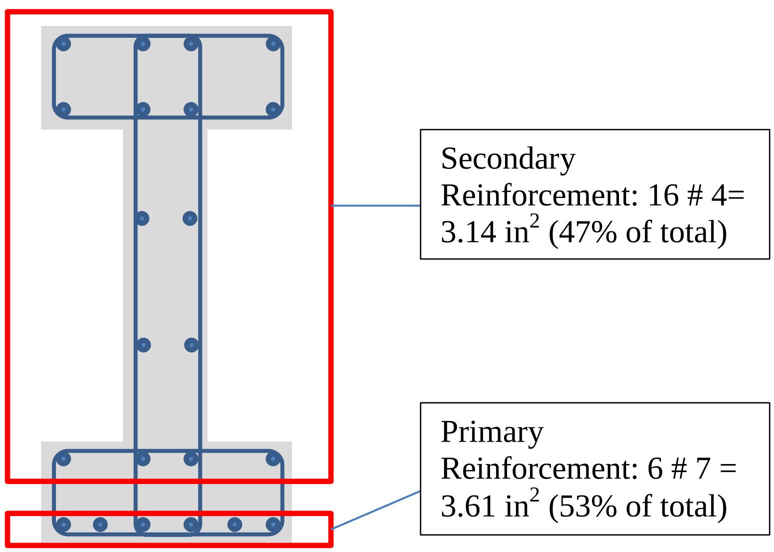

Figure 6 – Cross Section of Typical Spandrel Beam

ALTERNATE PATH CHECKS

Under an alternate path model, beams must be designed to resist a double-span condition. It is assumed that each row of floor beams shall be designed to withstand the load of the floor supported by it. Reinforcement must be continuous all the way to the supports, as opposed to being cut off at zones where it is not needed for gravity design. In addition, since beam connections over columns not removed will be subject to negative moments, reinforcement must be symmetrical. A preliminary design of the same cross section for a double span subject to progressive collapse check loads, using the Strength Increase Factors (SIF) listed in the GSA guidelines and using ultimate capacity not reduced by phi factors as listed in the guidelines, shows the required reinforcement ratio for flexural response will be in the order of 0.9%, still within economical design criteria and far below the balanced reinforcement ratio (2.9% for 4,000 psi concrete and 60 ksi steel).

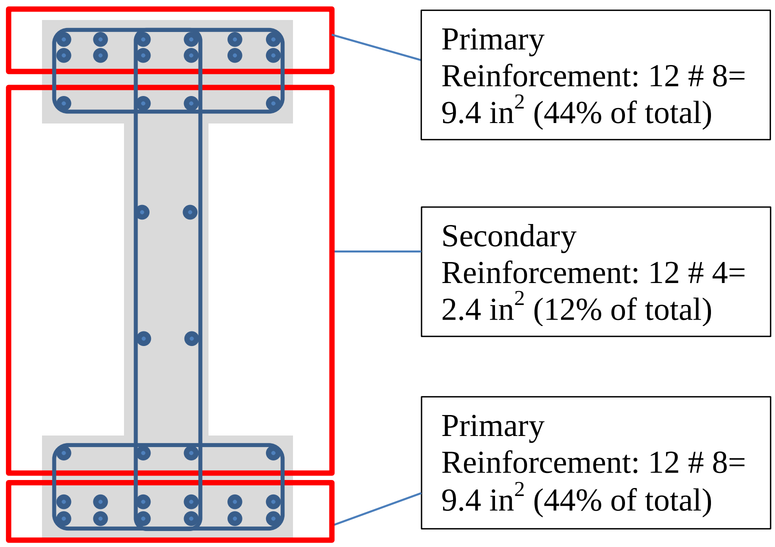

Figure 7 – Cross Section of Typical Spandrel Beam (Two Span Condition)

CATENARY ACTION (TIE FORCES)

When the support column of interests is removed, a number of tie forces develop in the structure. Figure **8** shows the schematic view of such forces in a typical building frame.

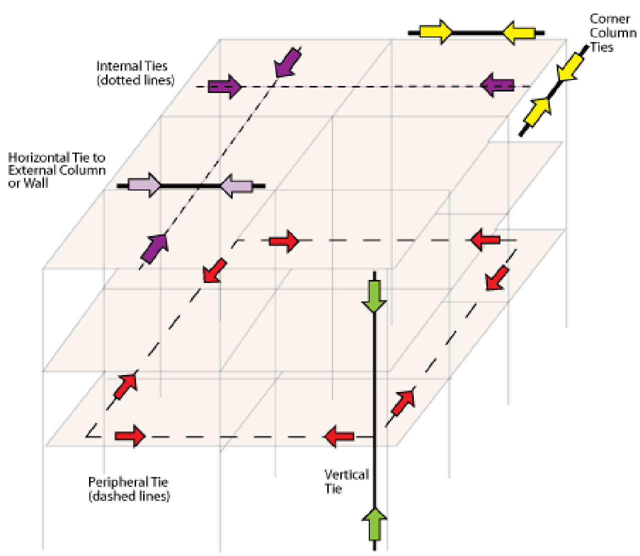

Figure 8 – Tie Forces Schematic (from UFC 4-023-03)

The typical precast beams designed in the previous paragraph for a double span bending moment condition shall be analyzed for catenary action. While only a combination of tests and high fidelity models could give a good estimate of which response mode is predominant (bending or axial), for the purpose of this study, which is to find out whether a conventional precast frame can be converted for progressive collapse with minimal cost, they are assumed to act separately.

The UFC guidelines state that members intended to perform in catenary action must be able to develop a minimum end rotation of 0.2 rad, resulting in a midspan deflection of 0.2 L (L being the typical span between columns, before any is removed). A simple compatibility analysis can provide a reasonable estimate of whether the precast beams defined above will reach equilibrium at the specific deformation set in the guidelines.

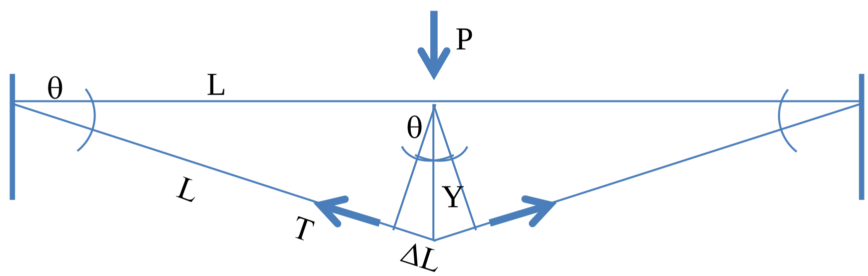

Figure 9 – Forces and Deformation in Catenary Action

Given a certain assumed catenary shape, the strain on the longitudinal reinforcement, the axial stress assuming elasto-plastic behavior, and the axial tensile force (T) can be calculated. By comparing the maximum vertical force (P) each configuration can take with the applied reaction in the connection divided by the maximum Demand Capacity Ratio, we can determine which configuration is needed to reach equilibrium. For the previous example, equilibrium would be reached at a 0.04 rad end rotation, equivalent to a 0.02 L maximum deflection, about 10% of the vertical deflection prescribed by UFC. Additionally, the predicted stress in the reinforcement steel would be in the order of 25 ksi, well below the elastic limit for 60 ksi steel (plus the 1.25 SIF factor prescribed by GSA). However, there are a number of factors not considered in this simplified model, such as:

- Column deformation and rotation at the beam connection.
- Out of plane membrane forces developed by the floor slab.
- Local deformation at the beam to beam connection.
- Local slippage or failure of some bars at heavily cracked / spalled concrete areas.
- Concrete tensile capacity (although concrete would be expected to crack for forces lower than the floor load divided by the DCR, it would potentially reduce deformations until cracking, when steel starts to carry the entire load).

The simplified model indicates that a precast spandrel beam designed for a double span condition has the potential to develop large catenary action forces, being able to carry the load of the removed column with additional reserves.

Since the UFC guidelines require demonstration that the tie force components are able to develop the required capacity at the prescribed deformation, without requiring the actual behavior to match that, an extension of the previous calculations to this case shows that when the end rotation reaches 0.2 rad, the longitudinal rebar is well into plastic range but with a strain of about 2%, well below the estimated ultimate strain of conventional reinforcement steel. It must be noted that the conventional definition of ultimate elongation, which is listed for each particular steel specification, is based on the local throat reduction at the test failure zone, and is not the same as the ultimate strain based on the original section properties. However, in this case, the predicted performance is well below the expected limits.

BEAM TO BEAM CONNECTION

For the Alternate Path solution to work, it must provide the same or higher moment capacity through the beam to beam connection. Conversely, if a Tie Forces model is adopted, the connection must develop the same or higher axial forces. The basic performance requirements for this connection are:

- Resiliency / Ductility
- Redundancy
- Reversibility.

Among the solutions which have been proposed, specifically for precast members, are field welded connections using embedded plates shop welded to reinforcement bars or studs. Assuming that multiple connection plates are provided in a symmetrical layout, the redundancy and reversibility conditions are met. However, such a connection has limited capacity to develop large plastic deformations, and therefore cannot be considered ductile.

Crawford[9] proposes the use of cables, either embedded into a floor slab or attached to the sides of a steel beam, to provide continuous catenary action through a removed support column. This concept can be readily applied to precast beams, but the need for a continuous cable or cables can be replaced with a local tensile tie device at the connection, splicing to the regular reinforcement of the beam. Additionally, by taking advantage of the reinforcement bars needed for constructability reasons, additional cost is minimized and conventional designs can be converted for use in progressive collapse projects with a minimal effort.

Another feature of typical precast beams is that double tees are often cast as solid rectangular sections near the supports, either for architectural or structural reasons. The vertical surface of the rectangular block’s edge can be used to install and anchor the tensile connectors in the field with no further modifications to a standard precast design and no aesthetic impact since the connectors and their anchors will be hidden.

In this study, high strength threaded rods were assumed due to ease of connection (requires only an oversized washer and a nut designed to keep local stresses in concrete low), but cables used in post-tensioning may also be used. In all cases, connection bars are not intended to be stressed, but rather to remain slack or snug tight unless a progressive collapse scenario occurs. Therefore, the precast structure as a whole is intended to perform like a regular structure for gravity loads, thermal deformations, and differential settlements without any special considerations. See Figure 10 for a view of the proposed connection.

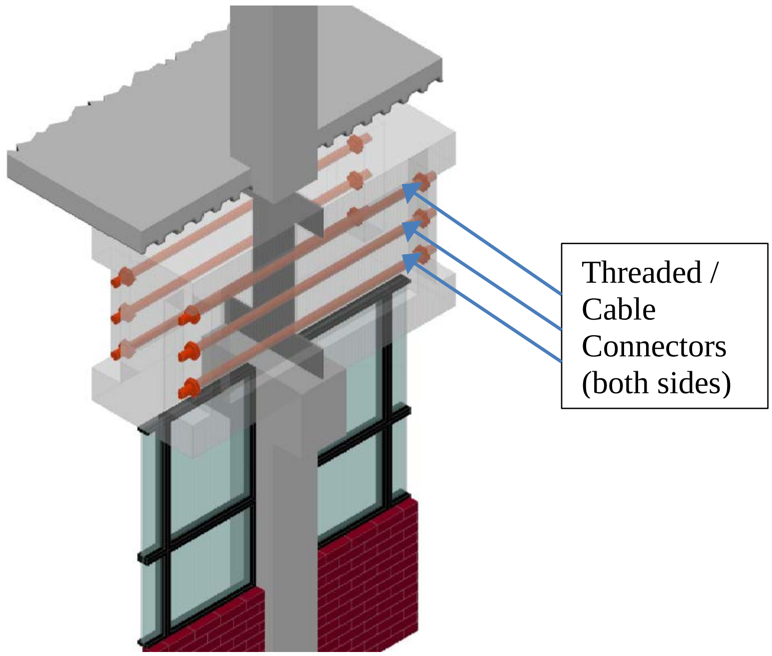

Figure 10 – Beam to Beam Cable or Threaded Rod Connectors

For the connection rods to be easy to install, they must be located within the thickened precast section. Therefore, their internal arm is reduced. Figure **11** shows the forces within and outside the connection zone.

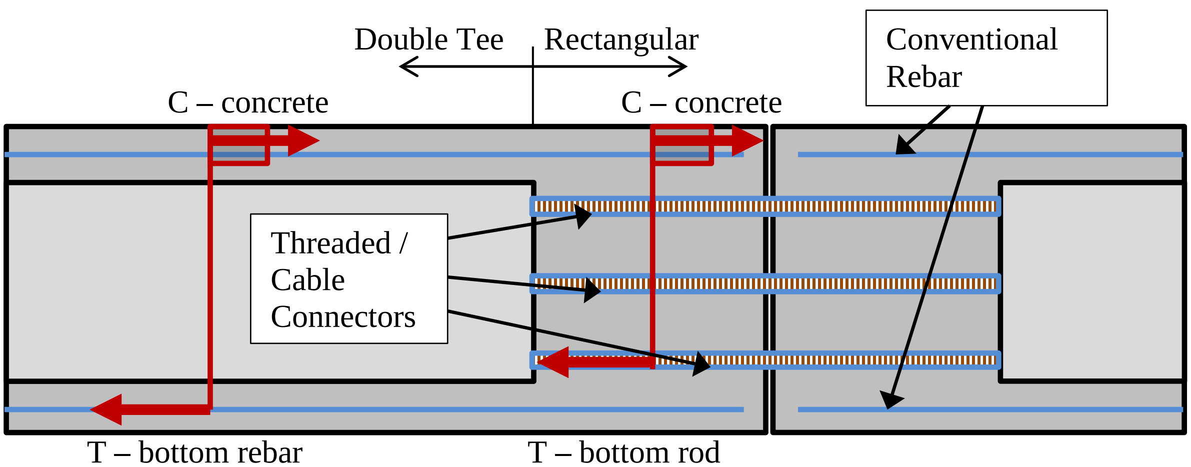

Figure 11 – Cable / Rod and Reinforcement Forces at Connection

At the beam to beam joint, the connection must also be able to receive the reaction of the column and split it between the two beams. As discussed above, the precast corbel may be considered as being removed together with the support column, and therefore it is not considered in this analysis. Loading the tensile connectors in shear will reduce their capacity to absorb axial forces, plus it will reduce the overall ductility of the connection. An H-shaped connector shear plate is proposed: the hole sleeves in the precast should be slotted horizontally to allow for the tensile connectors to elongate without loading the shear pins. The reduced section of the H plate ensures that no significant moment is transferred through it. The tensile ties should also be encased in an oversized sleeve to ensure the shear pins engage for vertical loads before the ties make contact with concrete. Figure **12** shows a conceptual view of the proposed connection.

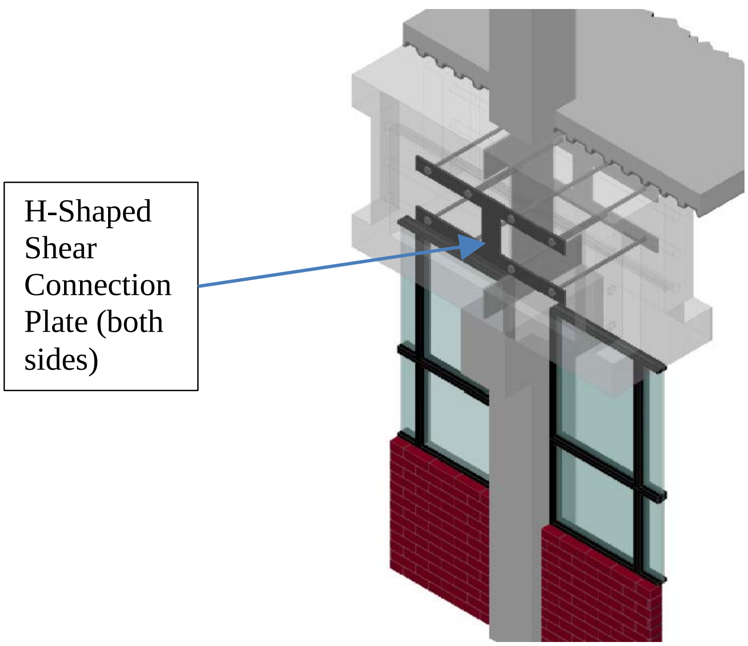

Figure 12 – Beam to Beam Shear Plate – Shear Pin Connectors

CONNECTION STEEL ALTERNATIVES

There are numerous high strength steels available in the post-tensioning and threaded rod market. However, they do not only differ in ultimate and yield strengths, but also in ultimate elongation and strains, resulting in widely diverging ductilities. Moreover, conventional definitions of steel types and grades often list ultimate elongations based on standard lengths of tested bar (typically 20 bar diameters), which does not accurately reflect the ultimate strain. The choice of a steel type over another depends on many factors, including structure geometry, predicted response, and even cost effective detailing. For example, a high grade post tensioning strand may provide a very high capacity and may be found to be ductile enough, but the standard trumpet anchors used in post tensioning may be too large for a limited width section. However, high grade threaded bars of the type commonly used in rock anchors and foundations may have a lower capacity, but can accommodate any washer and nut size as long as fracture stresses in concrete are acceptable (adequate confinement at anchor areas is key). Figure **13** shows typical stress-strain diagrams for a number of common steels:

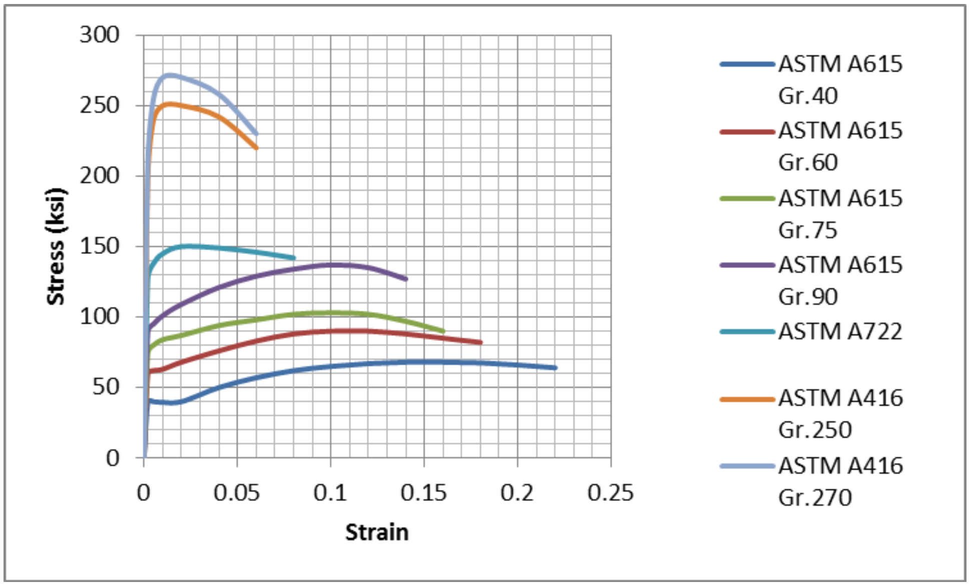

Figure 13 – Stress – Strain Diagrams For Typical Steels

Once an estimated rotation at the hinge has been calculated using catenary action, the strain demand for the connection bars can be calculated using geometry and strain compatibility, as follows:

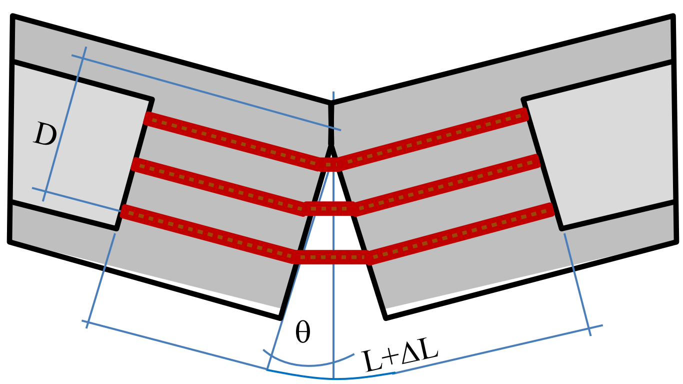

Figure 14 – Connection Bar Elongation

The elongation of the connection bar plays a part on the overall rotation of the beams. The designer must account for compatibility of elongations between the connection bars and the precast beams reinforcement when subject to catenary actions. The strain on the connection bars must also be adequate to develop the required moment capacity. Since the ultimate strain of the connection bars is limited by the type of steel chosen, but the bar elongation is a function of beam depth (distance from assumed point of contact at the top flange to centerline of bar under study) and the required hinge rotation to reach equilibrium under catenary action, the connection bars must have a minimum length, which also governs the size of the thickened rectangular section.

In the example, the required rectangular section length for relatively low grade (90 ksi) bars was in the order of 3 feet, compatible with standard precast practices and typical building aesthetic demands. However, if a prescribed rotation of 0.2 rad is used as specified in the UFC guidelines, the elongation demand grows and the minimum required length of the thickened section would be in the order of 12 feet, impractical to nail the connection bars in the field and requiring substantial modifications to a standard precast design. In cases like this, the designer may experiment with different steels of milder grade, allowing for larger ultimate strains and therefore allowing for use of shorter bars.

The contact point at the top flange must also be checked for concrete crushing under large deformations. While the best way to calculate its response is by means of full scale tests or high fidelity modeling, a number of measures can be taken to minimize the probability of a brittle crushing failure-for example, by confining the concrete near the flanges with closely spaced ties and by slanting the contact surface to match the predicted hinge rotation in the event of loss of the support column. Since beam to beam contact is not required for normal loading conditions, the space between flange ends can be filled with a lower compressive strength material, only intended to seal the interspace and sacrificial in the event of a progressive collapse scenario.

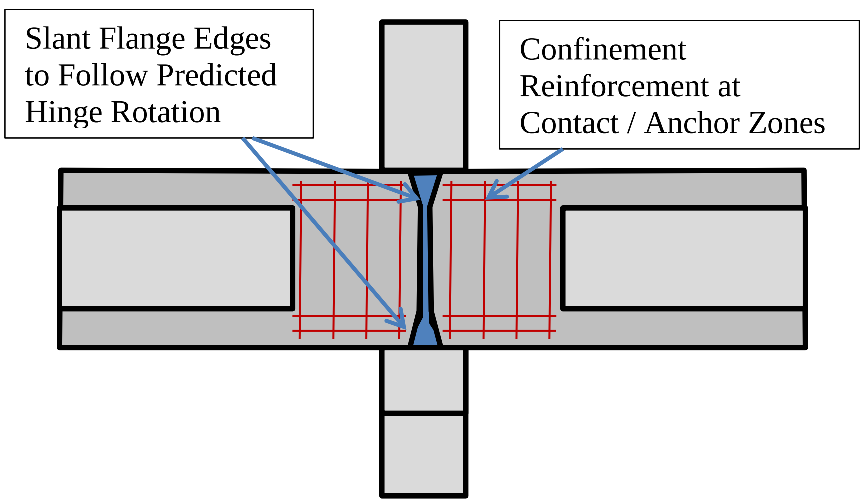

Figure 15 – Detailing to Minimize Concrete Crushing

CONCLUSIONS

Precast framing structures can be used for applications requiring consideration of progressive collapse, with minimal changes to standard designs for conventional loads. Design of vertical (column) and horizontal membrane (slab) ties does not present any special challenge. Design of horizontal beams and beam to beam connections to provide Alternate Path or Catenary

Action when a support column is removed can be achieved with minimal additional cost, no aesthetic impact, and use of standard, readily available components. Selection of the adequate type of steel for the beam to beam connections is key. Since designs intended to comply with GSA or UFC guidelines need to be submitted to the appropriate authorities for approval, additional developments in the shape of load tests and high fidelity Finite Element modeling is needed to properly support all design assumptions.

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