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THE EFFECTS OF THE EXTENT OF CRACKING ON THE DESIGN OF REINFORCED CONCRETE WALL STRUCTURES

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SUMMARY

The assumed extent of cracking and corresponding stiffness of a reinforced concrete wall structure influences the structural demands considered in its design. A short parametric study demonstrates that the historical approach of assuming that all elements within a structure are cracked when considering strength limit states can underestimate the demands throughout a nominally ductile structure by a significant amount versus those determined via rigorous determination of the extent of cracking. The study shows that the importance of the increased demands on deformation-controlled elements is dependent on the structural arrangement. Wall arrangements where the cracking strength exceeds that provided by the minimum reinforcement requirements (i.e. walls with flanges) require further investigation. Increased shear demand needs to be considered in the design of wall arrangements.

INTRODUCTION

The stiffness of reinforced concrete elements is dependent on the anticipated level of cracking for the given seismic demand. The need to consider the anticipated levels of cracking in the design process is directly identified in in Clause 6.9.1.1 of the recent amendment to the Concrete Structures Standard (NZS 3101 2017), which is repeated below.

> *6.9.1.1 Analyses to be based on anticipated levels of cracking*
> *Elastic analyses of seismic response, which are used to assess inter-storey drifts, periods of vibration and internal actions, shall make allowances for the anticipated levels of concrete cracking and tension stiffening of concrete between cracks. Where analysis indicates tensile stresses due to flexure and axial load are less than $0.55\sqrt{1.2f_c'}$ in the ultimate limit state, flexural cracking is unlikely to occur and the section properties shall be based on gross or uncracked transformed sections properties.*

Meanwhile, the seismic demand is dependent on the stiffness of the structure, as demonstrated by the Structural Design Actions Standard (NZS 1170.5 2004) spectral shape factor in Figure 1. In simple terms, for a constant seismic mass, the stiffer the structure the shorter the period, and in general the larger the lateral acceleration the structure must be designed to resist.

A common design approach is to assume that the entire lateral load resisting system is cracked when considering demands corresponding to the Ultimate Limit State (ULS). At a local level this may not be an issue as to mobilise the strength provided by the reinforcing steel the concrete must crack. However, an uncracked state locally may manifest in increased strength demands in already cracked regions elsewhere.

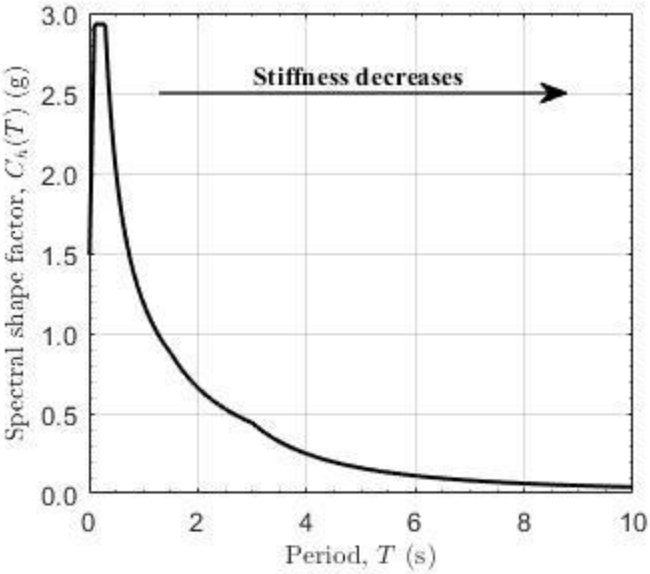

Figure 1. NZS 1170.5 spectral shape factor for site subsoil class C

The seismic demand in Auckland is low compared to other main centres in New Zealand and as such earthquake actions are generally able to be accommodated without significant structural ductility being considered. The "nominally ductile" design philosophy is often adopted on the basis that it is perceived that the ductile design provisions increase the design effort and detailing requirements to an extent that it is not justified.

The scope of this paper is limited to nominally ductile reinforced concrete wall buildings in the Auckland region which are designed using the strength demands determined via modal response spectrum analysis. These are of particular interest as they are generally regarded as not being subject to the additional design requirements of ductile structures such as capacity design and dynamic amplification etc. Instead, the design actions determined via an elastic analysis are typically used directly to apportion strength to each element without any consideration of mechanisms and failure hierarchies, though the validity of this approach is questionable (Brooke 2015). Because of this, broad and approximate analysis assumptions such as cracked stiffness modifiers can have an undue influence on the design outcome, whereas such assumptions are unlikely to be as significant when capacity design and dynamic amplification are considered.

EXAMPLE BUILDINGS

Example (fictitious) buildings have been derived to represent 5, 10, and 20 storey buildings. The interstorey height, h, is taken as 3 m giving overall building heights, H, equal to 15, 30 and 60 m for the 5, 10 and 20 storey buildings, respectively.

The buildings are constructed with 40 MPa concrete (f_c') and 500 MPa reinforcing steel (f_y). The buildings are importance level 2 (R=1.0) with a "nominal" ductility equal to 1.25 and structural performance factor equal to 0.925.

The lateral load resisting system of the example buildings comprises planar walls that are 8 m long. The axial compressive stress arising from gravity demands is assumed to be equal to $0.05f_c'$ at the bottom of the wall. The effective section properties of the cracked walls are equal to $0.29I_G$ considering a simplification of Table C6.5 of NZS 3101.

For each building, several different stiffness scenarios have been devised using the fully cracked properties as the baseline. The fundamental period of vibration (considering fully cracked properties) for each of these buildings has been determined via empirical methods in accordance with general guidance provided in the commentary to NZS 1170.5, i.e.

$$T = 0.0625H^{0.75} \quad (1)$$

The baseline properties have been further adjusted to achieve a Period Multiplier (PM) which is defined as the ratio of the fundamental period of vibration for the modified building to the baseline value and takes values between 0.5 and 4. The Period Modifier is achieved by altering the tributary seismic mass to provide the desired ratio with respect to the baseline value. In practice, this would be reflective of increasing or decreasing the number of walls for a given floorplate. The cracked periods of the 5, 10 and 20 storey buildings with PM=1 are given as 0.48, 0.80 and 1.34 s, respectively. It is worth noting that anecdotally, these baseline values are considered low, that is the example buildings would be considered to be unusually stiff versus those routinely encountered in modern design.

Note that this is a parametric study to investigate cracking phenomena in reinforced concrete buildings. The example buildings have been schemed to be as “realistic” as possible, however in this simple framework it is not possible to maintain a realistic scheme across all building sizes and periods without varying the structural scheme.

METHODOLOGY

The methodology for determining the anticipated extent of cracking is summarised below:

1. Initially the building is considered uncracked and the seismic demand for the uncracked period is determined via the Modal Response Spectrum Analysis (MRSA) method and (for simplicity) Square Root Sum of Squares (SRSS) in accordance with NZS 1170.5.
2. The storey with the largest demand to cracking moment ratio greater than 1 is reclassified as cracked. The stiffness of the now-cracked storey is updated, then the new period and corresponding demands are determined and applied to the structure. Each iteration between the uncracked structure and the final anticipated cracked state are classified as “intermediate states”.
3. The process is repeated until no further stories crack under the newly derived loads. The state of the structure at this point represents the anticipated level of cracking.

The above process is demonstrated graphically in Figure 2 for a 20-storey building with the upper bound period of 4.04 s or Period Multiplier equal to 3. The above process is repeated 4 times as the bottom 3 storeys progressively crack until the demand and cracking state reach an equilibrium. Also presented for reference are the cracking moment and the applicable bending strength derived considering minimum reinforcement requirements from NZS 3101;

Note that the progression of cracking is not necessarily sequentially upwards from the base of the building. In some taller buildings the higher mode demands can be significant and cause cracking to also initiate in the upper levels of the building.

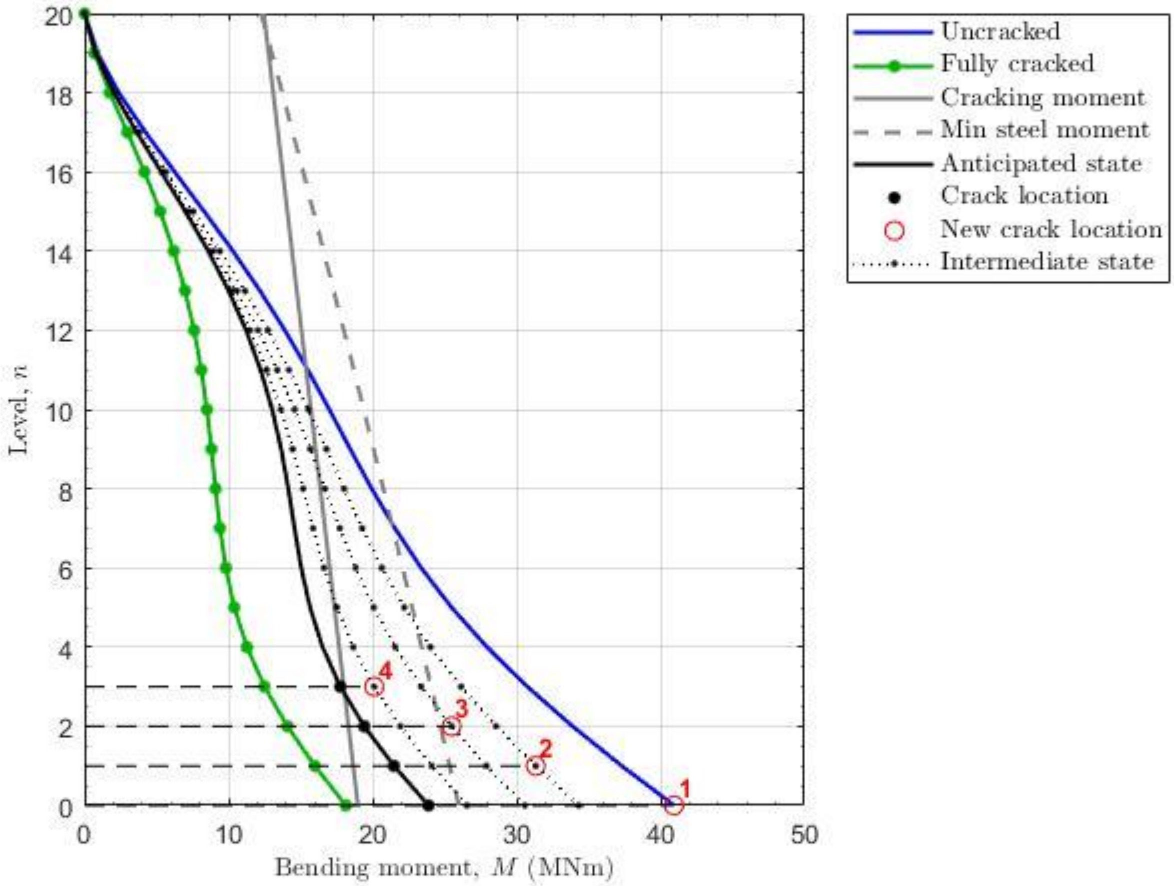

Figure 2. Methodology for determining the anticipated level of concrete cracking in the 20-storey example building T=4.04 s (i.e. PM=3)

RESULTS

Selected results for the 5, 10 and 20-storey buildings are presented in Figures 3, 4 and 5, respectively, and in each case use the same key as provided for Figure 3.

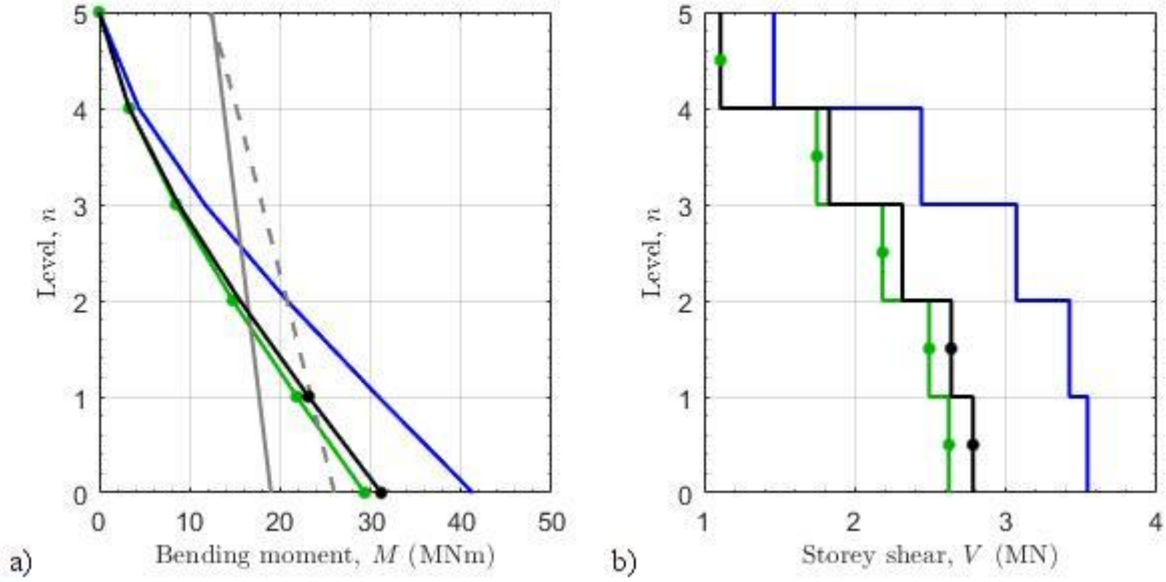

Figure 3. Demands under anticipated level of concrete cracking for the 5-storey example building T=0.48 s (i.e. PM=1) – a) bending moment, and b) storey shear.

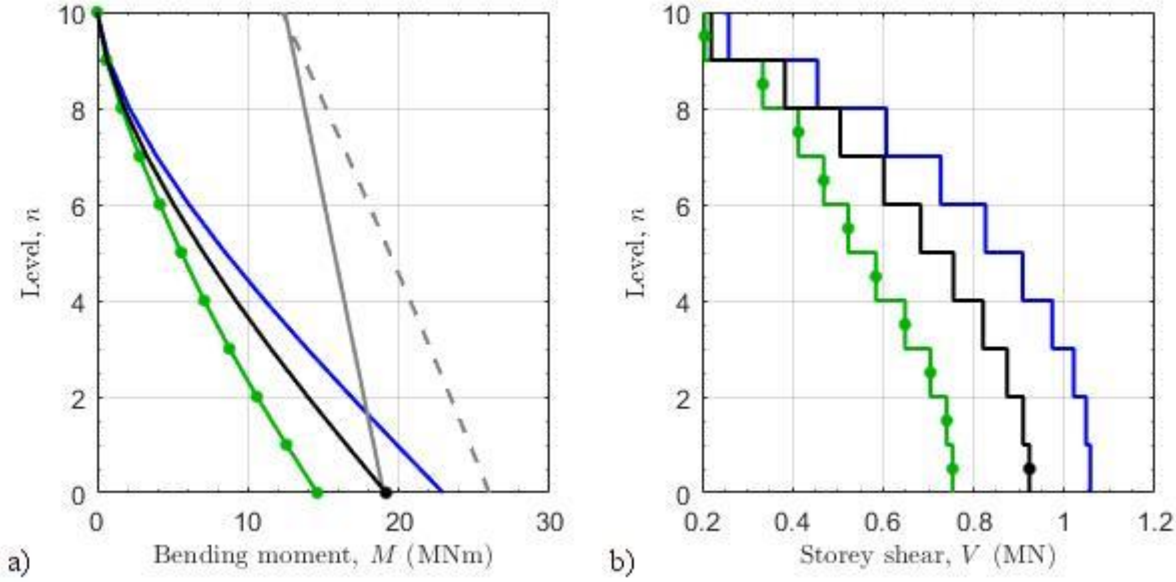

Figure 4. Demands under anticipated level of concrete cracking for the 10-storey example building T=0.80 s (i.e. PM=1) – a) bending moment, and b) storey shear.

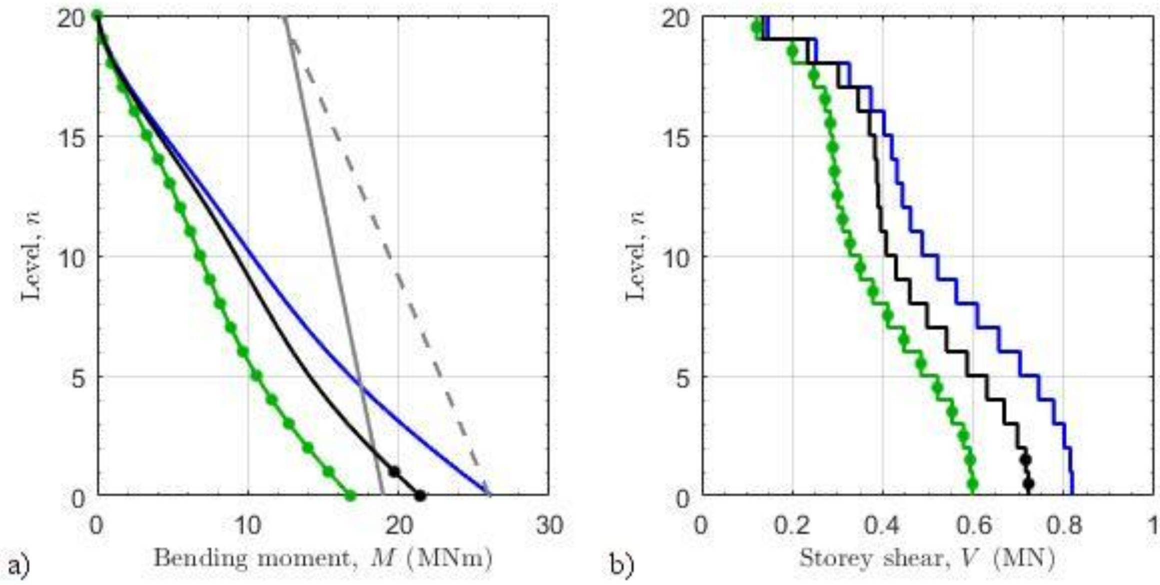

Figure 5. Demands under anticipated level of concrete cracking for the 20-storey example building T=2.69 s (i.e. PM=2) – a) bending moment, and b) storey shear.

Moment demand

To further investigate the phenomena at a parametric level, consider the ratio of anticipated to cracked moment at the base of the buildings as presented in Figure 6 a) with respect to period multiplier for the three building heights. Figure 6 a) demonstrates that the increase in base moment with respect to the cracked demand increases with number of storeys and decreases with increase in PM. The smaller the number of storeys the larger the effect of the first storey cracking on the fundamental period of the structure and thus the smaller the increase on the fully cracked demands. However, to be meaningful, the demands must be considered relative to the minimum reinforcement requirements contained within NZS 3101. The minimum reinforcing ratio is taken as $\rho_{l,min} = \sqrt{f_c'}/4f_y = 0.0032$ in accordance with clause 11.3.12.3(c). Figure 6 b) presents the ratio of both the anticipated and cracked demands to the dependable strength provided by minimum reinforcement. Figure 6 b) shows that for the planar wall structures considered, the increased base moments due to the anticipated level of cracking are inconsequential as they are only larger than the cracked moments when they are less than the strength provided by the minimum reinforcement requirements. The increase in moment demand at uncracked locations is not problematic for the example structures as the moment capacity determined with consideration of minimum reinforcement is always larger than the cracking moment, as shown by the $b = 0$ curve in Figure 7..

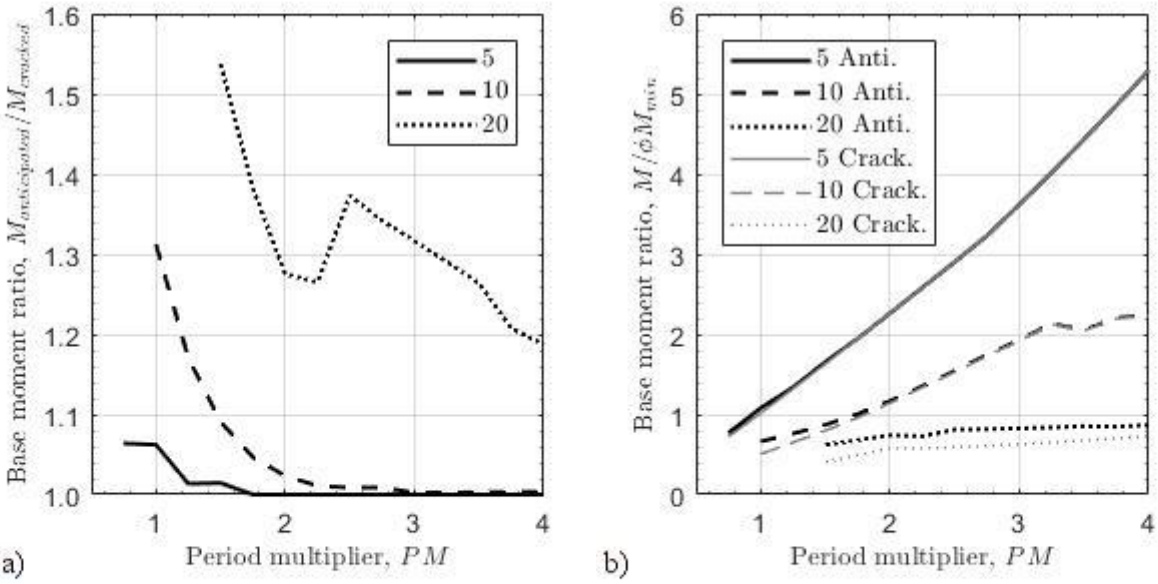

Figure 6. Base moment ratio with respect to the period multiplier of the NZS 1170.5 value – a) ratio of anticipated to cracked moment, and b) ratio of anticipated to minimum steel moment

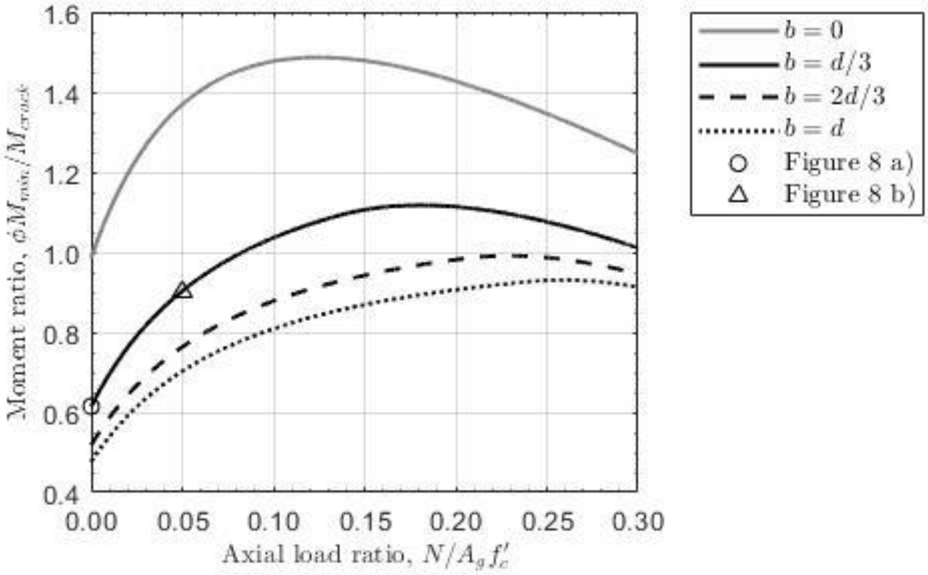

Figure 7. Ratio of minimum steel flexural strength to cracking moment for “C”-shaped walls

The ratio of bending capacity provided by minimum reinforcement to the cracking moment is not greater than one for all geometric variations of reinforced concrete walls with well-conditioned axial loads. Figure 7 presents the ratio of flexural strength provided by minimum reinforcement to the section cracking moment for a series of “C”-shaped walls. The dimensions of the walls are varied by the flange width, b, as a ratio of the wall depth, d. Figure 7 demonstrates that the moment ratio can be less than one and that the phenomenon is exacerbated by the length of the flange relative to the wall depth.

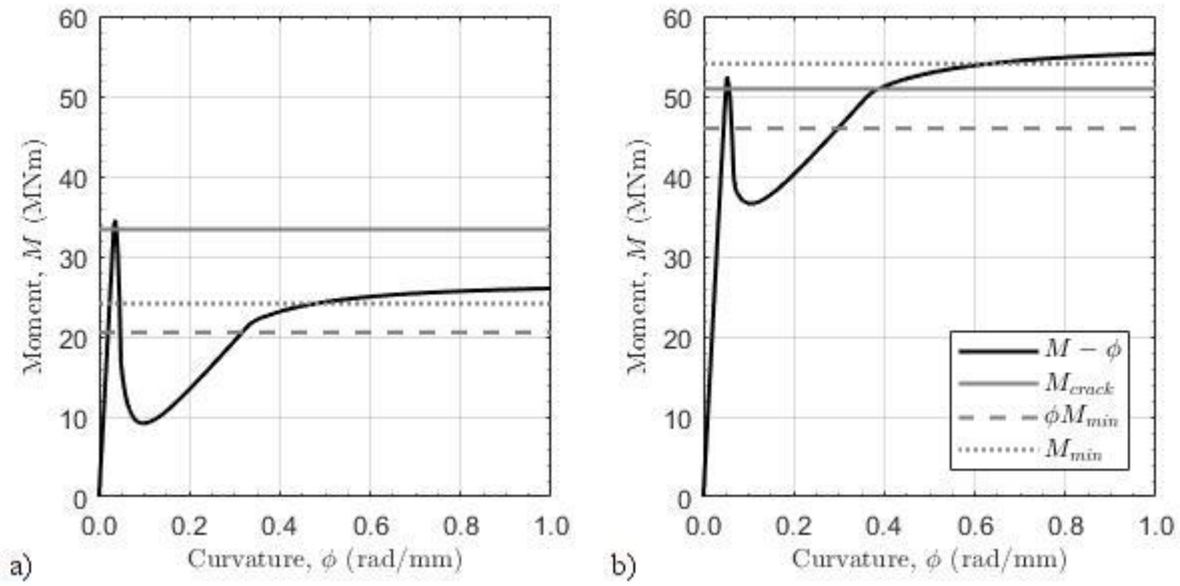

Figure 8. Moment-curvature response of a "C"-shaped wall with dimensions $b = d/3$ for different axial load ratios – a) $N/A_g f_c' = 0$, and b) $N/A_g f_c' = 0.05$

A section having cracking strength greater than that provided by minimum steel is counter-intuitive, so to demonstrate Figure 8 presents the moment-curvature response of two points along the $b = d/3$ curve determined using SAP2000 (CSI America 2021). Note that the moment-curvature analyses assume elastic-perfectly plastic behaviour for the reinforcing steel to ensure that the curves are consistent with the flexural strength determined using methods used in design. The two scenarios considered have axial load ratios equal to 0 (Figure 8 a)) and 0.05 (Figure 8 b)) which correspond to the top and bottom of the example buildings, respectively. Both Figures 8 a) and b) show that the flexural strength of the section at the onset of concrete cracking is greater than the dependable flexural strength provided by minimum reinforcement (φM_n) with the ratios of minimum reinforcement to cracking moment equal to 0.61 and 0.90, respectively. Cracking strengths larger than dependable flexural strengths are concerning because they will preclude distributed concrete cracking and are thus at odds with the assumptions which underpin the determination of plastic hinge rotation capacity in accordance with NZS 3101. The undesirable consequences of such behaviour have been demonstrated previously, for example in the fracture of reinforcement in walls of the Gallery Apartments building (Cooper et al. 2012)

The implications of the moment ratio are explored in Figure 9 for a 20-storey building with either planar or "C"-shaped walls (with $b = d/3$) and a target period of 5.39 s (PM=4). The planar wall structure (Figure 9 a)) designed for the fully cracked demands would be provisioned with minimum reinforcement and still have enough strength over the entire height of the building. The wall is anticipated to crack at the bottom four storeys and level 10. At all crack locations the capacity provided by minimum reinforcement is larger than the anticipated demand so designing considering only the cracked demand is compliant.

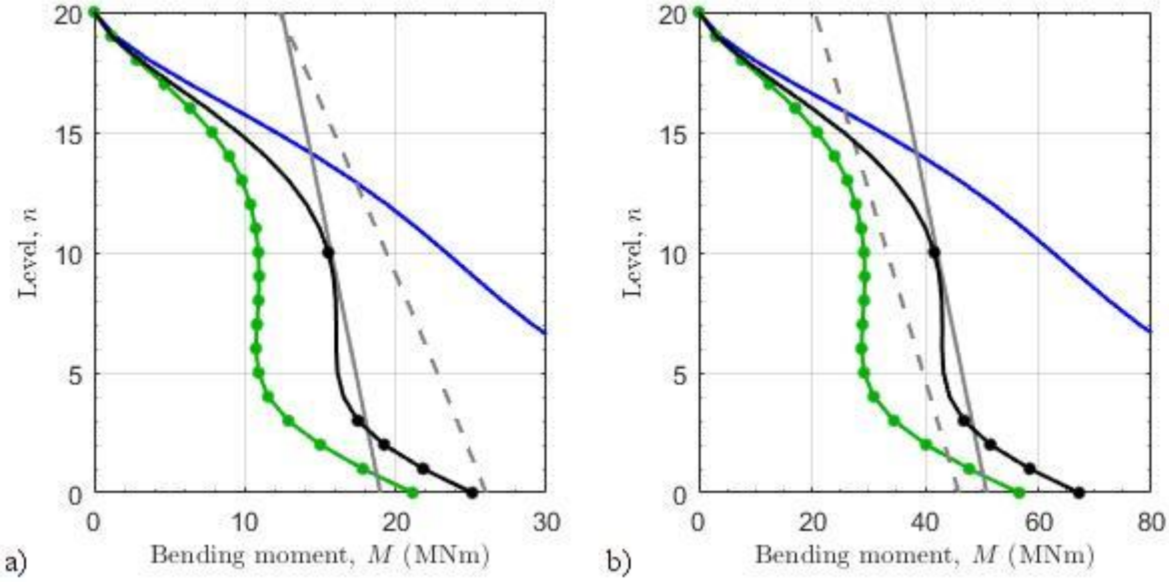

Figure 9. Bending moment demand for the 20-storey example building T=5.39 s (i.e. PM=4) - a) planar walls, and b) "C"-shaped walls where $b = d/3$ and $t = 0.0375d$

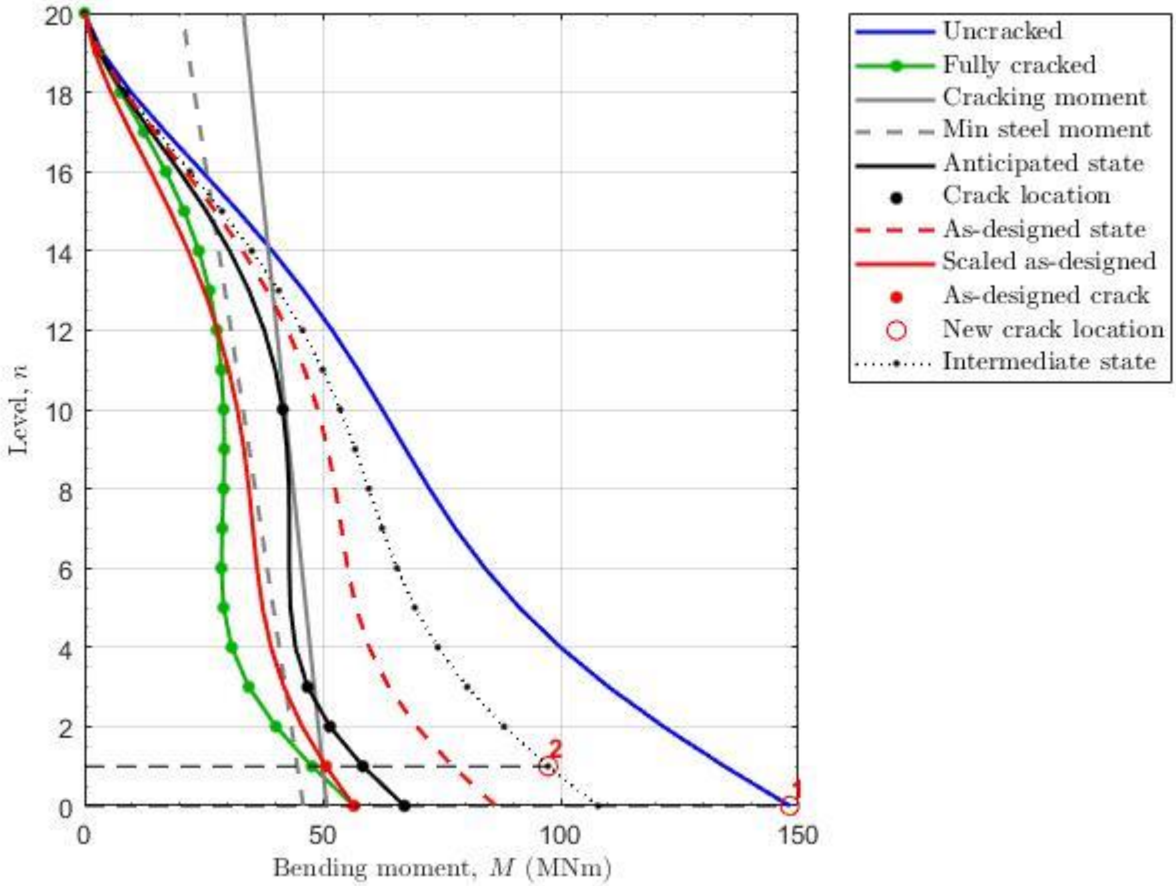

Figure 10. As-designed state of the 20-storey example building T=5.39 s (i.e. PM=4)

Meanwhile, the "C"-shaped wall structure (Figure 9 b)) cannot be designed for the fully cracked demand as the anticipated demands at cracked locations (black curve) exceed both the fully cracked demand (green curve) and the strength provided by minimum reinforcement (dashed grey curve). The result is that buildings with strength provisioned in accordance with fully cracked demands and minimum reinforcement requirements experience ductility demands much larger than those designed for. To demonstrate, consider Figure 10 where the anticipated cracking methodology is updated to include the state of the "as-designed" building where the strengths are provisioned in accordance with fully cracked demand. The base of the building is the first to crack and yield, thus creating a "fuse" with capacity equal to the larger of the fully cracked demands and the strength provided by minimum steel. With the demands limited by the strength at the base, the 2nd storey also cracks. With the two lowest storeys now cracked the demands reach an equilibrium where they cannot increase further, and no more storeys can crack. The scaled demands corresponding to this "as-designed" state are shown by the solid red line in Figure 10 and the flexural strength at the base of the building is equal to 56.6 MN. The ductility 1.25 demands at this stiffness state are shown by the dashed red line with demand at the base equal to 86.4 MN. Hence the force reduction due to ductility is equal to 1.25*86.4/56.6=1.9.

Shear demand

The increase in shear demand from the uncracked demands is always important and needs to be considered in the design. Figure 6 b) demonstrates that for the example buildings the shear demand can increase by up to 25%. Any increase in shear demand must be addressed in the design.

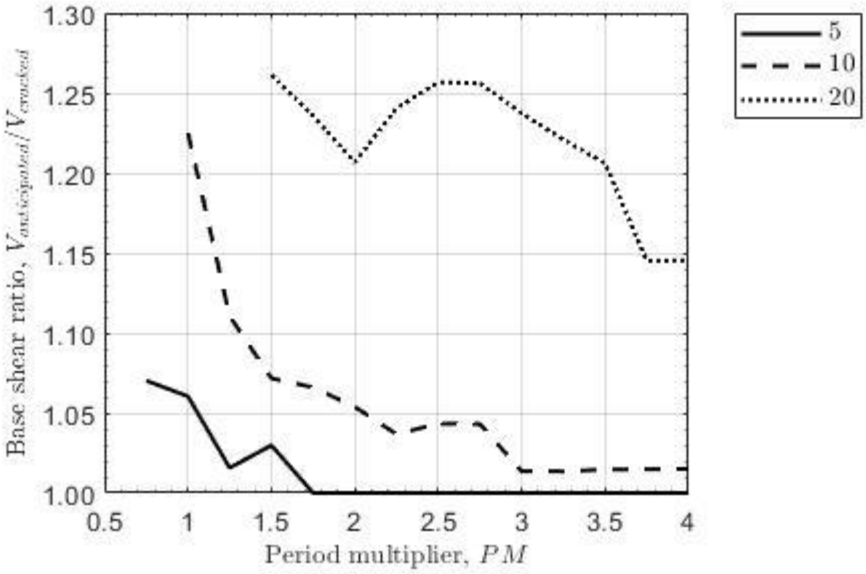

Figure 11. Ratio of anticipated to cracked base shear ratio with respect to the period multiplier of the NZS 1170.5 value

DISCUSSION

The methodology presented in this paper for determining the anticipated level of cracking in structures is likely to be impractical to undertake in a commercial environment for most real buildings. The methodology requires the designer to sequentially track cracking throughout a structure and conduct a modal analysis after each change in cracked state. Such an analysis is not considered practical as part of the design of larger, more complex buildings that generally include coupled wall systems rather than the simple examples presented herein.

This investigation highlights some areas of concern regarding the design of wall structures where the cracking strength exceeds that provided by minimum reinforcement requirements. The effects of building period, provisioned strength, force reduction factor and plastic deformations are all interrelated and contribute to a complex problem. Quantifying these effects is outside the scope of this simple conference paper. Further investigation is warranted incorporating nonlinear dynamic analysis of such structures.

CONCLUSIONS

This paper presents a short parametric study into the effects of the anticipated level of cracking in concrete shear wall buildings designed to be nominally ductile on the bending moment and shear demands.

Consideration of the anticipated level of cracking in a structure leads to larger bending moment demands in wall building when compared to the fully cracked demands. Design to fully cracked moment demand appears to be permissible for planar walls. Walls with flanges can have a cracking strength larger than that provided by minimum reinforcement requirements, a phenomenon that complicates the design process and requires further attention.

The shear demand in the walls can also increase and must be appropriately considered in the design approach for all structural arrangements.

Consistent design and analysis of nominally ductile buildings reinforced concrete wall buildings is difficult to achieve. The design issue addressed in this paper is complex and requires further attention. The intention of the paper is not to be a definitive, quantitative investigation but rather to qualitatively highlight a phenomenon that is often overlooked by designers. Further work is required to fully understand the problem and the implications for building design.

REFERENCES

Brooke, N. J. (2015). “How Nominal is Any Structure’s Ductility?” *Proc. New Zealand Concrete Industry Conference*, New Zealand Concrete Society, Rotorua, New Zealand, 10p.

Cooper, M. C., Carter, R., and Fenwick, R. C. (2012). *Final Report Volume 2: The Performance of Christchurch CBD Buildings*. Canterbury Earthquakes Royal Commission, Christchurch, New Zealand, 236p.

CSI America. (2021). *Computer program SAP2000 v23*. Computers and Structures Inc., Walnut Creek, California.

NZS 1170.5. (2004). *Structural design actions, Part 5: Earthquake Actions - New Zealand*. Standards New Zealand, Wellington, New Zealand, 154p.

NZS 3101. (2017). *Concrete Structures Standard* (NZS 3101:2006 inc. A1-A3). Standards New Zealand, Wellington, New Zealand.