TSUNAMI DESIGN USING NONLINEAR PUSH-OVER ANALYSIS

M. Baiguera[(1)], T. Rossetto[(2)], I.N. Robertson[(3)]

[(1)] *Research Fellow, EPICentre, Civil Environmental and Geomatic Engineering Department, UCL, UK, m.baiguera@ucl.ac.uk*
[(2)] *Professor, EPICentre, Civil Environmental and Geomatic Engineering Department, UCL, UK, t.rossetto@ucl.ac.uk*
[(3)] *Professor, Civil and Environmental Engineering Department, University of Hawaii at Manoa, HI, USA, ianrob@hawaii.edu*

Abstract

New tsunami design provisions in ASCE 7-16 Chapter 6 offer a comprehensive and practical methodology for the design of structures for tsunami loads and effects. While they provide prescriptive tsunami loading and design requirements, they also permit the use of performance-based analysis tools. However, the specifics of load application protocol, and system and component evaluation are not provided. The authors have recently proposed a nonlinear static pushover procedure for the design of structures for tsunami within the framework of ASCE 7-16 provisions. Through this approach, the user can both estimate the effective systemic lateral-resisting capacity of a building and the local component demand. This enables the identification of structural elements that may need to be strengthened to meet the ASCE 7-16 standard acceptance criteria. To demonstrate the design procedure, a prototypical reinforced concrete multi-storey building exposed to high tsunami hazard in the US Northwest Pacific coast is assessed. This is a building with sufficient height to provide last-resort refuge for people having insufficient time to evacuate outside the inundation zone. It is shown that both the prescriptive and pushover-based approaches highlight the need to strengthen the lateral-load resisting system of the building in order for it to satisfy the acceptance criteria of ASCE 7-16. The results of the nonlinear static pushover analyses show that the structural system has sufficient lateral strength to resist ASCE 7-16 prescribed tsunami loads. However, when component-based loading is considered, the exterior ground storey columns are observed to fail in shear, precipitating structural failure. This is in agreement with the ASCE 7-16 simplified systemic acceptance criteria, i.e. that the structure is unsafe for use as a refuge, and that it would require significant strengthening. However, the use of the tsunami nonlinear static analysis procedure allows a more targeted strengthening of the building, likely to result in significantly reduced costs.

Keywords: Tsunami Engineering, ASCE 7 Standard, Performance-Based Design, Tsunami Pushover Analysis

1. Introduction

The 2016 edition of the ASCE 7 Standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures [1], includes a new Chapter 6, Tsunami Loads and Effects, which contains comprehensive design provisions for tsunami resilience for use in the States of Alaska, Washington, Oregon, California, and Hawaii. This design standard has been included in the requirements of the 2018 International Building Code (IBC), and an extensive guide of the provisions with example applications is now available in Robertson [2].

The ASCE 7-16 tsunami design provisions apply to critical and essential facilities, tsunami vertical evacuation structures and other multi-storey buildings within the mapped Tsunami Design Zone (TDZ). This is the area vulnerable to being inundated by the Maximum Considered Tsunami (MCT), defined as having 2% probability of being exceeded in a 50-year period. ASCE 7-16 defines load combinations, importance factors and acceptance criteria for both structural system and component design. Fig.1 outlines the main steps for the evaluation of buildings. The acceptance criteria for the overall lateral-force-resisting system (LFRS) is based on a direct comparison between the tsunami base shear and the effect of horizontal earthquake forces. For the evaluation of an individual structural component (SC), the simplest approach is to ensure that the design strength is larger than the internal forces obtained using a linearly elastic, static analysis of the structure subjected to the prescribed tsunami loading cases. As an alternative, the standard allows for the use of performance-based criteria. This includes the adaptation of nonlinear static pushover analysis of ASCE 41-13 [3] to tsunami loading. However, no detailed guidance is provided as to how these performance-based methods should be performed.

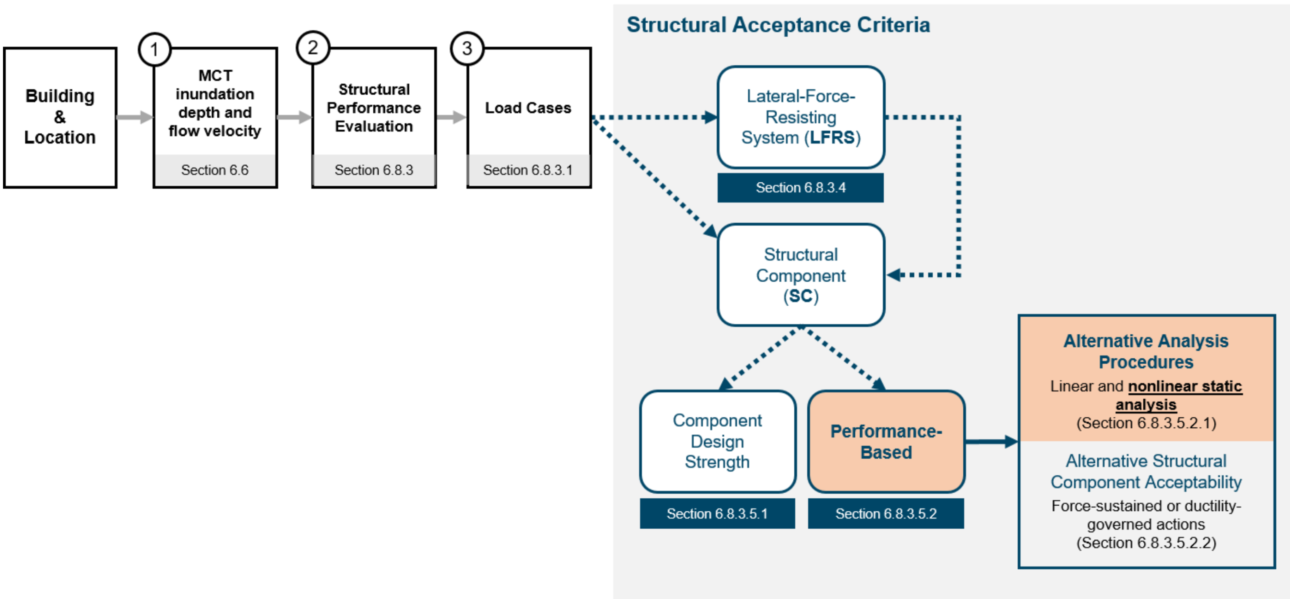

Fig. 1 – Tsunami nonlinear static analysis within the framework of ASCE 7-16 tsunami design provisions

Performance-based engineering methods for tsunami are much less developed than for other natural hazards, such as earthquakes. This is due to the complexity of understanding the impact of onshore tsunami inundation with coastal constructions and the challenges in developing inundation models that can simulate realistic tsunami loads and resulting effects [4]. Advances in physical modelling of tsunami and new field observations from recent events have led to the definition of new relationships for estimating structural loadings. These have been instrumental in establishing new analysis approaches that apply realistic tsunami loads to the structure and account for the material nonlinearity of structural elements. Macabuag et al. [5]

performed nonlinear static pushover analysis to assess the structural response of a reinforced concrete frame. Hydrodynamic forces were applied assuming a constant tsunami inundation depth and a lateral distribution of loads applied at each storey along the seaward columns. This approach, herein referred to as constant depth pushover (CDPO), is similar to a seismic pushover analysis, since the lateral tsunami force distribution is increased monotonically. Hence, this approach can be easily implemented in most structural analysis software. CDPO analyses have been employed to perform tsunami fragility studies [6,7]. Petrone et al. [8] developed novel analysis approaches, namely time-history dynamic analysis and a load-control variable depth pushover. The tsunami time-history procedure follows the same principles of a seismic time-history analysis (VDPO), apart from the input data, which is the tsunami force estimated from a simulated inundation time-history. In a VDPO, the tsunami inundation depth at the site of the structure monotonically increases, while the flow velocity is calculated assuming a constant Froude number. These procedures were also used in sequence with the corresponding seismic ones, to assess structures under sequential earthquake and tsunami [9,10]. For all methods, the estimation of the tsunami hydrodynamic force was based on experimentally-validated equations by Qi et al. [11], which account for the regime conditions of the flow impacting the structure and the density of the urban environment. In Petrone et al. [8], comparison of the results of time-history, CDPO and VDPO analyses highlight that, in terms of engineering demand parameters, (i.e. inter-storey drifts and column shear forces), the VDPO is in good agreement with the dynamic analysis, and consistently more accurate than CDPO. However, being a load-control analysis, the VDPO is not capable of capturing any post-peak branch in the pushover curve. This issue is overcome by the approach proposed in Baiguera et al. [12,13]: once the peak strength is reached, the analysis continues switching to response-control, where the top displacement is incremented; the corresponding tsunami force is calculated assuming the same inundation depth (and load pattern) as in the last step of the load-control phase of the analysis. This two-phase analysis approach is herein referred to as VDPO2.

The objective of this paper is to present a methodology whereby the VDPO2 can be applied for tsunami design of buildings located in the TDZ, following the ASCE 7-16 provisions. For this purpose, a prototypical RC frame selected from the design examples in [2] is used as a case study. The advantages of using a pushover analysis approach for design compared to the prescriptive acceptance criteria of the ASCE 7-16 provisions are discussed.

2. Tsunami hydrodynamic forces on buildings

The ASCE 7-16 Chapter 6 provides a practical methodology to calculate the overall tsunami load on a structure (F_T), which is estimated using the following hydrodynamic drag equation [1]:

$$F_T = 0.5\rho_s I_{tsu} C_d C_{cx} B(hu^2) \quad (1)$$

where ρ_s is the fluid mass density, I_{tsu} is the importance factor for tsunami forces, B is the building width, h is the inundation depth, u is the flow velocity, C_d is the drag coefficient based on the ratio B/h [Table 6.10-1 in ASCE 7-16], and C_{cx} is the proportional closure coefficient (with a minimum value of 0.7). The tsunami depth and flow velocity (h and u) vary according to time-history curves that are normalised to the maximum values at the building site. The maximum inundation depth h_{max} and flow velocity u_{max} are determined by applying the Energy Grade Line analysis [14]. Fig.2 shows the tsunami time-history curves for the case-study building presented later in this paper. It can be seen that the maximum lateral hydrodynamic force on the structure occurs when the velocity reaches its peak in each direction and the inundation depth is 2/3 of h_{max}. This is the most critical stage, indicated as Load Case 2 (LC2) in the provisions.

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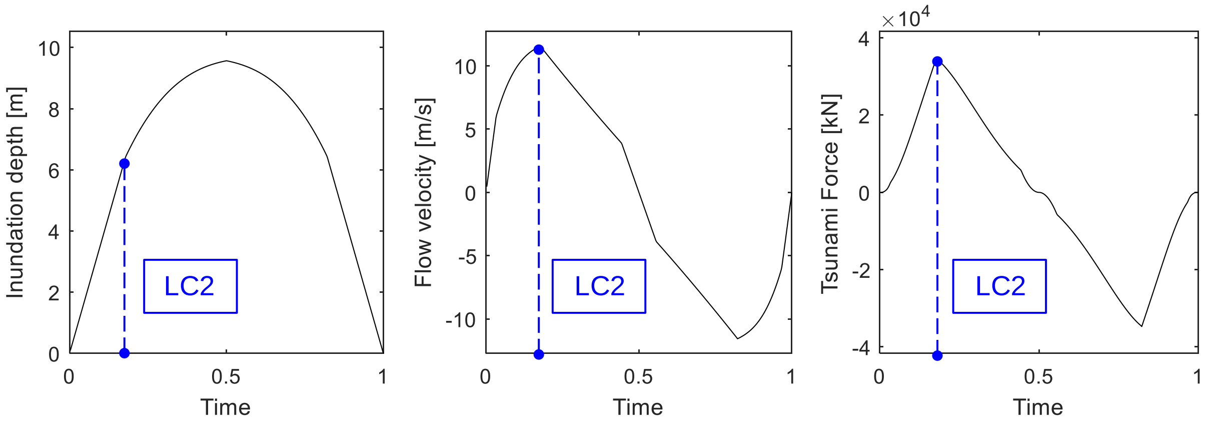

Fig. 2 – Tsunami inundation depth, flow velocity and force time-history curves at the building site in Seaside

3. Nonlinear pushover analysis procedures

The design methodology of the ASCE 7-16 provisions, illustrated in Fig.1, allows for the use of performance-based criteria to check the design of structural components, as an alternative to linear elastic analysis. The methodology proposed in this paper is based on the use of nonlinear static pushover analysis for tsunami design of buildings located in the TDZ, following the ASCE 7-16 and ASCE 41 provisions.

3.1 Analysis Procedures

The VDPO2 consists of a two-phase analysis procedure. In Phase 1, a load-control pushover analysis is conducted assuming the inundation depth and flow velocity increase at each time step in accordance with the ASCE 7-16 inundation depth and flow velocity time histories up to LC2, as shown in Fig.3. In Phase 2, the analysis switches to response-control pushover analysis, where the displacement is incremented, and the corresponding tsunami force is calculated assuming the same inundation depth as in the last step of Phase 1 of the analysis. The switch from Phase 1 to 2 occurs either when Phase 1 is completed (i.e. reaching LC2) or the analysis encounters a numerical convergence issue, whichever occurs first. In the latter case, the Phase 1 analysis is repeated up to the time step preceding the numerical convergence issue, and then Phase 2 is initiated. Throughout Phase 1 and 2 of the analysis, the tsunami hydrodynamic force on the structure is estimated according to Eq.1, which accounts for a varying C_d dependent on B/h.

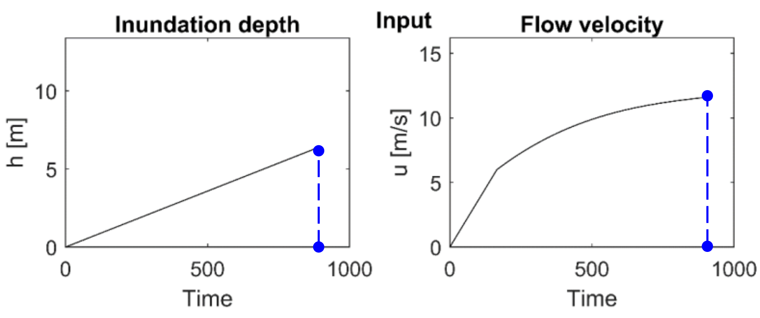

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Fig. 3 – Inundation depth and flow velocity time histories for Phase 1

3.2 Load distributions

ASCE 7-16 provisions assume that the hydrodynamic load distribution on the building is uniform. Different methods can be applied for distributing the hydrodynamic load over the height of the building. A typical approach used in past studies is to apply the loads at each storey level referred to hereafter as the "S" approach. The tsunami forces are calculated using a simple influence area approach, as illustrated in Fig.4 (left). This approach is in agreement with the ASCE 7-16 provisions. However, as illustrated in Fig.1, the ASCE 7-16 design methodology requires that every structural element be evaluated for the combined effects of systemic loads and component loads. Hence, a bespoke loading distribution is proposed in this study, referred to hereafter as the “C” approach. In the proposed C approach a portion of the total base shear is applied directly to the columns on the front of the building, with 5 load application points used along each column height (see Fig.4 right). This “distributed” load discretisation is the one recommended in [8], which they show to provide the best estimation of engineering demand parameters. To evaluate the impact of systemic and component loads in one single analysis, an increased drag component force (e.g., C_d = 2 for square columns and debris damming) is applied on a single exterior column, whilst redistributing the overall tsunami force to the remaining columns on the front of the entire building. For the selected exterior column, the component loads include hydrodynamic drag with debris damming effects and debris impact loading, as per the ASCE 7-16 provisions.

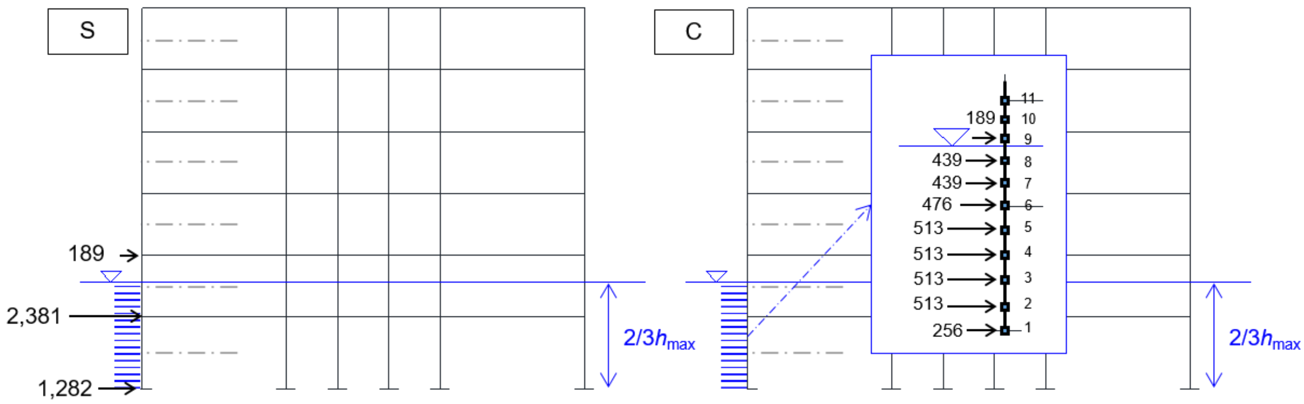

Fig. 4 – Loading discretisation methods S (Storey) (left) and C (Component) (right)

4. Case-study building

4.1 Prototype building

A six-storey office building is considered as a case-study (Fig.5). The building is located in Seaside, Oregon. It has been established that this coastal region is at high risk of being hit by a destructive tsunami following a large earthquake generated along the Cascadia subduction zone [15-18]. Fig.6 illustrates the building location within the 2,500-year probabilistic tsunami design zone map of Seaside. Based on the EGLA conducted in [19], the corresponding h_{max} and u_{max} at the building site are 9.57 m and 11.56 m/s, respectively.

The structure is classified as Tsunami Risk Category (TRC) II, and therefore it is not subject to tsunami provisions. However, the ASCE 7-16 encourages local jurisdictions to require tsunami design for tall TRC II buildings, to provide effective secondary alternative refuge. Chock et al. [18] established suitable building height thresholds for communities throughout the US Pacific coast, satisfying both the prescriptive acceptance criteria and a recommended height at least 3.66 m greater than the inundation depth. It can be seen that the upper three stories of the building are above h_{max}, i.e., they could function as a refuge according to the proposal of Chock et al. [18].

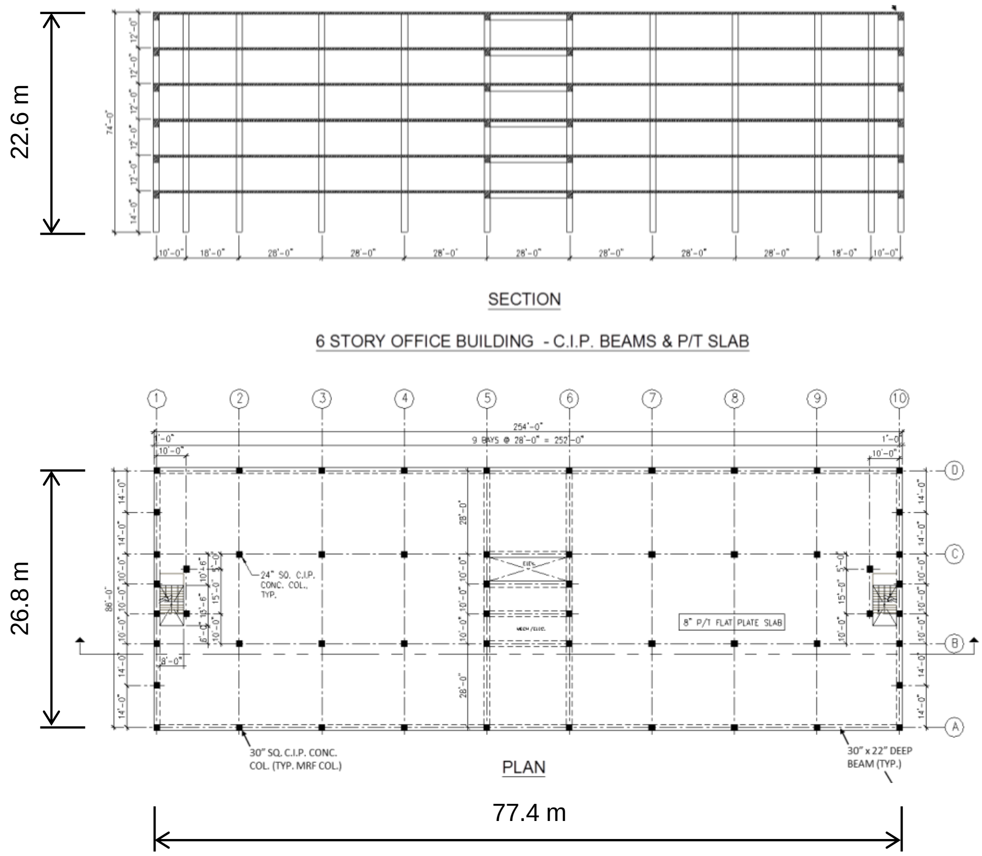

Fig. 5 – Prototype building structural geometry [19]

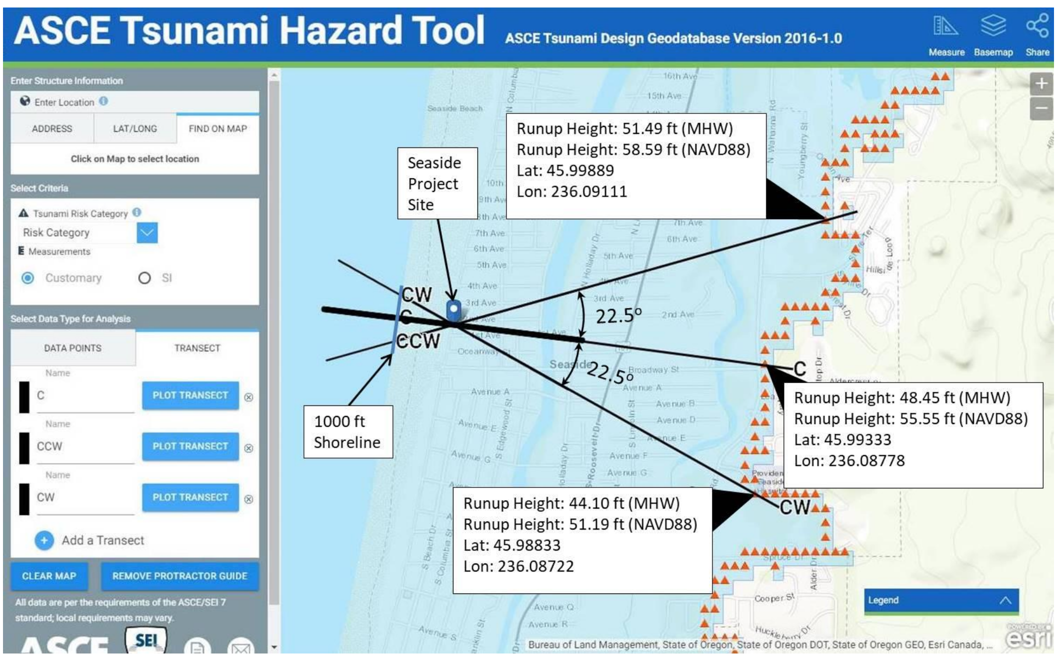

Fig. 6 – Transects for Energy Grade Line Analysis at building site in Seaside shown using the ASCE 7-16 Tsunami Geodatabase [20]

The case study building consists of RC special moment resisting frames (SMRF), a flat plate post-tensioned concrete floor system, and interior gravity load columns, as shown in Fig.5. It is designed for the ASCE 7 wind and seismic loads specified for Monterey, California [2]. The building design is appropriate for Seaside, which has similar seismic hazard to Monterey. Soil classification D for stiff soil is assumed for the building site. The lateral force resisting system consists of four SMRFs in the narrow direction (also the assumed tsunami flow direction) and two moment resisting frames in the wide direction (Fig.5). The size of the columns is uniform along the height of the building, i.e. 71.1x71.1 cm for the SMRFs, and 61x61 cm for the internal gravity load columns, while the size of beams is 76.2 cm wide by 61 cm deep. The concrete cover is 5 cm. In the SMRF columns, steel reinforcing ratio varies from 1.3% at the ground floor to 1% at the upper stories. Transverse reinforcement in the SMRF columns consists of ties with three 9.5-mm-diameter legs at every 10 cm in the column ends (71 cm long) and every 15 cm in the central column section. More details about the seismic design of the building can be found in [21]. Complete tsunami design examples for this building and others are provided in [2,19].

4.2 Finite element model

The case-study building is modelled in OpenSees [22] as a two-dimensional model replicating one half of the full structure. Fig.7 illustrates a sketch of the model that includes: one end moment resisting frame (with 8 columns), one interior moment resisting frame (with 6 columns), 6 exterior columns that form part of the transverse exterior moment resisting frames, and 6 internal gravity columns. All these components are linked by means of master-slave node control so as to simulate a rigid diaphragm at each floor level.

Beams and columns are modelled using force-based nonlinear beam-column elements. A distributed plasticity model is adopted, since the inelastic behaviour due to tsunami pressure can form at any point along the column height. A fibre approach is used for the cross-sections with five integration points along each element.

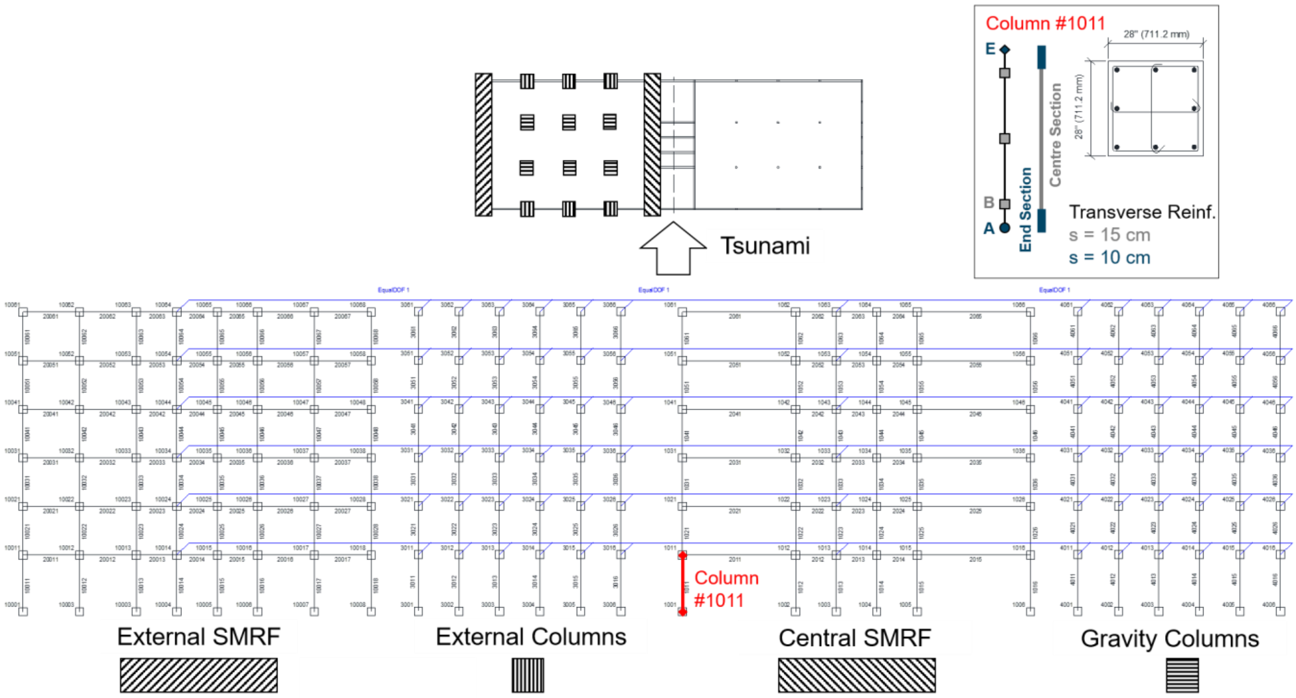

Fig. 7 – View of the two-dimensional finite element model (half of full prototype building)

In accordance with ASCE 41-13 Table 10-1, realistic in-situ nominal values are used for the concrete compressive strength (41.4 MPa) and reinforcing steel yield and tensile strengths (517 MPa and 776 MPa).

The constitutive material *Concrete04* in OpenSees, based on Uniaxial Popovics material [23] with an unloading and reloading stiffness model according to Karsan-Jirsa [24] and exponential decay for the strength, is employed to model confined and unconfined concrete. It is noted that *Concrete04* model simulates stiffness degradation. Concrete within the reinforcement cage is associated with a confined concrete constitutive law, while the cover concrete outside the reinforcement cage is modelled as unconfined. The steel stress-strain constitutive material is modelled using the Giuffre-Menegotto-Pinto model, named *Steel02* in OpenSees.

Previous studies [7-10] have shown that a typical collapse mechanism under tsunami loading is the occurrence of shear failure of columns. This often precipitates global failure if no strengthening measures are adopted. In this study, shear failure occurrence is tracked in all first-storey columns (i.e. those subjected to the highest shear demand), according to the formulation used in ASCE 41-13. It is noted that, both the end and central column sections are checked due to differences in their shear reinforcement (Fig.7). The OpenSees model does not evaluate shear failure, hence the shear check is performed as post-analysis.

5. Results

5.1 Prescriptive systemic acceptance criterion

ASCE 7-16 provides a simple criterion to evaluate the systemic tsunami design of a seismically-designed structure. This assumes that a building designed to resist high seismic loading (i.e. Seismic Design Criteria D, E or F), has sufficient inherent strength to resist the tsunami force [18]. Effectively, this implies that structural lateral force resisting system does not require additional lateral strength when:

$$F_T < 0.75\Omega_0 E_h \tag{2}$$

where Ω_0 is the system seismic overstrength factor, and E_h is the effect of horizontal earthquake forces. From the design of the prototypical building (E_h = 10,041 kN, Ω_0 = 3 for special MRFs, based on ASCE 7 Table 12.2-1), $\Omega_0 E_h$ = 30,123 kN. The applied tsunami force F_T = 34,692 kN at LC2 as per Eq.1 (see Fig.2) exceeds the limit of F_T = 0.75x30,123 = 22,592 kN from Eq.2. This indicates that the seismic lateral force resisting system would need to be strengthened to resist an earthquake force of $E_h = F_T/(0.75\Omega_0)$ = 15,418 kN, which corresponds to a 50% increase in seismic design base shear.

5.2 Nonlinear pushover analysis

The lateral capacity of the structure to resist tsunami loads is evaluated using the VDPO2. To draw a consistent comparison between the actual lateral tsunami capacity with the corresponding seismic one, a seismic pushover analysis is also performed. The seismic pushover is conducted using a lateral load distribution corresponding to the first mode response (fundamental period = 0.8 s; first mode characterised by 83% mass participation factor).

Fig.8 shows the total base shear-top drift curves from the seismic pushover analysis with the one from the VDPO with discretisation method S and C. The actual seismic lateral capacity (8419*2=16,839 kN) is significantly larger than the design one (5020*2=10,040 kN). However, it is substantially less than that predicted by the use of an overstrength factor Ω_0 = 3 (30,123 kN). The tsunami pushover curves show that, for both S and C loading conditions, the systemic tsunami capacity of the building is significantly larger than the overall tsunami at LC2 (F_T/2= 34,954/2 kN, shown as a thick dashed line in Fig.8). This assessment contradicts the results of the ASCE 7-16 systemic tsunami capacity acceptance criterion, and would indicate that the structure is safe for use as a refuge without additional strengthening.

However, ASCE 7-16 also requires that every structural element be evaluated for component loads. This assessment was done iteratively for each seaward column using loading discretisation C; the worst load combination is presented here (Fig.9). It can be seen that column shear failure occurs in all seaward columns.

As expected, the external SMRF column with the increased component loading is the first column to fail in shear (preceding the shear failure of the other seaward columns), indicating component failure results in a premature failure of the structure. Interestingly, the column shear failure occurs in their central sections (i.e., where the transverse reinforcement has a wider spacing) and not in the end sections where seismic design requirements for ductile response lead to increased transverse reinforcement.

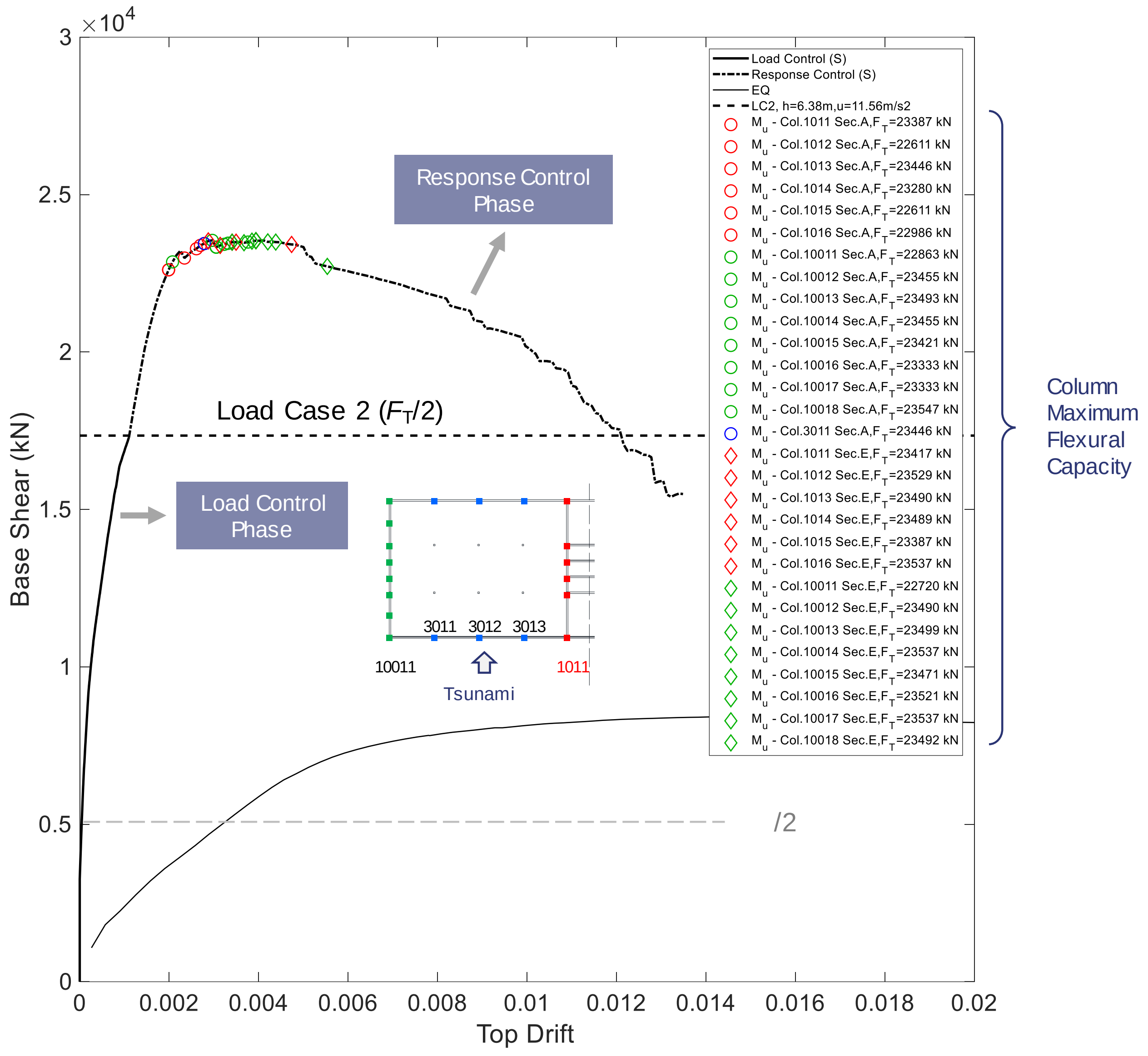

Fig. 8 – Base-shear vs. top drift curves from seismic PO and VDPO with load discretisation method S (Storey), with sequence of maximum flexural capacity and shear failure attainment (for ground floor columns)

If shear failure of the first column is assumed as the structural failure criterion, the resulting tsunami capacity of the building is almost a third of the design tsunami load. This analysis results in the same conclusions as the simplified ASCE 7-16 systemic tsunami capacity acceptance criterion, i.e. that the structure is not safe for use as a refuge without additional strengthening. However, the use of the VDPO provides information on what needs to be strengthened in order to improve the tsunami performance of the structure, i.e. the shear strength of the ground floor seaward columns, in this example case. For instance, by increasing the shear reinforcement in the central sections (spacing from 15 cm to 10 cm; see Fig.7) of all seaward columns, the columns would still fail in shear but the structure tsunami capacity would be double. The prescriptive ASCE 7-16 method would instead require the flexural and shear strengthening of all exterior columns at the first and second floors of this building [19].

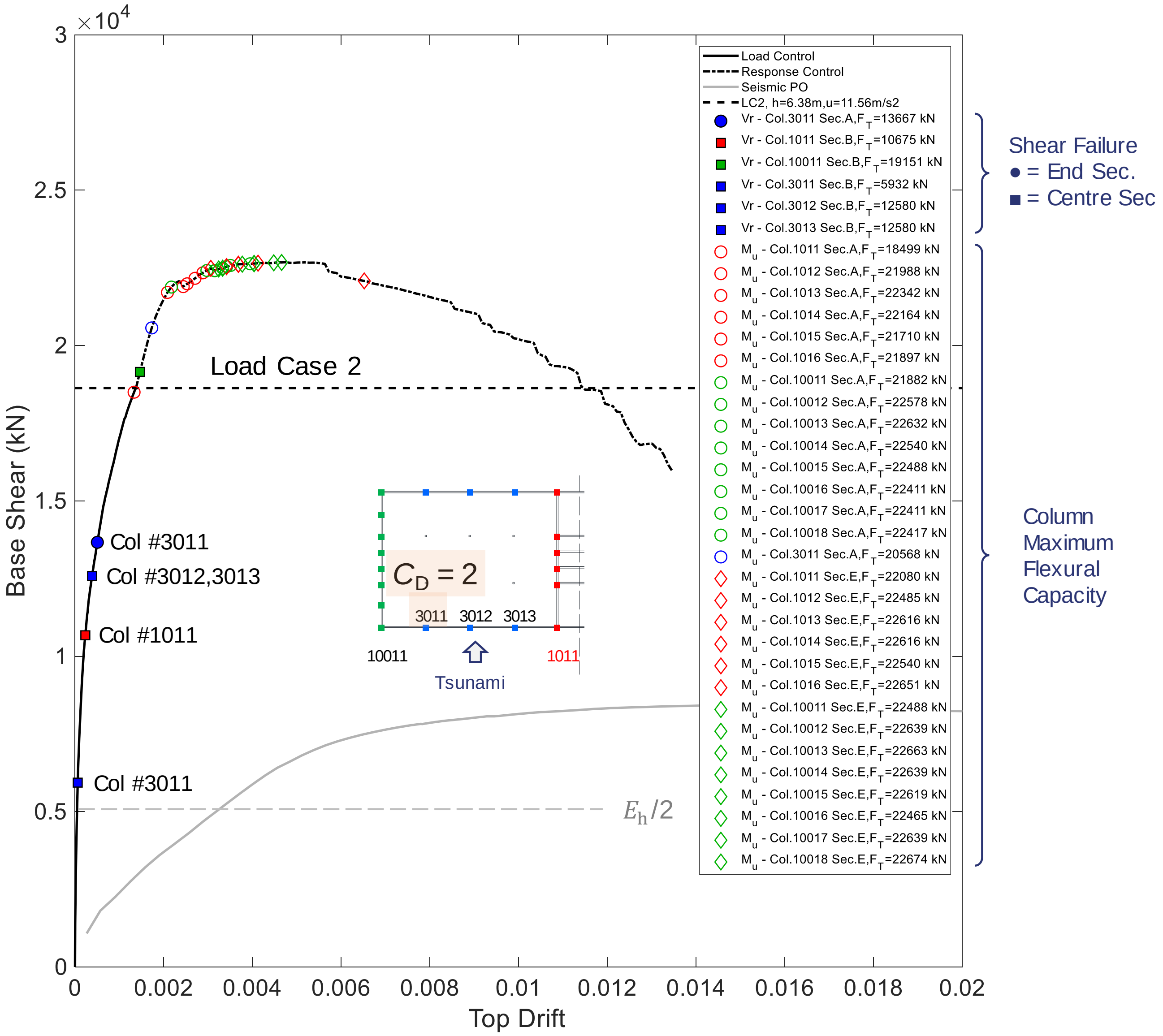

Fig. 9 – Base-shear vs. top drift curves from seismic PO and VDPO with load discretisation method C (Component), with sequence of maximum flexural capacity and shear failure attainment (for ground floor columns)

6. Conclusions

New tsunami design provisions in ASCE 7-16 Chapter 6 offer a practical methodology for the design of buildings to tsunami. While the use of performance-based analysis methods is permitted as an alternative to linearly elastic static analysis, no specific guidance is provided. This study presents the variable depth pushover analysis 2 phase approach (VDPO2) for the design of structures subjected to tsunami. The analytical results are compared to the simplified ASCE 7-16 systemic tsunami capacity acceptance criterion for a case study RC frame located in a high tsunami hazard area. The response of the structure was investigated applying two different tsunami load discretisation methods. For both load discretisation cases, the tsunami systemic capacity of the structure was seen to be sufficient to resist the ASCE 7-16 prescribed tsunami loads. However, when component loading was considered, the seaward ground storey columns were observed to fail in shear, precipitating structural failure. Because ASCE 7-16 requires that the same hydrodynamic conditions be considered during both incoming and outgoing tsunami flow, all seaward and landward ground storey columns would need to be strengthened in shear. Overall, the VDPO2 analysis provided the same result as the ASCE 7-16 prescriptive systemic acceptance criteria, i.e. that the structure is unsafe for use as a refuge, and that it would require significant strengthening. However, by applying the component loading procedure, the user can identify which structural elements need to be strengthened and what type of strengthening they require to meet the design acceptance criteria (e.g. ground floor columns that need more shear resistance). This approach has the potential for providing a more economical design as compared to the prescriptive ASCE 7-16 approach, which promotes an enhanced seismic design of the structure to meet the systemic acceptance criteria of the standard. The methodology is going to be further tested to check the cost savings that can be achieved through its implementation.

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