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ADVANCED FEA ASSESSMENT OF EXISTING CONCRETE FLOOR DIAPHRAGMS WITH LOW-DUCTILITY STEEL REINFORCEMENT

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Abstract

Cold-drawn steel reinforcement was commonly used in thin (c.60mm) floor diaphragm toppings in New Zealand during the 1990's and 2000's. The ductility capacity of cold-drawn steel is low compared to mild steel. Despite the known deficiencies of diaphragms constructed of these materials, their reliable strength and deformability must be assessed to quantify if the existing building is safe or require seismic retrofit.

Buildings rely on floor diaphragms to distribute inertial floor forces to walls and frames during earthquakes. Diaphragms are also required to transfer forces between wall and frames where displacement compatibility requires. Therefore, diaphragms are primary structural elements which are crucial to the structural stability of buildings. Detailed analysis is required to ensure diaphragms have sufficient strength and resilience to ensure the safety of building occupants during earthquakes.

This paper compares several methods of diaphragm analysis: deep beam analogy, strut-and-tie analysis, elastic shear friction/cohesion analysis and finite element implementations of modified compression field theory: total strain rotating crack model and the fixed crack Maekawa-Fukuura concrete model. The purpose of the study is to highlight the significance of assumptions made for various methods. Seemingly conservative assumptions of stiffness, tension & compression strength, steel elongation capacity, cyclic degradation may lead to a wide confidence interval including both highly conservative and unconservative results.

Construction information from an irregular diaphragm in a real-world building was analyzed using each modelling method. Static load application was used to develop the maximum envelope (backbone) behavior. Time-history analysis was utilized to demonstrate cyclic and hysteretic responses.

When determining the adequacy of critical existing reinforced concrete floor diaphragm, the use of non-linear finite element analysis is recommended to provide appropriately conservative results.

Keywords: Maekawa-Fukuura model; total-strain rotating crack model; strut-and-tie analysis; diaphragm; DIANA

1. Introduction

Cold-drawn steel reinforcement was commonly used in thin (c.60mm) floor diaphragm toppings in New Zealand during the 1990's and 2000's. The ductility capacity of cold-drawn steel is low compared to mild steel [1]. Despite the known deficiencies of diaphragms constructed of these materials, their reliable strength and deformability must be assessed to quantify if the existing building is safe or requires seismic retrofit.

Buildings rely on floor diaphragms to distribute inertial floor forces to walls and frames during earthquakes. Diaphragms are also required to transfer forces between wall and frames where displacement compatibility requires. Therefore, diaphragms are primary structural elements which are crucial to the structural stability of buildings. Detailed analysis is required to ensure diaphragms have sufficient strength and resilience to ensure the safety of building occupants during earthquakes. [2]

2. Modelling

2.1 Subject building

The study considered an existing 15 story building constructed in the early 2000's. The gravity structure comprised of reinforced concrete floors on steel deck supported by steel gravity beams and columns. The lateral force resisting system was concrete floor diaphragms and precast concrete walls jointed at each floor level and stitched together. The wall behavior was simplified for this study to comprise linear elastic springs representing cantilever shear walls to a height of 40m.

For convenience, physical geometries and materials were altered to ensure a uniform, fair comparison between analytical modelling approaches. As such, the results of this study do not represent the performance of the subject building, rather the study focuses on the difference between assessment methods.

2.2 Subject diaphragm

The subject building's floor comprised of concrete-steel composite construction (utilizing tray-deck cold formed profile). Gravity, thermal, creep and shrinkage actions were excluded for simplicity. The subject diaphragm was a 100mm thick concrete 2D plane element with 665 cold-drawn steel mesh reinforcement, as shown in Fig. 1. Steel beams are treated as fully composite with the concrete floor.

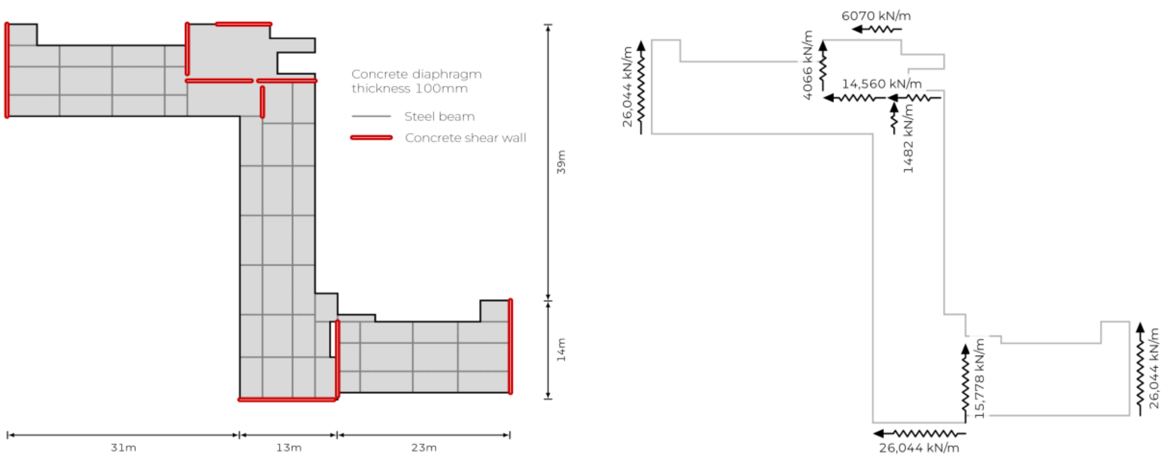

Fig. 1 – Diaphragm geometry and support stiffness

2.3 Support conditions

Linear springs were assigned at the location of each shear wall, similarly for each analysis method. The subject building's response at the ultimate limit state was known to be near to elastic, with total displacement ductility for the system close to 1.0. It was considered appropriate for this study to neglect non-linear behavior of supporting shear walls. Wall stiffness was chosen to be equal to the elastic response of 40.5m tall cantilever walls using the gross second moment of area.

Wall supports were assigned as line springs (kN/m/m) except for the deep beam model that required a lumped spring equal to the summed line spring values. Computed support stiffnesses are shown in Fig. 1 as lumped springs for each wall line.

2.4 General material properties

Probable (mean) properties were derived using the November 2018 Engineering Assessment guidelines section C5 [1] for specified 30MPa concrete and 665 steel mesh reinforcement.

Table 1 – Probable concrete material properties

Engineering property	Symbol	Value
Unconfined compressive strength	$f'_{c\_prob} = 1.5 f'_c$	45 MPa
Uni-axial tensile strength	$f'_{ct\_prob} = 0.55\sqrt{f'_{c\_prob}}$	3.69 MPa
Youngs modulus	$E_{c\_prob} = 4700\sqrt{f'_{c\_prob}}$	31.53 GPa
Strain at ultimate limit state compression	ε_{cu}	0.003
Material weight	$g.\gamma_c$	24 kN/m^3

Table 2 – Probable steel reinforcement material properties

Engineering property	Symbol	Value
Yield tensile strength	f'_{y_prob}	600 MPa
Ultimate tensile strength	f'_{u_prob}	720 MPa
Youngs modulus	$E_{s\ prob}$	200 GPa
Strain at ultimate limit state	ε_{su}	0.015

Table 3 – Probable steel beam material properties

Engineering property	Symbol	Value
Yield tensile strength	f'_{y_prob}	320 MPa
Ultimate tensile strength	f'_{u_prob}	500 MPa
Youngs modulus	$E_{s\ prob}$	205 GPa
Strain at ultimate limit state	ε_{su}	0.100

2.5 Cracked reinforced concrete material properties

Additional material properties were required to allow the use of more advanced analysis methods.

2.5.1 Maekawa concrete model

The finite element analysis program utilized for this study implements a complex non-linear crack model. The constitutive model based on total strain was developed following the Modified Compression Field Theory, originally proposed by Vecchio & Collins [3] in 1986. The total strain-based crack models follow a smeared approach for the fracture energy. The three-dimensional extension to this theory is proposed by Selby & Vecchio [4], which was implemented by the FEA software. The concrete stress-strain relationship of the total strain model shown in Fig. 2 is a verification cyclic test for the properties utilized in this assessment. Note unloading is not to the origin (secant unloading) rather, the sophisticated Maekawa model incorporates damage memory which effectively introduces elemental elongation during hinging cycles. The rotating or fixed total strain model with Maekawa Cracked Concrete compression curve and JSCE Tension Softening curve in tension were applied. [5]

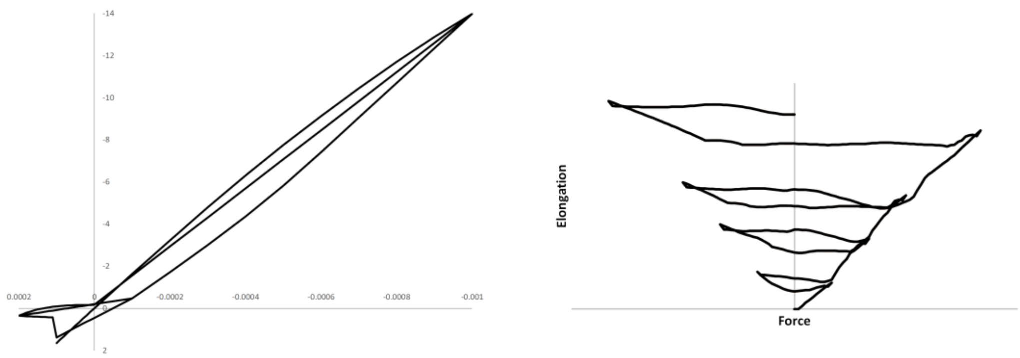

Fig. 2 – Demonstration of Maekawa concrete hysteresis output and arbitrarily loaded material elongation

2.5.2 Concrete Tension Behavior

Concrete tension capacity should not be neglected when tension stiffness contributions increase strain demands in other, more critical parts of the structure. An assumption of 0 MPa concrete tension strength can be used to model cracked reinforced concrete, however this assumption has serious consequences to the structural stiffness of the system. No tension strength in the concrete means that at the first load step, the full concrete element exhibits evenly distributed cracking and stiffness degradation throughout the diaphragm.

2.5.3 Under-reinforced concrete

The main objective of this study is to determine the potential performance of lightly reinforced diaphragms, namely the use of 665 cold drawn steel welded wire mesh. A significant problem with the performance of this historic construction type is that the concrete tension capacity is significantly higher than the steel tensile capacity. The author defines this condition as under-reinforced.

Under-reinforced concrete has an abrupt stiffness reduction when the first crack forms. For typical geometries, the resulting crack spacing will be much wider than the length required to bond (develop) the steel to the concrete which causes significantly high stress concentrations in the steel crossing the crack. It is demonstrated that a 1m crack spacing for the subject diaphragm will concentrate strains in the steel, compared to the average strain of the diaphragm, by a factor of 10. The CEP-FIP 1990 Model Code [6] was utilized to create the estimated peak strain profile shown in Fig. 3 (this was used in preference to the FIB 2010 [7] simplified method).

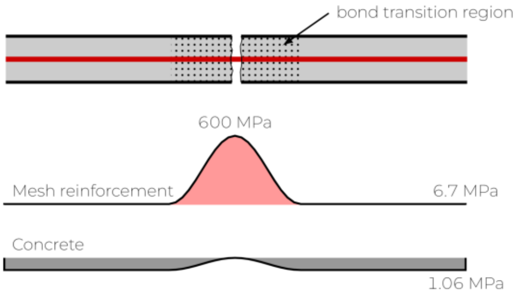

Fig. 3 – Stress concentrations in mesh reinforcement at the crack bond-transition zone

Once the peak strain profile could be correlated to the average strain profile, the steel material backbone curve was altered to match the concrete cracking behavior. Performance can then be assessed at any point where the FEA model converges. This is because steel reinforcement will shed and redistribute forces if the reinforcement reaches fracture strain caused by stress concentrations across modelled cracks.

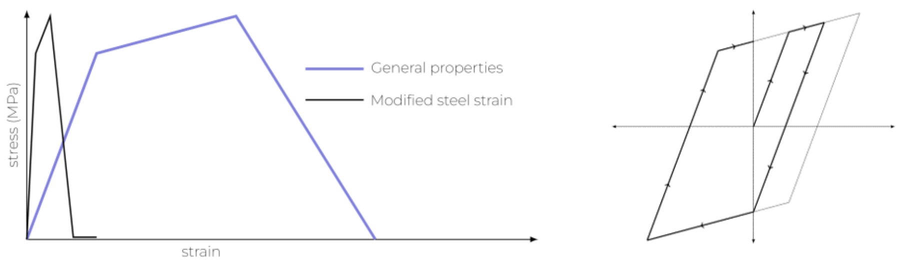

Fig. 4 – Stress-strain backbone for mesh reinforcement modified to reduce strain capacity due to reinforcement stress concentrations at concrete crack locations

2.6 Beam model

A simple elastic beam section was modelled along the typical centerline of the subject diaphragm. Rigid offsets were utilized for areas within adjacent orthogonal beam depths. Spring supports were applied to nodes along the beam where walls were located perpendicular to the beam line.

2.6.1 Performance criteria

The beam is considered to have reached the ultimate limit state when either:

- bending moment demand reaches the flexural strength;
- beam shear strength is reached in B-regions; or
- shear friction strength is reached in D-regions.

2.6.2 Limitations

Deep beam analysis is very simple and has limited application for diaphragm strength assessment of existing buildings.

- Beam-shear strength assessment and assumption of plane stress distribution is only valid within B-regions which are uncommon within typical building geometries
- Shear distortion is not incorporated within regions where the diaphragm changes direction
- Complex geometry, such as penetrations, cannot be modelled

2.7 Strut & tie model

An in-plane grillage was created with a spacing in both direction of 1.0m as described by J.Scarry [2]. Rectangular section properties were assigned for orthogonal widths of 1000mm and diagonal widths of $75\% \frac{1}{\sqrt{2}} 1000mm$ = 530mm – both with a thickness of 100mm. A pseudo-elastic pinned truss analysis was run by considering diagonal members as compression-only struts.

2.7.1 Performance criteria

The strut & tie model is considered to have reached the ultimate limit state when either:

- orthogonal tie force reaches the tension strength of the mesh reinforcement;
- orthogonal or diagonal strut force reaches the compression strength of the concrete; or
- shear friction capacity is reached along the support condition [8].

2.7.2 Limitations

Strut & tie diaphragm analysis has many limitations; whether implemented as a macro, hand method of analysis or more sophisticated, plane truss computer analysis.

- Tension ties have the same stiffness as a compression strut.
- Stiffness does not change as a function of stress i.e. no stiffness degradation as the concrete cracks and so a force-displacement plot is not possible.
- Analysis results are only sensible at the ultimate limit state.
- Due to the low ductility capacity of cold-drawn mesh reinforcement, mesh strain demand is incompatible with strut & tie analysis at the ultimate limit state.

2.8 Cohesion friction model

The cohesion friction model is an elastic finite element analysis using membrane stiffness relationships between integration points. Elastic concrete material properties were utilized for this finite element model.

2.8.1 Performance criteria

Membrane analysis is considered to have reached the yield capacity when:

- Maximum compression strains reach 0.003
- Maximum tension stresses reach the tension yield strength of the steel reinforcement
- Maximum shear stresses reach the shear-friction-cohesion strength capacity [8]

2.8.2 Limitations

- Does not include softening due to concrete cracking, therefore the neutral axis remains near the centroid.
- Can only determine a capacity close to a yield limit state suitable for diaphragms that remain essentially elastic.

2.9 Total strain rotating crack model

The total strain rotating crack model is implemented by DIANA finite element analysis software as discussed in section 2.5.1. The model is essentially an extension to modified compression field theory which incorporates 3-dimensions and non-linear material behavior. Shear capacity is intrinsically checked within the modified compression field theory by ensuring the compression field and steel reinforcement remain within the acceptable performance criteria.

2.9.1 Performance criteria

Performance can be assessed at any material strain for any state of interest – these are typically:

- Orthogonal total strain – steel reinforcement tension
- Principle compression strain – concrete compression

2.9.2 Limitations

- Fracture energy and softening functions are sensitive to the selection of crack spacing
- Analysis loses accuracy as the mesh density differs from the crack spacing
- Crack orientation is recalculated at every load step (likely producing conservative results).

2.10 Total strain fixed crack model

The total strain fixed crack model is a further extension to the rotating crack model which locks the crack location to the angle it first appeared in the run. The Maekawa-Fukuura model considers up to 6 cracks per mesh element depending on damage history and user selection of minimum crack angle spacing as well as slip along the crack interface.

2.10.1 Performance criteria

The performance criteria are identical to the rotating crack model.

2.10.2 Limitations

- An additional parameter is required to model the residual shear stiffness of the crack interface. This parameter is a percentage of elastic shear modulus which can be assigned typical values ranging from 0% to 1%. Minimal guidance was found to select an appropriate value; however results are not highly sensitive to this value for similar loading patterns.

3. Analysis Results

3.1 Body acceleration

Each analysis subjected the diaphragm mass to an acceleration in the Y direction. The magnitude of acceleration was selected so that each model met the performance criteria discussed in section 2. The reported values are the total force resisted ($F = m.a$) and normalized reactions at each support. Force-deflection curves shown in Fig. 6 report local deformation of support 3 compared to support 1 (counting walls from the left hand side).

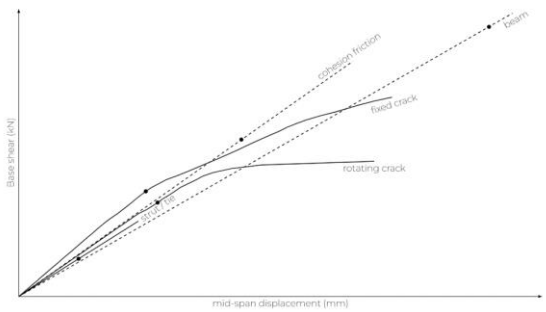

Fig. 5 – Backbone curve result for body acceleration. The performance criteria were met at the location marked on each curve.

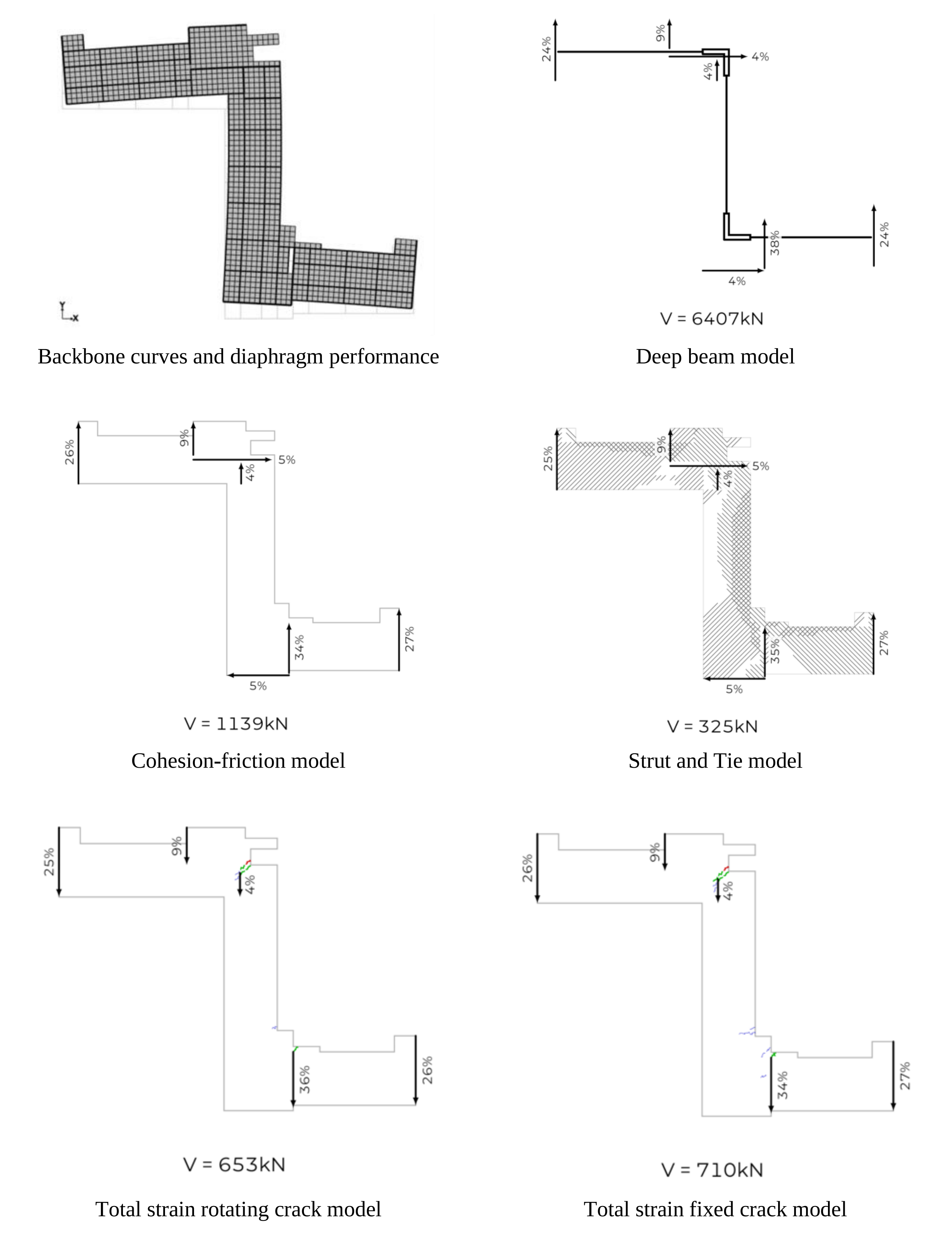

Fig. 7 – Body acceleration analysis results (orthogonal support reaction omitted for clarity)

Both deep beam and cohesion friction analysis could be further limited to loading which reaches the concrete tensile stress i.e. just prior to initiation of cracking. However, this lower limit is still significantly higher than the non-linear procedures.

3.2 Earthquake shaking response

Each model was subjected to boundary acceleration to the diaphragm in the Y direction. Similarly to the first analysis method (section 3.1 Body acceleration), the earthquake record was scaled until the diaphragm met the performance criteria discussed in section 2. The reported value is a normalized scale factor of the record, the peak force resisted and normalized reactions at each support at the time increment of the peak force.

The earthquake record was originally the El Centro 1940 N-S direction, artificially modified to match the design spectrum from NZS1170.5 as subsoil class C. This method was selected as the fairest comparison between models as there is only minor variation in peak acceleration across a variety of spectral periods as shown in Fig. 8.

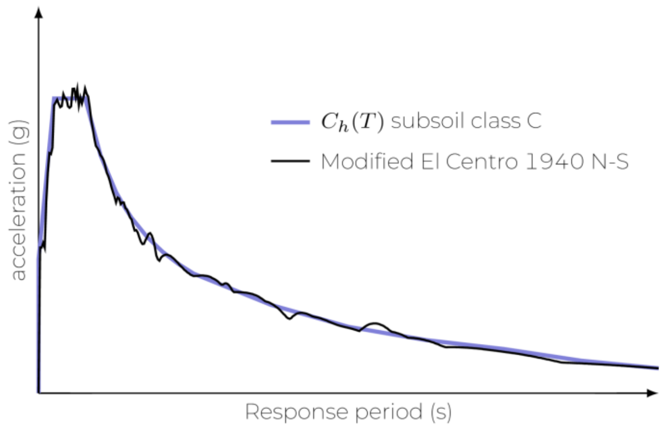

Fig. 8 – Response spectrum of the acceleration record used for time-history analysis

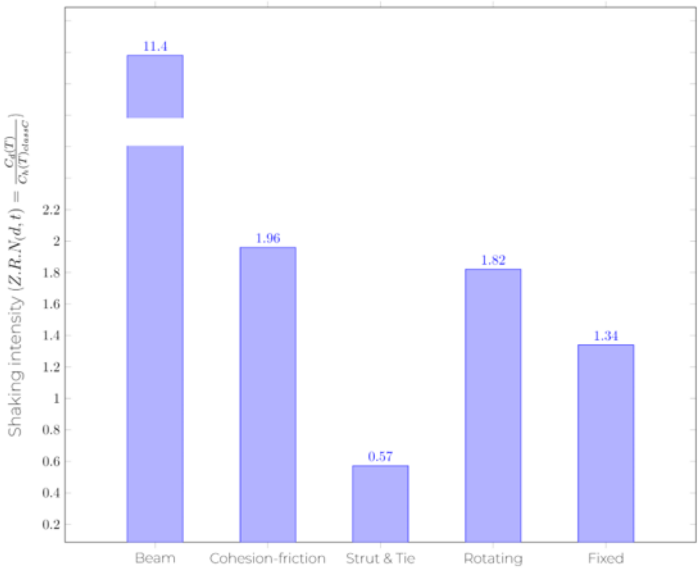

Fig. 9 – Ground shaking intensities at which performance criteria were met for each analysis model

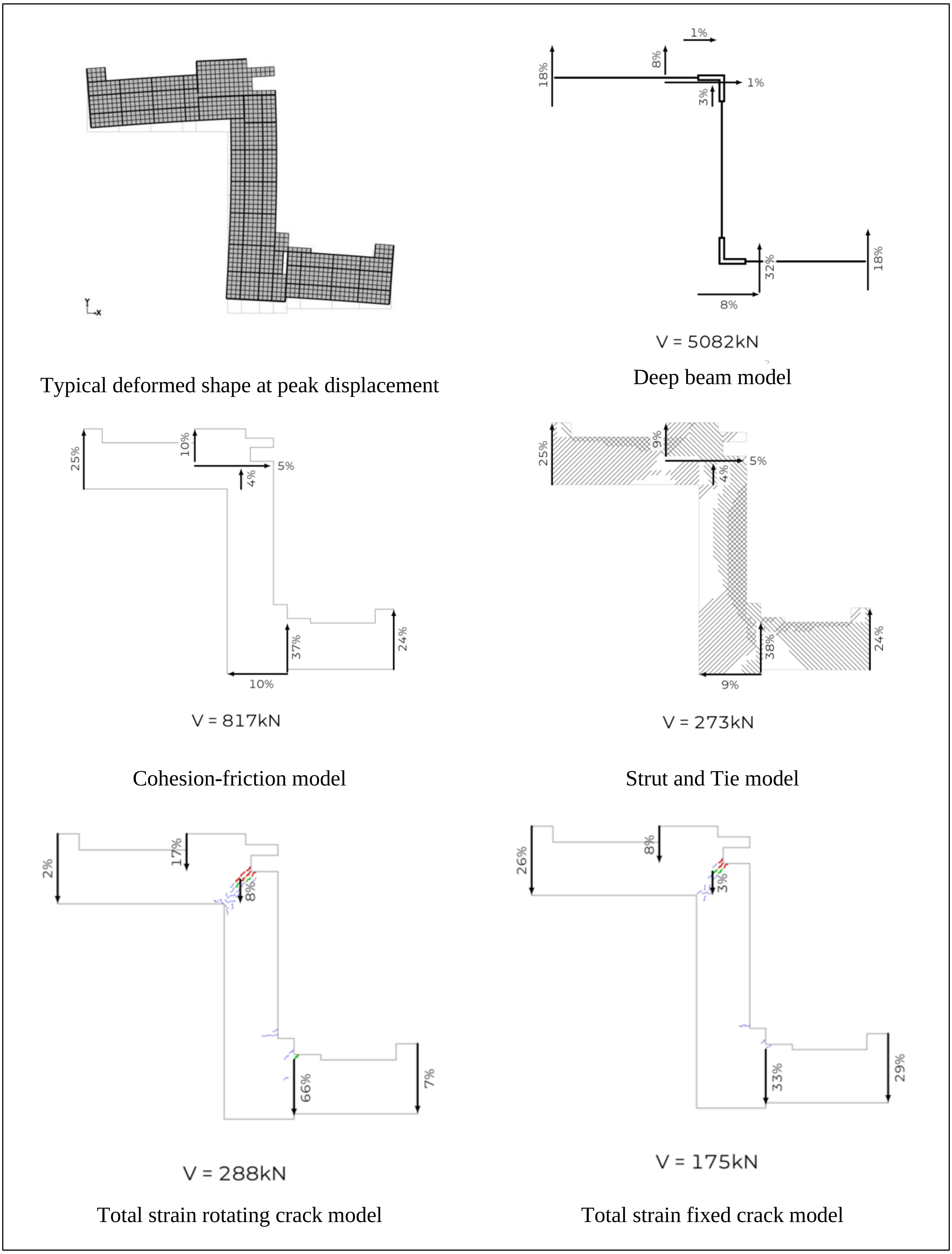

Fig. 10 – Time-history analysis results

The reported base shear in **Error! Reference source not found.** are of less interest than the scale factor used for the earthquake shaking record which are reported in **Error! Reference source not found.**.

All failure modes were the same in nature to the body acceleration loading results.

Again, the beam model grossly overestimates the diaphragm capacity because local discontinuities cannot be accounted for.

The cohesion friction model gave a capacity about 4 times higher than the strut & tie model for both analyses.

The concrete crack models gave greater capacity in the time-history analysis than the static analysis when compared to the results of the other analysis types.

4. Conclusions

Deep beam analysis is unsuitable for irregular diaphragms.

Strut and tie analysis consistently rated the diaphragm with the lowest capacity. Cohesion friction analysis consistently rated the diaphragm with the highest capacity.

Rotating and fixed total strain crack models produce capacity curves due to a non-linear relationship between stiffness and strength. Peak reinforcement strains crossing cracks can be incorporated in a model capable of producing a performance assessment. Resilience can be explicitly checked due to incorporation of non-linear material properties for earthquake shaking at greater intensities than the earthquake intensity used to check the ultimate limit state. As a result of this, the total strain models can also predict the damage sustained by the structures for different seismic intensities and offer very useful insights in the evaluation of the life safety hazards during earthquakes and some useful information regarding the downtime and repair costs.

5. Recommendations

Consider using a total strain cracking concrete model for analysis of diaphragms with low ductility reinforcement to give better certainty of available resilience.

Further research is recommended to test cracking behaviour of under-reinforced concrete sections for correlation with the total strain crack model. It was noted during preparation of this paper that under-reinforced concrete members may be significantly prevalent in the existing building stock (not only diaphragms) when considering probable tensile concrete strengths.

Further guidance is required to fully implement the fixed crack model i.e. parameters for damage functions and residual shear capacity of crack planes.

6. Acknowledgments

The author would like to acknowledge our clients that have utilized our services in the past year. Their loyalty and trust in our expertise has enabled exploration and full implementation of highly advanced analysis methods which provide greater certainty to seismic ratings.

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