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Robustness assessment using nonlinear analysis methods: a parametric study

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Abstract

In the present study, static and dynamic nonlinear analyses were undertaken on representative planar frames with a range of different geometrical and mechanical parameters, with the aim to assess whether analysis methods drawn from current seismic design principles can be successfully applied to the case of a typical robustness scenario, where a column is suddenly lost due to an accidental action. It was shown that static nonlinear methods of analysis can provide good approximation of the structural response. Seismic detailing was found to significantly improve robustness, with higher benefits seen at low ductility levels, and similar trends to what is currently known within earthquake engineering were obtained. The geometric nonlinearity was found to be critical, allowing for a realistic representation of the structural performance. Increasing the number of storeys was shown to enhance progressive collapse resistance, while gravity loads tend to increase progressive collapse and ductility demands; on the contrary, equivalent viscous damping was shown to reduce the ductility demand. Removal time was found to be of primary importance on the structural response, with inertial effects significantly being reduced when the removal time is longer.

Keywords: disproportionate collapse, ductility, nonlinear analyses, robustness, progressive collapse, pushdown analyses, seismic detailing.

1 Introduction

Following the collapse events that happened over the last fifty years (e.g. Ronan Point building in 1968, the A.P Murrah Federal Building in 1995 and the World Trade Centre in 2001), the phenomenon of progressive collapse has emerged as an occurrence where the triggering cause and the resulting collapse are characterised by a disproportion in size, also known as disproportionate collapse [1]. Based on this concept, ro-

bustness has been defined as the insensitivity to local failure [2], with several methodologies proposed for the design and assessment of civil engineering structures.

Several similarities have been identified between such methodologies and those currently used for designing seismic resistant structures, including the need for ductile requirements, which allow large plastic deformations in the structural design to prevent collapse under strong ground motions [3]. It has been shown, for instance, that if adequate seismic strengthening and detailing was in place, the impact of damage due to blast scenarios which happened over seismically active regions (such as the case of the A.P Murrah Federal Building) could have been reduced, resulting to survivability [4].

Limited attempts have been made to quantify such benefits, particularly because currently there are no universally accepted strategies [5]. As in other design situations, the existing ones can be classified into performance-based (direct), which evolve from design objectives and structural analysis; and prescriptive (indirect), which implicitly address structural integrity through strength, continuity and ductility [6].

Design requirements for accidental actions within Eurocode 1 [7] are prescriptive, classifying buildings into consequence classes based on risk, with each class corresponding to additional required measures including horizontal ties and risk assessments. There are different strategies recommended, which however are not fully developed, with robustness seen as a strategy focused on limiting the extend to localised failure. Eurocode's strategy, has been termed inconsistent, aimed at preventing rather than limiting localised failure [6], while assessment carried out on existing tying and anchorage provisions, highlighted their inappropriateness on robustness [8]. Prescriptive requirements have thus been characterised as unrelated to real performance, with the lack of ductility deemed as a significant limitation of current code provisions [9].

Alternatively, the American standards specify both direct and indirect design methods for progressive collapse. Within ASCE 07-10 [10], the Alternate Path Method (APM) and Specific Local Resistance method (SLR) are proposed as direct methods, while indirect methods include considerations of strength, continuity and ductility. Under APM, primary structural components are removed and the ability of the remaining structure to remain intact is assessed. Local failure is thus allowed and collapse is avoided by alternative load paths. Conversely, SLR is focused on identification and resistance of key structural components which are prone to an extreme event. As the magnitude and location of an extreme event cannot precisely be estimated, a threat-dependent design offers limited appropriateness for extreme events other than the one specified [11]. APM therefore remains the most common method for assessment, with guidance given both within UFC 4-023-03 and GSA 2003 [12, 13].

In this paper, the implementation of analysis methods drawn from current seismic design principles is investigated to a column loss event, and the key results are presented. Nonlinear static analyses methods are initially carried out to assess the robustness resistance of planar frames with respect to a range of parameters, while nonlinear dynamic analyses are then used to assess the predictiveness of the results. The study aims to assess whether such methods can be successfully applied for struc-

tural robustness, demonstrates the significance of certain aspects whose interpretation is not straight forward, and highlights the need for further experimentation and clear guidance.

2 Methods of analysis for structural robustness

Different methods of analysis, with various degrees of complexity and accuracy, have been used to assess the performance of structures under a robustness scenario.

Linear elastic analyses are the simplest option, where equivalent static loads are applied, based on the assumption that deformations remain sufficiently small and building layouts are simple [14]. Alternatively, if temporary loads are to be accommodated by large deformations of the connections, nonlinear analyses are required to allow for geometric and material non-linearities, giving a more precise representation of the structural response [3]. In particular, within static nonlinear (pushdown) analysis, the resisting capacity of a frame is assessed by increasing displacement at the removed column location [15]. Similarly, to account for inertia effects associated with the structural response, dynamic analyses are the most accurate assessment methods, where a step-by-step numerical integration is performed on the equations of motion [16].

Bajaj [17] argued that static analyses may result in over-conservative designs, as nonlinearity and inertia effects are accounted through load factors and are therefore not optimised, while on the other hand dynamic analyses are complex. Khandelwal [5] proposed three variants of the pushdown analysis; the Uniform Pushdown, the Bay Pushdown and the Incremental Dynamic Pushdown, in which the loads are monotonically increased on the entire structure, on the damaged bay and conducting successive nonlinear dynamic analyses on the bays of interest, respectively. The authors reported Uniform Pushdown being the least realistic (as the collapse could not necessarily be attributed to the damaged bay), Bay Pushdown was found sufficiently reasonable and Incremental Dynamic Pushdown the most realistic, but costly in terms of computational time. Marjanishvili [18] discussed the advantages and disadvantages of each method concluding that different advantages from each method should be incorporated into a progressive analysis procedure, which evolves from simple to complex methodologies.

Pushdown analyses therefore remain the most common analyses methods, due to the implementation of nonlinear behaviour and reduced computational demand. Although these analyses are relatively simple, great care must be taken when representing material behaviour, and detailed finite elements must be used, without excessively increasing the computational cost [14].

2.1 Assessment considerations

There are several considerations that are absent or inadequately addressed within current literature and codes of practice, limiting the effectiveness of analysis methods in

the professional practice. A selection of five key aspects is discussed below, and the results of numerical investigations carried out on each of them are presented in the next section.

2.1.1 Demand

During performance assessment, the capacity of a structure (obtained from nonlinear static analysis) is typically compared to the demand associated with an extreme event, and checked against acceptance criteria. In earthquake engineering, such demand is usually obtained through elastic or inelastic response spectra. In the case of robustness, there are currently no methodologies available that explicitly define demand. As a consequence, current analysis methods may result in under- or over-conservative designs. For instance during seismic design, beam-column connections are typically provided with rotational ductility to resist cyclic ground motion events. During a blast scenario however, loading is typically monotonic rather than cyclic and therefore using the same connections may be inappropriate [19]. Characterisation of the demand is therefore significant during design and assessment.

2.1.2 P- Δ effect

While both UFC and GSA procedures require representation of the geometric non-linearity within pushdown analysis, there has been lack of consensus within literature. Iribarren [20], as well as Choi and Kim [21], suggested that P- Δ effects must be accounted for, to allow for a realistic representation of the catenary effect and the increased resistance of the beams to collapse, improving the results of pushdown analysis. On the other hand, Marjanishvili and Agnew [22] argued P- Δ effects must be neglected, as catenary forces develop at large deformations, which are more likely to cause failure due to excess rotation before any catenary force can be activated.

2.1.3 Load combination

Current research has shown that the load combination used for assessment is of primary importance. If inappropriate load is applied, it may result to under- or over-estimations of the overall structural response [20]. With reference to Table 1, the required dynamic increment factor (DIF) under GSA is constant for the whole structure and all static procedures, despite that nonlinear effects are already accounted for in nonlinear static analysis. This is not the case with the newer version of UFC 2009, where the DIF is selected as a function on nonlinear behaviour and is applicable only to the bays adjacent or above the lost member. Furthermore the UFC procedure considers double the live loads compared to GSA, and lateral loads are included in the analysis. It is therefore critical that assessment is carried out to quantify the effects of different load combinations.

Table 1: Recommended loads for progressive collapse

GSA 2003			UFC 4-023-03	
	Gravity	Lateral	Gravity	Lateral
Static	2(G+0.25Q)	-	(1.2G+0.5Q) Ω_N^*	0.002 ΣP^{**}
Dynamic	G+0.25Q	-	1.2G+0.5Q	0.002 ΣP

* Ω_N =DIF for bays adjacent or above removed element, and 1 elsewhere;

DIF = $1.08 + 0.76/(\theta_{pra}/\theta_y + 0.83)$ for steel, $1.04 + 0.45/(\theta_{pra}/\theta_y + 0.48)$ for concrete, θ_{pra} is the plastic rotation angle and θ_y the yield rotation.

** ΣP = Sum of gravity loads dead and live acting on a floor.

2.1.4 Column removal time

Removal time (τ) has been characterised as determinant on the dynamic response of a frame when a column is suddenly lost [20]. Several authors have modelled the column removal as instantaneous, including the work of Rahai [23]. Alternatively, Bajaj argued that in real-life situations there is no such thing as an instantaneous removal [17]. A removal over a sufficiently small time interval must therefore be considered, based on experimental assessment or guidance within codes of practice. Furthermore, it has to be appreciated that different unforeseen scenarios (i.e fire, blast) are associated with different collapse times and when assessing only the ‘instantaneous’ case it is implicitly assumed that a sufficiently small value of τ will represent the worst case scenario, which can lead to overdesign.

GSA and UFC specify $\tau = T_1/10$, with T_1 being the period of vibration associated with the vertical motion. There is no indication about the mass to be considered for evaluating T_1 , while at the same time the requirement for convergence tests is not addressed. Since the dynamic response is related to vibration frequencies associated with inertial forces, it is meaningful to take into consideration the mass participating [24]. Similarly, convergence tests are critical to ensure that the deformed shape is adequately captured and the dynamic response of the structure is accurately evaluated. Code interpretation therefore remains significant, necessitating the requirement for assessment.

2.1.5 Equivalent viscous damping

Several authors suggested that equivalent viscous damping must be taken as 5% of critical damping within a dynamic analysis [22, 23], however there has been no clear indication by codes of practice. If damping is overestimated, structures can suffer higher losses than predicted, while on the other hand, an underestimation can lead to conservatism.

Figure 1: Structural frame model

3 Numerical investigations

Static and dynamic nonlinear analyses were undertaken with the commercial software SAP2000 [25] on representative planar frames for the typical robustness scenario where a column is suddenly lost. Figure 1 shows the n -storey 3-bay moment-resisting frame used for our parametric study, in which the dashed column has been removed (either before the static analysis, or to initiate the dynamic analysis), while G , Q and ψ_2 represent permanent load, variable action and combination coefficient, respectively.

Pushdown analyses (corresponding to push-over analyses in earthquake engineering) were initially performed by first removing an internal column and then loading the structure until failure [26]. The global pushdown (capacity) curves were used to compare sectional ductility μ_s (associated with detailed design of the beam-to-column connections) with structural ductility μ (quantified through the finite element analyses).

Nonlinear dynamic analyses were then carried out with a simulation of the column removal, and time histories of displacement and reaction forces were obtained at the removal location, and were used to compare the ductility demand μ_d against the available capacity μ_c . Ductility demand amplification A_{dyn} and required overstrength R were introduced to assess the predictiveness of static methods of analysis:

$$A_{dyn} = \frac{\mu_d^{(dyn)}}{\mu_d^{(stat)}}; \quad R = \frac{F^{(dyn)}}{F^{(stat)}}, \quad (1)$$

where $\mu_d^{(dyn)}$ and $\mu_d^{(stat)}$ are the values of ductility demand evaluated through dynamic and static analyses, respectively; similarly, $F^{(dyn)}$ and $F^{(stat)}$ are the maximum support reactions given by dynamic and static analyses.

3.1 P- Δ effect

In the initial stage of our nonlinear static analyses, the effects of P- Δ nonlinearity were investigated at different values of sectional ductility by varying the moment-curvature relationship of plastic hinges. As shown from Figure 2(a), when P- Δ is considered, a hardening phase is observed in the pushdown curves, with a significant drop in the ultimate displacement as well as an increase in base reaction, which can be attributed to the catenary action within the beam. This is explained by considering that at large deformation gravitational loads are resisted through the vertical component of the axial force in the beam rather than flexure, causing this behaviour.

Similarly, Figure 2(b) confirms that in order to realistically represent structural response, geometric nonlinearities have to be considered when the structural ductility is evaluated. Additionally, it appears that frames designed against progressive collapse will experience significant benefits from increasing the sectional ductility only at low ductility levels ($\mu_s < 6$), while at higher levels ($\mu_s > 6$) minor benefits will be seen. This poses a question on how different an earthquake resistant structure would behave, suggesting an interesting future investigation directly comparing the two cases.

Figure 2: Pushdown curve (a) and Structural ductility (b) with and without P- Δ effects.

3.2 Damping ratio

Figure 3(a) compares in the same force-displacement graph the results of the nonlinear static analysis (black line) with the dynamic responses obtained with different levels of equivalent viscous damping ratio ζ_0 from 0.01 to 0.15 (coloured lines), where a nominal value of 0.05 represents the typical damping level for civil engineering structures. Similarly, Table 2 summarises the values of $\mu_d^{(stat)}$, $\mu_d^{(dyn)}$, A_{dyn} and R which are obtained from the force-displacement curves at each analysis.

It is observed that in all cases the structure does not fail, with ductility demands

Table 2: Analysis results

Viscous damping	ζ_0	0.01	0.05	0.1	0.15
	$\mu_d^{(stat)}, \mu_d^{(dyn)}$	1.5, 3.8	1.5, 2.7	1.5, 2.2	1.5, 1.8
	A_{dyn}	2.5	1.8	1.5	1.2
	R	1.2	1.1	1.1	1
Imposed load	ψ_2	0.3	0.5	0.6	0.8
	$\mu_d^{(stat)}, \mu_d^{(dyn)}$	1.1, 2.0	1.5, 2.7	1.8, 3.2	2.5, 4.7
	A_{dyn}	1.8	1.8	1.7	1.9
	R	1.2	1.1	1.1	1.1
Storeys	n	4	6	8	10
	$\mu_d^{(stat)}, \mu_d^{(dyn)}$	2.3, 4.0	1.5, 2.7	1.3, 2.1	1.1, 1.9
	A_{dyn}	1.7	1.8	1.6	1.7
	R	1.1	1.1	1.1	1.2
Removal time	τ/T_n	0.1	0.5	2.5	12.5
	$\mu_d^{(stat)}, \mu_d^{(dyn)}$	1.5, 2.7	1.5, 2.7	1.5, 2.0	1.5, 1.6
	A_{dyn}	1.8	1.8	1.3	1.1
	R	1.1	1.1	1.1	1

below the capacity. As expected, increasing the damping results in a reduction of ultimate displacement and a consequent decrease in ductility demand. This is attributed to the energy dissipated by viscous mechanisms, leading to less energy dissipation required by the plastic hinges.

Interestingly, the lower the value of ζ_0 , the larger the initial reduction in base reaction due to the sudden column removal. Increasing ζ_0 , the initial plastic deformation decreases, and is followed by a subsequent reduction in the amplitude of oscillations within the elastic domain clearly seen in Figure 3(b).

Observing the dynamic response, the highest reduction in dynamic ductility demand $\mu_d^{(dyn)}$ occurs between $\zeta_0 = 0.01$ and 0.05 , with a reduction of 1.1 (30%). Damping is therefore critical and must be taken into consideration within dynamic analysis, to allow for a realistic representation of the energy dissipation of the structure. Implementing a value of $\zeta_0 = 0.05$ within analysis can significantly reduce detailing requirements up to 30%. It must be appreciated, that the current uncertainties within assessment will play a determinant role and if the required level of damping is higher than $\zeta_0 = 0.05$, greater benefits will be seen. Further experimentation is therefore required to provide recommendations and guidance for assessment.

With reference to the static pushdown curve, an increase is observed in A_{dyn} with decreasing values of ζ_0 . This is justified by the fact that damping was not embedded within determination of the performance point.

It is shown, that the pushdown curve (dotted line) rests above $F^{(dyn)}$, suggesting that an amplification factor up to 1.8 (assuming $\zeta_0 = 0.05$), would provide an overestimation in terms of strength required. Considering that this overestimation would

Figure 3: Force-Displacement response (a) and displacement time history (b) for different equivalent viscous damping ratio.

depend on the level of damping, it is arguable whether it would be overcome by neglecting P- Δ and catenary effects. Implementation of an equivalent Single Degree of Freedom (SDoF) representation is therefore required, as well as clear guidance on the level of damping to be used for assessment, to reduce amplification factors and fully appreciate the benefits of the pushdown analysis.

3.3 Percentage of imposed load

Figure 4(a) shows the force-displacement response of nonlinear static and dynamic analyses when different imposed load is considered at constant $\zeta_0 = 0.05$.

The higher the load, the more critical the situation as the ultimate displacement

increases, requiring the structure to be more ductile. This is caused by an increase in the support reaction, requiring higher level of energy dissipation in the inelastic response after its loss. Interestingly, a ductility of 4.7, allows the structure to withstand 80% of the live load and survive the accidental event.

Figure 4: Force-Displacement response (a) and displacement time history (b) for different imposed load.

At high ψ_2 , the initial reduction in base reaction is less, and there is a higher initial plastic deformation with the hinges dissipating the energy, leading to less oscillations within the elastic domain (hysteresis loop), as clearly seen within Figure 4(b). Similarly, as ψ_2 decreases, the amplitude of oscillations increases, suggesting that damping becomes more critical.

Considering the dynamic response, increasing ψ_2 from 0.3 to 0.8, causes a significant increase in ductility demand of 2.7 (135%). With reference to the pushdown curve, A_{dyn} and R also reduce as ψ_2 increases, with a discontinuity seen at $\psi_2 = 0.8$,

and the high magnitude of A_{dyn} is attributed to the dynamic effects not being considered within pushdown analysis. Lastly, it is shown that the global pushdown curve rests above $F^{(dyn)}$ for $0.3 \leq \psi_2 < 0.8$ and below $F^{(dyn)}$ for $\psi_2 = 0.8$, suggesting that, for the case under consideration, an amplification to overcome ductility demands would overestimate strength requirements for imposed loads being less than 80%.

3.4 Number of storeys

Figure 5(a), summarises the results of the static and dynamic nonlinear analyses at different storeys and constant values of $\zeta_0 = 0.05$ and $\psi_2 = 0.5$.

Increasing the number of storeys, reduces ductility capacity and demand, with the structure required to be less ductile. The increased number of elements and hinges, causes the hinge formation to vary with height, resulting in yielding and ultimate displacements to increase. As the increase in ultimate displacement is of higher magnitude, it causes a corresponding increase in ductility capacity. Similarly, to explain the drop in ductility demand, it must be appreciated that with higher number of storeys, more plastic hinges can occur. Regardless of the capacity and ductility assigned to these hinges, this increase in the number of hinges enables alternative load paths to become available.

It is observed that the structure becomes stiffer with n , which can be attributed to a higher redundancy level. Furthermore, the area under the curves increases with n , suggesting that more energy is dissipated in the inelastic response. Considering the dynamic response, plotting the displacement time histories (Fig. 5(b)), it is shown that a higher initial plastic deformation occurs at low n , with the energy being dissipated to the plastic mechanisms, leading to less oscillations. On the other hand, increasing n , reduces initial plastic deformation, and more oscillations are required, highlighting the significance of damping.

The highest reduction in $\mu_d^{(dyn)}$ occurs between $n = 4$ and 6 with a value of 1.3 (33%) while there is a reduction of 0.8 (30%) between $n = 6$ and 10, suggesting that similar benefits are seen due to the change in geometric property.

Considering the pushdown analyses, A_{dyn} ranges from 1.7 to 1.8, which is attributed to the dynamic effects not being considered. It is shown, that the global pushdown curve, rests near $F^{(dyn)}$ for $n = 4$, and above $F^{(dyn)}$ for $n = 6, 8$ and 10. At low n , implementation of an amplification would therefore provide a good approximation of the dynamic response, while at high n the pushdown curve would overestimate the strength required.

3.5 Removal time

Figure 6(a) shows the results of the dynamic analysis at different column removal times τ in comparison with T_n (the period of vibration with 85% of the mass participating), and constant values of $\zeta_0 = 0.05$, $\psi_2 = 0.5$, and $n = 6$.

Figure 5: Force-Displacement response (a) and displacement time history (b) for different number of storeys.

It is shown that removal time plays a critical role on the dynamic response of the structure, with ultimate displacements and ductility demand significantly being reduced with decreasing τ . As the column is lost, there is a sudden reduction in vertical base reaction, which becomes more critical as τ falls below T_n due to the inability of this reaction to be distributed to other supports within the given time. This causes an initial high level of plastic deformation, with the hinges dissipating the energy and is evident by the increase in area under the plot. This deformation is then followed by a higher elastic domain and more oscillations (Fig. 6(b)), underlining the significance of damping at low removal times.

It is shown, that the rate of increase of $\mu_d^{(dyn)}$, reduces with decreasing τ , with negligible variation between $\tau = 0.1T_n$ and $0.5T_n$, suggesting that as long as the mass participating is taken into consideration, the value of removal time proposed by GSA

Figure 6: Force-Displacement response (a) and displacement time history (b) for different removal times.

and UFC is satisfactory, small enough to capture the dynamic response of the system, while at the same time large enough to allow for a higher time step to be used within the analysis. Furthermore, for $\tau = 0.1T_n$, $\mu_d^{(dyn)} = 2.7$ while $\mu_d^{(stat)} = 1.5$, which accounts for 44% underestimation of ductility demands by the nonlinear static (pushdown) analysis.

If τ is significantly larger than T_n , the dynamic effects and inertial forces are limited, with A_{dyn} and R decreasing; and if the removal time is an order of magnitude higher than T_n , the static response occurs. It is also shown, that the global pushdown curve rests above $F^{(dyn)}$ for $\tau = 0.1T_n$, $0.5T_n$ and $2.5T_n$. It is therefore appreciated that at large values of τ the pushdown analysis will provide a good indication of the dynamic response without the need for an amplification factor. On the other hand, it can underestimate up to 44% ductility demands at ‘instantaneous’ events. Implement-

ing an amplification factor up to 1.8 would be sufficient in terms of ductility demand, however it would be overconservative in terms of strength required.

4 Conclusion

Ductility is widely acknowledged for its contribution towards seismic resistant structures with limited attempts made to quantify its benefits on robustness. Current design codes are prescriptive, necessitating the requirement for performance-based design and a widely accepted strategy for assessing structural robustness. Pushdown static analyses are the most common methods of analysis, however there is still no clear definition for the demand associated with blast events. The parametric study carried out to investigate, the robustness of frame structures with ductile connections has demonstrated that:

- Seismic detailing, ensuring a higher level of sectional ductility, can significantly enhance the performance of buildings against progressive collapse, however if the ductility is increased after a certain extent, only minor advantages will be seen.
- Pushdown analyses are simplified approaches that can provide good approximations of the structural response and reveal the effects due to the nonlinear behaviour hidden within elastic analyses. Pushdown analysis tend to be over-conservative in terms of strength, requiring ductility demand amplification factors ranging from 1 - 2.5.
- Equivalent viscous damping can significantly reduce detailing requirements when included in the analysis, and becomes critical at high-rise buildings and low removal times. It was shown that an increase of ζ_0 from 1% to 5% of critical damping can reduce ductility demands by 30%.
- Vertical loading can significantly enhance the possibility for progressive collapse. An increase of the imposed load from 30% to 80%, resulted in 135% increase in ductility demands. Clear guidance is therefore required on loads to be considered during pushdown analysis assessment.
- Increasing the number of storeys enhances progressive collapse resistance, due to the alternative load paths becoming available at increased height. In particular, increasing the number of storeys from 4 to 6, resulted in 33% reduction in ductility demand.
- Removal time plays a critical role on dynamic structural response, with inertial effects significantly being reduced at longer removal times. For removal time an order of magnitude higher than T_n , the nonlinear static analysis provided an accurate estimation of the ductility demand while for ‘instantaneous’ removal ($0.1T_n \leq \tau < 62.5T_n$), ductility demand was underestimated up to 44%.

Possible future extension of the proposed approach can allow a direct comparison with seismic resistant structures, leading to an optimised structural design against earthquake and robustness loading scenarios.

5 List of symbols and abbreviations

Symbol	Units	Description
A_{dyn}	—	Ductility demand amplification
$F^{(dyn)}$	kN	Maximum dynamic base reaction
$F^{(stat)}$	kN	Maximum static base reaction
G	kNm^{-1}	Permanent load
Q	kNm^{-1}	Variable action
R	kN	Overstrength
T_1	s	Period of vibration associated with mode for vertical motion
T_f	s	Fundamental period of vibration
T_n	s	Period of vibration with 85 % mass participation
T_1	s	Period of vibration of first vertical mode
n	—	Number of storeys
ζ_0	—	Equivalent viscous damping ratio
μ	—	Structural ductility
μ_c	—	Ductility capacity
$\mu_d^{(dyn)}$	—	Dynamic ductility demand
$\mu_d^{(stat)}$	—	Static ductility demand
μ_s	—	Sectional ductility
τ	s	Column removal time / time factor
ψ_2	—	Combination coefficient for imposed load

Abbreviations

APM	Alternate Path Method
ASCE	American Society of Civil Engineers
DIF	Dynamic increment factor
SDoF	Single degree of freedom oscillator
SLR	Specific Local Resistance Method
UFC	Unified Facilities Criteria

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