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Non-Linear Pushover Analysis of Ordinary Moment Resisting Frame Structures in Australia

Andreas K Gustafson[1]

1. Corresponding Author, Managing Director Optimised Engineering Solutions, 39 Hume Drive, Bundamba QLD Australia. Email: andreas@optimisedengineering.com.au

Abstract

The introduction of AS1170.4 2007 in Australia has outlined specific seismic design and performance criteria for building structures and their structural response. Further, combined with the release of AS3600 2018, specifically section 14 design for earthquake actions, the seismic design and analysis of reinforced concrete structures has become more onerous for ductile response. The increased detailing requirements for ductile sway frame structures in particular results in more costly structures both in terms of section size and reinforcement quantity. This has compounding effects for space and cost viability for commonly built commercial structures in Australia such as retail spaces and carparking structures. From a professional engineering office perspective, the increased detailing associated with a higher ductility class is not cost effective or practical, and the choice of analysis method is based entirely on reducing cost whilst meeting performance criteria. For retail structures that don't contain shear core or wall structures, moment resisting frames (MRF's) are the only lateral bracing against seismic loads. Typically, these MRF's in Australia are constructed with small columns, one way band beams, and one-way slabs which are post-tensioned for deflection control. Any stair or lift cores are often limited in number or isolated in positions across the floor plate that provide little overall structural bracing. Therefore, more sophisticated methods of analysis are necessary to achieve a viable and practical design for these structures in the low seismic risk region of Australia. This paper presents the use of non-linear push over analysis with the Capacity Spectrum Method for one-way band beam and slab sway frame structures built throughout the state of Queensland. The method may be used to demonstrate that standard detailing to the main body of AS3600 achieves a minimum ductility of 2 (limited ductility) without the need for higher ductility class detailing to section 14 to satisfy seismic demand. A mixed-use retail building is presented with transfer beams and is analysed using a 2D and 3D frame pushover analysis.

Keywords: Sway frame structures, Non-linear analysis, Monotonic Pushover Analysis, Concrete Structures, earthquake

1 Introduction

Sway frame structures are a structural system that does not rely on rigid shear walls or core boxes to provide resistance against lateral loads. Typically, the framing system comprises of reinforced concrete columns and beams that are rigidly connected in one or more directions. The use of post tensioned concrete (PT) one-way band beams and slabs is common practice for carpark and retail structures in Australia, especially for major commercial shopping centres. The rigidity of the system relies upon the rotational stiffness of intersecting column and beam/slab joints through the frame and can be either pinned or rotationally rigid at the footing level. The deflection response of such frames is entirely governed by flexural action as opposed to shear raking or shear deformation. It should be noted that very little guidance and research are available for one-way or two-way flat slab frames. Most research describes the behaviour of beam frame systems, which may have some relevance to slab sway frames. Building codes universally classify these framing systems as moment resisting frames (MRF’s). MRF’s can be further classified into different subcategories depending upon detailing, structural ductility, and required seismic capacity. AS3600 has the following categories for MRF’s:

1. Ordinary Moment Resisting Frame (OMRF)
2. Intermediate Moment resisting Frame (IMRF)
3. Special Moment resisting Frame (SMRF)

OMRF’s are considered to have limited ductility and require no special seismic detailing outside of the main body of AS3600 and section 14.4. IMRF’s are considered moderately ductile and require specific detailing to section 14.5 of the standard. SMRF’s are fully ductile and require special detailing. As the need for seismic capacity increases so does the corresponding reinforcement detailing. It is therefore necessary to adopt a more rigorous approach to seismic analysis and design in Australia that is proportionate to the seismic risk profile, which would enable rational member sizing and less onerous reinforcement detailing whilst maintaining the necessary structural integrity against earthquake events. AS1170.4 and its commentary released in 2021 allow for performance-based analysis methods to be adopted in lieu of simplified approaches.

The results of two analyses on a structure with mixed use function, will be presented to demonstrate that OMRF’s are inherently flexible and given that maximum displacement demands in Australia are relatively low, higher levels of ductility may not be necessary. Finally, standard reinforcing details adopted for the structure connections will be presented demonstrating that no special detailing is required to achieve at least a limited ductile response for OMRF’s in line with section 14 of AS3600.

2 Example Building

The example retail building is a mixed use multi storey frame with both retail and car parking functions. The MRF is constructed of reinforced concrete with high level foundations. It is representative of common construction practice utilising post-tensioned one-way floor slabs and one-way band beams with reinforced columns. The structure is classified as importance level 3 to Australian Standards and the NCC, with site soil class Ce soil. The frame is the primary load resisting system of the building with articulated infill walls and façade isolated from the main frame.

Figure 1 presents the typical floor framing plan and figure 2 a cross section through one of the frames. Figure 3 presents the column and beam reinforcing details, and figure 4 the typical slab reinforcing layout. The OMRF action can be separated into two directions: one OMRF is the direction of the band beams, and the other an OMRF in the direction of the slab spans. The frame in the slab direction is more flexible than the band beam direction. Band beams intersect columns and slabs span between the band beams as shown in figure 1. A summary of the typical beam and column details is as follows:

- Typical band beams 450DP x 1700W, N40 concrete, figure 3
- Typical columns 500 x 500, N50 concrete 8N24 bars, figure 3
- 600&700 diameter columns, N50 concrete 10N28 bars, figure 3
- Transfer beams 1500DP x 2400W, N40 concrete, 8 x 5 strand tendons, 5N28 bars top and bottom, figure 3
- Post tensioned slabs, 180 & 200 thick, N40 concrete, 4 strand tendons at 1m centres, figure 4

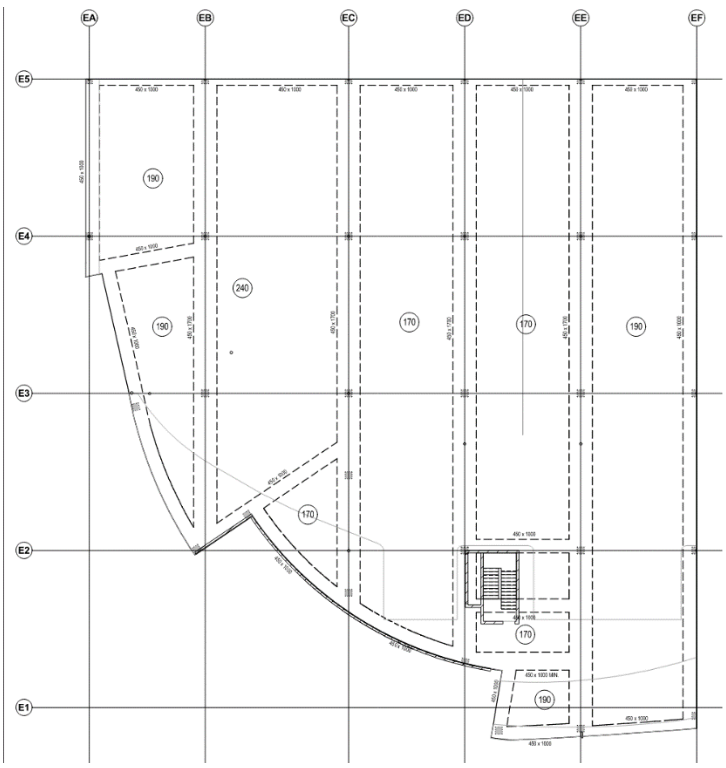

Figure 1. Typical Floor Framing Plan

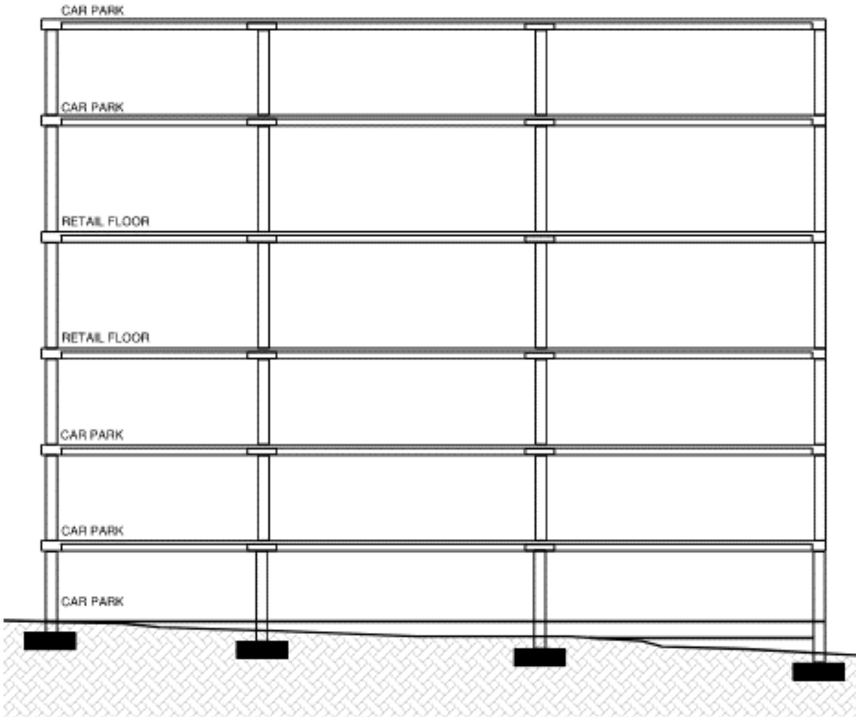

Figure 2. Typical building Cross Section

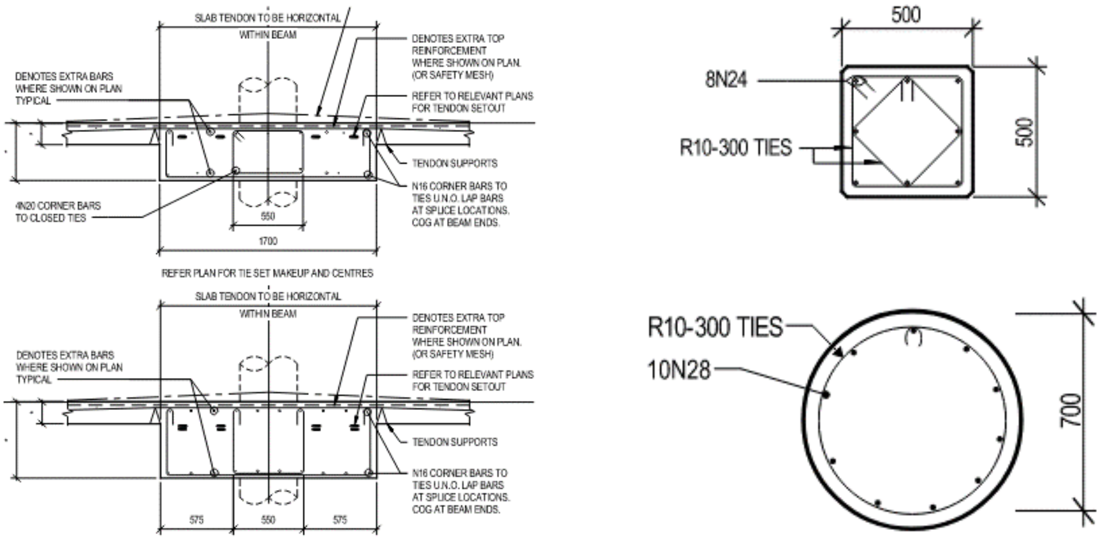

Figure 3. Typical Band Beam and Column Reinforcing Details

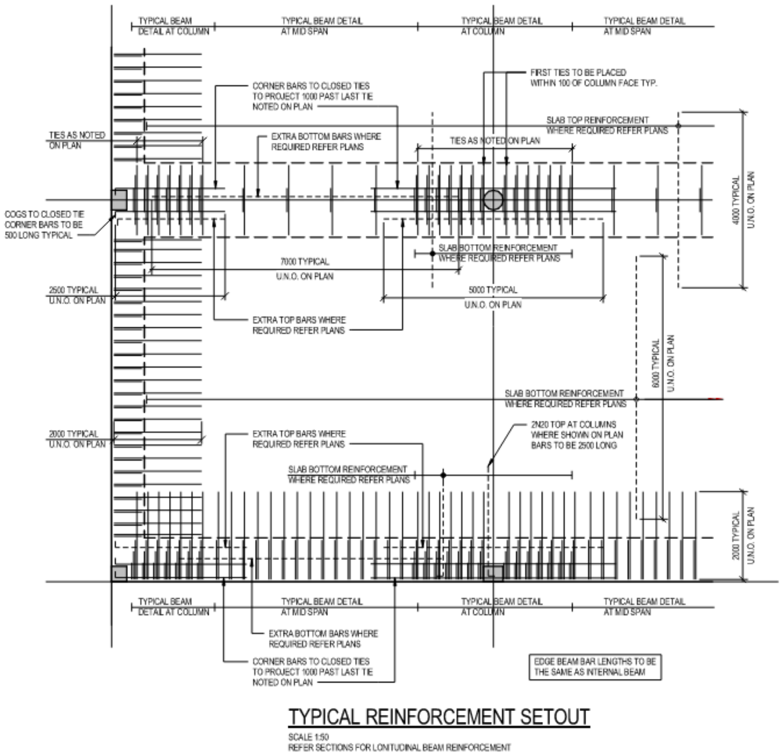

Figure 4. Typical Slab Reinforcing Layout

3 Non-Linear Pushover Analysis method

The non-linear push over method is not new and has been incorporated into many international design codes. Its use in regions of moderate to high seismicity is commonplace and forms part of the displacement-based analysis methods for performance-based design. The method has its advantages over more simplified methods such as more accurate estimates of displacement during member cracking. In addition, it takes advantage of the direct material properties available in structural members rather than assuming a linearly increasing relationship between a structural section and a load. The linear model results in ever increasing stiffness demands and heavy reinforcement detailing. For this reason, simplified methods of calculating seismic forces and base shears are ineffective and cause over estimation of structure sizing and material quantities. They also have no meaningful way to quantify actual ductility.

AS1170.4 allows for the direct calculation of ductility using a non-linear pushover analysis, defined in clause 6.5. The recent update and revision to the standard's accompanying commentary was released in 2021 by the Australian Earthquake Engineering Society and now contains a brief explanation of the push over method. Given the relatively low risk seismic classification of Australia, adopting excessive reinforcement detailing to achieve a "collapse prevention" should not be necessary. Seismic damage is directly related to drift and material strain rather than inertial force induction, which neatly ties into the benefits of non-linear analysis. The capacity of a structure or frame can be classified as displacement capacity, and the input seismic signal as the displacement demand. A capacity curve (also known as a push-over curve) can be generated from the analysis for the entire structure or individual storeys. Figure 5 presents a typical push-over curve & ADRS diagram and highlights the inelastic nature of structures.

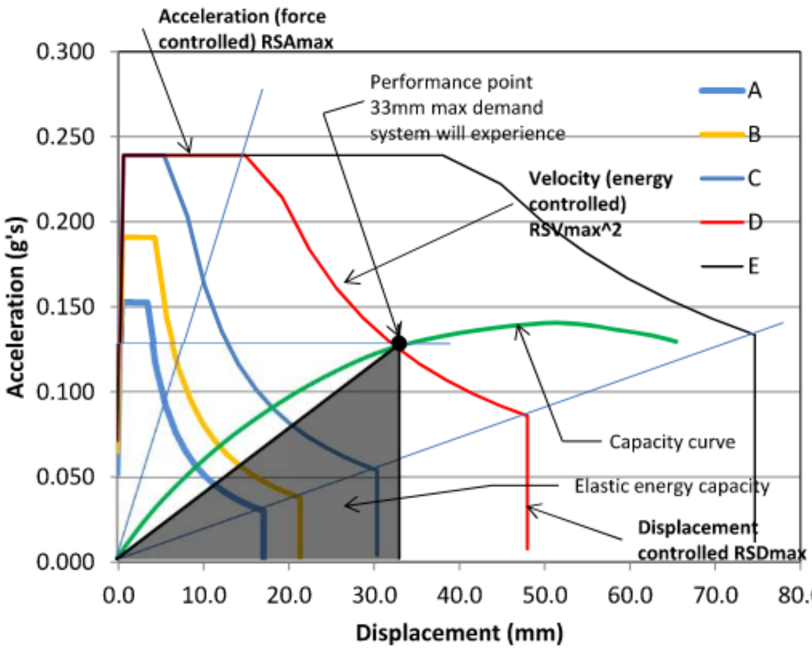

Figure 5. Pushover Capacity Curve & ADRS Diagram

The final pushover curve representing the entire structure can be converted to the acceleration displacement domain by dividing the base shear by the effective mass and calculating the effective displacement. The point where the capacity curve intersects the demand curve is the performance point, as shown is figure 5. A first tier and second tier check recommended by Wilson & Lam (2006, 2008) can be conducted to assess the capacity of the structure. In the first tier check the effective displacement is compared with the Peak Displacement Demand, and if the effective displacement is less than the PDD the structure is deemed satisfactory. In the second tier check the pushover curve is transformed into the acceleration displacement domain and then overlaid on the acceleration displacement response spectrum curves.

The hinge properties specific to beams and columns are calculated using a moment-curvature analysis for each section. This generates the “backbone” curve for each hinge and represents its inelastic capacity. The hinge locations in the frame members are illustrated in figure 6.

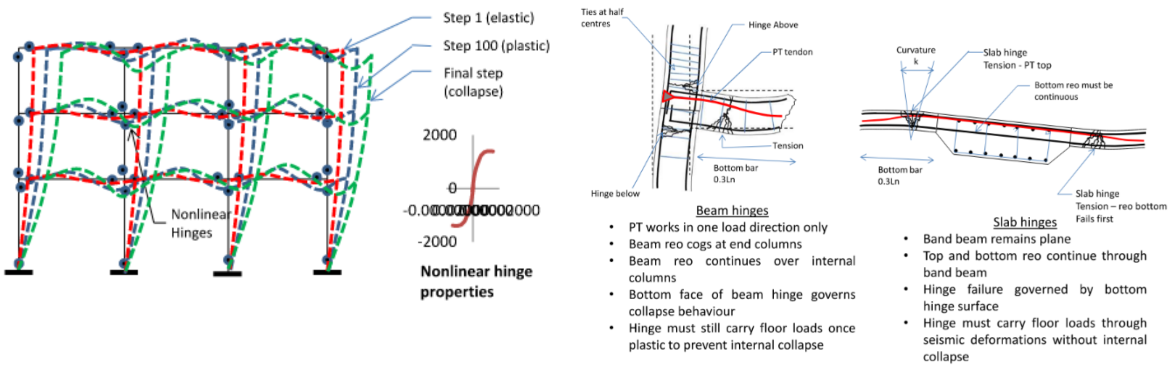

Figure 6. OMRF Hinge Locations and Frame Behaviour

The principles above were used to develop a specific routine procedure in professional practice for assessing the seismic performance of OMRF's. They intend to address and extend the provisions contained in AS1170.4 and the requirements of AS3600 for detailing PT one-way beams and slabs as OMRF's.

Design procedure

1. The frame is designed and proportioned for dead and live loads to Australian standards. Columns are proportioned such that the design axial compression load is at or less that 30% of the section capacity. This ensures the ductile response of column hinges.
2. The frame is then checked under lateral wind load and additional reinforcing at member intersections is added to provide sufficient strength under load reversal.
3. The structure is idealised into two discrete MRF's (single 2D plane of column grids) in the band beam direction and the slab direction.
4. A natural frequency analysis is carried out on the MRF's to establish the modal load patterns to be used for the push-over analysis. Torsional modes are ignored in the 2D frame analysis and are captured in the MDOF analysis with response spectra.
5. Plastic hinges are defined at the top and bottom of columns, and at each end of the beams. This includes the columns fixed to footings, with hinges sitting in the column section above the footing connection.
6. The properties of each hinge are calculated using moment-curvature analysis of section properties. Slab hinges are calculated at the slab intersection with band beams in the slab OMRF direction. This is the likely point at which slab hinges will form, as shown in figure 6.
7. The 2D frame models are analysed in ETABS with the non-linear push-over method, in several patterns including modal shape patterns, with hinge properties as defined in step 6. The gravity loads represent the initial starting load state of the pushover analysis. The models are pushed until failure of the structure occurs either by a series of hinges reaching ultimate capacity or a soft storey collapsing.
8. The push-over capacity curve is constructed from the push-over models and compared to the ADRS curves as shown in figure 5. If the displacement capacity from the structure is less than the PDD, the structure is re-designed. If the PDD is not exceeded, the structure is deemed sufficiently ductile (limited ductility achieved) for earthquake actions.
9. The Push-over analysis steps 4 to 8 are carried out again for a 3D OMRF model using the same hinge properties. The Model is built with frame elements to idealise the beams and slabs.

4 Results and discussion

The results of the analysis of the example retail structure highlighted in section 2 are presented below for the two-dimensional and three-dimensional models. The analysis was carried out using moment-curvature for hinge properties and ETABS to perform the push-over analysis. A spreadsheet called "curvature" was created to efficiently generate backbone plastic hinge curves. The calculated moment curvature lines were transferred to bilinear lines (Priestly et al., 2007) for ease of input into ETABS. The spreadsheet uses stress-strain curves for concrete, high tensile wire strand, and N500 grade reinforcing bar and mean rather than characteristic material strengths, Menegon, Tsang, Wilson, and Lam (2015).

Figure 7 below summarises the moment curvature capacities of the typical columns. The columns have an axial load ratio of between 25% and 30%, and the average ratio of ultimate curvature to yield curvature is about 2 for the hinge sections. Level 3 columns have the lowest axial load and the highest curvature capacity consistent with the literature and engineering texts. The lowest level columns have the smallest available curvature.

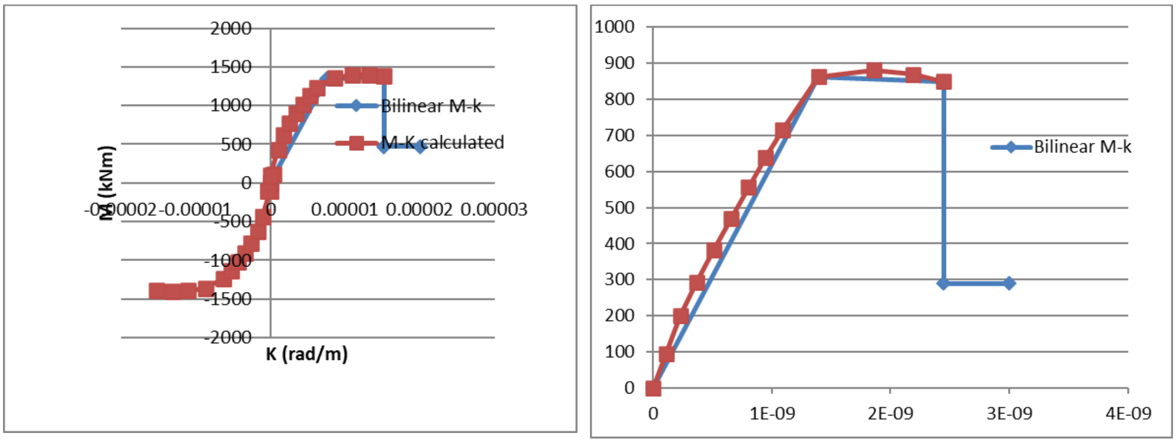

Figure 8. Typical Column Moment-Curvature Results

The moment curvature relations for the typical slabs and beams are shown in figure 8 and demonstrate an asymmetrical behaviour due to the presence of "draped" post tension cables. Additional bottom steel in the slab direction where the slab intersects the band beam has been added to ensure ductile behaviour because the post tension cables are in the top fibre only. The moment curvature capacity for the top fibre in tension is significantly large and has curvature capacity well above a ratio of 2.

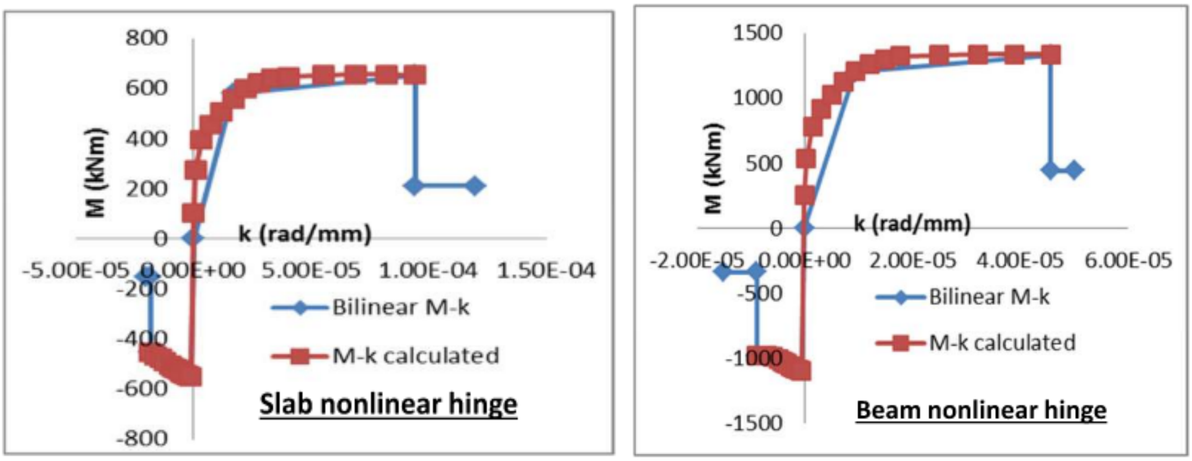

Figure 8. Typical Beam and Slab Moment-Curvature Results

The properties of each member were input into the three-dimensional and two-dimensional models in ETABS with 4 lateral load patterns. The load patterns were normalised from 0 to 1 based on the maximum value at a given level from the modal analysis. Non-linear geometric effect such as P-delta were included in the analysis to account for local and global deformation effects. For clarity figure 9 depicts the various states, IO (Immediate Occupancy), LS (Life safety), CP (Collapse Prevention), on a moment-curvature plot to identify the magnitude of inelastic strain. The objective for each hinge is to not exceed the CP state.

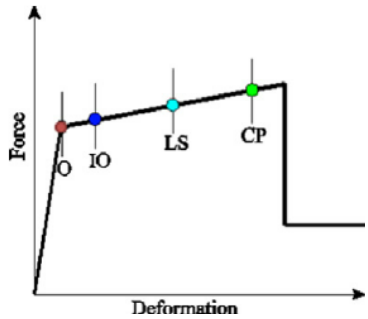

Figure 9. Hinge States Plotted Against Moment-Curvature

Figure 10 presents the CSM results for the 2D push-over analysis is the slab and beam direction for one isolated grid. Figure 11 presents the CSM results for the 3D push-over analysis. For the 2D models the Slab direction experienced first collapse at a top floor displacement of 280mm, with the failure mechanism being slab hinges on level 3. Slab hinges at this displacement reach post-crushing failure at level 3 in two locations. Localised hinge failures terminate the analysis once the slab hinges on the retail floors reach ultimate capacity. The higher storey heights at the retail level govern the collapse mechanism of the structure. The failure displacement is 6 times greater than the maximum demand displacement for class C soil at 45mm. The slab direction performance point crosses the demand curve at 45mm displacement. For the beam direction the structure reaches a max top floor displacement of 205mm, with the failure mechanism being column hinges on level 2 and 3. The columns reach post crushing state at level 2 and collapse state at level 3 at this maximum displacement. None of the beam hinges exceed the elastic capacity. The beam direction performance point crosses the demand curve at 45mm, with the failure displacement being 4.5 times higher than the demand displacement. Therefore, the structure is deemed satisfactory for seismic load and satisfies AS1170.4 for limited ductile behaviour based on these results. It is noted that the demand curves have been multiplied by a factor of 1.5. The slab frame direction governs the collapse behaviour of the structure, and incorporating additional bottom steel in the slab hinges is justified. The analysis indicated that a ductility of 2 can be achieved with N12-300 bars lapping appropriately either side of the slab hinge and very marginally increases the reinforcement tonnage for the project.

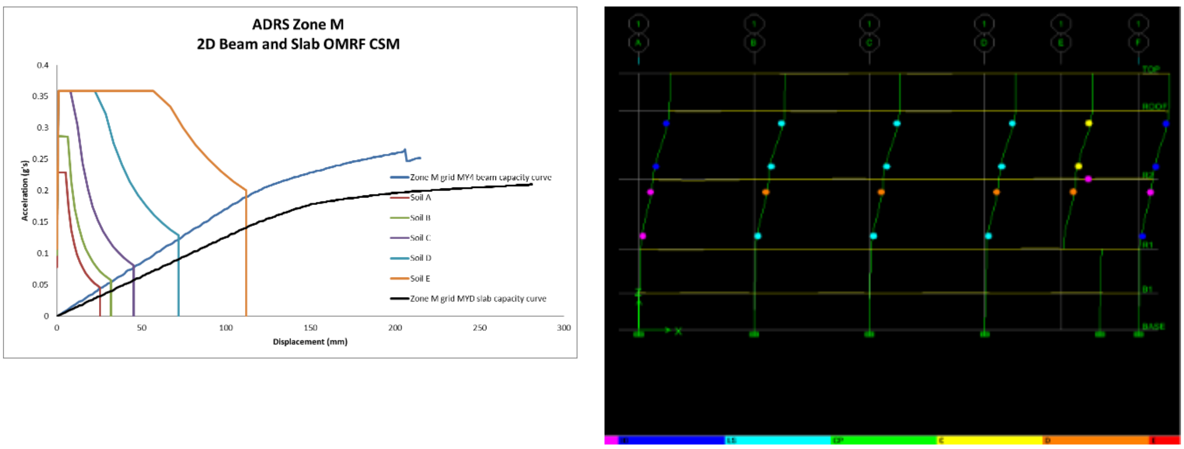

Figure 10. 2D Push-Over Analysis CSM results

In the beam direction greater stiffness is observed from the pushover curve than compared to the slab direction. The failure mechanisms in this direction occur in the lower storey; this is due to more heavily loaded columns supporting transfer levels. The Lower storey columns produce internal instability, indicating that a major load path of the structure has failed suddenly. This collapse mechanism occurred at a top-level displacement of 205mm, with column hinges in B1 basement above the retail transfer floor failing first. The transfer columns in the retail space above the car park begin to approach ultimate capacity at this displacement, demonstrating that the transfer structure at these locations is the likely collapse mechanism. There are multiple transfer beams and columns at each of the upper levels above B1 which redistribute force unsymmetrically. The transfer beam hinges remained elastic at the collapse displacement, and do not experience yielding. Typically beams with smaller depths than those of the transfer beams only just begin to experience yield after the columns have failed. Deflection capacity is once again 4.5 times greater than the maximum demand displacement, indicating that the structure appears to satisfy earthquake demands. Therefore, the structure is deemed to comply with AS1170.4 in the beam frame direction. Minimum bottom steel (6N16) in the beams over the supports was found to be adequate to achieve the necessary limited ductility response.

In the slab direction the structure experienced first collapse at a top floor displacement of 190mm; the failure mechanism was typically narrow edge beam hinges or interior slab hinges. As expected, the bottom fibre zone of beam/slab hinges fails due to the significantly lower ductile behaviour generated by the asymmetric reinforcement. In addition, localised hinge failures terminate the analysis once the model can no longer produce static equilibrium of internal forces for that non-linear displacement. The failure displacement is 4 times greater than the maximum demand displacement for Australian earthquake spectra at the performance point for class C soil. Therefore, the structure is deemed to comply and satisfies AS1170.4. The slab direction governs the collapse behaviour of this structure and therefore must contain minimum ductility in both the top and bottom fibre of the slabs and at any critical hinge locations. It was found that minimum bottom reinforcement of N12-300 was adequate to achieve the necessary ductility.

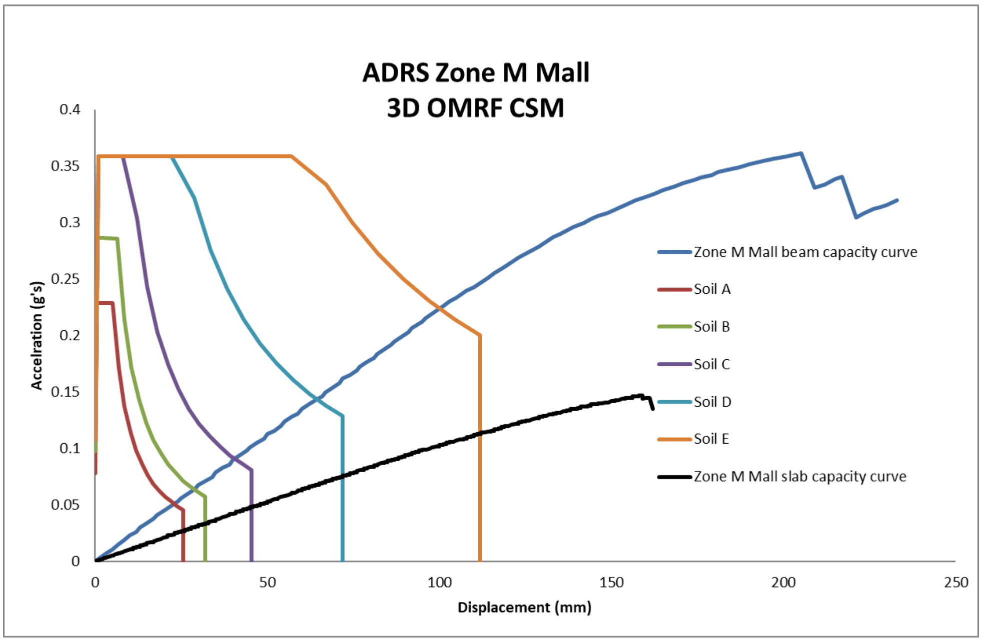

Figure 11. 3D Push-Over Analysis CSM results

Most of the columns were utilised at about 40% of their plastic curvature capacity across the structure. Slab hinges yielded well before beam hinges in both cardinal directions of the push-over analysis. The resulting capacity curves and ADRS overlays show that the structure performs satisfactorily under seismic actions for all soil classes even with demand curves increased by 1.5 times. Testing and analysis have shown that OMRF's are considerably flexible and possess enough ductility to meet the displacement demands of low to moderate risk seismic events, Han, Kwon, and Lee (2004). Characteristically OMRF's show a stable energy dissipation capacity without experiencing abrupt strength deterioration. This is predicted well by the capacity spectrum method, even though the structures are designed entirely for gravity and wind loads. This demonstrates that standard reinforcement detailing for OMRF's provides at a minimum a ductility capacity of 2, and that the provisions in section 14 of AS3600 2018 for OMRF's is likely adequate. Storey drifts for other soil classes are likely to be high and will potentially affect the serviceability of secondary elements.

Whilst the results of this study are promising, it is noted that he models utilised in this study were constructed of beam line elements only. These were modelled in the slab direction as discrete slab widths at column lines, with nothing in between. Further work is needed to assess the slab hinge behaviour using non-linear shell elements to capture hinge behaviour away from column lines. In addition, span to depth and beam geometry ratio needs to be investigated further, as the limited guidelines for beams do not readily apply to one-way slabs. The beam to column width for band beams is another factor that affects the hinge capacity. No testing of this type of connection exists in the literature, and therefore warrants further work to establish the validity of this study.

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