Appropriateness of Current Regulatory Requirements for Ensuring the Robustness of Precast Building Typologies

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Abstract

The phenomenon of progressive collapse can be likened to the failure of a house of cards where structural damage propagates beyond the locality of the initial damage and to an extent disproportional to the original cause. Insufficient consideration of the structure's potential for progressive collapse has widely been seen as responsible for some of the most high profile structural collapses of the last 60 years. The need to rigorously consider and mitigate for the risk of such collapse occurring is often seen to be more imperative within the design and detailing of pre-cast concrete structures, which is mainly due to their segmental nature of and associated inherent lack of structural continuity. Aimed at highlighting the need for a more quantitative design methodology, the paper evaluates the suitability of commonly advocated measures for ensuring structural robustness in pre-cast building typologies.

Using a non-linear 'push-down' simulation the suitability of existing tying and anchorage force provisions are evaluated, with such prescriptive detailing rules often adopted by design engineers to justify a suitable level of structural robustness. This computational assessment enabled a quantitative assessment of the performance of pre-cast framed buildings subjected to a sudden column loss event. The findings highlight a need for current design and detailing practice to take more appropriate account of the nonlinear response of components and joints incorporated within multi-storey buildings.

Introduction

Progressive collapse refers to the phenomenon where localised damage brings about wider and even total structural collapse. The failure of the Ronan Point flats in London (1968), the collapse of the Alfred P Murrah Building in Oklahama (1995) and the destruction of the World Trade Centre towers (2001) are perhaps the seminal incidents that highlighted the potential vulnerability of structures to such events. In response, the major international design codes incorporated a number of regulations designed to assure that adequate attention is provided within the design, detailing and construction of buildings so that the resulting structures can be considered to have an adequate 'robustness' or insensitivity to local failure events (Starrosek, 2008). However, despite the existence and application of such rules, a number of studies and publications (Beeby, 1999) have suggested that although practising engineers inherently understand the need to prevent against disproportionate collapse they still lack: the analytical tools, assessment methodologies, appropriate metrics and explicit design guidance that would enable them to ensure the risk of disproportionate collapse is adequately considered and suitably mitigated. Starrosek (2007) actually suggests that the lack of a widely accepted quantitative method for assessing a structure's ability to resist collapse following an event means that the scope for efficient and repeatable structural design, optimisation and effective review is currently limited.

One such potential method is through the use of a non-linear 'push-down' computational assessment procedure. This method is directly analogous to 'push-over' analysis, which is often used as part of contemporary seismic design to determine the ultimate performance of a structural system. This is achieved by increasing the load multiplier in step increments until the plastic failure of the assembly can be identified and has previously been successfully adapted for use within the robustness assessment of both steel and RC building typologies (Kim 2009, Lee 2011). However, no such similar analysis appears to have been conducted for common precast building types. This is despite the fact that such buildings are often intuitively considered to be less robust than similar, alternative steel or in-situ reinforced concrete structures. Such analysis is perhaps then more imperative for buildings constructed in this way, because of the need to demonstrate that the individual structural components used will be effectively and robustly 'stitched' together.

This paper conducts a series of non-linear push down assessments to better understand the response of typical precast framed structures to a column loss event, quantifying their performance relative to existing robustness requirements. Such analysis will also allow for an assessment of the widely adopted prescriptive 'tying' and 'anchorage' rules, currently deemed sufficient for the avoidance of disproportionate collapse within many of the major international codes. Such assessment is necessary following concerns (Izzuddin 2008) that such rules and details are (in reality) unrelated to the actual actions imposed and structural performance necessary following such a damage event. Specifically, the existing provisions currently exclude any consideration of the necessary ductility demands placed on the connections and structural members (Izzuddin 2008).

Limitations of Current Design Methods

The UK, US and European (ODPM 2004, ACI 2008, CEN 2006) design regulations all contain specific provisions addressing the need to design against disproportionate collapse. All adopt similar procedures in which the buildings are classified based upon their intended use, size and the level of risk that any potential structural collapse may present to the public. This process of building classification defines the appropriate level of structural robustness that must be achieved following the design, detailing and construction processes. However, the subsequent and necessary definition of the required structural performance always appears to be: highly qualitative, aspirational and subjective in nature for each of the regulatory guidance documents.

Starrosek (2007) highlighted this fact, also suggesting that the lack of a more quantitative performance requirement limits the engineer's ability to assess, evaluate or optimise one design method or structural solution against another. Consequentially an engineer is not currently able to quantifiably demonstrate that his/her building will adequately perform in a manner deemed appropriate to its risk classification. For example the designer cannot currently assess how much closer or further away from an acceptable robustness the design will become by adopting one structural form, transfer structure or connection detail over another.

Such limitations within the existing design codes effectively force the design engineer to instead achieve regulatory approval by demonstrating that their adopted design and detailing philosophy is in line with one of the ‘approved’ design strategies available. Through employing such design methods, the resulting building is adjudged (by the pertinent design codes (ODPM, ACI, CEN)) to be sufficiently robust without any need to: assess, measure or justify the resulting structural performance of the construction. This paper however, aims to assess the suitability of two of these strategies, each of which is discussed below.

Prescriptive Tying

Perhaps the most commonly adopted of the available methods is where the engineer ensures that the structural elements and any resulting joints detailed are in line with the prescriptive 'tying force' provisions provided by the codes. The philosophy is based on the assumption that through the use of such details, the designer will consequentially improve the indeterminacy of the structure, localising any damage that may occur, by taking advantage of the alternative load paths established. This is accepted however, without a subsequent need to demonstrate or justify these mechanisms by explicit calculation or computational assessment. Recent studies (Izzudin 2008) have questioned this approach, querying whether the tying provisions defined suitably allow for the true structural actions and effect that such elements and joints will be required to resist following a partial building collapse. Given the period during which they were developed (following the Ronan point collapse in 1968) and the simplicity of the resulting equations, it is unlikely the expressions developed were intended to account for the complex dynamic and non-linear effects induced in reality. The lack of any compulsory regulation requiring the engineer to demonstrate that the adopted construction details are suitably ductile to allow for the resulting large deformations induced, is perhaps the starkest indication that these design expressions do not rigorously consider the true performance requirements for buildings exposed to accidental load conditions.

Alexander (2004) also introduced that for certain structural typologies the philosophy of ensuring structural redundancy via the provision of adequate joint continuity may actually contribute to a progressive collapse event. This work argues that in the event of the loss of structural stability, excessive tying may actually have the effect of 'dragging' out or down elements above or below the region in which the member has been removed or destroyed, questioning the blanket insistence on the use of continuous vertical ties. This 'pull down' phenomenon was actually observed on an experimental concrete panel high rise block constructed and tested by the Building Research Establishment (HMSO 1968). However, no further detailed experimentation, modelling or quantification of this effect appears to have been subsequently conducted. As such, there is little understanding of which building types, layouts or details might be most susceptible to its realisation.

This paper asserts that the suitability of stipulating tying provisions without having first demonstrated that such detailing rules provide a suitable performance in relation to the likely structural actions and ductility demands for which they are included should be questioned. An assessment to check that the final building is not susceptible to any secondary ‘pull-down’ effects is also prudent, with the paper therefore aiming to demonstrate the suitability, or otherwise, of common 'fully tied' precast concrete framed structures in meeting these additional design requirements.

Effective Anchorage

A further, code compliant (ODPM, CEN), robustness design strategy is that of demonstrating that 'effective anchorage' exists at the structural connections between elements, with such a design approach restricted to buildings classified as being of a lower risk of disproportionate collapse (i.e. 2A, 2Lower etc). This provision is again prescriptive in nature, with similar concerns to those expressed in relation to the existing tying requirements again applicable. That is the suitability of the current guidance in relation to calculating and stipulating acceptable loads that the effective anchorage details must resist. For example, the overly simplistic requirement for the connections to resist a (presumably factored) force equivalent to the dead weight of the member it supports (BSI 2010) is again unlikely to correctly allow for the true structural action, ductility requirements and dynamic effects that will be experienced during a collapse or damage event. In a manner similar to that which currently exists for the contemporary tying expressions, little information is at this time provided into how such guidelines have been derived and more significantly in regards to their validity.

Additionally, engineers often fail to appreciate that the use of effective anchorage and prescriptive tying rules represent two distinct design approaches to ensuring a building's robustness. A prominent example of such a misunderstanding appears to be present within the latest European national guidance document for precast structures (BSI 2010). The specific clause requires all precast floor, roof and stair members to be 'effectively anchored' regardless of the building's robustness class, stating that such anchorages must be designed so that they are capable of transmitting the *"dead weight of the member to that part of the structure that contains ties".* Such a requirement however, is incongruous and contradictory to the currently accepted approach to robustness design. For example an engineer is entitled to design a lower class structure without the inclusion of vertical ties, with the engineers also possibly having adopted one of the two alternative design strategies available to them (see Research Methodology). If either of these philosophies is instead adopted, it would then be possible that no part of the structure would have to contain vertical ties. How then could the necessary anchorage regulation be met?

Research Methodology

The major international design codes allow the adequate robustness of buildings to be demonstrated through the use of any one of four potential design approaches. These include meeting the prescriptive 'tying force' or alternative 'anchorage' provisions discussed, with the anchorage provisions only relevant to the UK and European regulations for class 2A and 2Lower buildings respectively. Alternatively the engineer may also achieve compliance by ensuring that either the 'Notional Member Removal' or 'Key Element' provisions have instead been met.

Most pertinent to this study are the assessment methods that can be adopted as part of a notional member design. This requires the engineer to demonstrate that following the loss of any vertical load bearing member, the remaining structural components will have sufficient 'capacity' to transfer any resulting actions, through the establishment of suitable alternative load paths. The provision however is currently commonly, with some suggesting unsuitably (Izzudin 2008), applied using conventional design checks. Adopting such a simplistic approach will again fail to account for the complex non-linear geometric and material effects induced by the occurrence of partial structural collapse. Despite this current practice however, the notional column design provision is essentially performance based and as such has been found to allow for the more appropriate consideration of the progressive collapse phenomenon (Kim 2009, Lee 2011). This is because if the correct assessment methodology is used, the engineer becomes able to assess the actual capacity of the structural system. The term 'capacity' is taken to refer to the critical property preventing structural collapse and may therefore relate to element strength, deformability, ductility, stability or stiffness.

This study asserts that because no robustness performance metric is currently defined within the design regulations, and because a building designed using any of the available design approaches can be considered to be adequately robust, it must therefore also currently be the case that a building designed using one of the possible design strategies should be equally robust to a similar building designed using an alternative strategy. As such, it should be that a building designed using the prescriptive tie or anchorage rules will be able to sustain the actions imposed on it under an assessment conducted to meet the notional column removal provisions. This therefore presents an opportunity to assess the adequacy of current tying and effective anchorage rules for ensuring the insensitivity of typical precast concrete building to a progressive collapse. This will be done through the use of a non-linear push down computational study, with such assessments having historically been shown to be suitable for robustness assessment and design against disproportional collapse (Kim 2009, Lee 2011). This analysis will also be capable of investigating both the suitability of the joint details and the building's susceptibility to any secondary effects that may compound the advancement of a progressive collapse. It will thus allow the key concerns associated with tied and anchored buildings to be addressed and evaluated in a quantifiable manner.

Nonlinear Pushdown Analysis of Model Structures

Guidance does exist (GSA 2010) with regards to conducting this type of computational progressive collapse analysis and assessment. Such analysis should be performed by instantly removing vertical load bearing structural elements and then by assessing the structure's residual ability to accommodate such damage. However, software packages that are capable of carrying out such analysis, in order to correctly account for instantaneous changes in the stiffness matrix and building geometry are rarely commercially viable, and thus available to a practising design engineer. As such the GSA (2010) guidelines also allow for alternative (yet 'equivalent'): linear static non-linear static (pseudo-dynamic) and simplified dynamic assessment procedures to be adopted within the progressive collapse assessment of buildings. These analysis methodologies are then more easily carried out using more widely available software packages.

The non-linear static analysis method defined within the GSA regulations requires a stepwise increase in regards to the amplitude of applied vertical loads, until the maximum amplified loads are reached or a collapse is observed (Marjanishvili 2006), with the computational analysis essentially becoming a vertical derivative of a seismic 'push-over' analysis. This assessment technique is utilised as part of this study, as it is capable of allowing for the non-linear material and geometrical effects currently believed to be absent from the contemporary prescriptive design techniques. Although the 'push-down' method cannot capture the instantaneous dynamic effects associated with aspects such as column loss events or debris loading for example, studies (Izzuddin 2008, Marjanishvili 2006) have shown that the application of factored ‘equivalent’ static load cases are capable of accounting for such actions and effects.

Interestingly, the GSA guidelines (GSA 2003) allow for and define a load controlled, non-linear static analysis, in which the load is applied to the structure in at least ten incremental steps from zero up until the total specified loading. The resistance of the structure against such loading is assessed, with the output forces, moments, shears and deformations each then compared against the relevant acceptance criteria. However, it has been shown that such load controlled push-down generally involves numerous analysis re-runs, is sensitive to the chosen load steps and tolerances and is also unable to converge to a solution when the ‘load factor’ begins fall, i.e. when building collapse is progressing (Marjanishvili 2006). The load factor refers to a measure of performance utilised as part of similar studies considering the non-linear push-down assessment of multi-storey buildings (Kim 2009, Lee 2011, Marjanishvili 2006). The metric essentially quantifies what proportion of the load case the plastically deformed or ‘collapse-arrested’ structure can transmit to the foundations through the alternative load paths it can establish and is defined as:

$$\text{Load Factor} = \frac{\text{Equivalent Applied Load}}{\text{Total 'Linear Static' Load}} \tag{1}$$

In this way a load factor >1.0 represents a building that would not collapse before the required design load conditions have been exceeded and as such can be considered to be suitably robust.

Design and Analysis of Structural Models

The adopted analysis models have been designed to represent a precast framed structure with a $7.5 \times 7.5\text{m}$ structural grid and a floor to floor height of 3.8m. Models consisting of two, four and ten storeys were analysed for a 'tied' frame design. Alternative models adopting effectively anchored connections were also considered for the two and four storey structures, with the elevations for the analysed structures illustrated within Figures 1 ((a)-(c)). The precast column and beam elements are designed in accordance with the EC2 design code (CEN, 2006) for $C40/50$ and $f_y = 500\,N/mm^2$ grade concrete and reinforcement respectively. The resulting beam and column sections are depicted as part of Figure 2 ((a)-(b)).

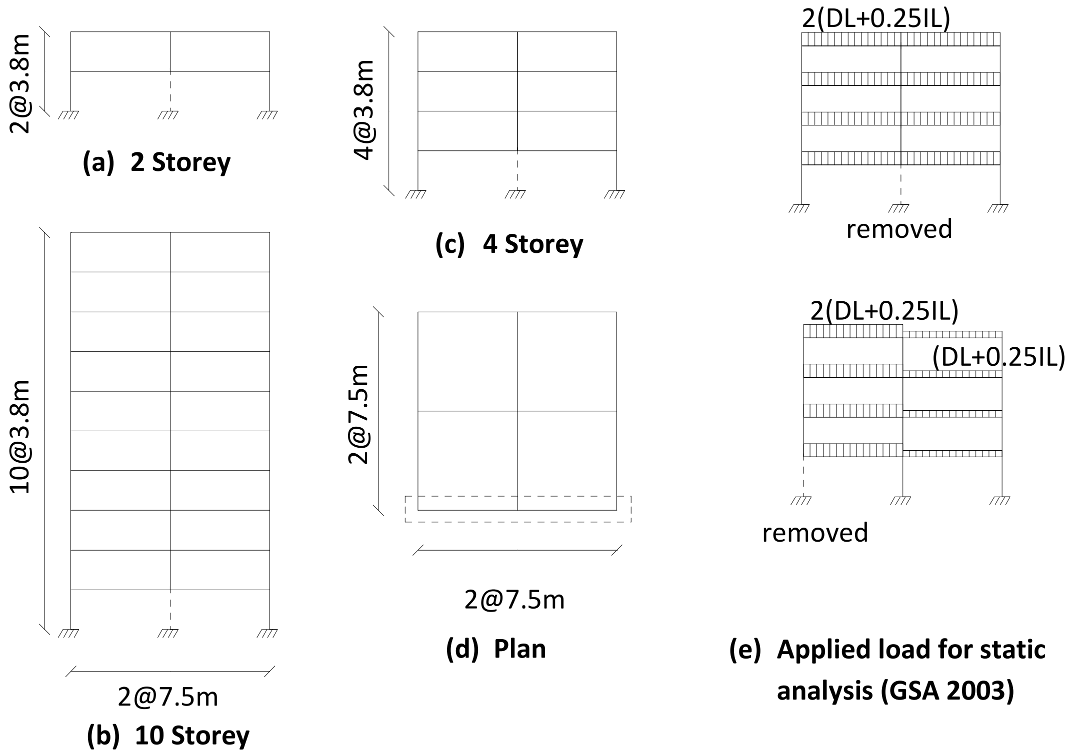

Figure 1 Elevation, Plan View and Loading Conditions for the Analysis of Computational Structures

A superimposed dead load of 6kN/m^2 and an imposed load of 2.5kN/m^2 were applied to each of the models analysed. In addition, a notional horizontal lateral load, equivalent to 1.5% of the characteristic weight of the structure, was also applied in order to represent the non-verticality of the precast column members. The lack of application of such a minimum level of horizontal load appears to be a significant limitation of similar previously undertaken studies (Kim 2009, Lee 2011). However, this load is important in correctly accounting for and identifying any potentially detrimental secondary geometrical effects that may occur.

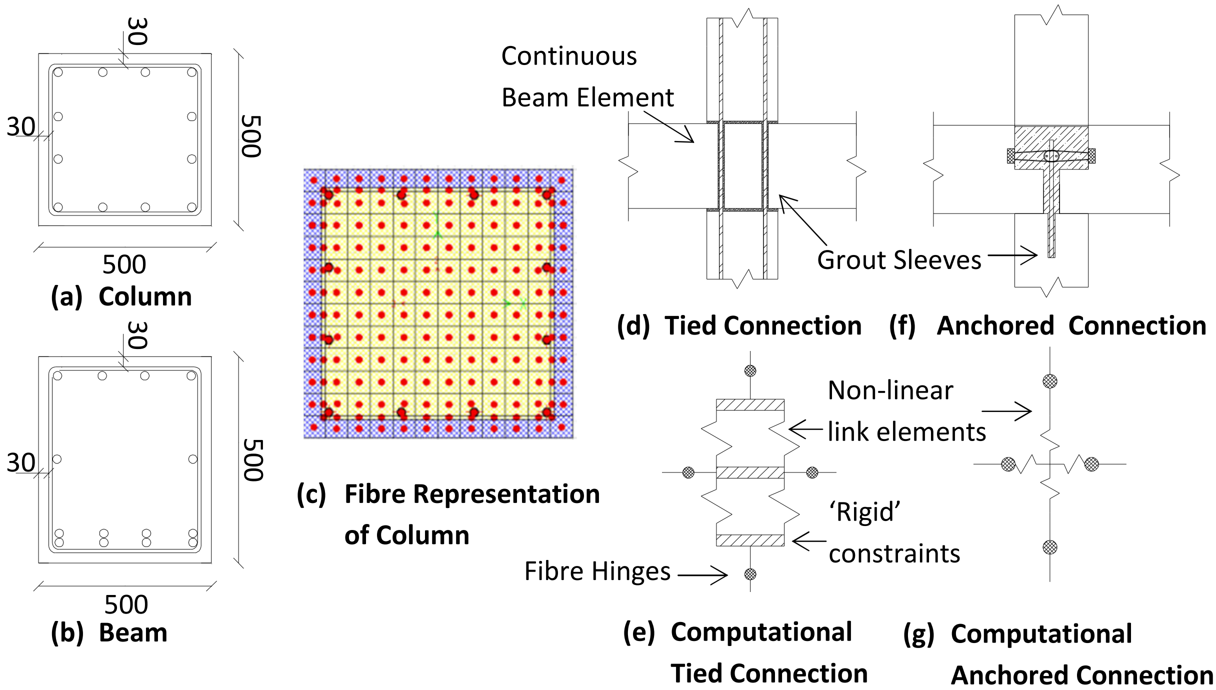

Figure 2 Structural Sections, Connection Designs and Computational Equivalents

Further, in order to account for the dynamic effects associated with a sudden column removal event, the recommendations given within the GSA (2010) guidelines require a dynamic amplification factor of 2 to be applied to the spans for which the column is removed. An un-amplified dead load is also applied to the remaining spans, with an imposed load reduction factor of 0.25 applied in both cases. The resulting load combinations are illustrated as part of Figure 1 (e). Because the chosen building types have a simple and repetitive layout, only two critical structural damage scenarios are considered necessary for investigation. These were the removal of central and corner column elements at the critical lower level of the structure (Figure 1 (e)).

Because of the identified limitations associated with the load-controlled push-down method currently prescribed within the GSA recommendations, the vertical push-down analysis was instead carried out through the adoption of a displacement controlled assessment. That is, the vertical displacement at the position (or node) where the column is removed was incrementally increased with the corresponding vertical load to this displacement then calculated. This allows the load factor to be similarly evaluated, although the analysis can be more expediently run, as well as being significantly less likely to diverge, when compared to the load controlled alternative (Kim 2009).

Effective use of the proposed non-linear, static robustness assessment procedure is of course highly dependent on the adopted representation of the plastic properties of each component, as well as their connections, as part of the computational model (Inel 2006). That is, our understanding of the ultimate inelastic deformation capacities of the components detailed in terms of their geometric and mechanical characteristics should be captured as part of the assessment. The required non-linear load-deformation relationships have, in previous studies (Kim 2009, Lee 2011), been based on those values published within seismic design guidance, such as ASCE 41-06 (2007). However, these values do not account for the effect of significant variations in the axial forces applied to the components. Such forces and variation though, will be much more prominent and critical within a progressive collapse simulation than for the seismic assessments for which the values were derived. This is because such forces will significantly affect (in potentially both a beneficial and detrimental manner) the rotational behaviours and thus capacities of the elements and connections. Therefore, a much more effective method of capturing the structural behaviour of the RC elements was considered to be through the use of 'fibre-hinge' analytical elements. In this representation, the element's cross-section is subdivided into a number of elementary layers or 'fibres' to which the appropriate material models can then be assigned (Figure 2 (c)). By dividing the structural cross section in this way it is possible to determine an effective representation of the non-linear moment-curvature relationship for the structural component in a manner that suitably accounts for the proportion of axial load applied. The non-linear load deformation characteristics derived in this way were then also validated against relevant experimentally derived values (Panagiotakis 2001). The associated structural behaviour was then incorporated within the computational models as 'hinge' elements that are specified at the locations where the applied lateral and gravity loads are considered to produce maximum effects. That is the plasticity of the structural components (modelled as a $P-M_2-M_3$ hinge) is assumed to be lumped at the centre and ends of the beam and column elements.

The load deformation characteristics relating to the precast connection details were determined by consideration of the behaviour of the details illustrated within Figures 2(d) and 2(f). Such details are commonly adopted in UK structural design in order to meet tied and effective anchorage conditions respectively. For the vertical continuity tying requirements, the load that the connection is required to resist (as a tensile force), is determined by consideration of the equivalent axial compressive load that the column removed resists prior to its loss. However, this load only relates to that action which results from the application of the accidental load case and only for the load that is from the storey which would have been directly supported by the removed column. The resulting detail (Figure 2(d)) incorporates H25 reinforcing bars which are equally spaced about the centre point of the column, with the bars also fully anchored and lapped with the reinforcement within the precast column. The connection was modelled using non-linear 'link' elements and constraints as illustrated within Figure 2(e) in order to assess the suitability of the connection with respect to its rotational capacity.

To demonstrate the suitability of an effective anchorage connection (Figure 2(f)), it was only necessary to demonstrate that a lateral force equal to the dead weight of the horizontal member it supports can be resisted, with all the applied loads again factored as required, under accidental conditions by the European code (CEN 2006). For the detail considered, the reinforcing bar grouted into position is designed to act as a type of cast in steel billet, i.e. it acts in shear. However, because of the insufficient lap/anchorage of the bar it cannot be considered to have any rotational capacity. Therefore this connection type was modelled as non-linear link element, which was specified to lose load bearing capacity once the code stipulated axial limit had been reached (Figure 2(g)).

The Performance of Tied and Anchored Precast Framed Buildings

The response of the chosen precast building typologies to the nonlinear static push-down analyses conducted is presented within Figures 3(a) and 3(b) for the structures subjected to a column loss event at the centre and corner of the building's end bay respectively. The plots show the load factor (Eq. (1)) against the imposed deflection at the location at which the column has been removed. Because the maximum strength of structures in each case does not exceed a load factor of 1.0 none of the structural typologies considered would satisfy the recommendations of the GSA (2010) guidelines.

It was observed that for buildings of 10 storeys adopting a tied design, and for cases in which a corner column was removed, the precast framed structure 'yielded' at a load factor of around 0.58 with plastic hinge failures observed to occur initially, and as would be expected, at the point of maximum moment due to the induced cantilever. A much higher initial yield (0.75) and increased maximum strength (0.81-0.84) was observed for the structural models in which the central column was removed. Such a response should be expected because in the cases where a corner column has been removed, the push-down load is only being resisted by one, rather the two bays that act for the central column case. Interestingly, an improved performance was seen for both the central and corner column load cases as the number of storeys was increased for the tied buildings. This appears to be because of a combination of effects. Firstly, the increased axial load appears to act so as to improve the moment rotation capacity of the plastic hinges. In addition, the taller buildings also have more structural members and the presence of more components in the building/model appears to inherently increase the number of alternative load paths which are available to resist and redistribute the induced loads.

As can be seen from Figure 3(a) a much more suitable overall building response to the column loss events was seen for the 'tied' rather than the 'anchored' building types. This is because after reaching ultimate strength a much more gradual saw toothed falling branch is seen until failure, with every instantaneous drop in strength relating directly to a plastic hinge reaching its ultimate strain limit and the loss of residual plastic strength. In contrast, the load factor plot observed for the anchored low rise buildings was observed to be almost 'elastic' and 'brittle' in its nature. This is because framed structures resist progressive collapse essentially through the action of the vertical ties in tension and the rotational capacity/ductility of the beam to column connections. Because the anchored connections modelled only offer restraint in one constrained axial direction, they are consequentially ineffective in arresting the building collapse for the low rise structures considered.

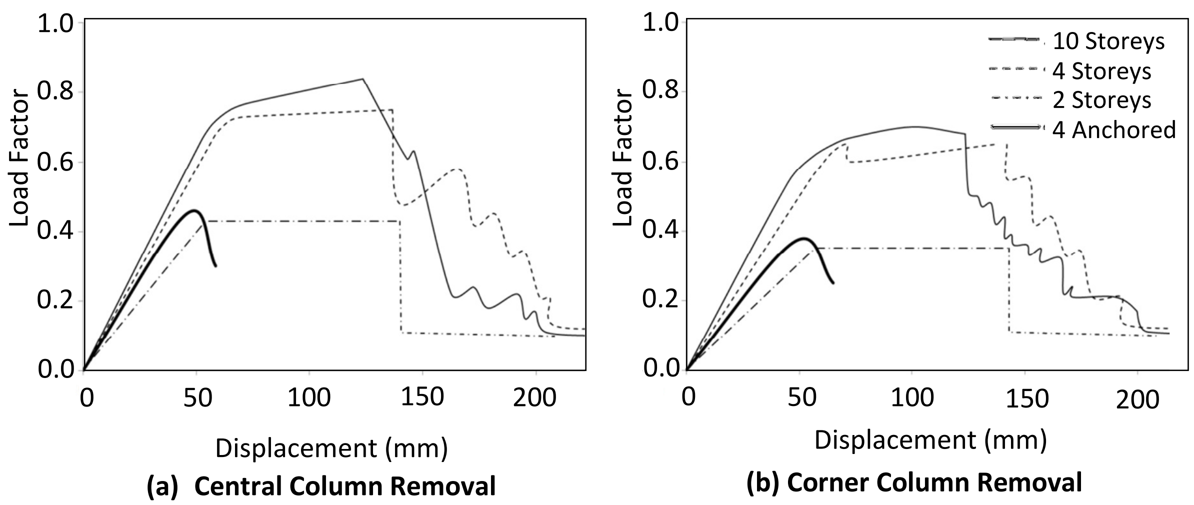

Figure 3 Load-Displacement Relationships of Model Structures

Conclusions and Proposals for Future Work

For the simplistic precast framed structures considered none were found to meet the GSA (2010) robustness performance regulations. All of the two and four storey structures investigated could also be classified as 'susceptible' to progressive collapse, according to the performance metric proposed by Marjanishvili and Agnew (2006). However, none of the buildings considered showed any indication that a secondary, detrimental 'pull down' effect due to the use of ties would induce or hasten the collapse sequence. The resulting behaviour and therefore 'performance' of the tied structures though is considered to be directly related to and significantly affected by the chosen tying detail Figure 2(d). However, a larger amount of investigation into the sensitivity of building performance to the nature of the precast tied connections to be used is required before any firm conclusions in regards to the suitability of the current prescriptive tie design methodology and detailing rules, as they apply to precast framed structures, can be drawn.

Further, this study also provides no indication of in what manner the measured robustness of the structure will change in response to variations in: span length, storey height or plan shape. It is proposed therefore that such variables should be considered and incorporated as part of any future, similar studies, so as to further inform any necessary corrections to the existing robustness design regulations and guidance. In addition, analogous investigation of the performance of alternative precast cross wall construction typologies and the effect of utilising and suitably modelling for segmental and flexible floor diaphragms (e.g. Prestressed Hollowcore floor units) would also be of great significance to ensuring the suitable design of robust precast building typologies in the future.

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