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COMPARISON OF SEISMIC ASSESSMENT GUIDELINES USING A CASE STUDY REINFORCED CONCRETE WALL BUILDING

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ABSTRACT

There are several seismic assessment standards and guidelines available around the world that can be used to identify vulnerable buildings. The assessment procedures and criteria in these documents are different, and thus, the assessment outcomes for a particular building, if assessed using different standards, can also be different. In this study, provisions of the linear static and non-linear static analysis procedures of three prominent seismic assessment documents, the American Society of Civil Engineers /Structural Engineers Institute standard ASCE 41 (2017) [1], the New Zealand Seismic Assessment Guidelines (2017) [2], and the European Standard EN 1998-3 (2005) [3] (also known as Eurocode 8 Part-3 or EC8-3) are discussed and compared, highlighting some of their similarities and differences. A reinforced concrete (RC) wall building used in FEMA P-2006 (2018) [4] for demonstration of ASCE 41 provisions is taken as the case study building for comparison of the assessment provisions. The linear and non-linear static analysis procedures specified in the three documents are applied to the case study building and the assessment outcomes are compared. The assessment results are found to vary across the analysis methods and guidelines. However, the critical governing vulnerability for the building is found to be the same. It is observed that with the simplifying modelling assumptions, coupled with the inherent conservatism in the assessment using linear static analysis, a more conservative outcome is obtained using the linear static methods as compared to the non-linear static methods. Overall, EC8-3 provisions are found to be the most conservative of all three guidelines considered for the assessment of the example building.

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INTRODUCTION

A significant stock of buildings exist worldwide that have either not been designed for seismic loading or, if seismic loading was considered, the standards to which they were designed have since been rendered obsolete and insufficient with the latest research and knowledge. Such buildings can pose a risk to life safety, and a socio-economic burden on society, as they are potentially vulnerable to severe damage or collapse in future earthquakes. Realizing this concern, procedures have been developed around the world to assist in identifying deficient buildings and to enable informed decision-making regarding either retrofitting or removing them. Different assessment techniques employed worldwide vary in the level of complexity, and even the simpler ones vary amongst themselves in the level of effort required. As a result, the assessment outcome is prone to differences and inconsistency when different guidelines are used. Another important aspect that can lead to inconsistent assessment outcomes is the assumptions allowed by the guidelines - even if the same guideline or standard is used, the assumptions employed by the assessors can significantly influence the results of the assessment. In this study, the possible differences in outcome as a result of applying the provisions of different international standards/guidelines for seismic assessment are examined. Previous studies have highlighted differences in seismic assessment approaches. For example, Lupoi et. al. [5] used the then-current version of NZ seismic assessment guidelines, [6], ASCE (FEMA-356) pre-standard [7], and Japanese standard [8] to assess three building models (two 2-dimensional and one 3-dimensional) and compared the results with the experimental observations. They highlighted the inconsistencies due to the different provisions in the guidelines and how each differed from the experimental findings. Mpampatsikos et. al. [9] evaluated the seismic assessment provisions of EC8-3, and the Italian Seismic Code [10], for the assessment of Reinforced Concrete Frame Buildings. They demonstrated the differences due to various assumptions by analyzing 3-dimensional models of two existing buildings in Italy. Araujo and Castro [11] compared the provisions of the American Society of Civil Engineers (ASCE) standard ASCE 41-13 [12] and EC8-3 for seismic assessment of steel moment frames. They compared the performance levels, compliance criteria, data collection requirements, analysis techniques, and acceptance criteria suggested by the two documents with case study buildings. In our study, the provisions for detailed/comprehensive assessment using three documents are compared: Tier-3 assessment using ASCE 41-17 (referred hereafter as ASCE 41), the New Zealand Seismic Assessment Guidelines, including its revised concrete provisions in section C5 (2018) (referred hereafter in the document as NZ guidelines), and EC8-3.

The focus of the study is to demonstrate the similarities and differences in the assessment outcome. First, the performance assessment philosophies of the three guidelines are summarized. Then, some aspects of the linear and non-linear static analysis methods suggested by the three guidelines are discussed, highlighting the differences between them. A three-story RC wall example building used in FEMA P-2006 [4] (referred hereafter in the document as FEMA P-2006) is used as the case study building. Assessment using the Linear Static Procedure (LSP) and Non-Linear Static Procedure (NSP) of ASCE 41-13 [12] is carried out by FEMA P-2006. Though the assessment is based on ASCE 41-13, changes in ASCE 41-17

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with respect to the previous version are highlighted by FEMA P-2006. Provisions of NZ guidelines and EC8-3 are applied to this building, and the results are compared with those reported in FEMA P-2006. The following sections present a concise version of a more detailed report [13].

PERFORMANCE ASSESSMENT

The assessment of a building is aimed at understanding its expected performance in future earthquakes and identifying vulnerabilities, which may then be taken up for retrofitting. Assessment is usually done at certain predefined levels of seismic hazards, for specific performance levels. The specification for the levels of the seismic hazards and the performance criteria differs from guideline to guideline and may often be decided by the commissioning or the regulatory authority.

ASCE 41 defines certain basic performance objectives for existing buildings (BPOE), and seismic hazard levels (BSE) for carrying out assessment. Table 1 shows the BPOE and the hazard levels depending on the seismic risk category of the building. Structural performance objectives are defined as Immediate Occupancy, Damage Control, Life Safety, Limited Safety, and Collapse Prevention. Objectives for non-structural performance are Operational, Position Retention, Life Safety, and Hazards Reduced. Hazard levels BSE-1E and BSE-2E typically correspond to corresponding to 20% and 5% probability of exceedance in 50 years (annual probability of exceedance (APoE) of 1/225 and 1/975, respectively). In general, two performance levels are selected for structural and non-structural performance assessment, each corresponding to the two hazard levels.

Table 1: Basic performance requirements for existing buildings (Adapted from ASCE 41).

Risk Category	BSE-1E	BSE-2E
I and II	Life Safety Structural Performance	Collapse Prevention Structural Performance
	Life Safety Non-structural Performance	Hazards Reduced Non-structural Performance
III	Damage Control Structural Performance	Limited Safety Structural Performance
	Position Retention Non-structural Performance	Hazards Reduced Non-structural Performance
IV	Immediate Occupancy Structural Performance	Life Safety Structural Performance
	Position Retention Non-structural Performance	Hazards Reduced Non-structural Performance

Like ASCE, EC8-3 also defines certain performance levels. They include the limit states of Damage Limitation, Significant Damage, and Near Collapse. The standard defines the expected performance of the structural and non-structural components for each performance level. All, two, or a single performance level may be required for carrying out the assessment depending upon the authority implementing the standard. The corresponding hazard levels may also vary, but the recommended hazard levels are 20% in 50 years, 10% in 50 years, and 2% in 50 years (APoE of 1/225, 1/475, and 1/2475, respectively) corresponding to the three performance levels.

Unlike ASCE 41 and EC8, the NZ guidelines solely focus on life safety. When using the NZ guidelines, assessment is with respect to a minimum life safety performance expected of a similar new building. The assessed capacity of a building is limited by the capacity of any component, failure of which is expected to lead to what the guideline refers to as a "significant life safety hazard". The assessed capacity of a building, divided by the Ultimate Limit State (ULS) demand, for a similar new building determines the score, which is reported as a percentage, %*NBS* score. ULS demand is the design seismic demand level for a similar new building, depending upon the importance (or IL) category of the building, calculated per New Zealand standard NZS 1170.5:2004 [14]. The lowest of the scores across different components (or weaknesses as they are defined by the NZ guidelines) becomes %*NBS* rating. NZ guidelines define a new building as follows: “A building is considered to be a new building until all of it is complete and ready for use”. *NBS* stands for New Building Standard.

A building rated to be at 100%*NBS* is expected to provide a minimum life safety performance expected of a similar new building. A building rated less than 100%*NBS*, say at X%*NBS*, is expected to provide the same life safety performance at X%ULS demand as the 100%*NBS* building at ULS. Allowances are inbuilt in the assessment methodology to give confidence that a minimum level of life safety performance is met at higher levels as well; however, the degree of confidence reduces with the increase in demand [2]. This is reflected in the colour coding in Figure 1. As a result, the assessed capacity of components is limited to below the non-degrading response, with some margin. This margin is larger for the components/mechanisms identified as Severe Structural Weakness (SSW). They are the components with step change response, with higher consequences in terms of life safety, and with low confidence in their assessed capacity. Examples of SSW could be non-ductile columns with high axial load, or shear-controlled interconnected walls, carrying high axial and lateral loads.

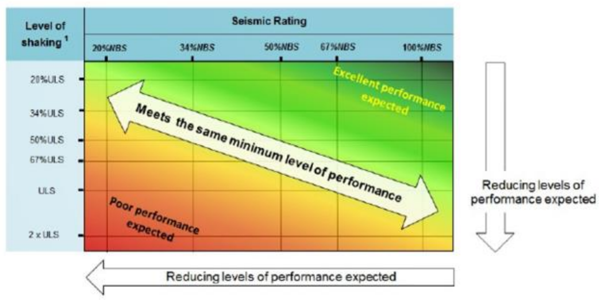

Figure 1: Indicative relationship between seismic performance, earthquake rating, and level of shaking (NZ guidelines).

Another subtle difference that highlights a distinction in assessment philosophies is how the assessment outcome is reported. In the cases of ASCE 41 and EC8-3, a hazard level is defined, and the assessment outcome is reported in terms of a demand-to-capacity ratio (DCR) of components reflective of their adequacy for the desired performance objective. However, in the case of NZ guidelines, %*NBS* is a form of capacity-to-demand ratio (CDR) defined as a percentage. In other words, the %*NBS* rating of a building reflects the demand at which the building provides at a minimum the same life safety performance as a similar new building at the ULS demand. For example, a building rated at 34%*NBS*, when subjected to 34% ULS shaking, is expected to perform at least at the same minimum level as expected of a similar new building subjected to 100% ULS shaking [2]. The difference is subtle, however, it is important to note that the focus of the New Zealand guidelines is gauging the life safety performance of a building with respect to a similar new building, rather than determining shortfalls in individual component capacities to achieve a desired performance objective, as in the case of ASCE 41 and EC8-3.

The above difference can be explained further with the help of a pushover response. Figure 2 represents a pushover curve for an arbitrary building. The red marker indicates the target displacement as determined by ASCE 41 for the selected hazard level. The cyan marker in Figure 2 indicates the displacement at the assessed rotation capacity of the critical member, which is indicated by the cyan marker in Figure 3. In the case of ASCE 41, evaluation is based on the comparison of the rotation demand for the critical component at the target displacement, to the rotation capacity corresponding to the required performance level shown in Figure 3. However, in the case of NZ guidelines, evaluation is done by comparing the building capacity (limited by the rotation capacity of the critical component), with the demand spectrum, following an approach similar to the capacity spectrum method [15]. The building pushover response is converted to the capacity curve of an idealized equivalent Single Degree of Freedom (SDOF) system, and compared with the demand spectrum, which is reduced based on the damping (as a function of available ductility) in the system, as shown in Figure 4, and later in Figure 5 as well.

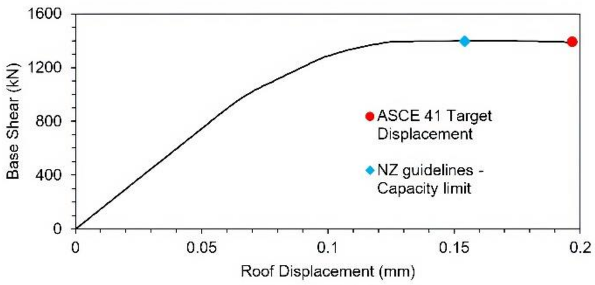

Figure 2: Pushover curve of an example building.

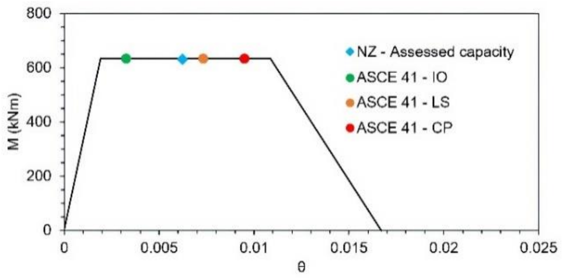

Figure 3: Moment-rotation backbone for the critical member in the example building (IO- Immediate Occupancy, LS-Life Safety, CP-Collapse Prevention).

ANALYSIS METHODS

All three guidelines specify linear/non-linear and static/dynamic procedures for carrying out structural analysis with certain limitations on the use of linear and non-linear static methods. In addition to the above methods, the NZ guidelines recommend using a Simple Lateral Mechanism Analysis (SLaMA) as the first step in a seismic assessment.

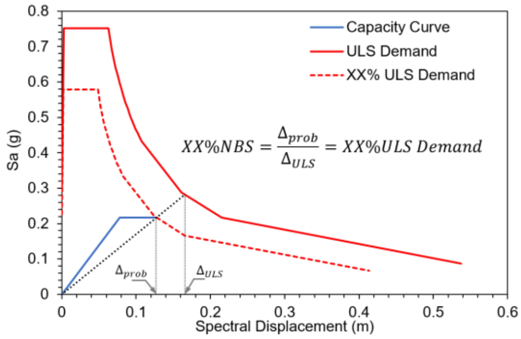

Figure 4: Calculation of %NBS score of the example building (Adapted from NZ guidelines).

SLaMA

SLaMA is a mechanism-based nonlinear analysis technique that provides information about the formation of a lateral mechanism and the available strength and deformation capacities. The strength and deformation capacities of the lateral load resisting system are assessed following a hierarchy of strength from the component level to the system level. The mechanism is represented by base shear vs. roof displacement response, which is then converted to an idealized capacity curve for an equivalent SDOF system, as suggested by the NZ guidelines. The capacity curve is compared with the demand spectrum, which is reduced (from the elastic spectrum) based on available hysteretic damping assessed from the global ductility derived from SLaMA. NZ guidelines suggest estimates of hysteretic damping based on past studies [16], [17], [18], and [19]. It follows an approach similar to the capacity spectrum method [15]. For a low-rise, simple regular building, SLaMA may be sufficient for assessment. For more complex buildings, SLaMA is still recommended to be the first step and can guide further evaluation using other analysis methods.

Linear Static Methods

Linear static analysis methods include both linear static and linear dynamic methods. The linear static methods include the Equivalent Static Method in the NZ guidelines, the Linear Static Procedure (LSP) in ASCE 41, the q-factor method, and the Lateral Force Method in EC8-3. The salient aspects of the three linear static methods considered in this study are summarized in Table 2.

In the Equivalent Static Method in the NZ guidelines, the base shear capacity of a building is determined, which is then compared with the Ultimate Limit State (ULS) seismic demand calculated based on the first mode response, adjusted for higher mode effects (if substantial) and reduced based on the assessed global ductility. The assessor is required to have an understanding of the failure mechanism and available ductility, which can be estimated using SLaMA. This helps in estimating K_μ, and S_p . K_μ, is a factor that reduces the ULS elastic demand based on available ductility, and, S_p is the structural performance factor, which is a scaling factor for the ductile systems. The capacity of the building determined from the analysis (and limited by the component with the least capacity) divided by the ULS demand determines the building rating. Usually, only the primary lateral structure is modelled. The gravity structure is assessed based on its drift capacity and the assessed drift capacity of the lateral structure. If the primary gravity structure governs, the rating is revised accordingly.

Table 2: Linear static analysis methods.

Parameter	NZ Guidelines (Equivalent Static Method)	ASCE 41 (Linear Static Procedure)	EC8-3 (Lateral Force Method)
Base shear demand	$V = S_a W \frac{S_p}{K_\mu}$ V is scaled until the probable strength capacity of the component considered to be a potential life safety hazard is reached	$V = C_1 C_2 C_m S_a W$	$V = \lambda S_a W$
Applicability	• Regular building • First mode dominant (Building height <30m) • Uniformly distributed ductility demand (e.g. beam sway mechanism for frame structure) or when Ductility demand ≤ 2	• Regular building • First mode dominant (limit on fundamental period) • DCR< lesser of (3 or m-factor)	• Regular building • First mode dominant (limit on fundamental period) • (DCR max/DCR min) < 2.5 for ductile (deformation-controlled) actions
Acceptance criteria	Checked against ULS demand for %*NBS*	Ductile – $m\kappa Q_{CE} > Q_{UD}$ Brittle – $\kappa Q_{CL} > Q_{UF}$* *Maximum force that can be transferred from ductile mechanisms	Ductile – $\Delta^{\#}_{cap} > \Delta_{UD}$ Brittle – $Q_{CL} > Q_{UF}$* # Evaluated after diving the mean properties by the confidence factor *Maximum force that can be transferred from ductile mechanisms, calculated by multiplying mean properties with the confidence factor
Secondary seismic/ Primary gravity components (not modelled)	Separately checked based on the drift demands	Checked for capacity as above	Checked for capacity as above

For the linear static procedure (LSP) in ASCE 41, a building is subjected to a “pseudo seismic force”, which is defined as the force that is expected to give the same deformations in the building in a linear analysis as are the expected deformations in the building in its yielded state, when subjected to the design earthquake motion. The seismic force is distributed along the height by assuming a first-mode profile specified by the standard. The analysis is carried out, and force demands in individual components are determined. Force demand-to-capacity ratios (DCR) are calculated for all structural components. If certain irregularities are present, a limit is imposed on the maximum DCR beyond which the linear methods are not permissible. LSP is not recommended if the fundamental period of the building is greater than or equal to 3.5 times Ts, the characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment to the constant velocity segment of the spectrum. There are additional limiting considerations concerning irregularities and building layout, specified in ASCE 41.

For checking acceptance in LSP, actions on the building components are identified as either deformation-controlled (ductile) or force-controlled (brittle). DCRs are calculated again, but the capacities are adjusted based on whether the action is deformation or force-controlled. The comparison is based on the force capacity and demand, irrespective of whether the action is force-controlled or deformation-controlled. Probable force capacities of the deformation-controlled actions are increased based on the inherent ductility in individual components by multiplying with the respective capacity modification factors, m. For force-controlled actions, demand is limited by the maximum force transferred to the force-controlled component by the yielding structure based on the expected strength of components transferring force to the force-controlled components/mechanisms. Capacities of the force-controlled mechanisms are calculated based on the lower-bound strength. This is different than the capacity of the deformation-controlled actions, which are based on the probable strengths. A knowledge factor, k, (≤1) is also multiplied to the capacity to keep an allowance for uncertainty in the capacity determination. Since elastic deformations from LSP are intended to be estimates of the expected non-linear deformations, the gravity structure can be checked against the computed drifts directly, if it has not been modelled.

The lateral force method (EC8-3) is similar in philosophy to the LSP of ASCE 41. The distribution of forces along the height is based on the fundamental mode shape, which may be assumed as linearly increasing along the height (proportional to the product of mass and height). Similar to ASCE 41, EC8-3 specifies actions as ductile or brittle, which are the same as deformation-controlled or force-controlled, actions respectively. Force DCR for all actions are calculated to check the applicability of the linear methods. For applicability of the linear methods, the ratio of maximum to minimum DCR amongst all deformation-controlled actions with DCR greater than 1 shall be less than a limiting value, which is typically considered as 2.5. For the linear static method, the fundamental period of a building, in both directions, must be less than the lower of 2 sec or 4Ts, where Ts is as defined earlier. In addition to the above two conditions, the method is applicable for buildings that are first mode dominant and comply with the criteria for regularity in elevation as specified by the standard EN 1998-1:2004 [20].

For acceptance checks of the force-controlled actions, the procedure is similar as in the case of LSP of ASCE 41. However, the confidence factor (akin to the knowledge factor of ASCE 41) is applied to reduce the lower-bound strength of materials to calculate the capacity, while the demand (determined by the capacity of the ductile mechanism transferring force to the brittle mechanism) is enhanced by applying the confidence factor to the mean material properties. For deformation-controlled actions, unlike LSP of ASCE 41, DCR is calculated based on the deformation capacity and demand. Deformation demand is directly available from the analysis, while the deformation capacity is determined based on the provisions of the guidelines using the mean properties, reduced by the confidence factor. The gravity structure can be assessed by checking the drift capacity against the drift demand.

Another linear static method suggested by EC8-3 is the q-factor method. It is similar to the Equivalent Static Method defined in the NZ guidelines. In this method, the demand is reduced based on the expected available global ductility factor (or q-factor).

Non-linear Static Methods

The salient aspects of the non-linear static methods are summarized in Table 3. In the case of NZ guidelines, the non-linear static pushover analysis (NLSPA) is carried out by applying 'push' to the structure until the assessed capacity of a component is reached, failure of which is deemed to be a significant life safety hazard. The analysis is performed in the two horizontal directions separately, but the effects are required to be combined for elements where bi-axial effects are expected to be significant. This is achieved by considering 100% of effects in one direction with 30% from the other orthogonal direction. The P-Delta and accidental torsion effects are also included, if required.

The NZ guidelines recommend at least one of each of the two categories of lateral load profiles to be considered. In the first category, at least one of the following three is to be selected: (1) A modal pattern based on the fundamental mode shape or (2) a lateral load pattern depending upon the story mass and story height (approximate first mode shape) or (3) a pattern proportional to the story shear distribution obtained by combining modal responses (to include at least 90% of total seismic mass) from a response spectrum analysis of the building. The first two are applicable if the mass participation in the fundamental mode is more than 75%. The third is applicable when the fundamental period of the building exceeds 1 second. The second category includes two patterns: (1) a load pattern with loads proportional to the mass at each story and (2) a load pattern that changes as the structure is displaced, considering the properties of the yielded structure. NLSPA can be carried out by explicitly modelling the full force-displacement backbone of individual components. Alternatively, a simplified analysis may be carried out by modelling a bilinear elastic-plastic force-displacement relationship for components.

Assessment using the nonlinear analysis methods in New Zealand guidelines is similar to the capacity spectrum approach [15]. The pushover response is converted into the capacity curve of an equivalent single-degree-of-freedom (SDOF) system. The capacity curve is then compared with the demand spectrum, represented as an Acceleration-Displacement response spectrum (ADRS) spectrum. The demand spectrum is reduced based on damping, which is estimated as a function of ductility. For the estimation of ductility, yield deformation is to be estimated. Based on the NZ guidelines, there is no strict way of estimating the yield deformation. One of the ways is assuming elastic-perfectly plastic idealization, balancing the areas between the actual and the idealized curve, as shown in Figure 5.

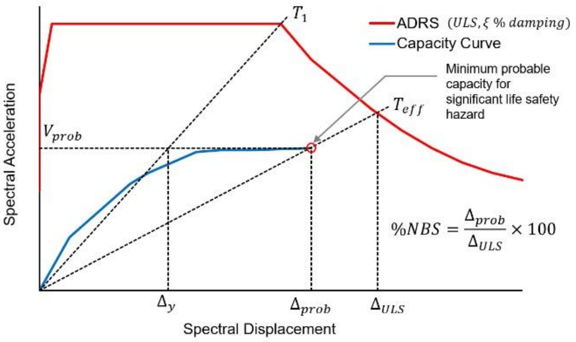

Figure 5: Nonlinear Static Pushover Assessment using the ADRS plot (Adapted from NZ guidelines).

If higher modes are deemed to be significant, i.e., the sum of modal mass participation factors in the first two translation modes in each direction is less than 60%, the internal actions are also to be checked with the results of a modal response spectrum analysis.

Nonlinear Static Procedure (NSP) is permitted only when sufficient information is available to justify a knowledge factor of 1. Further, NSP is permitted if the global ductility factor, $\mu_{strength}$ is below the threshold maximum value, μ_{max}. If the global ductility factor exceeds the maximum value, it is deemed that dynamic instability will become important and a Non-Linear Dynamic (Non-linear Time History) Procedure (NDP) is recommended. This is an important distinction as compared to the New Zealand guidelines where the component deformation capacities are kept lower than the degrading response, and the pushover response does not extend into the degrading branch.

The recommended load pattern is based on the fundamental mode shape in the direction under consideration. Base shear vs roof displacement response is to be determined up to 150% of the target displacement given, which is given by

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \tag{1}$$

T_e is the effective period defined as:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \tag{2}$$

Here, T_i is the elastic period of the building, K_i is the elastic lateral stiffness of the building and K_e is the effective lateral stiffness of the building based on the idealized pushover response, as shown in Figure 6. The effective lateral stiffness is taken as secant to 60% of the effective yield strength, V_y. Δ_d is the displacement corresponding to the maximum base shear V_d or the target displacement, whichever is smaller. The first two line segments are drawn such that the areas below and above the actual pushover curve are the same and $V_y \leq V_d$. The third line segment of the idealized curve is drawn passing the point on the actual pushover where base shear is 60% of V_y.

Table 3: Non-linear static analysis methods.

Parameter	NZ guidelines (NLSPA)	ASCE 41 (NSP)	EC8-3 (NLSPA)
Target displacement	Displacement at which a mechanism is formed or a significant life safety hazard is identified to occur	$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$	$\delta_t = k.\Gamma.S_e(T^*)\left[\frac{T^*}{2\pi}\right]^2$
Load profile	First mode (or Mass & Height proportional)/Multi mode shear distribution based (T>1s) And Mass proportional/Force pattern changing with yielding of structure	First Mode	First Mode (or Mass & Height proportional) and Mass proportional
Applicability	Higher mode effects are not significant (alternatively, in combination with Modal response analysis)	$\mu_{strength} < \mu_{max}$ Higher mode effects are not significant (alternatively, in combination with Modal response analysis)	Higher mode effects are not significant (alternatively, in combination with Modal response analysis)
Modelling	Both primary and secondary seismic elements are modelled Full backbone curve using strength degradation and residual strength	Both primary and secondary seismic elements are modelled Full backbone curve using strength degradation and residual strength	Both primary and secondary seismic elements are modelled Full backbone curve using strength degradation and residual strength
Acceptance criteria	Building Capacity curve is checked against ULS demand ADRS for %*NBS*	Deformation controlled actions – $\kappa\Delta_{CE} > \Delta_{UD}$ Force controlled actions – $\gamma\chi(Q_{UF} - Q_G) + Q_G \leq Q_{CL}$ In general, the demand for a force-controlled action is limited by the maximum force that can be transferred by the yielding structure	Ductile – $\Delta^{\#}_{cap} > \Delta_{UD}$ Brittle – $Q_{CL} > Q_{UF}$* # Evaluated after diving the mean properties by the confidence factor *Maximum force that can be transferred from ductile mechanisms, calculated by multiplying mean properties with the confidence factor

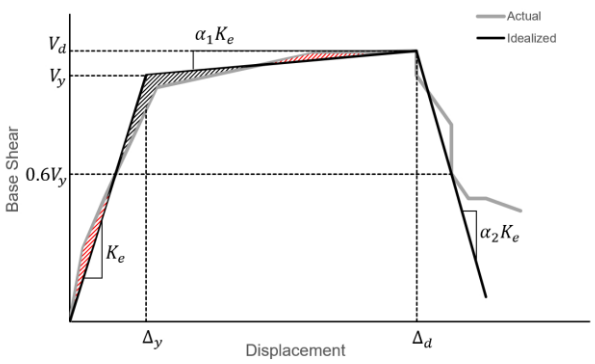

Figure 6: Idealized pushover response (Adapted from ASCE 41).

Acceptance criteria of the deformation-controlled actions are based on the deformation DCR corresponding to the performance level the building is being assessed for. Force-controlled actions for which non-linear behavior has not been directly modelled, are assessed based on DCR calculated using lower bound strength and demand calculated from the pushover analysis adjusted for the criticality of the action and the performance level, as shown in Table 3.

As in the case of NZ guidelines, per ASCE 41 as well, the analysis results in the orthogonal directions can be combined by considering 100% of actions in one direction with 30% of actions in the other direction.

EC8-3 also specifies two loading patterns to be considered: (1) a 'modal' pattern with the lateral forces proportional to the forces based on the first mode, which can be approximated as proportional to the product of story seismic mass and story height, and (2) a 'uniform' pattern with loads proportional to the mass at each story irrespective of height.

For determination of the target displacement, the building pushover response is converted to the pushover response of an equivalent single degree of freedom (SDOF) system by using a transformation factor based on the seismic mass and displacements at each story, as detailed in EC8-3. It is then idealized in an elastic-perfectly plastic manner such that the areas under the actual and the idealized curve are the same, balanced by areas above and below the actual curve, as shown in Figure 7. If there is a large difference between the calculated target displacement and the displacement corresponding to the mechanism formation (point A), the target displacement is treated as point A for idealization of the curve. In other words, the pushover response up to the target displacement is idealized as bilinear elastic-perfectly plastic.

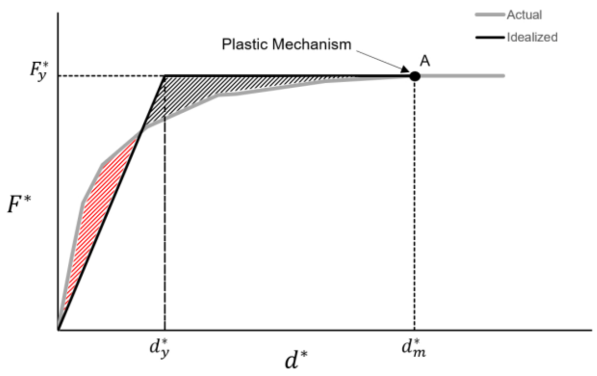

Figure 7: Idealized pushover response (Adapted from EN 1998-1:2004).

The target displacement of the equivalent elastic SDOF system is determined as:

$$\delta_{t,SDOF} = k.S_e(T^*)\left[\frac{T^*}{2\pi}\right]^2 \quad (3)$$

The target SDOF displacement is then multiplied by the transformation factor, Γ to determine the target roof displacement:

$$\delta_t = k.\Gamma.S_e(T^*)\left[\frac{T^*}{2\pi}\right]^2 \quad (4)$$

Acceptance of a deformation-controlled action is in terms of the deformation DCR, which is based on the demand from the analysis at the target displacement and the capacity corresponding to the performance level under consideration, calculated using mean properties divided by the confidence factor. For a force-controlled action, acceptance is in terms of the force (or strength) DCR, based on the demand from the analysis and the capacity based on the lower bound properties, divided by the confidence factor as shown in Table 3.

Note that per EC8-3, there is no limitation on knowledge (or confidence) to employ the non-linear static procedure as in the case of ASCE 41. Per EC8-3 as well, the analysis results in the orthogonal directions can be combined by considering 100% of actions in one direction with 30% of actions in the other direction.

APPLICATION OF ASSESSMENT GUIDELINES ON THE EXAMPLE BUILDING

ASCE 41 Application in FEMA P-2006

FEMA P-2006 includes an example concrete shear wall building that has been evaluated using LSP and NSP of ASCE 41-13 with the mention of revisions in ASCE 41-17, wherever applicable. The example building is a three-story building with a single basement constructed in the 1950s, located in Seattle, Washington, USA. The size of the building in plan is 36 m x 18 m (120 ft. x 60 ft.), with a uniform grid spacing of 6 m (20 ft.) in both directions. The height of each story, including the basement, is 4.2 m (14 ft.). The basement has retaining walls all along its periphery. There are two walls in the longitudinal direction and a single wall in the transverse direction. This wall in the transverse direction (grid D in Figure 8) is discontinued at the ground floor level and is supported by two columns below in the basement. The rest of the structure is identified as the gravity load supporting structure and consists of moment frames with columns located in grids of 6 m (20 ft) in either direction. The typical floor plan and an elevation section of the building are shown in Figures 8 and 9.

In FEMA P-2006, Tier-1 screening of the building per ASCE 41-13 is conducted. It is found that the building lacks redundancy in the transverse direction, as only one shear wall is oriented in that direction. The wall also fails the quick shear check per ASCE 41-13, and since it is also discontinued at the ground floor level, it is identified as a vertical discontinuity. Two new walls are added at the periphery in the transverse direction on grid A and grid G, as shown in Figure 8. No modification is proposed for the discontinued wall due to the space requirements of the client in the basement. With these additional walls, analysis for the Tier-3 evaluation of the building is conducted. The BPOE selected is life safety (LS) and collapse prevention (CP) at BSE-1E and BSE-2E hazard levels, respectively.

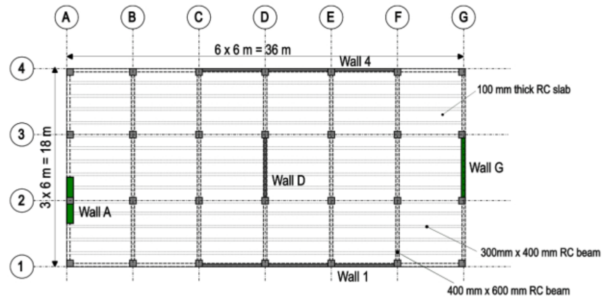

Figure 8: Floor plans of the building (Walls at grids A and G are new walls) (Adapted from FEMA P-2006).

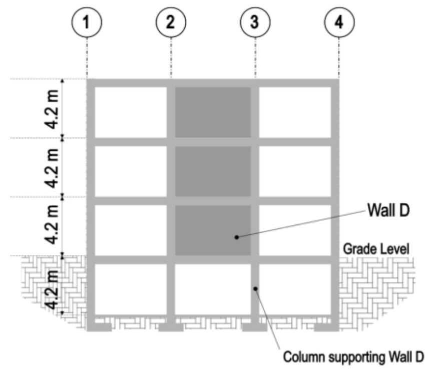

Figure 9: Elevation along Grid D (Adapted from FEMA P-2006).

In FEMA P-2006, for LSP, only the lateral force system (walls) is considered. The distribution of forces along the height is considered per ASCE 41, assuming the forces at each story to be proportional to the story mass and height from the base. A rigid diaphragm based analysis is done to calculate the force distribution in individual walls. Torsion due to an accidental eccentricity of 5% of the building's plan dimension is included in the analysis. Only the superstructure (above grade slab level) is included in the analysis. The columns supporting the discontinuous wall at grid D in the basement are analyzed for the forces transferred by the wall at grid D. The floor diaphragm at the ground floor level is also checked for adequacy in transferring the loads from the discontinued wall to the peripheral retaining wall. The adequacy of the wall-diaphragm connection is also checked. Typical calculations for the gravity load supporting columns have also been carried out. Note that a few issues with the published example in FEMA P-2006 are identified during the current study. These issues are discussed in the detailed report [13]. Assessment outcome per ASCE in this paper is presented after accounting for some of the issues.

For NSP application in FEMA P-2006, the entire building is modelled. The walls and the columns are modelled to behave in a non-linear manner, while the beams are elastic and the diaphragms are elastic and flexible.

In this study, the provisions of SLaMA, and the linear static and non-linear static analysis methods of the NZ guidelines. and EC8-3 are applied to the FEMA P-2006 example building.

SLaMA

As a first step for assessment per NZ guidelines, SLaMA is conducted. For conducting SLaMA, the primary lateral systems are identified in each direction. In this case, the primary lateral systems are the walls in both directions. To remain consistent with FEMA P-2006, the analysis of the superstructure alone (above ground floor level) is considered. The variation of forces along the height is also considered to be the same as in the case of FEMA P-2006, which is proportional to the product of the seismic mass and story height. It is found that the walls in the longitudinal direction are governed by shear while the walls in the transverse direction are governed by flexure. For convenience of notation, in the rest of the paper, the wall on grid 1 is called Wall 1, the wall on grid 4, Wall 4, and so on.

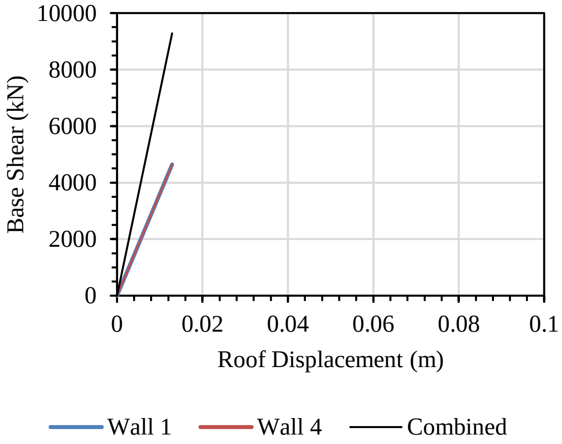

Figure 10(a): Base shear vs deformation (Longitudinal direction).

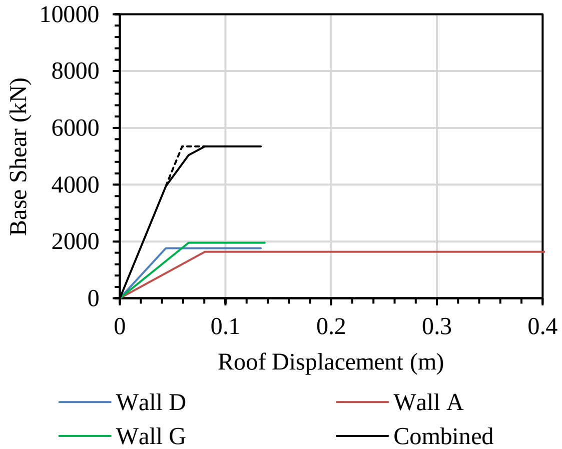

Figure 10(b): Base shear vs deformation (Transverse direction).

The base shear vs. roof displacement curves of the lateral system in each direction are shown in Figures 10(a) and 10(b). Rotation capacities of the walls are calculated based on the estimates provided in the NZ guidelines. The longitudinal walls 1 and 4 are identified to be shear-governed, with horizontal reinforcement being 0.1%. NZ seismic assessment guidelines allow some inelastic rotation capacity in shear-governed walls with low axial loads. However, if the horizontal reinforcement in walls is less than 0.15%, the inelastic rotation capacity is to be considered zero, which is the case for walls 1 and 4. Their yield (and ultimate) deformation capacity is hence considered to be their estimated yield deformation capacity, reduced in proportion to the ratio of their shear capacity to the shear corresponding to their moment capacity at yield. Figures 11(a) and 11(b) show the comparison of demand and capacity in both directions, shown in the Acceleration Displacement Response Spectrum (ADRS) format for the idealized SDOF system. Demand Spectrum is based on ULS for IL-2 category building, which corresponds to a return period of 500 years (Annual probability of exceedance 1/500). In the longitudinal direction, the capacity of the building falls short of the demand (%*NBS*=85), while in the transverse direction, the capacity exceeds the demand (%*NBS*=124). However, note that SLaMA does not account for torsion, which is to be included in the other analysis methods employed.

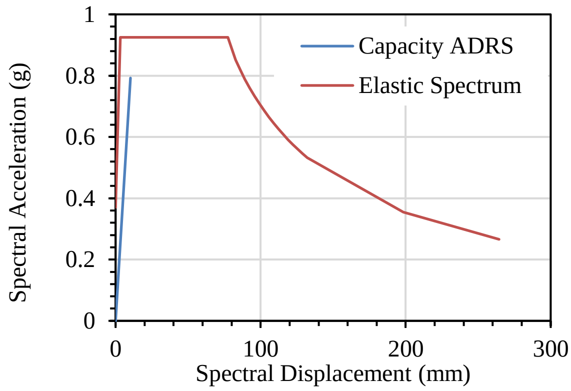

Figure 11(a): ADRS (Longitudinal direction).

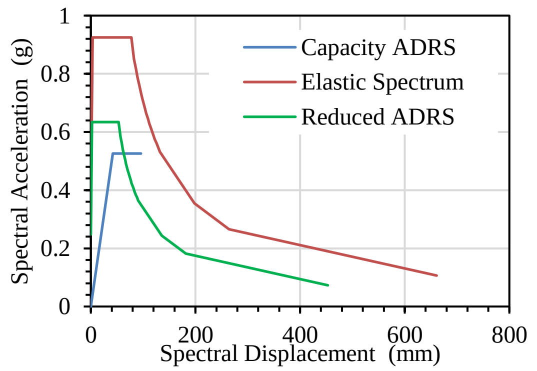

Figure 11(b): ADRS (Transverse direction).

Since no inelastic deformation capacity is considered for walls in the longitudinal direction, the capacity curve is only the linear elastic branch. The stiffness as well as the strength of these walls are large as compared to the transverse direction.

In the transverse direction, Wall D has the largest stiffness but the least strength and post-yield deformation capacity of the three walls. As the capacity curve is limited by the deformation capacity of Wall D, the additional ductility provided by the new walls A and G does not get utilized. As a result, the system as a whole is expected to increase in stiffness and strength through addition of Walls A and G but not in ductility. This highlights an important distinction between SLaMA and the assumptions in FEMA P-2006 using ASCE 41. Per FEMA P-2006, Wall D has the lowest stiffness of all the walls based on the assumed stiffness values, however, SLaMA uses a 'secant to yield' approach, which results in Wall D being the stiffest. Such a difference in relative stiffnesses may end up in a different assessment outcome altogether in some cases. Note that Wall G has a shear span-to-length ratio of less than 2. Accordingly, its drift capacity is considered as 1%, as recommended by the NZ guidelines. However, the guidelines also recommend that squat walls that have the detailing as per the limited ductile or ductile walls may be considered as slender walls for estimation of their drift capacities. Wall G, being a new wall detailed with confined boundary regions, is expected to have a larger drift capacity. However, here, its drift capacity is conservatively considered as 1%. It is shown that this choice does not change the outcome.

Linear Static Analysis

LSP application based on ASCE 41 is covered by FEMA P-2006. The linear static analysis provisions of the NZ guidelines and EC8-3 are applied to the example building. As in the case of FEMA P-2006, for the linear static methods, only the superstructure above the basement is analyzed by carrying out rigid diaphragm based analyses.

The applicability of the linear methods is first verified. Though there is a discontinuous Wall D, but since only the superstructure is analyzed, this discontinuity is ignored while considering the applicability of the linear static methods. Forces in the walls are computed considering 'secant-to-yield' stiffness and accidental torsion in each direction (10% eccentricity for the Equivalent Static Method and 5% eccentricity for the Lateral Force Method). The capacities and demands are calculated as specified by the respective guidelines. For EC8-3, the performance objectives selected are the Limit State of Significant Damage (SD) and the Limit State of Near Collapse (NC).

Note that the distribution of forces at different floor levels for the Equivalent Static Method (NZ guidelines) is slightly different from that for the Lateral Force Method (EC8-3). For the equivalent static method, 8% of the base shear is applied at the roof level and the remaining 92% is distributed at all floor levels using the linear profile, like in the Lateral Force Method. Here, the linear load profile as in the case of the Lateral Force Method is considered for the Equivalent Static Method as well. Equivalent Static Method profile leads to a larger shear span of walls. Since the base shear capacity in the transverse direction is governed by the moment capacity of Wall D, the Equivalent Static Method profile leads to a decrease in the base shear capacity (and hence, %*NBS*). However, the resulting change is less than 3%. In the longitudinal direction, the base shear capacity is governed by the shear capacity of Walls 1 (and 4). Hence, the change in the shear span makes no difference to the outcome.

Apart from the lateral system, the basement columns supporting Wall D, the diaphragm at grade level, and the diaphragm to Wall D connection are checked separately based on the force transferred by Wall D. The gravity system is checked based on the calculated drift demands. Foundation checks have not been carried out. The results from the analyses are summarized in Table 4. Also shown are the results obtained from ASCE 41 after accounting for some of the issues observed in the FEMA P-2006 LSP example.

The results summarized in Table 4 show that the assessment, using the linear static methods of all three guidelines, identifies that the building is deficient in the longitudinal direction due to inadequate shear capacity of walls 1 and 4. NZ guidelines specify buildings with %*NBS*<67% as earthquake-risk buildings, and those with %*NBS*<34% as earthquake-prone buildings. Any building rated to be earthquake-prone is to be mandatorily retrofitted or removed within a defined timeframe [21]. However, buildings with %*NBS* ≥ 34% are tolerable. The governing %*NBS* score for the lateral system in the longitudinal direction is more than 69%. So, based on the lateral system, the %*NBS* score is tolerable. DCR values suggest that the lateral system in the longitudinal direction does not meet either of the two performance objectives in the case of ASCE 41 and EC8-3.

In the transverse direction, the most critical component in the lateral system is identified to be Wall D in the case of NZ guidelines and EC8-3. While it is shear-governed for the assessment carried out using EC8-3, it is deemed to be flexure-governed in the case of NZ guidelines. This is due to the lower calculated shear strength based on EC8-3. Shear strength considering mean material strengths for wall D based on NZ guidelines, ASCE 41 and EC8-3 are 2323 kN, 1726 kN, and 1303 kN, respectively. Further, since shear in walls is identified as a brittle mechanism, per EC8-3, for acceptance checks, lower bound strength is considered, which pushes the DCR higher. Per the NZ guidelines, Wall D is recognized as a squat wall, and accordingly, its rotation capacity is limited to 1% based on the axial load it carries and its transverse reinforcement ratio. The %*NBS* score is thus limited by its moment capacity in the case of Equivalent Static Analysis and its deformation capacity in the case of SLaMA. Note that in FEMA P-2006, Wall D is initially deemed to be flexure-governed based on force DCR, calculated with the probable strengths, and later, only the acceptance check for flexure is applied. However, shear in Wall D per ASCE 41 is displacement-controlled and m-factors for flexure and shear are different. As a result, when checked for shear, the deformation DCR for shear is higher than the DCR for flexure. Hence, the governing mechanism is found to be shear in Wall D for the collapse prevention performance level. The governing mechanism for the life safety performance level is flexure in Wall A. Based on the lateral system in the transverse direction, the %*NBS* score is tolerable per NZ guidelines, however the DCR values suggest that it does not meet any of the two performance objectives in the case of EC8-3, while meeting both performance objectives in the case of ASCE 41.

The diaphragm - Wall D connection at grade level has been conservatively assessed to be deficient per ASCE 41 in FEMA P-2006. In the FEMA document, the floor beam at gridline D is considered insufficient to act as a collector as the tension reinforcement in the floor beam is not relied upon. Shear resistance is only considered to be provided by two 6m long floor slab-wall D interfaces. For consistency with FEMA P-2006, for assessing the connection using NZ guidelines and EC8-3 here, the floor beam at gridline D is not considered to act as a collector due to insufficient embedment of reinforcement and /or limited capacity to resist bending plus axial tension. Also, in FEMA P-2006, the diaphragm is assessed for overall capacity by considering it to act as a beam, bending in plan, with part of basement walls acting as flanges. Any adjoining soil resistance is neglected. The diaphragm is found to be deficient. Keeping with the assumptions in FEMA P-2006, the same outcome results from a strut-tie model per NZ guidelines and EC8-3

Per all three standards, the diaphragm and the diaphragm-wall connection at grid D are found to be deficient. However, in the case of EC8-3, the DCR for Wall D in shear is greater than the DCR for the diaphragm and its connection to Wall D, implying that shear failure of Wall D is expected to preclude the failure of the diaphragm or its connection with Wall D. However, in the case of NZ guidelines and ASCE 41, due to higher calculated shear strength of Wall D, the diaphragm is assessed to be weaker than Wall D.

Table 4: Summary of analysis results-linear static methods (Governing component is shaded in Grey colour).

S. No.	Component	NZ guidelines SLaMA	NZ guidelines Equivalent Static Method	EC8-3	ASCE 41
1.	Lateral Force Resisting System (Longitudinal Direction)	*%NBS* = 85% (ULS) Wall 1,4 (shear)	*%NBS* = 69% (ULS) Wall 1,4 (shear)	Max. DCR = 2.64 (NC) Wall 1,4 (shear) Max. DCR = 1.77 (SD) Wall 1,4 (shear)	Max. DCR = 1.80 (CP) Wall 1,4 (shear) Max. DCR = 1.16 (LS) Wall 1,4 (shear)
2.	Lateral Force Resisting System (Transverse Direction)	*%NBS* = 174% (ULS) Wall D (deformation capacity)	*%NBS* = 85% (ULS) Wall D (moment capacity)	Max. DCR = 1.71 (NC and SD) Wall D (shear)	Max. DCR = 0.64 (CP) Wall D (Shear) Max. DCR = 0.54 (LS) Wall A (Flexure)
3.	Diaphragm Capacity	*%NBS* = 78 %		DCR=1.46 (NC and SD)	DCR=1.22 (CP and LS)
4.	Diaphragm-Wall Connection	*%NBS* = 85 %		DCR=1.60 (NC and SD)	DCR=1.32 (CP and LS)
5.	Columns at Grid D - Compression	Not governing		DCR = 0.71 (NC and SD)	DCR = 0.87 (CP and LS)
6.	Columns at Grid D - Tension	Not governing (Uplift-Not enough resistance to cause tension)		Not governing (NC and SD) (Uplift-Not enough resistance to cause tension)	Not governing (CP and LS (Uplift - Not enough resistance to cause tension)
7.	Gravity Columns in Transverse Direction	Not Governing (Expected to undergo inelastic deformation)		Not Governing (Expected to undergo inelastic deformation)	Not Governing (Expected to undergo inelastic deformation

Further, in the case of NZ guidelines, failure of the diaphragm results before yielding in any of the walls, and since Wall D is the stiffest of all the walls in the transverse direction, the behavior of the lateral system is elastic when the diaphragm fails. As a result, the demand is pushed up to the elastic demand (zero plastic deformation), thus lowering the *%NBS* rating considerably. However, the NZ guidelines recommend that the diaphragm capacity need not be taken greater than due to the building overstrength. Hence, if the diaphragm has a capacity just higher than the moment capacity of wall D, inelastic action will not occur in the diaphragm. Therefore, the demand for assessing the diaphragm at grade level and its connection to wall D is considered as the overstrength capacity of Wall D.

Note that EC8-3 and ASCE 41 limit the demand for force-controlled actions as the maximum force that can be transferred by the non-linear response of the building. The seismic force applied on wall D exceeds its moment capacity for both hazard levels in the cases of ASCE 41 and EC8-3. Thus, the demand on the diaphragm is limited to the moment capacity of wall D. Hence, the DCR is the same irrespective of the hazard (and performance level). Per NZ guidelines, *%NBS* scores for the diaphragm and diaphragm-Wall D connection are greater than 34%, and thus within the tolerable range. DCR values also suggest that the diaphragm does not meet any of the performance objectives in the cases of ASCE 41 and EC8-3.

The columns at gridline D, under Wall D, are acceptable in compression. In tension, due to the limited resistance from the foundation, uplift is expected to occur before the tension capacity of the columns is reached. The building is least stiff in the transverse direction, and hence, the gravity columns are checked for drift demands in the transverse direction and are found to experience yielding but be within the drift limits at the onset of axial failure.

Often, the component stiffnesses for linear elastic methods are based on stiffness multipliers to arrive at the effective stiffness of members, like ASCE 41 effective stiffness values or stiffness modifiers suggested in the commentary to New Zealand concrete structures standard, NZS 3101.2:2006 [22]. Secant-to-yield stiffnesses can also be estimated using yield rotation estimates based on the recommendations of NZ guidelines. However, the secant to yield stiffness calculated using NZ guidelines' provisions often leads to lower values as compared to the NZS 3101.2:2006 stiffness multipliers. Which approach is to be used is open to debate. Suggestions have been made to align the NZS 3101.2:2006 stiffness multipliers with the NZ guidelines' secant to yield estimates for members expected to yield [23]. Since forces in the members are directly proportional to their elastic stiffnesses when using the linear analysis methods, the force distribution and the analysis outcome depend on the estimation of the elastic properties of the members. ASCE 41 specifies estimates of the effective section properties for carrying out analysis using LSP based on component test results. For example, estimates of the effective stiffness of columns with low axial loads are derived from reversed cyclic tests [24]. Effective section properties are expected to give sufficiently accurate estimates of the building period and the distribution of internal forces in a building. EC8-3 specifies that equivalent stiffness shall be calculated based on the 'secant to yield' stiffness. It is calculated as $M_y L_v / 3\theta_y$, where M_y is the yield moment, L_v is the shear span and θ_y is the estimated yield rotation, per provisions of the standard. For NZ guidelines, both stiffness multipliers using NZS 3101.2:2006 and the secant to yield stiffness are estimated based on the estimates of yield rotation per provisions of the guidelines. Figure 12 shows the relative stiffness of walls, calculated using ASCE 41 effective section properties, 'secant to yield' stiffness per EC8-3, NZS 3101.2:2006 stiffness multipliers, and 'secant to yield' stiffness calculated using the estimated yield deformations per NZ guidelines.

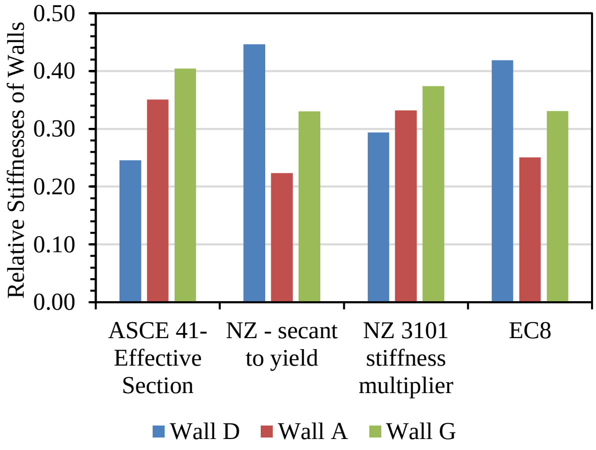

Figure 12: Relative stiffness of walls in transverse direction (Total for three walls add to 1).

Figure 12 demonstrates that the relative stiffnesses, and hence, the force distribution amongst the walls, is drastically different between the 'secant to yield' approach of NZ guidelines and EC8-3 from that calculated considering the effective section properties of ASCE 41 and NZS 3101.2:2006 stiffness multipliers. While Wall D is the stiffest considering 'secant-to-yield' stiffness, it is the least stiff using the effective stiffness approach. Since Wall D has been shown to be the most critical of the walls in assessment in the case of NZ and EC8-3 guidelines, such a difference can lead to a different assessment outcome. Note that in Table 4, the critical component in the transverse direction is shown to be Wall D based on secant-to-yield stiffness estimates. However, based on NZS 3101.2:2006 stiffness multipliers, the critical wall in the transverse direction is found to be Wall A due to different estimates of the relative stiffnesses of walls, as shown in Figure 12. But, %*NBS* increases from 85% to 95%. %*NBS* in the longitudinal direction remains almost the same.

Also, the properties of the two new walls, A and G, in the FEMA P-2006 example may have been so selected that they are stiffer than Wall D so that their ductility is utilized. However, as can be seen from SLaMA, it is the ductility of Wall D that governs the ductility of the lateral system in the transverse direction, while the new, more ductile walls only add to strength and stiffness without achieving their full ductility capacity. Per EC8-3, since Wall D is deemed to be shear governed with low estimated shear capacity, Wall A and Wall G do not achieve even half of their strength before Wall D fails in shear. Hence, the choice of strengthening measures taken may strongly depend on how the stiffness of the walls is calculated. In addition to the difference in the relative stiffnesses, the absolute stiffness (or effective stiffnesses) also differ from one standard/method to another. Figure 13 shows the effective stiffness (as a fraction of gross section stiffness) calculated using ASCE 41 effective section properties, NZS 3101.2:2006 stiffness multipliers and 'secant to yield' stiffness calculated per NZ guidelines and 'secant to yield' stiffness calculated per EC8-3. This also affects the calculation of drift demands for the assessment of gravity structure.

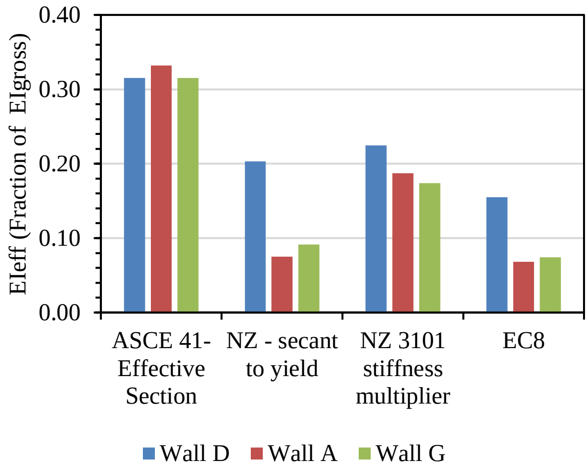

Figure 13: Effective stiffness of walls in transverse direction.

Another important aspect is the treatment of uncertainty concerning the material properties. Both ASCE 41 and EC8-3 specify a knowledge (or confidence) factor depending upon the extent of inspection/testing, whereas, the NZ guidelines do not specify the use of such a factor. Such factors introduce some degree of conservatism in assessment.

Non-linear Static Analysis

For non-linear static analysis, a 3D model is developed for analysis using the Python version of the open-source framework OpenSees [25], i.e., OpenSeesPy. Figure 14 shows the OpenSeesPy model. Unlike FEMA P-2006, where the basement walls are modelled, they are not included in the model here as they are very rigid in-plane compared to the superstructure above. However, to model the out-of-plane flexibility of walls, linear springs are modelled along the length of the basement walls at the base (ground level), as shown in Figure 15. Column footings are considered fixed in translation while the rotational degrees of freedom are released. Columns supporting Wall D are allowed to uplift in tension once the resistance due to the dead weight of the footing, load on the basement floor, and overburden soil is exceeded. Walls have been modelled with 1D displacement-controlled fibre elements to capture the flexural behavior of walls A, D, and G, which are expected to yield in flexure and linear elastic elements for highly squat walls 1 and 4 in the longitudinal direction, which are dominantly shear controlled. Rigid, elastic elements are connected at either end of the fibre wall elements to model the rigid offsets along the length of the wall, as indicated in Figure 14. The columns are also modelled using the displacement-controlled fibre elements. Beams were initially modelled using linear elastic beam-column elements; however, it was found that uplift in the basement column supporting the discontinuous wall D was leading to demands in excess of the strength of the adjacent beams and hence lumped elastic-perfectly plastic hinges have been introduced at the ends of beams at grid D at all stories to capture nonlinear behavior in those beams. Secondary beams have not been modelled.

It is noted that assessment using the linear elastic methods shows the diaphragm at grade level to be inadequate, as summarized in the last section; however, for the non-linear analysis, it is assumed that diaphragm issues have been addressed and do not govern. Hence, the diaphragms are modelled using elastic shell elements.

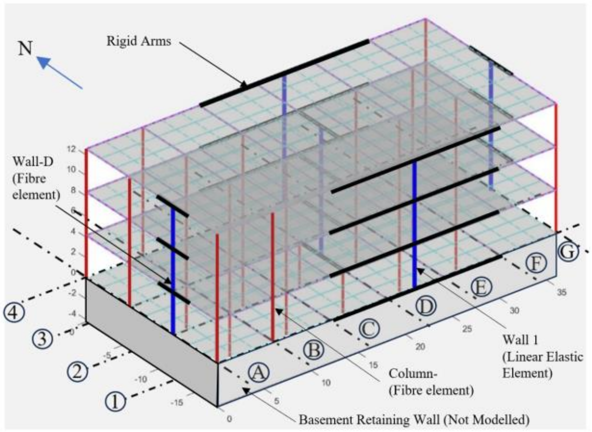

Figure 14: OpenSeesPy model with columns (Red), Beams (Magenta), Shear walls (Blue with rigid arms in Black) and Diaphragm (Cyan).

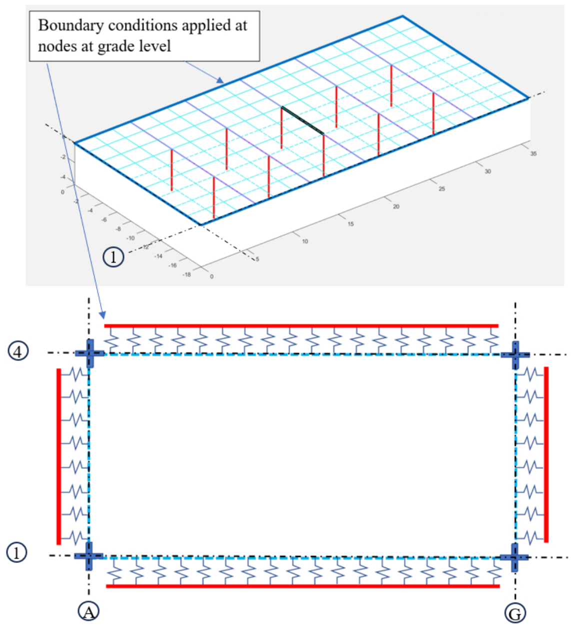

Figure 15: Boundary conditions at grade level – Springs in retaining wall out of plane direction, other degrees of freedom are fixed.

The Kent-Part-Scott compressive strength model [26] with the unloading/reloading stiffness based on Karson and Jirsa [27] and no tensile strength is used to model the confined and the unconfined concrete. This material model is implemented in OpenSeesPy as material '*Concrete01*'. Peak compressive stress and strain for the confined concrete are calculated based on Mander et. al. [28]. For the unconfined concrete, compressive stress is assumed to drop to zero at a strain of 0.005. For reinforcement, OpenSeesPy material '*Steel01*' with bilinear stress-strain relationship and kinematic strain hardening is considered. To model the shear force-displacement behavior of walls, the tri-linear relationship prescribed by ASCE 41 for walls, as used in FEMA P-2006 for all walls, has been considered; however, with a gentler post-capping slope, as shown in Figure 16. Recall that walls 1 and 4 are highly squat walls with transverse reinforcement of 0.1%. Per the NZ guidelines, their inelastic rotation capacity shall be taken as zero. ASCE 41 also treats walls with less than 0.15% transverse reinforcement as force-controlled. So, the inelastic shear force-deformation behaviour shown in Figure 16 is not expected for Walls 1 and 4. Note that while inelastic behaviour is used to model the walls, the capacity of the building is kept limited to the point where inelastic deformation sets in the walls (i.e., only the initial and cracked stiffness segments are considered) as shown by the solid blue curve in Figure 16. Similarly, using EC8-3 provisions, walls 1 and 4 are considered force-controlled and assessed accordingly.

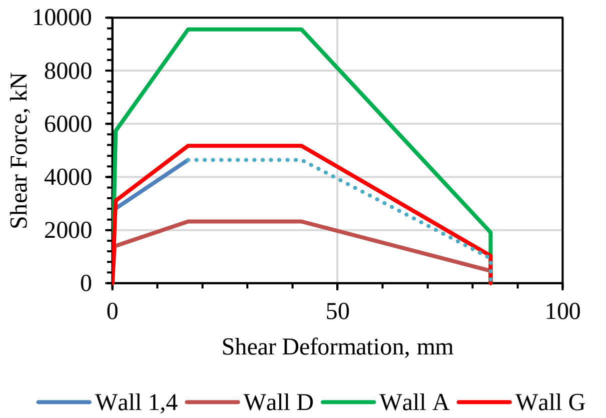

Figure 16: Force-deformation relationship for shear springs.

As discussed earlier, per NZ guidelines for the non-linear static pushover analysis, two load patterns (one in each category) must be considered. For this example, the first of each category is selected. The lateral load profiles are termed Load Profile-1 (Mass x height proportional) and Load Profile-2 (Mass Proportional). The results shown here are only for load profile-1, i.e., load proportional to seismic mass × height. For assessment using NZ guidelines, the pushover response is limited to the point corresponding to the onset of inelastic displacement in Walls 1 and 4, as shown in Figure 17. Also, since most of the base shear capacity is provided by the walls and the overall pushover response is highly dominated by the wall behavior, it can be assumed that there is no global ductility in the longitudinal direction. Pushover response may still be idealized in a bilinear manner; however, only the elastic branch is considered for assessment. An alternate idealization could be to consider to consider linear behaviour to the point where Wall 1 (and 4) reaches its peak strength, and inelastic action initiates. Both alternatives are shown in Figure 17. The alternative treatment will result in a lower stiffness and thus may lead to a higher %*NBS* score. However, for this example, since the walls are very stiff, both approaches result in the same outcome. Figure 18 shows the capacity curve and the demand spectra in ADRS format for both alternatives.

There is a slight increase in the base shear capacity as compared to SLaMA due to the contribution of secondary structure, which was neglected while carrying out SLaMA. Base shear capacity increases from 9284 kN to 10687 kN (profile-1) and %*NBS* score has also increased from 85% to 96%.

Figure 19 depicts the pushover curve in the transverse direction. The red circle in Figure 19 indicates the roof drift at which the uplift of the column on grid D sets in. The uplift starts at a roof drift as low as around 12 mm, leading to the redistribution of forces and reduction in rotation demand on Wall D.

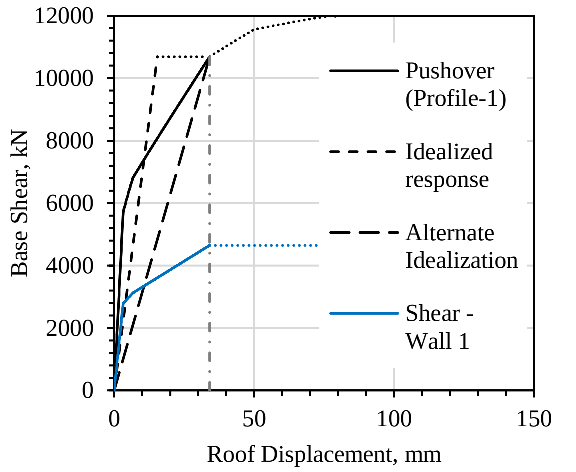

Figure 17: Pushover curve (Longitudinal direction).

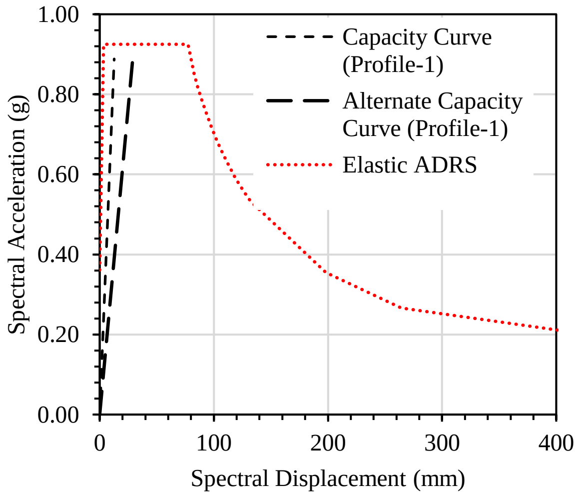

Figure 18: Capacity curve and demand Spectrum (Longitudinal direction).

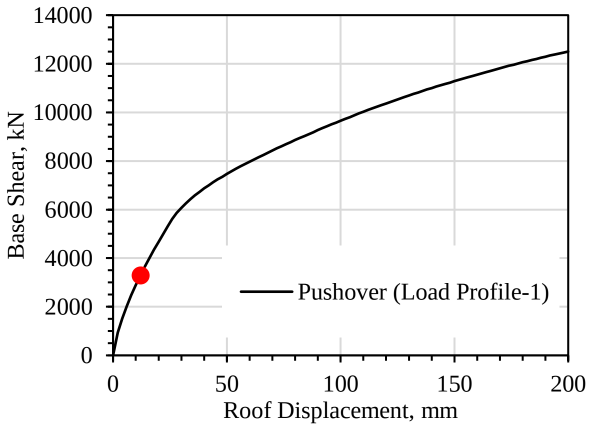

Figure 19: Base shear vs roof displacement - Transverse direction (Red circle denotes the point at which uplift of column at grid location -D starts to uplift).

Recall that Wall G, which is deemed to be a squat wall per NZ guidelines, has its ultimate rotation capacity conservatively estimated to be 1%. The drift capacity of the building in the transverse direction is limited by the drift capacity of Wall G. However, as shown later, the %*NBS* score is greater than 100%, thus this conservative assumption is not critical to the outcome of the assessment.

The truncated pushover curve limited by the drift capacity of Wall G and the idealized bilinear curve are shown in Figure 20. The pushover curve is idealized such that the areas under the actual and the idealized curves are equal. Figure 21 shows the capacity curve and demand spectrum.

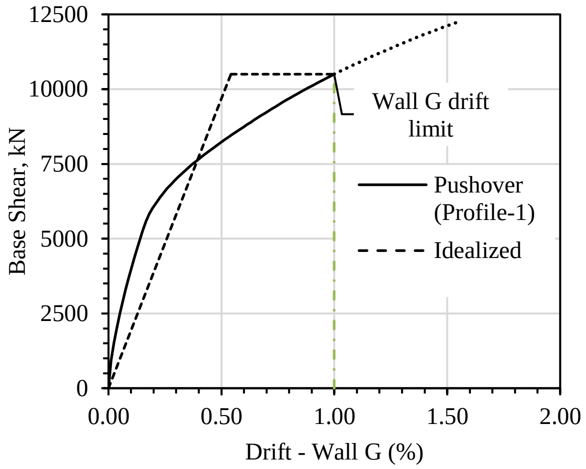

Figure 20: Pushover curve (Transverse direction).

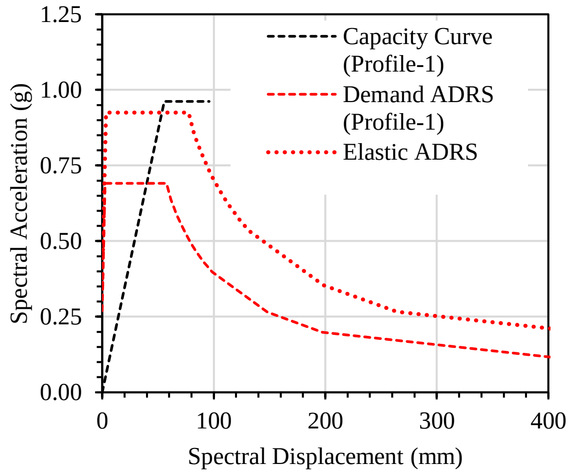

Figure 21: Capacity curves and demand spectra (Transverse direction).

The %*NBS* rating assessed based on the transverse direction is in more than 100%, with an increase from 124%*NBS* using SLaMA to 153%*NBS* using a nonlinear static analysis. The base shear capacity in the transverse direction, based on SLaMA, is 5346 kN. From Figure 20, it is evident that the base shear capacity (10499 kN) has increased substantially as compared to the outcome of SLaMA (and the Equivalent Static Analysis). This is due to a few reasons. First and foremost, the strengths of walls A and G have increased by considering the increased strength of the confined concrete in the walls. Secondly, for Wall G and Wall D, only nominal rectangular sections (as pointed out in the detailed report [13]) are considered for calculation of strengths in the case of the linear methods versus the consideration of the entire wall section, including columns at the ends, for the nonlinear methods keeping it consistent with FEMA P-2006. Thirdly, the contribution of only walls A, D, and G is considered for linear static methods. However, the contribution of the secondary frame structure, other than the walls, is also included in the base shear capacity in the non-linear static analysis. Figure 22 shows the contribution of different components to the base shear capacity. Since the basement column supporting Wall D is allowed to uplift in tension, force in Wall D redistributes to the adjacent frames, taking their contribution to around 17% of the total base shear.

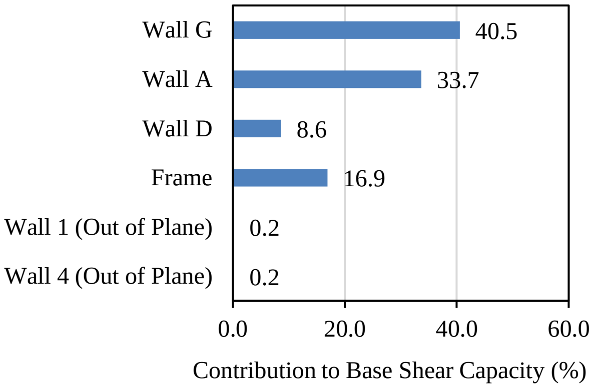

Figure 22: Contribution of different components to the base shear capacity (%) (Load profile-1).

The same OpenSeesPy model is used for applying EC8-3 provisions for the non-linear static pushover analysis. Note that there is a considerable difference in the shear capacity of walls determined from EC8-3 and NZ guidelines, and accordingly, the shear force vs. deformation relationship for the walls requires modification. However, in this section, since the intent is to discuss the difference between the application of assessment procedures, the modelling details are kept the same for the assessment using EC8-3 provisions as in the case of NZ guidelines to eliminate the influence of very low shear capacity calculated using EC8-3 provisions as compared to the NZ guidelines. Pushover analyses are carried out in both orthogonal directions, and the target displacements are computed for the two load profiles per EC8-3 recommendations. EC8-3 specifies that results from the pushover analysis in the two orthogonal directions be combined; however, considering the example building is a building with walls arranged orthogonally, analysis results are considered separately for each direction only. Figure 23 shows the pushover curve for load profile-1 in the transverse direction, i.e., proportional to the seismic mass × height at each story.

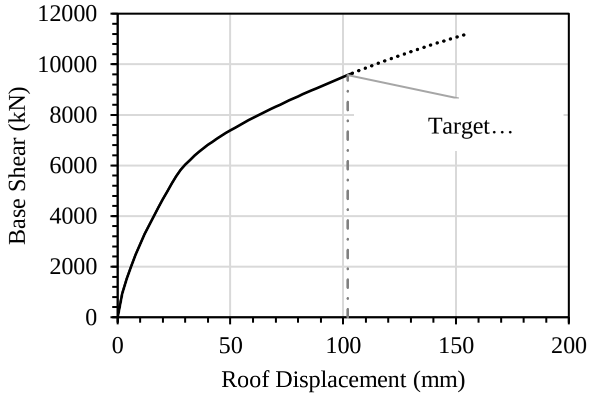

Figure 23: Pushover curve in transverse direction (Load profile-1).

EC8-3 recommends that the capacities of the force-controlled mechanisms be based on the mean properties divided by the partial factor and the confidence factor. Since, in the NSP application in FEMA P-2006, the knowledge factor is considered as 1, for comparison here also, it is assumed that the confidence factor is 1 for carrying out the assessment using EC8-3. For deformation-controlled mechanisms, the deformation capacities of the walls are calculated based on EC8-3, and the demand is based on the analysis results.

Table 5 summarizes the target displacements, base shear demand, and DCR ratio for load profile-1 for 'Near Collapse' and 'Significant Damage' performance levels, respectively. All walls satisfy the acceptance criteria for the limit state of significant damage. However, for the limit state of near collapse, the DCRs of Wall 1 and Wall 4 are slightly above 1.

Table 5: EC8-3 - 'Near collapse' Performance level (2% in 50 years) - Load profile-1.

Performance Level	Direction	Target Displacement, mm (Drift)	Base Shear Demand, kN	Component	Action	DCR
Near collapse (APoE – 1/2475)	Longitudinal	38.6 (0.30%)	10941	Wall 1/4	Shear	**1.05**
					Flexure	Elastic
	Transverse	102.0 (0.80%)	9571	Wall A	Shear	0.36
					Flexure	0.36
				Wall D	Shear	0.66
					Flexure	Elastic
				Wall G	Shear	**0.88**
					Flexure	0.44
Significant damage (APoE – 1/475)	Longitudinal	14.5 (0.12%)	7942	Wall 1/4	Shear	**0.81**
					Flexure	Elastic
	Transverse	52.0 (0.40%)	7490	Wall A	Shear	0.28
					Flexure	0.25
				Wall D	Shear	0.56
					Flexure	Elastic
				Wall G	Shear	**0.71**
					Flexure	0.31

For comparison with the EC8-3 results, Table 6a shows DCRs for different walls based on non-linear static analyses for NZ guidelines. For NZ guidelines, since the philosophy differs from the other two standards, the following is done to show the results in the form of DCR. Since the walls in the longitudinal direction are shear-controlled with no inelastic deformation capacity, the DCR is the ratio of the shear capacity to the elastic shear demand at ULS, which is taken simply as the inverse of %*NBS* score as a ratio. For the transverse direction, since %*NBS* score is greater than 100%, for calculating DCR, the pushover response is limited to a "target" displacement such that %*NBS* score is approximately equal to 100%, as shown in Figures 24a and 24b. The DCR values in shear are the shear force DCRs, and the DCR values in flexure are the deformation DCRs.

Table -6a: DCRs based on NLTH analysis per NZ guidelines - Load profile-1.

Direction	Component	Action	NZ Guidelines NLSPA (ULS APoE – 1/500)
Longitudinal	Wall 1/4	Shear	**1.04**
		Flexure	Elastic
Transverse	Wall A	Shear	0.29
		Flexure	0.10
	Wall D	Shear	0.63
		Flexure	Elastic
	Wall G	Shear	0.70
		Flexure	0.46

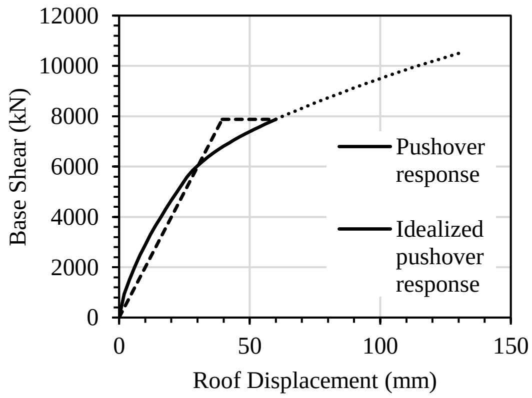

Figure 24(a): Truncated pushover response such that the capacity is almost equal to the demand - Load profile-1.

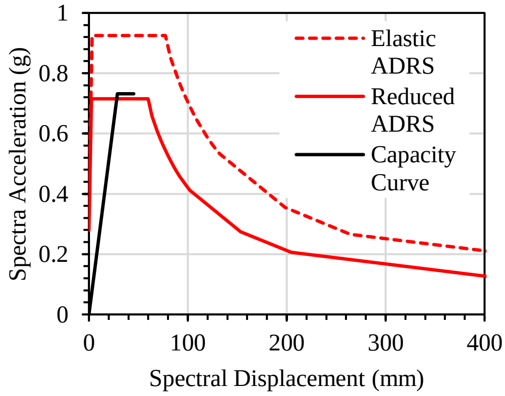

Figure 24(b): Capacity curves and demand spectra such capacity is almost equal to demand - Load profile-1 (Transverse direction).

DCRs for assessment using NSP per ASCE 41 for the Collapse Prevention performance criterion are reproduced from FEMA P-2006 in Table 6b. Note that per FEMA P-2006, walls 1 and 4 are force-controlled. Their DCR values reported are force DCRs. In the transverse direction, walls A, D, and G are assessed to be deformation-controlled. DCR values for these walls in shear are reported as shear deformation DCRs, while those in flexure are plastic deformation DCRs.

Table -6b: DCRs based on NSP analysis per ASCE 41 (from FEMA P-2006).

Direction	Component	Action	ASCE 41 NSP (Collapse Prevention APoE – 1/975)
Longitudinal	Wall 1/4	Shear	**0.93**
		Flexure	0.46
Transverse	Wall A	Shear	0.04
		Flexure	0.06
	Wall D	Shear	0.15
		Flexure	Elastic
	Wall G	Shear	0.08
		Flexure	0.06

Table 7 summarizes the assessment outcomes based on the three non-linear static methods of analysis. For the ease of comparison with %*NBS*, instead of DCR, CDR values (inverse of DCR) are provided. It is observed that for both performance objectives in the case of EC8-3 and ASCE 41, the lateral systems in both directions meet the acceptance criteria, except for the longitudinal direction for the limit state of near collapse, per EC8-3. However, the CDR value is close to 1. Per NZ guidelines as well, in the longitudinal direction, %*NBS* score is slightly below 100%. However, it is greater than 100% for the lateral system in the transverse direction. Governing mechanism in the longitudinal direction is the same for all three guidelines. However, in the transverse direction, the governing mechanism is different. While it is the limiting rotation capacity of Wall G in the case of NZ guidelines, it is the shear capacity of wall G in the case of EC8-3. In the case of ASCE 41, the governing mechanism identified is shear failure in Wall D at level 2. The reason for this departure seems to be treatment of the support condition of columns supporting Wall D. While the columns are allowed to uplift in this study, it is apparently not the case in FEMA P-2006.

DISCUSSION

In this section, the similarities, and differences in the assessment outcome for the example building using the provisions of the three guidelines are discussed. The governing mechanisms for the lateral systems in both directions and the associated assessment outcomes for both linear and non-linear static procedures are summarized in Table 8. (For the ease of comparison with %*NBS*, instead of DCRs, CDRs (inverse of DCRs) are given in Table 8).

Longitudinal Direction

The assessment done using the *linear static analysis* procedures of the three guidelines identifies the lateral system as deficient in the longitudinal direction, with the governing mechanism being shear in Wall 1 (or 4). The CDR is the lowest for EC8-3 amongst the three guidelines.

Table 7: Summary of the assessment outcome based on the non-linear static methods.

Direction	Component / Outcome	NZ Guidelines NLSPA	EC8-3 NLSPA	ASCE 41 NSP (FEMA P-2006)
Longitudinal Direction	Governing component	Wall 1 / Wall 4 (Shear)	Wall 1 / Wall 4 (Shear) (NC and SD)	Wall 1 / Wall 4 (Shear) (CP and LS)
	Outcome	%*NBS*=96	CDR= 0.95 (NC) CDR = 1.18 (SD)	CDR = 1.08 (CP) CDR = 1.28 (LS)
Transverse Direction	Governing component	Wall G (Deformation capacity)	Wall G (Shear)	Wall D (Shear failure at level 2) (CP and LS)
	Outcome	%*NBS*=153	CDR = 1.09 (NC) CDR = 1.37 (SD)	CDR = 6.67 (CP) CDR = 12.50 (LS)

NC - Near Collapse, SD - Significant Damage, CP – Collapse Prevention, LS – Life Safety

Table 8: Summary of identified governing mechanisms and assessment outcome.

		Linear Static Methods			Non-Linear Static Methods		
	NZ SLaMA	NZ Equivalent Static Analysis	EC8-3 Lateral Force Method	ASCE 41 LSP	NZ NLSPA	EC8-3 NLSPA	ASCE 41 NSP (FEMA P-2006)
Governing mechanism (Longitudinal direction)	Wall 1 / Wall 4 (Shear)	Wall 1 / Wall 4 (Shear)	Wall 1 / Wall 4 (Shear)	Wall 1 / Wall 4 (Shear)	Wall 1 / Wall 4 (Shear)	Wall 1 / Wall 4 (Shear)	Wall 1 / Wall 4 (Shear)
Assessment outcome (Longitudinal direction)	%*NBS* = 85	%*NBS* = 69	Min CDR = 0.37 (NC)	Min CDR = 0.56 (CP)	%*NBS*=96	Min CDR= 0.95 (NC)	Min CDR = 1.08 (CP)
Governing mechanism (Transverse direction)	Wall D (Deformation capacity)	Wall D (yield moment capacity)	Wall D (shear)	Wall D (Shear)	Wall G (Deformation capacity)	Wall G (Shear)	Wall D (Shear failure at level 2)
Assessment outcome (Transverse direction)	%*NBS*= 124	%*NBS* = 97	Min CDR = 0.56 (SD and NC)	Min CDR = 1.56 (CP)	%*NBS*=153	Min CDR = 1.09 (NC)	Min CDR = 6.67 (CP)

NC - Near Collapse, SD - Significant Damage, CP – Collapse Prevention

Recall that the demands for assessment are different following the three guidelines. The demand is highest for the limit state of Near Collapse, per EC8-3, and corresponds to APoE of 1/2475 (2% in 50 years). In contrast, the demand for the Collapse Prevention performance level, per ASCE 41, corresponds to an APoE of 1/975 (5% in 50 years). ULS demand per NZ guidelines corresponds to an APoE of 1/500. Further, the estimated shear capacity of Wall 1 (or Wall 4) using EC8-3 is lower than that estimated using ASCE 41 and NZ guidelines. The above two reasons explain the difference in CDR values. While %*NBS* score is tolerable in the case of NZ guidelines, the walls do not meet the acceptance criteria per ASCE 41 and EC8-3.

Again, based on the non-linear static analyses, all guidelines identify the governing mechanism in the longitudinal direction as the failure of Wall 1 (or Wall 4). Though the results indicate CDR >1 per ASCE 41 and <1 for EC8-3 and NZ guidelines, all three outcomes are relatively close as compared to the results from the linear static analysis.

If EC8-3 and ASCE 41 results are compared, there are two counteracting effects at play. First, the demand on Wall 1 is lower for Collapse Prevention damage, per ASCE 41. This is because of the lower hazard level per ASCE 41, as compared to EC8-3, and also because of the lower target displacement per ASCE 41, which results from the specific way the pushover curve is idealized per ASCE 41. Recall from the "Analysis Methods" section earlier in the paper (and Figures 6 and 7) the difference in the idealization procedure of ASCE 41 as compared to EC8-3, which assumes an elastic-perfectly plastic idealization. Figure 25 compares the idealized pushover response in the longitudinal direction following ASCE 41, and EC8-3 approaches for the demand corresponding to an APoE of

1/2475. The target displacement values are annotated in the figure. Also shown is the idealized linear response based on the NZ guidelines and the displacement corresponding to the performance point. There is a significant difference in the target displacement, and thus, the demand is considerably lower based on the ASCE 41 idealization procedure as compared to EC8-3.

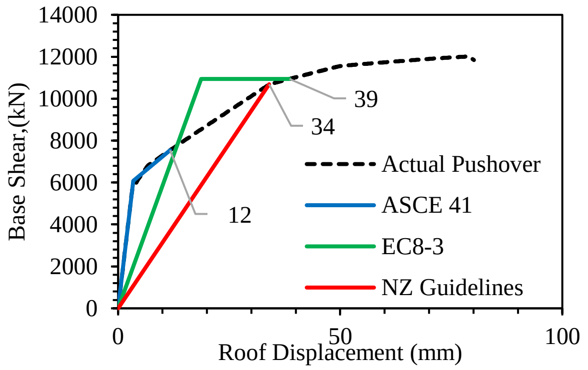

Figure 25: Idealized pushover curves-Longitudinal direction (Load profile-1).

However, there is a counteracting second effect. Note that the effect of lower shear strength based on EC8-3 provisions is not captured here in the non-linear static analysis. The analysis is done considering shear strength per NZ guidelines to eliminate the influence of very low shear capacity calculated using EC8-3 provisions. Because of this consideration, the lower bound capacity of Wall 1 (4403 kN) is higher as compared to 3728 kN (838 kips) per ASCE 41 reported in FEMA P-2006. This difference in capacity counteracts the effect of higher demand for assessment per EC8-3 to some extent.

For the assessment using NZ guidelines, recall that the idealized pushover response is assessed to be linear elastic up to the shear failure of walls 1 and 4. %*NBS* score is assessed to be 96%, which is tolerable.

Transverse Direction

In the transverse direction, governing mechanisms vary for different analysis methods. Based on the linear static analyses in the transverse direction, the governing component is Wall D per all three guidelines. However, while the moment capacity of Wall D governs the outcome in the case of NZ guidelines, the shear capacity of Wall D governs in the case of EC8-3 and ASCE 41. While Wall D meets the acceptance criteria in the case of ASCE 41 (CDR>1), it does not in the case of EC8-3. There are two main reasons for this difference. Firstly, the estimated shear capacity of Wall D using EC8-3 is lower than that estimated using ASCE 41. Secondly, shear in Wall D per EC8-3 is force-controlled and, thus, assessed more conservatively compared to ASCE 41, which treats Wall D as deformation-controlled. Further, the treatment of force-controlled actions is more conservative in EC8-3, as compared to ASCE 41. The confidence factor per EC8-3 is applied to conservatively assess the capacity as well as to enhance the demand.

For the equivalent static analysis, per NZ guidelines, the base shear capacity is conservatively assessed to be limited by the yield moment capacity of Wall D. %*NBS* score is thus slightly below 100%, which is tolerable.

In the linear static analysis discussed above, the starting assumption is that Wall D is fixed at the grade level, disregarding the potential uplift of the column supporting Wall D. However, if the demands on the supporting columns are considered, it would be evident that the tension column would uplift, thus limiting the demand on Wall D and changing the governing mechanism for the building.

For the assessment based on the non-linear static analyses, the reason for the different governing mechanisms between ASCE 41 and the other two guidelines is the different treatment of the support condition of columns at grid D. While the columns supporting Wall D are allowed to uplift in the model employed here for the assessment per NZ guidelines and EC8-3, it does not seem to be the case in FEMA P-2006 study. Again, note that the effect of lower shear strength based on EC8-3 provisions is not included in the non-linear static analysis. However, per EC8-3, shear is force-controlled. Thus, the assessment is based on its lower bound force capacity, which results in the shear in Wall G being the governing mechanism per EC8-3, while the assessment according to the NZ Guidelines is limited by the deformation capacity of Wall G. Further, the difference in the idealization of the pushover response based on the three guidelines also leads to a difference in the demand levels. The CDRs indicate that the lateral system in the transverse direction meets the acceptance criteria based on non-linear analysis methods per ASCE 41 and EC8-3. %*NBS* score is also greater than 100%.

Overall, the lateral system in the longitudinal direction, i.e., Walls 1 and 4, governs the assessment outcome. It is tolerable per NZ guidelines, acceptable per ASCE 41, but slightly falls short of acceptance for the limit state of near collapse per EC8-3.

CONCLUSIONS

FEMA P-2006 provides the seismic assessment of an example RC wall building using ASCE 41. The current study extends this example building assessment by also applying the NZ seismic assessment guidelines and EC8-3 to the same building. The linear static and the non-linear static analysis methods of the three guidelines are compared.

Key similarities and differences identified from completing the assessments using the three guidelines are summarized below.

- The NZ guidelines focus solely on life safety, comparing the capacity of the building with a similar new building to assign %*NBS* scores. In contrast, ASCE 41 and EC8–3 allow for the assessment in relation to different performance objectives at different corresponding demand levels.
- For linear static analysis, different methods of stiffness calculations (i.e., effective stiffnesses (Table 10.5) used in ASCE 41, stiffness multipliers based on NZS 3101.2:2006, versus secant-to-yield stiffness in the case of NZ guidelines and EC8-3) may lead to a different load distribution between the lateral systems, and hence, the identification of different components as critical. This may subsequently result in different retrofitting solutions.
- Per the NZ guidelines, there is no explicit consideration of components (or actions) as force-controlled (or brittle) and deformation-controlled (or ductile). Rather, it is implicit in the calculation of building capacity (and ductility) by either carrying out SLaMA or non-linear analysis with the expected component inelastic capacity.
- In the case of ASCE 41 and EC8-3, the comparison for force-controlled action is based on the lower bound strength (and reduced based on the knowledge or confidence factor). Further, EC8-3 applies the confidence factor to increase the demand as well as to reduce the capacity of the force-controlled actions, thus adding additional conservatism. There is no consideration of the knowledge (or confidence) factor per NZ guidelines.

- The different guidelines use different shear capacity models for walls. EC8-3 gives the lowest shear capacity of the three guidelines. Further, shear in walls is considered a brittle (or force-controlled) mechanism per EC8-3. ASCE 41 allows wall shear to be a deformation-controlled action if the wall has a low gravity load and transverse reinforcement ratio of more than 0.15%. NZ guidelines also allow some post-yield deformation capacity for shear-governed squat walls based on the axial load and the transverse reinforcement ratio.
- Different assumptions concerning the idealization of pushover curves and the calculation of target displacements can lead to different outcomes. ASCE 41 idealization approach follows the pushover response more closely than the elastic-perfectly plastic idealization used by EC8-3. This leads to a higher elastic stiffness for ASCE 41, thus in lower target displacement than EC8-3.

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NOTATIONS

$\%NBS$	Percentage of new building standard as calculated by application of NZ guidelines
C_0	Factor to relate the displacement of an equivalent SDOF with the roof displacement of the MDOF system (ASCE 41)
C_1	Modification factor to relate inelastic displacements to displacements obtained from the elastic response (ASCE 41)
C_2	Modification factor to represent the effect of pinched hysteresis shape, cyclic degradation, and strength deterioration on maximum displacement response (ASCE 41)
C_m	Effective mass factor to account for higher modal participation effects (ASCE 41)
CF	Confidence factor determined based on the extent of information (EC8-3)
DCR	Demand to Capacity Ratio
$d*$	Pushover displacement of an idealized SDOF system (EC8-3)
d_m^*	Displacement corresponding to the ultimate strength of an idealized SDOF system (EC8-3)
d_y^*	Yield displacement of an idealized SDOF system (EC8-3)
E	Modulus of elasticity
$F*$	Base shear of an idealized SDOF system (EC8-3)
F_y^*	The ultimate strength of an idealized SDOF system (EC8-3)
g	Acceleration due to gravity
I_{eff}	Effective moment of inertia
I_{gross}	Moment of inertia of gross section
k	factor to modify the target displacement for short period non-linear system (greater than 1). It is 1 otherwise (EC8-3)
K_e	Effective lateral stiffness of the building (ASCE 41)
K_i	Elastic lateral stiffness of the building (ASCE 41)
K_μ	Factor to reduce demand based on available ductility (NZ guidelines)
L_v	Shear span (EC8-3)
m	Component capacity modification factor for ductile actions (ASCE 41)
M_y	Yield moment (EC8-3)
Q_{CE}	Expected capacity (ASCE 41)
Q_{CL}	Lower bound capacity (ASCE 41)
Q_G	Demand due to gravity loading (ASCE 41)
Q_{UD}	Demand for ductile (deformation controlled) actions (EC8-3)
Q_{UF}	Demand for brittle (force-controlled) actions (EC8-3)
S_a	Response spectrum acceleration at the fundamental period of the building in the direction considered
$S_e(T^*)$	Elastic spectral acceleration ordinate at T* (EC8-3)
S_p	Structural performance factor (scaling factor for ductile systems) (NZ guidelines)
$T*$	Period of idealized Single Degree of Freedom (SDOF) system (EC8-3)
T_e	Effective period calculated from idealized pushover curve (ASCE 41)
T_i	Initial period of the building (ASCE 41)
V	Base shear demand
V_d	Maximum Base shear in pushover response (ASCE 41)
V_y	Yield strength of the building in the direction under consideration (ASCE 41)
W	Total seismic weight of the building
Γ	Transformation factor to convert the pushover response of the building to the pushover response of an equivalent SDOF system (EC8-3)
γ	Load factor depending upon the criticality of action (ASCE 41)
Δ_{cap}	Displacement capacity of deformation-controlled actions (EC8-3)
Δ_{CE}	Displacement capacity of deformation-controlled actions (ASCE 41)
Δ_d	Displacement at the maximum base shear in pushover response (ASCE 41)
Δ_{prob}	Assessed displacement capacity of an idealized SDOF system (NZ guidelines)
Δ_{UD}	Displacement demand determined from the analysis (EC8-3)
δ_t	Target displacement for pushover analysis
$\delta_{t,SDOF}$	Target displacement of elastic Single Degree of Freedom (SDOF) system (EC8-3)
θ_y	Estimated Yield Rotation (EC8-3)
κ	Knowledge factor determined based on the extent of information (ASCE 41)

λ Correction factor to relate the first mode mass to total seismic mass (EC8-3)

λ_f Partial safety factor on material properties (EC8-3)

μ_{max} Maximum Threshold value of Global ductility factor for the Non-Linear Static Procedure to be applicable (ASCE 41)

$\mu_{strength}$ Global ductility factor (ASCE 41)

χ A factor for adjusting action caused by the response for the selected performance level (ranges from 1 to 1.3) (ASCE 41)