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IMPLEMENTATION OF TIE FORCES FOR PROGRESSIVE COLLAPSE RESISTANCE OF A 12-STOREY BUILDING

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ABSTRACT: In the US, many important government and military buildings have special design requirements intended to minimize the potential for Progressive Collapse (PC). In Canada, design of government buildings to resist PC is in the process of being implemented and the requirement is currently assessed on a case-by-case basis. Per the project design requirements, this building needed to provide resistance against PC. At the time the project design commenced, there was no CSA design standard for improving the resistance of buildings against PC. Therefore, PC was addressed following the design guidelines for Tie Forces of the US Department of Defense (DoD) Unified Facilities Criteria (UFC) 4-023-03 *"Design of Buildings to Resist Progressive Collapse"*. This standard applies to American military buildings 3 stories or higher, and is the most widely used criteria for Progressive or Disproportionate Collapse in the US. The design to resist PC following the UFC document is threat independent. The Tie Forces approach was selected since it could be most easily adapted to a reinforced concrete flat plate/flat slab structure – the most common form of construction for high rise buildings in Eastern Canada – and the building Occupancy Category permitted this method to be used. In the Tie Forces approach, the entire building is "tied" together with horizontal and vertical tension elements incorporated into the structure to enhance continuity, ductility and structural redundancy to enable re-distribution of loads from damaged areas into un-damaged areas. The additional tension elements must be properly interconnected and anchored, using similar detailing required for seismic design. Critical load-bearing components must be designed to respond in a ductile manner under dynamic loading conditions. This paper will discuss the challenges of incorporating the detailing and additional tension strength requirements of the UFC into the 12-storey flat plate/flat slab reinforced concrete building structure.

1. Progressive Collapse

Progressive collapse (PC) is defined in the commentary of the American Society of Civil Engineers Standard 7-05 *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-05) as "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it."

Probably one of the most famous events was the 1968 collapse of the Ronan Point apartment building where an accidental explosion in a corner kitchen on the 18th floor blew out the exterior wall panel and failure of the corner bay propagated up and down to cover almost the complete height of the building. After this event, England was the first nation to address PC explicitly in their building standards. PC due to terrorist attacks is a relatively rare event, as it requires the combination of abnormal loading and a structure that lacks continuity, ductility and redundancy. As a result, there have not been many PC events recorded In the Western nations. One of the few and more severe cases of PC from a malicious attack is the Oklahoma City bombing of the Alfred P. Murrah Federal Building in 1995. It is estimated that 42% of the Building was destroyed by PC and only 4% by the explosion or blast. This attack claimed 168 lives and left over 800 injured. While the probability of occurrence is low, the consequences of not having a building capable of reducing the potential for PC could be catastrophic. Therefore, design requirements have been developed and implemented to protect important military and federal facilities in the US. To date final policies have not been adopted in Canada for implementing design to resist PC for government and military buildings. Currently each facility is assessed on a case-by-case basis by the Canadian Department of National Defence (DND) based on a threat risk assessment and the building occupancy.

It is important to point out that, as stated in (Ellingwood, Smilowitz, Dusenberry, Duthinh, Carino, 2006) there have been numerous cases of PC of buildings during construction and data suggest that buildings under construction have a higher probability of sustaining collapse. However, the design approaches against PC mentioned in this paper relate only to PC of completed buildings in service.

1.1. Design Approaches to Resist Progressive Collapse

Prevention or mitigation of PC is generally achieved using two different methods: 1) indirect design and 2) direct design. The indirect method consists of improving the structural integrity of the building by providing redundancy of load paths and ductile detailing. The direct method is divided into two approaches: Specific Load Resistance (SLR) and Alternate Path (AP), with the latter being the most widely used in the US. The following paragraphs provide a brief description of these approaches.

1.1.1. Indirect Design: Tie Forces

The Tie Forces approach is the most popular and widely used indirect design method against PC and it is only allowed in the UFC 4-023-03. In this approach, the building is “tied” together by adding horizontal and vertical tension elements with the goal to allow the structure to transfer loads from a damaged area into an undamaged area using catenary or membrane action. This is illustrated in Figure 1.

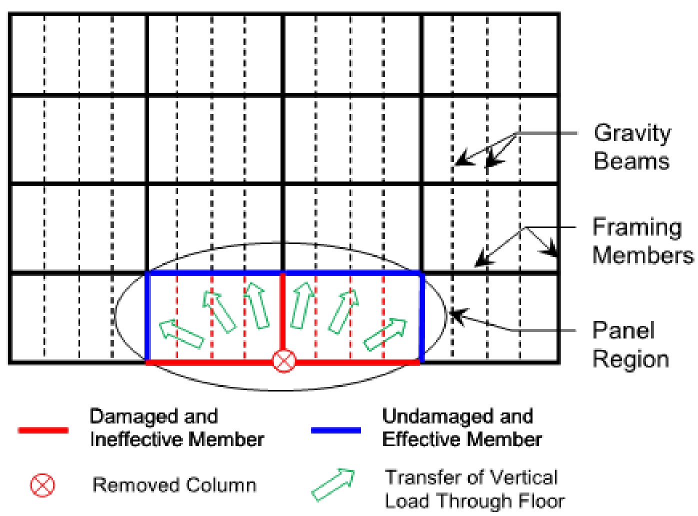

Figure 1: Tie Forces Transfer of Loads from Damaged Areas (From Appendix C UFC 4-023-03)

There are three types of horizontal ties that must be provided using the floor and roof system of a given structure: 1) Longitudinal, 2) Transverse, and 3) Peripheral. Vertical ties are also required in columns and load-bearing walls. Tie forces may be provided by existing structural elements that have been designed using conventional design methods or by additional components. Typically, for reinforced concrete slabs, the tie forces are provided using the reinforcing steel inside the slab. All Tie Forces must be continuous and satisfy the strength, location and distribution requirements of the UFC 4-023-03.

1.1.2. Direct Design: Alternate Path Analysis.

In the AP method, the designer must show that the building is capable of bridging over a removed structural element and that the resulting extent of damage does not exceed the damage limits. Generally, there are three procedures for performing an AP analysis: Nonlinear Dynamic (NLD), Nonlinear Static (NLS), and Linear Static (LS). Of the three, LS and NLD methods are the most popular.

Linear Static (LS): This is the simplest of the three procedures to apply. For the analysis, the applied load is enhanced by a "Load Increase Factor" that approximately accounts for both dynamic and nonlinear effects. The enhanced load is applied to the linear static model that has been modified by removal of a column, wall section or other vertical load-bearing member. The calculated internal member forces (actions) due to the enhanced loads are compared to the expected member capacities, which include a capacity increase factor (CIF, similar to the "*m*-factor" in ASCE 41) that accounts for the expected ductility. Components are also checked for brittle failure modes using force-controlled actions.

Nonlinear Dynamic (NLD): In this case, the un-modified load case is directly applied to a materially- and geometrically-nonlinear model of the structure. In the first phase of the dynamic analysis, the structure is allowed to reach equilibrium under the applied load case. In the second phase, the column or wall section is removed almost instantaneously and the software tool calculates the resulting motion of the structure. The resulting maximum member deformations are compared to the deformation limits based on component type and construction. Dynamic nonlinear analysis explicitly includes nonlinearity and inertial effects and therefore no correction factors are needed.

1.1.3. Direct Design: Specific Local Resistance.

This method requires that the structure or specific critical components of it (columns, load-bearing walls, transfer girders, etc.) provide sufficient strength and ductility to resist a specific abnormal loading condition. One of the most common examples is the hardening of building columns and load-bearing wall elements to resist blast loads and vehicle ram attacks determined from vulnerability assessments specific to that particular building. The intent, of this approach is to minimize the potential for collapse by reducing the potential for failure of critical elements.

1.1.4. Direct Design: Enhanced Local Resistance (UFC 4-023-03).

This method requires that shear capacity of critical column or load-bearing walls and their connections to the lateral force resisting elements be greater than the end reactions associated with the flexural capacity of the component. The goal is to provide a ductile and more controlled failure mode in the event of an overload.

1.2. Existing Guidelines for Design against PC in the US and Canada

Existing US and Canadian building codes do not address PC explicitly. Standards such as ASCE 7 and ACI-318, CSA A23.3 include references to improve structural integrity but do not provide quantifiable or enforceable requirements to resist PC. Only two US agencies have developed guidelines that provide quantifiable and prescriptive requirements to reduce the potential for PC. These guidelines are the *General Services Administration, Alternate Path Analysis and Design Guidelines for Progressive Collapse - 2013,* and the *Department of Defense (DoD) Unified Facilities Criteria Design of Buildings to Resist Progressive Collapse, UFC 4-023-03 – 2013.*

In April of 2012, the Canadian Standards Association (CSA) released S850-12, *Design and Assessment of Buildings Subjected to Blast Loads,* which is the first Canadian standard that specifically addresses design of buildings against PC. This document provides criteria for the analysis and design of new buildings and assessments of existing buildings to resist blast loads. In chapter 10, the S850-12 provides design recommendations for enhancing structural integrity and minimizing the potential for PC. The standard uses a threat-dependent approach and only direct methods of design are permitted. To mitigate the risk for PC designers may choose to design important primary and secondary components to resist the effects of specific Design Basis Threats (DBT) or alternatively, an AP analysis may be performed for components that cannot provide the required level of resistance against the DBT. The AP method is largely based on the UFC 4-023-03 approach.

2. Building Description

The building consists of a 12-storey tower founded on a larger three-storey podium base structure. The tower, with a footprint of 1420 m^2, provides accommodation units above Level 2. The podium structure, measuring approximately an additional 1500 m^2 beyond the tower footprint, houses an eating facility, kitchen, bar, recreational/gathering areas, mechanical and service spaces, and loading dock and storage facility. The mechanical rooms are located in the first and twelfth storey.

Offset load bearing elements, columns and walls are not desirable in PC design when using the Tie Forces approach. It is not specifically forbidden to have offset load bearing elements, however, it becomes very difficult to comply with the vertical tie continuity requirements at offsets since this would require a more complex and expensive vertical tie system such as cables or rods that provide a continuous vertical tension tie from the 1st floor to the roof. Therefore, to maintain continuity of the columns and load bearing walls and allow for larger spans in the gathering spaces, the podium structure was offset from the tower footprint.

Reinforced concrete construction was determined to be the most economical for complying with the structural loadings required by the National Building Code (NBC) while also meeting the PC and force protection requirements for the structure. The primary structural system consists of reinforced concrete flat plate two-way slab construction for the tower levels, and a reinforced concrete flat slab with drops for the podium levels. Drop panels and column capitals were incorporated into the podium structure to allow longer spans for the assembly use spaces, beams frame the perimeter of large openings and slab steps as required. The podium structure is less repetitive than the tower so standard stick framed formwork will be used.

Flat slab construction is the most commonly used method of reinforced concrete construction in Eastern Canada. This method of construction is conducive to the use of fly form systems and results in a thinner floor structure, reducing the overall height of the building. Fly forms are being used for the tower structure only. Locally skilled labour is more familiar with flat slab construction than with reinforced concrete beam supported slabs and moment frame systems, which are not common in high-rise construction in Canada, but are more widely used in the USA.

The primary lateral load resisting system consists of reinforced cast-in-place concrete shear walls forming the elevator and stair shafts with an additional shear walls on the west side of the building across from the elevator core and forming exterior walls of the podium structure. The exterior wall cladding consists of precast panels supported on the slab edge directly adjacent to the columns above Level 3 slab and metal cladding over reinforced concrete and metal stud framing for the first three levels of the podium. Reinforced concrete walls are located on the building faces that were deemed most vulnerable to threats.

A dynamic modal analysis was conducted using ETABS modeling software for design of the lateral load resisting elements. A rendering of the building is shown in Figure 2 below. Access to the site is very tight on three sides due to its close proximity to adjacent building restricting possible lay down areas during construction. The site grade slopes aggressively across the width of the building.

Figure 2: Building 3D Rendering

3. Overview of UFC 4-023-03 Design Requirements and Applicability to the Building Design

One of the main reasons the client chose to follow the UFC 4-023-03 established design guidelines to incorporate PC resistance was to maintain a threat-independent design. The release date of the CSA S850-12 standard was after initial planning and onset of design for the 12-story building. They were aware of this CSA document but did not want the design criteria for PC to be based on any particular level of threat. Threat levels are not static – they vary constantly with time influenced by escalation of domestic and international affairs.

A threat risk assessment is typically conducted for the project to address the hazards and consequences. PC is a rare occurrence in Western nations making it very difficult to reasonably assess the probability of occurrence for a specific hazard or group of hazards. As a result, the risk assessment of PC is reduced to a consideration of consequences. In the UFC 4-023-03 consequences are generally measured in terms of loss of life, deeming the building's use and occupancy level as the most important criteria for consideration. Only buildings and portions of buildings that are three stories or more are required to provide PC resistance. Buildings are assigned an occupancy category (OC) using a similar arrangement as the ASCE 7 occupancy classifications. The PC design requirements are then determined based on the OC of the building. Table 1 provides a summary of PC design requirements for each OC in the UFC 4-023-03.

The tower portion of the building has 12 stories and the podium portion three stories, therefore, both must be designed for resistance against PC. For PC purposes, the building was classified as Occupancy Category II (OC-II). For OC-II buildings, resistance against PC may be incorporated using the Tie Forces approach and the Enhanced Local Resistance (ELR) method as required in Option 1. Flat plate systems such as the one used for the building are a good option for implementation of Tie Forces because the system already requires steel reinforcement spanning in both directions spread throughout the floor slabs which can be used as tie force elements. AP on the other hand, requires more time consuming structural modeling, and components must achieve flexural capacities and ductility levels that could be problematic to implement with a flat plate system. Non-linearity of frame type structures can be modeled using pre-defined plastic hinges placed at specific locations on frame elements, which allows complete AP NLD analyses to be performed relatively fast. This allows designers to iterate between different concepts as is often required in design efforts, in a time and cost efficient manner. For flat-plate systems, more complex models with non-linear multi-layers shell elements, fine meshing, and stress-strain curves are required in order to more accurately model non-linear dynamic behavior of slabs. These models can be time consuming and do not lend themselves for an iterative design approach. Thus, it was determined that the Tie Forces provided the most cost effective and efficient design method for satisfying the requirements for PC.

Table 1 UFC 4-023-3, OC and Design Requirements

Occupancy Category	Design Requirement
I	No Specific Requirements
II	Option 1: Tie Forces (TF) for the entire structure and Enhanced Local Resistance (ELR) for the corner and penultimate columns or walls at the first story. OR Option 2: Alternate Path (AP) for specified column and wall removal locations.
III	Alternate Path for specified column and wall removal locations and Enhanced Local Resistance (ELR) for all perimeter first story columns or walls.
IV	Tie Forces and Alternate Path for specified column and wall removal locations and Enhanced Local Resistance for all perimeter first story columns or walls.

4. Incorporation of Tie Forces and ELR Design

Horizontal internal ties were incorporated using the bottom layer steel reinforcement in both directions. The UFC 4-023-03 has continuity, location and connection requirements for horizontal internal and peripheral ties. For example, horizontal ties must be lapped only with Class B splices and non-contact splices are not allowed. In addition, the horizontal ties must distributed in the floor slab in such way that no more than twice of the required tie strength is concentrated on the column strips, and all horizontal internal ties must be connected to the peripheral ties with seismic hooks. However, because in different structures Tie Forces may be provided by different elements, the UFC 4-023-03 does not specifically indicate or recommend whether top of bottom bars in a flat plate slab are to be used.

In 2-way slab design typically bottom bars are required over the entire suspended slab area spanning between lines of support. Therefore, it was logical to use the bottom bars as the internal tie bars since they could be easily adapted to comply with the continuity (splices), location (minimum spacing) and connection (hooks and detailing) requirements of the UFC 4-023-03. All horizontal internal ties were anchored to the outermost peripheral tie bar around the building perimeter and around openings. Figure 3 illustrates the typical connection detail for internal ties into peripheral ties at a beam or wall.

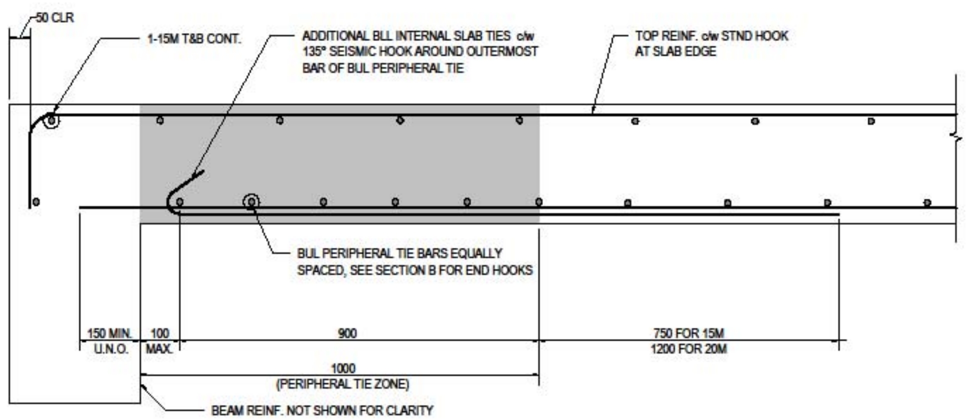

Figure 3: Typical Connection Detail at a Beam - Internal Ties to Peripheral Ties

Peripheral ties were added around the perimeter of the building, at applicable openings and around sub-areas per UFC 4-023-03. The floor layout of this building included two stairs shafts, two elevator cores, a crane opening, steps in slab elevation and changes in slab thickness. Peripheral ties must run adjacent to walls or beams and the internal ties must hook around the outermost bar of the peripheral ties. At the same time, the slab bottom bars need to extend into the supporting walls or beams to anchor the reinforcement. To meet both requirements additional hooked bars were added which spliced with the internal ties outside of the peripheral tie to provide the required anchorage of internal ties to peripheral ties as detailed in Figure 3.

In addition, the design loads for different areas of the building varied drastically especially at Levels 2 and 3 due to transitions in use between assembly and residential occupancy and heavy snow accumulation on the roof of the podium. Per the UFC 4-023-03, heavy-loaded areas (with respect to adjacent areas) can be sub-divided by providing a peripheral tie around it. If the sub-division is not made, then tie force requirements based on the heavy-loaded area must be used throughout the whole floor, which can add a significant amount of steel reinforcement to the design. Therefore, multiple sub-areas were used through Levels 2 and 3 to minimize the amount of steel reinforcement for tie forces, resulting in a complex peripheral tie layout. A partial Level 2 plan is shown in Figure 4. Because of the various sub-areas, a significant increase in end-hook connections and detailing was required to connect the internal ties to the peripheral ties at sub-areas. Figure 5 shows internal ties connecting to a sub-area peripheral tie zone at a change in slab thickness between the tower slab and the podium slab.

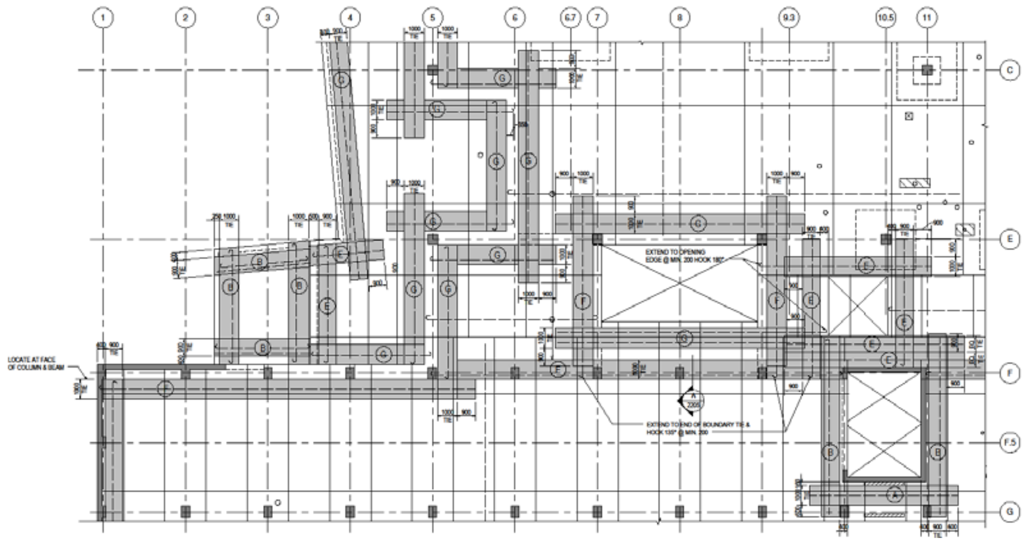

Figure 4: Partial Plan Peripheral Tie Layout at Level 2 (Peripheral Tie zones in gray areas)

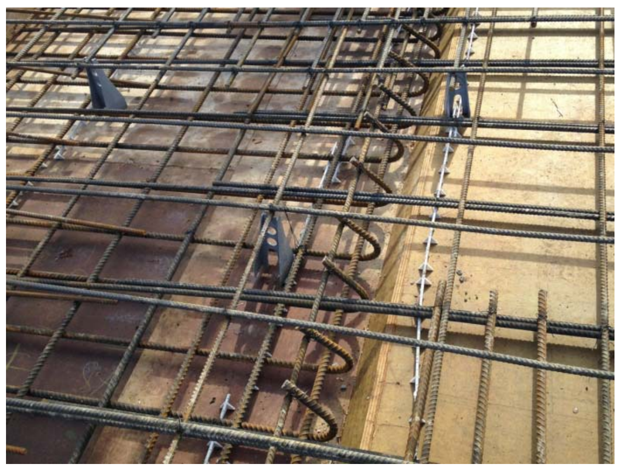

Figure 5: Internal Ties Hooking to PT at Transition in Slab Thickness

Vertical ties were provided by the columns and walls in the form of elongated vertical bar splice lengths into the columns above. The vertical ties are continuous from the roof level down to the first column or wall-supported floor above the foundation.

ELR – Critical load bearing walls and columns were detailed to have extra shear capacity to satisfy the ELR requirements to provide a nominal level of protection for perimeter columns. Closely spaced closed ties with minimum 135 degrees hooks into the column were placed in corner and penultimate columns to provide a design shear strength that exceeds shear demand associated with the flexural demands of the columns.

Preliminary estimates indicate that there will be an increase of approximately 30% in the slab reinforcing for the Tie Forces design. There is also an increase to labour costs due to special detailing and installation requirements and placement of the additional reinforcing.

5. Design and Construction Challenges

A number of constructability and coordination issues arose as the design progressed. This project was a modified design-build contract, therefore minimizing construction cost was a high priority and the contracted design timeline was aggressive. The PC design requirements put a number of restrictions on the design for all disciplines. A thorough pre-planning and design development process was paramount to identify and resolve design and construction related issues early in the design process. A significant effort was invested to continuously coordinate between the design and construction teams. All stakeholders were involved from the onset of the design process. Adherence to the design requirements for PC were continuously reviewed throughout design and tender process. The project was developed using Autodesk Revit Architecture, Structure, and MEP, and Civil 3D in collaboration with Navisworks project review software to facilitate coordination and identify conflicts.

Punching Shear Capacity: The UFC 4-023-03 method for Tie Forces was initially developed largely based on beam-supported slabs under large deflections where the punching shear would not be a concern in the event of a local column failure. The Tie Forces approach was developed assuming that the undamaged portions of the structure around the column/wall removal would be capable of carrying the redistributed forces. Although the tie forces may be used for flat plate design, the UFC 4-023-03 does not require column areas to be checked for punching shear failure modes. However, it is the intent of the UFC that in a PC event, the column strip will be able to carry the loads from the area around the damaged column and transferred it to undamaged portions of the slab. Even though it was not specifically required by the UFC document, the slabs were checked for punching shear with individual columns removed (double spans) for the load case of 1.2D + 0.5L following the UFC 4-023-03 design criteria for PC. Therefore, shear reinforcement (Figure 5) was added as required to ensure that a column would not punch through the slab in the event of a single column removal. The shear studs were only required under the PC load condition. Column capitals were added in locations where shear studs could not achieve the required shear resistance (Level 12 floor supporting the mechanical room).

Figure 5: Reinforcing at a Typical Perimeter Column

Coordination of slab penetrations/openings: In PC design no penetrations/ sleeves or openings are permitted in the peripheral tie zones around the slab perimeter and around large openings – stair and elevator shafts and the crane opening. Post drilled or cored openings should be avoided since internal ties may not be cut without compromising the integrity of the tie system and locating rebar prior to coring can get expensive. Regions around the elevator and stair shafts are typically used for running services so intense coordination was required to avoid conflicts between required penetrations and peripheral tie zones. Peripheral ties were located to avoid known openings. Sleeve shop drawings were required from the mechanical, electrical and sprinkler subtrades for review prior to installation and all openings for mechanical and electrical services were coordinated in-house pre-installation.

Sleeves were required to be placed prior to rebar installation to minimize cutting of rebar and maximizing continuity of the internal slab ties. In-slab conduit and inserts were not permitted without prior review and approval. To maintain the required distribution and continuity of internal ties, openings must be smaller than the maximum spacing allowed for internal tie forces. The maximum spacing for internal ties is equal to 20% of the span in the direction of the tie. In addition, a tie element (rebar) is required on each side of the opening and continue full length of the slab between peripheral ties rather than just a splice length beyond the opening edges. This approach results in an increase in rebar volume at openings. Continuous communication with subtrades was vital to convey an understanding of placement restrictions for this project.

Nonstandard Lap-Splice requirements: Lap splice lengths for internal ties and peripheral ties in columns and tie reinforcing are established on material over-strength values of 1.5 for concrete compressive strength (f'_c) and 1.25 for steel reinforcement yield strength (f_y). This results in nonstandard lap lengths and end-hooks not commonly used by rebar fabricators and installers. Lap splice locations are also restricted in the UFC 4-023-03 for internal and peripheral ties. Laps must be placed away from the centre lines of support within the centre 60% of the slab span, which differs from the typical approach that requires bottom bars in a two-way slab to be spliced at the supports or on column lines. Therefore, additional coordination with rebar fabricators and installers was required during preconstruction meetings and reiterated on site to ensure that proper splices and end hooks were used.

Peripheral/Internal tie continuity: With a flat plate or slab construction, the hooking requirements of internal ties to peripheral ties is more difficult to achieve than for a beam-supported slab. Internal slab ties are required to hook around the last bar of the peripheral tie it terminates at along the slab edges with minimum a 135 degree hook. This means that the bottom upper layer (BUL) bars would hook downwards while the bottom lower layer (BLL) bars would hook upwards. With no perimeter beam, it is not possible to hook the BUL bars down to achieve the double hooked configuration. In order to meet the intent of the UFC for anchorage, the BLL peripheral ties were shifted inwards to allow the BUL internal ties to extend a hooked development length beyond the outermost peripheral tie bar as detailed in Figure 6.

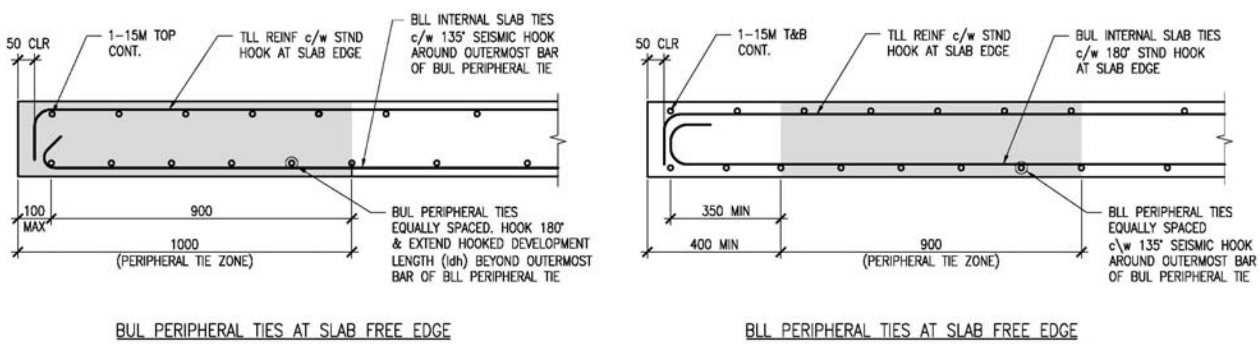

Figure 6: Internal Ties hooking to BUL and BLL Peripheral Ties at slab edge

Precast panel supports: The precast sub-contractor proposed double (2-bay) precast wall panels to minimize the number of crane lifts. In doing so, the number of panels reduced from 410 to 230 panels, a 44% decrease, which would significantly reduce crane lift time and installation costs. This also reduced the required on-site storage time for the panels which is a benefit given the limited site storage available. The Tie Force method for calculation of peripheral tie requirements assumes uniform distribution of cladding along the building perimeter and requires at least one support at each column line. The precast element design is typically developed by a sub-contractor as a delegated specialty design item after the final design of the structure. The peripheral ties in the slabs were originally designed following the UFC 4-023-03 requirements to carry only one bay (4.15m) of tributary weight as a single point load at the centre of an 8.3m span. If the panels were designed to span two bays, then the dead load and the peripheral tie requirements are doubled and the originally designed peripheral tie bars and support conditions would not meet PC requirements.

In order to meet the PC design requirements for the slabs, a third mid-panel gravity connection was required for the doublewide panels at the intermediate column locations. It is not typical to have three (3) gravity supports on a single precast panel as this creates an indeterminate support condition and it is not possible to assess how much of the panel dead load would be distributed to the centre support due to uncertainty in the possible variation in support elevations and methods of shimming. Thus, a solution was required that provided a centre support under PC load conditions but relied on only two supports for installation and normal building load conditions and did not overload the centre support.

The decision was made to shim snug the mid-panel support after installation and after the panel had been allowed to deflect and creep for minimum 28 days. In doing so the mid panel support would take minimal gravity load during normal load conditions while allowing the dead load to be immediately supported by the slab adjacent to each column during a PC event.

Since this change was implemented during tender, the structural capacity of the suspended slabs, columns and footings was reviewed to carry the dead weight of the double precast panels with only two supports during the panel erection/installation phase and under normal design load conditions. After excavation, assessment of the rock quality the geotechnical engineer upgraded the footing bearing capacity which allowed a small cushion for footing capacity. The PC load condition governed the design for punching shear so there was some reserve capacity under normal load conditions.

6. Summary

The design to prevent PC for this project required significant coordination and investment early in and throughout the design process to identify and resolve potential design and construction related issues. Effective collaboration between all stakeholders and innovation in design process has been paramount to the project success. It was highly beneficial to involve the construction team early in the process to align design concepts and construction constraints and realities, in an attempt to identify critical issues related to site material selection and constructability. Considerable effort was made to convey understanding of non-typical construction detailing requirements and restrictions for the Tie Force construction to all levels of the construction team to minimize site errors.

The Tie Forces approach in the UFC 4-023-03 is vague on specific requirements for flat slab/flat plate design. Therefore, extrapolation beyond the requirements in the UFC along with engineering judgment and innovation in the design was required to develop clear details that met the intent of the PC requirements, maintained tie force continuity, and minimized impact on construction costs.

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