Displacement-based Seismic Assessment: Practical Considerations

Weng Yuen Kam, Umut Akguzel & Rob Jury

Beca Ltd, New Zealand.

Stefano Pampanin

University of Canterbury, Christchurch, New Zealand.

2013 NZSEE Conference

ABSTRACT: Assessing the seismic capacity of an existing building is fundamentally different from designing a new building for seismic actions. Seismic assessment requires a clear understanding and reliable evaluation of the existing load paths, the probable inelastic deformation mechanisms, the probable "collapse mechanism", and the available ductility/displacement capacity of the structure. Displacement-Based Assessment (DBA) (or the Simple Lateral Mechanism Analysis, SLaMA) procedure is as a practical and effective seismic assessment tool, providing an excellent understanding of the way in which a building is likely to perform in earthquakes. This paper builds on the DBA/SLaMa method published in the literature (Priestley 1996, 2007 and NZSEE 2006) showing practical implementation for realistic buildings of complex configurations. Several key challenges that require further development are briefly discussed.

Keyword: Displacement-based assessment DBA, Simple Lateral Mechanism Analysis SLaMA, seismic evaluation, existing buildings

1 SEISMIC ASSESSMENT OF EXISTING BUILDINGS

1.1 Introduction

Assessing the seismic capacity of an existing building is fundamentally different from designing a new building for seismic actions. Poor seismic performance of buildings is generally due to inadequate ductility/displacement capacity and poor load path, rather than inadequate lateral strength. These deficiencies are generally consequences of a lack of seismic design, a lack of consideration of capacity design principles, and poor detailing.

Therefore, a realistic seismic assessment requires a clear understanding and reliable evaluation of the existing load paths, the probable inelastic deformation mechanisms, the probable "collapse mechanism", and the available ductility/displacement capacity of the structure.

In this paper, a displacement-based assessment (DBA) procedure is shown as a practical and effective seismic assessment tool. It is noted that displacement-based assessment may be achieved using direct hand calculation methods (Priestley, 1996, Priestley *et al.* 2007 and NZSEE, 2006) or sophisticated non-linear computer analysis (ASCE-41, 2007). The focus of the paper is on the practical implementation of the hand-calculation method for realistic buildings of complex configurations. Several key challenges that require further research and development are briefly outlined.

1.2 Force-based or displacement-based seismic assessment?

Seismic assessment of existing buildings has been traditionally based on force-based methodology, owing to the background of force-based seismic design familiar to most practitioners. However, it is accepted that material damage is function of strain, curvature, rotation and displacement imposed on structure by the external forces, be it seismic, wind or gravity (e.g. Priestley *et al.*, 2007 and Blume *et al.*, 1961). Similarly, it is apparent that displacement-based parameter such as inter-storey drift is better in quantifying damage for secondary structure (e.g. stairs, non-ductile columns) and non-structural elements (partitions, façade). Displacement based procedures also offer the only practical method of assessing structural systems of varying available ductility in the same direction that are commonly encountered in existing buildings.

The conventional force-based seismic design and assessment methodology is generally based on satisfying the strength demand-to-capacity ratio and specific seismic detailing provisions for adequate ductility capacity of critical components. Therefore, an existing structure may be considered satisfactory (say 100% new building standard, *%NBS*) if it satisfies the general strength requirement with some assumption of the structural ductility inherent to the building. The likely behaviour and performance of the structure under the design-level earthquake shaking is not required to be known and may only be described in broad terms (e.g. life safety).

One common misgiving from the public of the seismic assessment is the mismatch of the predicted seismic performance and the observed damage of buildings an earthquake. For example, the acceleration response spectra from the 22 February 2011 Christchurch earthquake would indicate most of the buildings in the CBD would have experienced 2 to 3 times the elastic design acceleration as shown in Figure 1a. The conventional force-based seismic assessment of buildings which survived 22 February 2011 earthquake may often yield very low *%NBS* score, yet these buildings remain standing.

In our opinion, a displacement-based assessment may reduce some of the discrepancies between the assessed and the observed performance. At the very least, a displacement-based assessment would seek to quantify explicitly the maximum expected inter-storey drift and the maximum ductility demand on critical elements; which therefore allows a better correlation to the expected damage and performance.

A preliminary review of the displacement response spectra from the 22 February 2011 earthquake (Figure 1b) and the observed damage of reinforced concrete buildings suggests the spectral displacement demand is a better indicator of damage (Kam *et al.*, 2012). Similar observation was made on the relationship of observed earthquake damage and spectral displacement demand in the 4 September 2010 earthquake (Pampanin *et al.*, 2011).

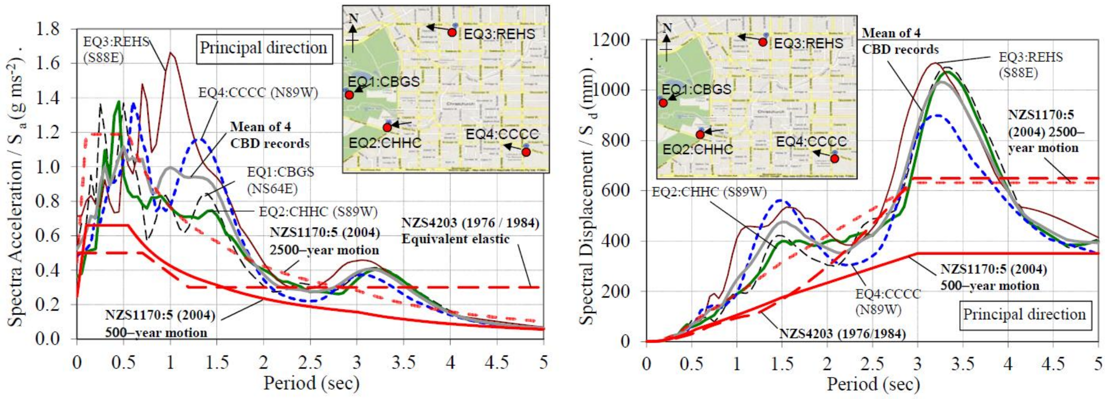

Figure 1: 22 February 2011 Mw 6.2 earthquake response spectra (5%-damped) in the Christchurch City Central and the NZS1170:2004 design spectra (red solid) for Christchurch (soil class D, R=20km): a) Acceleration response spectra, b) pseudo-displacement response spectra.

1.3 Strength or displacement performance limit state

Under current interpretation of the 2004 Building Act and the NZSEE recommendation (2006), a building is considered earthquake-prone building (EPB) if it reaches the ultimate limit state capacity under a 33% seismic shaking expected for the site for a new building i.e. <33%NBS. In our opinion, a narrow interpretation of the 33%NBS based on the lateral strength limit state alone may result in un-conservative or incorrect risk assessment. This is schematically illustrated in Figure 2 below, as taken from Kam (2011). It is noted that Figure 2 does not include the S_p factor as per NZS1170.5, and the S_p factor would somewhat compensate for the lower displacement capacity of brittle system by imposing a higher strength requirement.

From a force-based assessment perspective, only case F & G with the lateral strength < 67%NBS and <33%NBS are considered earthquake-risk and earthquake-prone buildings respectively. From a seismic performance perspective, cases E, F and G are seismically-vulnerable as they will be vulnerable to collapse under moderate-to-large earthquakes (with >67%NBS displacement demand). On the other hand, cases C and D are expected to perform better, despite having a lower strength capacity.

Figure 2 emphasizes the importance of holistic view of the seismic performance after the retrofit interventions. Cases C and D, while do not attain 67%NBS lateral strength, are more preferable for their ductile response and sufficient deformation capacity.

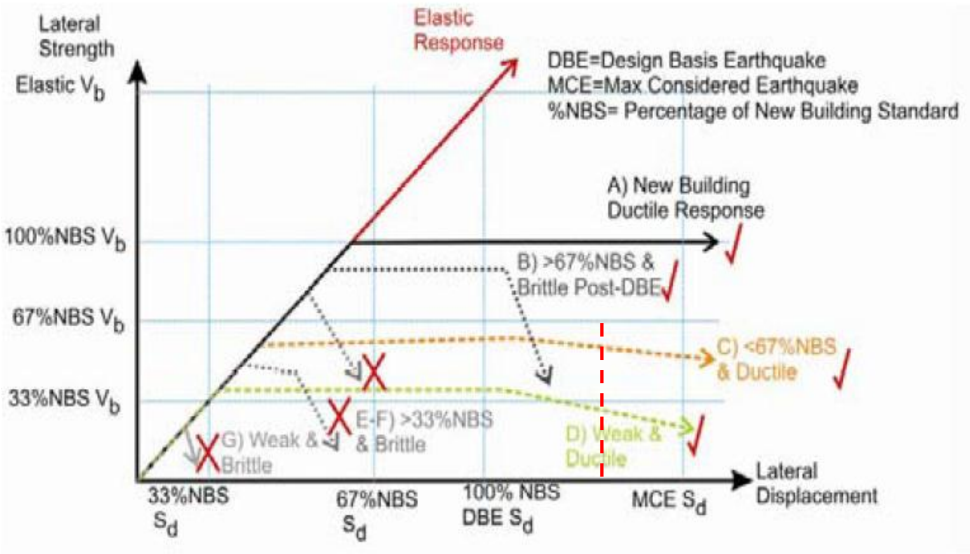

Figure 2: Seismic retrofit objectives (as *%NBS* requirement) in terms of lateral strength and displacement requirements. V_b = lateral strength / base shear. (from Kam, 2011).

2 PRINCIPLES OF DISPLACEMENT-BASED ASSESSMENT (DBA)

In principle, the displacement-based assessment (DBA) procedure compares the lateral displacement capacity of a building with the expected lateral displacement demand. The substitute structure single-degree-of-freedom (SDOF) approximation is used to characterise a building as an equivalent linear system responding to the displacement capacity, Δ_{ult}. This is conceptually illustrated in Figure 3.

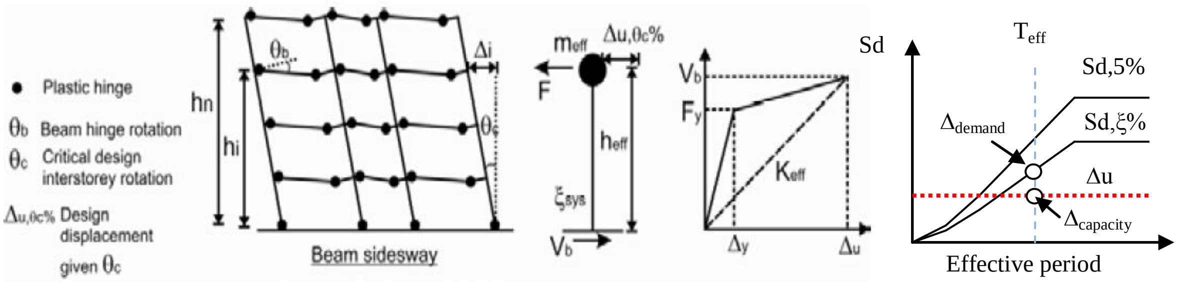

Figure 3: Fundamentals of displacement-based assessment. Refer to the next section for further terminology definition.

Existing literature (Priestley, 1996; NZSEE, 2006, EAG 2012) has provided some guidelines on the DBA procedure especially for reinforced concrete structure. For several seismic assessment projects undertaken by Beca, a broader procedure was adopted. The key steps of the procedure is summarised in Figure 4.

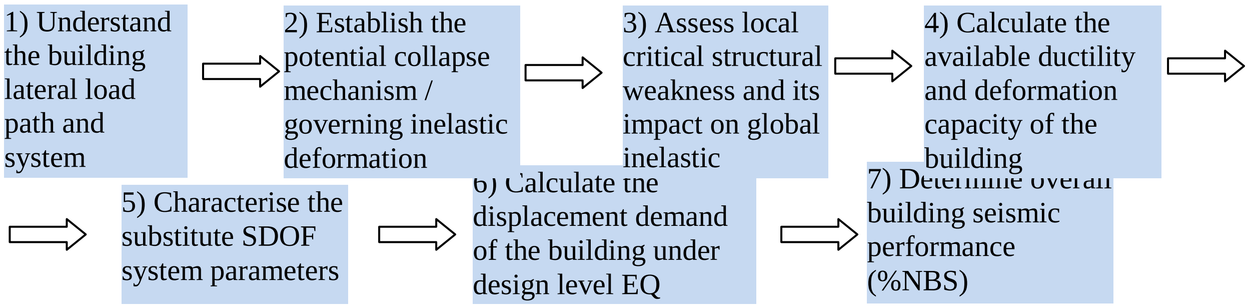

Figure 4: Flowchart procedure of the displacement-based assessment.

3 EXAMPLE: DISPLACEMENT-BASED ASSESSMENT (DBA) OF WALL STRUCTURE

3.1 Background

In order to illustrate the basic displacement based assessment (DBA) procedure conceptually, an example four-story reinforced concrete wall building with gravity concrete frames is to be assessed using the DSA approach. The DSA methodology is presented in a step-by-step fashion. Due to the space constraints, reference is made to literature in sections, equations or figures wherever necessary. The floor plan and the layout of the reinforced concrete walls are shown in Figure 5.

It is also assumed that the engineer would have completed the preliminary assessment work including a detailed review of structural drawing review and necessary site visual and intrusive inspection, to form a good understanding of the building lateral load resisting system.

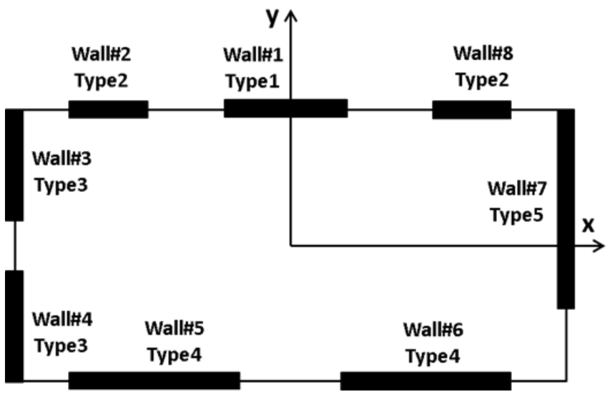

Figure 5: Hypothetical wall structural plan for DBA example. Internal gravity frames not shown.

3.2 Step #1: Determine the hierarchy of strength and governing failure mechanism

Step #1.1: *Calculate the flexural and shear capacity of each element*:

Form a view of the critical section of each of the load resisting elements based on a drawing review and a preliminary estimate of the potential plastic zones. Calculate the axial-flexural and shear capacities at the critical section. Conventional moment-curvature analysis can be used to determine the moment capacities and curvature limits for each cantilevered wall (see Figure 6). The shear capacity of the wall, φV_u can be calculated using conventional shear assessment equations which includes the degradation of concrete shear contribution due to flexural ductility demand (e.g. NZSEE, 2006).

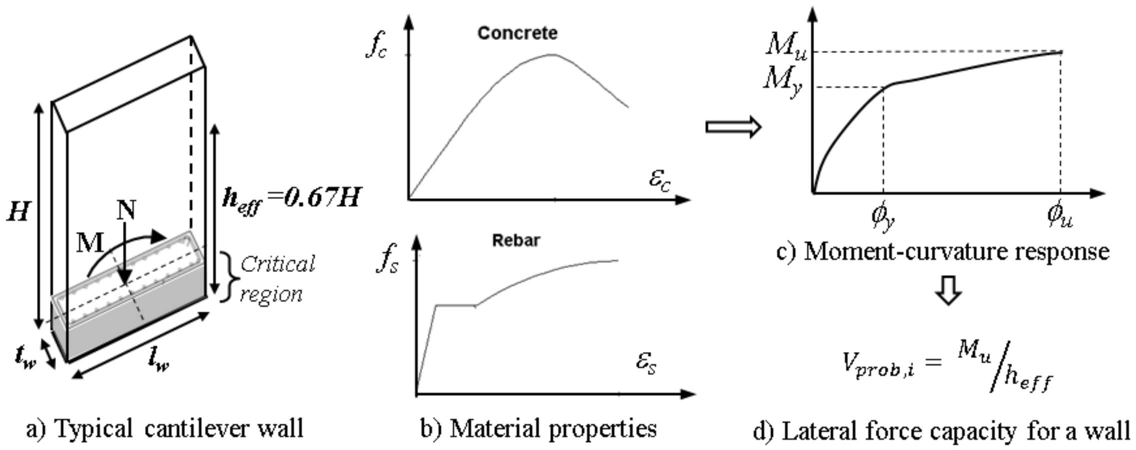

Figure 6: Section analysis of a wall element

Step #1.2: *Determine the governing inelastic mechanism of each element using a hierarch of strength analysis (hand calculation for pseudo-pushover).*

For a cantilevered reinforced concrete wall, the governing inelastic mechanism is determined by comparing the probable base flexural capacity with the probable shear capacity multiplied by the effective height of the wall. The achievable base shear for each wall in each direction is the minimum of $\varphi M_u / H_{eff}$ and φV_u.

Step #1.3: *Calculate the post-elastic displacement (deformation) capacities for each element*: The yield displacement, plastic hinge length, plastic curvature, plastic displacement and ultimate displacement capacity at the effective height of the building can be calculated by the relationships as given in Priestley *et al.*, 2007 and EAG, 2012.

Priestley *et al.* (2007) recommends a reduction of the calculated shear resistance as a function of the flexural ductility demand associated with the flexural hinge at the base. Therefore, Step 1.3 and 1.2 may be iterative as the calculated curvature ductility demand may subsequently change the governing inelastic mechanism at the base from flexural hinge to flexural-shear failure.

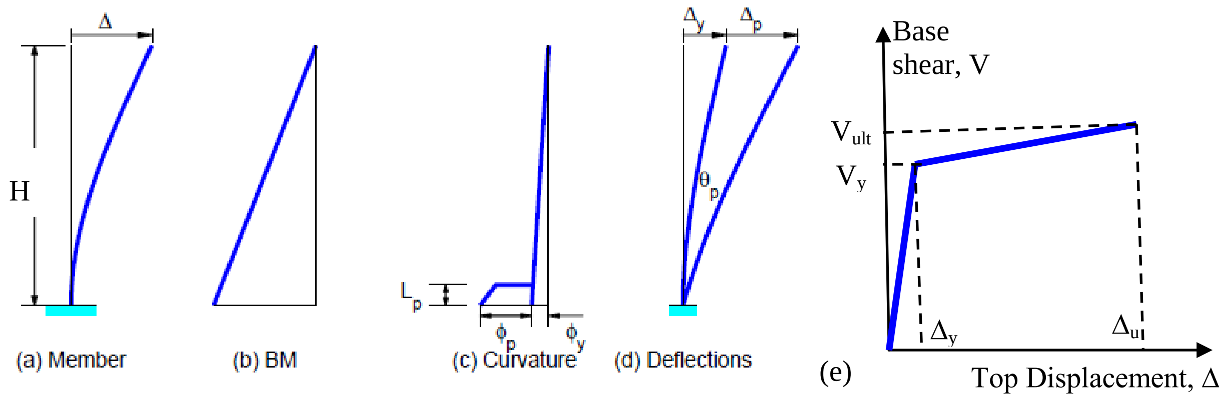

Figure 7: Calculate the post-elastic displacement (deformation) capacity of flexural cantilevered member.

3.3 Step #2: Estimate the lateral strength of the system

Once the governing inelastic mechanism and its corresponding base shear strength and displacement capacity at effective height have been determined, a bilinear idealisation of the push-over capacity curves for the cantilevered walls can be plotted (e.g. Figure 8). The lateral strength of the building in each principal direction is computed by the super-positioning of all walls' base strength contributions at the critical displacement capacity.

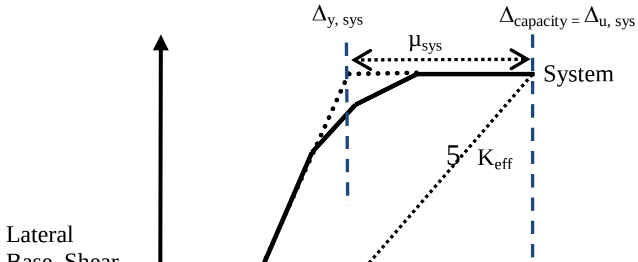

Figure 8: Push over capacity curves for individual elements and the system for the Y-loading direction.

3.4 Step #3: Assessment of Critical Structural Weaknesses

At this stage, the critical structural weaknesses (CSWs) that would impact on the global displacement and capacity of the building are assessed. Specific calculations would be required to check whether the critical load path is integral to sustain the inelastic mechanism represented by the push-over capacity curves. For the cantilevered wall system, potential CSWs to be checked include a) horizontal diaphragm-to-wall capacity, b) wall foundation capacity, c) inelastic torsion stability from plan irregularity and d) torsion amplification for accidental eccentricity for elements at the edge.

3.5 Step #4: Assessment of the system achievable ductility

The achievable ductility of the system, μ_{sys} can be estimated by the ratio of the ultimate displacement capacity and the yielding displacement of the system. For the example with walls with multiple yield displacements as shown Figure 5, a bi-linear approximation may be adequate as shown in Figure 8. The calculated μ_{sys} needs to be checked against the material standards requirements to ensure μ_{sys} is below the specified structural ductility limit for the available detailing. If the reinforced concrete wall has non-ductile detailing such as plain bar lap splices, unconfined boundary ends etc., a relatively conservative μ_{sys} limit = 2.0 should be used.

3.6 Step #5: Characterise the substitute structure SDOF properties

At this step, the building global response is characterised as an equivalent SDOF system, using the substitute structure approach (Priestley *et al.*,2007). In this approach, structural effective period, T_{eff} is directly related to the effective stiffness, k_{eff} while k_{eff} is a function of achievable lateral base shear (V_{prob}) and achievable lateral displacement capacity ($\Delta_{c,\xi}$).

The steps are illustrated in Figure 9 in a sequence to compute: a) displacement profile shape, b) critical displacement based on deformed shape and yield displacement, c) effective seismic mass, d) achievable damping and ductility, e) effective period and stiffness. More information can be found in Priestley *et al.*, 2007.

Several technical challenges in adopting these equations which were derived for direct displacement based design (Priestley *et al.*, 2007) for seismic assessment of complex structure are discussed in the next section.

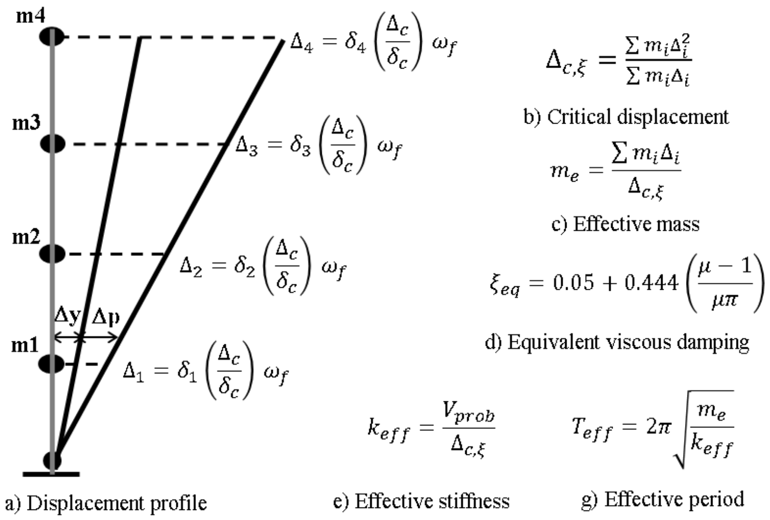

Figure 9: Summary of substitute structure properties for the wall structure example

3.7 Step #6: Compare structural displacement capacity against demand

Determine the structure spectral displacement demand at effective height compared with displacement capacity. The structure spectral demand, $\Delta_{d,\xi,}$ equals to the product of the site hazard spectral displacement, $\delta(T)$ and the spectral reduction factor K_ξ .

K_ξ accounts for the energy dissipation contribution from hysteretic and elastic viscous damping. NZSEE, 2006 and Priestley *et al.*, 2007 recommends the following equation for $K_\xi = (7 / (2+\xi))^{0.5}$ where ξ is the computed equivalent viscous damping for the system.

As the New Zealand Loading Standards NZS1170.5 does not yet incorporate an explicit displacement design spectrum, the pseudo-displacement spectra ordinates, S_d (T) can be generated by dividing the acceleration spectral ordinates, S_a (T) by ω^2, where $\omega = 2\pi/T$ = the angular frequency.

3.8 Step #7: Percentage of new building standard *%NBS* at Ultimate Limit State

The seismic performance of the building is therefore the ratio between the lateral displacement capacity and the expected lateral displacement demand of the equivalent SDOF system:

$$\%NBS = \Delta_{c,}/(\Delta_{d,\xi}\, S_p)$$

where S_p is the structural performance factor from NZS 1170.5:2004.

It is apparent that cantilevered walls system with shear-critical inelastic mechanism, the %NBS can be directly calculated using the ratio of achievable base shear capacity and required base shear demand, as per force-based assessment procedure.

4 ISSUES FOR PRACTICAL IMPLEMENTATION OF DBA

A number of the fundamentals of the displacement-based assessment (DBA) procedure are based on the direct-displacement based design (DDBD) procedure for new buildings (e.g. Priestley *et al.*, 2007). As such, there are a number of limitations of the DBA procedure for realistic buildings of complex configurations which may not have a well-defined deformed shape or ductile inelastic sway mechanisms. Several of these issues are discussed in the following paragraphs with some suggestions to mitigate them.

4.1 Complex system

The DDBD is prescribed for structures with well-defined and regular lateral load resisting systems and principal bracing lines. Therefore, for complex structures with mixed lateral load resisting systems (e.g. concrete wall and steel frames) and for irregular-shaped structures, the current DBA procedure may be limited.

For complex structures with significant 3D effects (bi-directional loading effects, plan and vertical irregularity, semi-rigid diaphragm), the DBA procedure can be used in parallel with limited 3D elastic analysis to form a view of the initial lateral load distribution in the system based on relative stiffness. The elastic analysis result can be used to assess the likely inelastic mechanism and hierarchy of failures. Nonetheless, the principles of DBA as outlined in Section 2 should be applied.

If necessary, hand-calculation based DBA can be used with computer-based non-linear analysis to determine the sequence of inelastic mechanism and the overall non-linear pushover capacity curve of the building. In our opinion, a simplified hand-calculation based DBA as outlined in the preceding sections should be carried out to form an understanding of the likely inelastic mechanism prior to any detailed non-linear analysis of the building. An understanding of the likely inelastic mechanism would allow a simpler non-linear model that focuses on the necessary non-linearity.

Buildings with plan eccentricities but with otherwise good well distributed lateral load resisting systems in both directions will be unduly penalised by standard elastic methods of analysis (eg. elastic modal). It is suggested that provided there is a viable method of taking torsional induced effects perpendicular to the direction of loading, hand methods as outlined above have a better chance of predicting seismic behaviour than a complicated elastic based computer analysis.

4.2 Coupling beam contribution in reinforced concrete wall system

In many existing reinforced concrete wall systems, unintentional coupling elements can provide significant lateral strength and stiffness to the overall system. One approach is to ignore the contribution of the coupling beams and assume a flexural or shear "hinge" would form at the ends of the coupling beams. The second approach is to explicitly calculate the overturning moment resistance from coupling beams and the associated dispalcement limits based on the available detailing.

Priestley *et al.* (2007) provides some guidance on the estimation of yield and ultimate displacements at the effective height for system with coupling beams. The overturning moment resistance from coupling beam, $M_{OTM,cb}$ can be calculated using first principles (as shown in Figure 10 below), where $M_{OTM,cb} = \Sigma V_{cb,i} \times L_{span.cb}$ and $V_{cb,i}$ is the limiting capacity of the coupling beam from either flexural or shear behaviour.

We note that failure of any or all of the coupling beams may not necessarily be indicative of the full capacity of the structure.

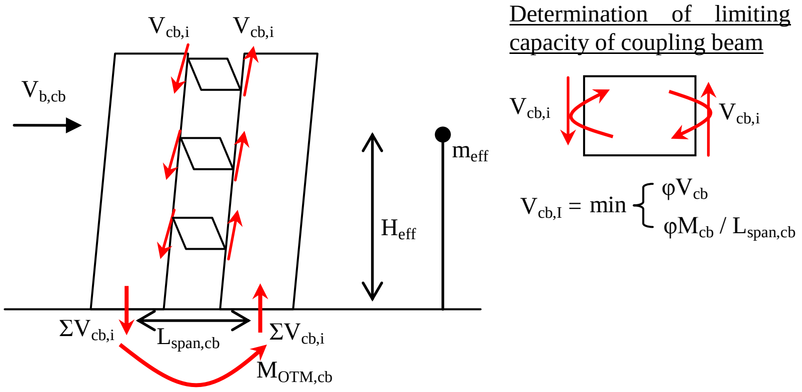

Figure 10: Example of calculation of coupling beams contribution in RC wall system.

4.3 Building without effective floor and roof diaphragms

The DBA procedure outlined in the preceding section relies on the assumption of rigid diaphragm to distribute the seismic inertia to all lateral load resisting elements. Separate analysis may be required to check the diaphragm in-plane capacity to distribute the lateral loads.

For some structure without an effective diaphragm (e.g. Figure 11), each component will need to be assessed as an independent system with tributary mass being the contributing seismic mass. The limiting displacement of the overall system will be governed by the limiting component's displacement capacity and the displacement flexibility of the diaphragm.

Figure 11: Example of flexible roof diaphragm.

4.4 Relationship between local inelastic mechanism and the displacement capacity\

By identifying the weakest link within the system, the plastic mechanisms can be identified and based on the ductility and deformability of the plastic mechanisms, the likely inelastic mechanism and collapse mode can be determined. It is noted that if the lateral load path is not well assessed, then any analysis method will be futile.

If the governing inelastic mechanism comprises non-ductile or brittle failure mechanism such as buckling of steel braces, reinforced concrete beam-column joint shear failure, foundation bearing failure, timber-ply nailed wall failure, the associated displacement and ductility capacity of the inelastic mechanism may be difficult to assessed with great accuracy.

The NZSEE (2006) guidelines provide guidance on the ultimate displacement capacity for the identified inelastic mechanism (e.g. Figure 12 for reinforced concrete frames). ASCE-41 (2007) provides some additional guidance on the achievable displacement capacity for various types of components and inelastic mechanisms. Further research is required to allow the adoption of the ASCE-41 acceptance criteria for New Zealand applications but the material contained within ASCE-41 is nevertheless very valuable in assessing the capability and capacity of structural systems.

The NZSEE (2006) guidelines and Priestley (1996) propose a "Sway Index" calculation to assess the global sway mechanism (beam-sway versus column sway) for a non-ductile reinforced concrete frames. Alternatively a more refined hierarchy of strength analysis can be used to determine the inelastic mechanism of the beam-column joint connection at every floor level.

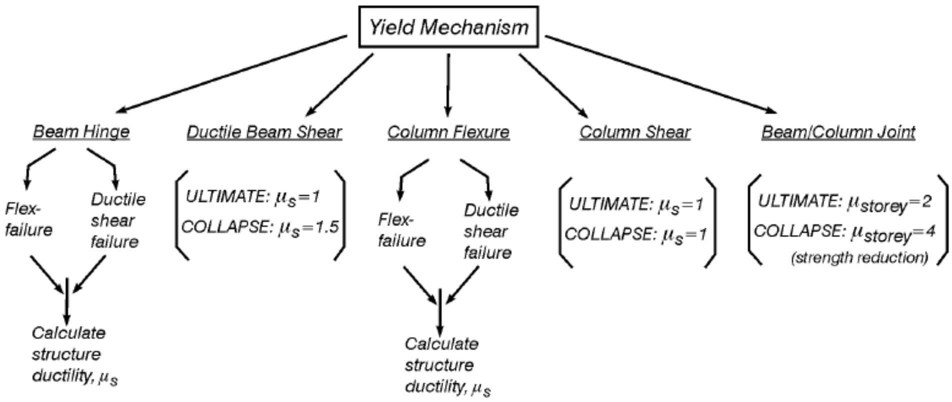

Figure 12: Displacement capacity for various inelastic mechanisms of reinforced concrete frames as recommended by NZSEE (2006).

4.5 Deformed shape

The building's probable inelastic deformed shape profiles depends on the critical local inelastic mechanism and the possible load redistribution as the building responds in the non-linear range. The existing literature (Priestley *et al.*, 2007) provides some guidance on the deformed shape profiles for regular cantilevered walls, moment-resisting frames and steel braced frames. For unknown or mixed inelastic mechanism (as illustrated in Figure 13c), the estimation of the probable inelastic deformed shape profiles is a challenging issue for the DBA procedure. Figure 13-bottom illustrates simplistic upper and lower bounds of the deformation capacity by assuming a beam-sway and a column-sway (soft-storey) profile respectively.

As discussed in Section 4.1, for complex system, the DBA may be very conservative and may need to be used in combination with some computer 3D analysis to form a better understanding of the load distribution. It may be necessary to temper the assessment with some engineering judgement of the assessed collapse mechanism.

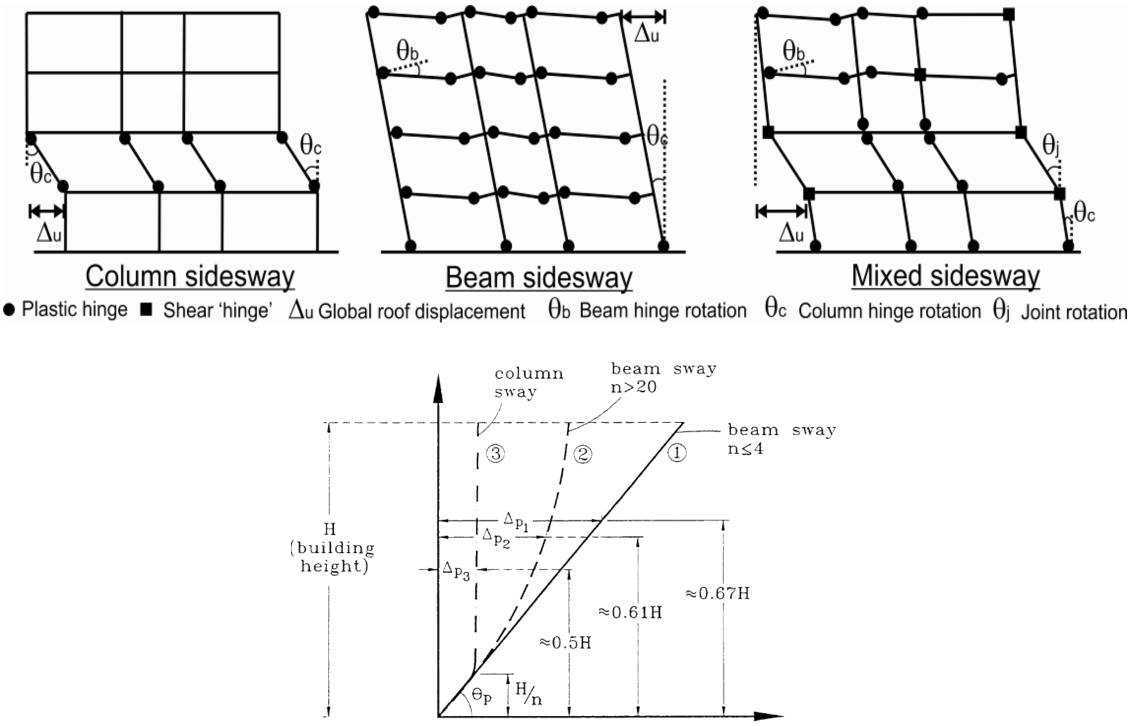

Figure 13: Inelastic mechanism for RC frame building: TOP: a) column sidesway; b) beam sidesway; c) mixed sidesway. BELOW: Inelastic deformed shape profiles (from Priestley, 1996).

4.6 Damping and Reduction of Seismic Demand

Priestley *et al.* (2007) provide some guidelines on the damping and reduction of spectral demand for seismic assessment. Spectra demand reduction factor for equivalent damping of the system K_ξ , is given by the following equation:

$K_\xi = (7 / (2+\xi_{sys}))^{0.5}$ (far-field earthquake e.g. Auckland)

$K_\xi = (7 / (2+\xi_{sys}))^{0.25}$ (near-fault earthquake e.g. Wellington)

where ξ_{sys} = equivalent viscous damping factor for the system (in each principal direction)

Several methods can be used to estimate the ξ_{sys} values based on achieved structural ductility μ and the expected inelastic mechanism. However, it is difficult to extrapolate the local ductility capacity to the global energy dissipation, ξ_{sys} for mixed-inelastic mechanism. We have adopted a base-shear contribution weighted average approach to compute the achieved global energy dissipation, ξ_{sys} for mixed-inelastic mechanism (e.g. steel braced frame coupled with concrete shear walls).

4.7 Treatment of uncertainties

There are significant uncertainties in both the displacement and capacity of a structure, and the displacement demand from the seismic hazard. Some researchers have adopted a probabilistic approach to account for these uncertainties on the seismic risk assessment (e.g. Sullivan and Calvi, 2011) which are more generally mathematical and theoretical.

In practice, we have adopted a more pragmatic bounded sensitivity analysis in which reasonable upper and lower bounds of key variables are assessed as part of a sensitivity analysis to form a view of the range of expected seismic performance. Key variables to be considered in DBA are as discussed in Section 3.

As such, the seismic assessment result is generally reported as a range to reflect the uncertainties in the inputs and assumptions. It also allow the engineer to determine which assumption that can be refined with further analysis, intrusive inspection, material testing etc. in order to best reduce uncertainties in the assessment result.

4.8 Other issues

There are several other issues with the DBA procedure that requires further thinking and consideration which will be excluded from the current paper due to space constraints:

- *Higher modes effect.* DBA relies heavily on the first mode behaviour and some modifications will be needed to account for higher mode effects. Priestley *et al.* (2007) for example, provides a recommendation to amplify the shear demand of cantilevered concrete shear walls for higher mode effects.
- *Performance*-based seismic assessment: DBA allows a relatively quick assessment of the building performance at various seismic demand levels. Further guidelines on the performance limit states corresponding to performance objectives will be required.
- Comparisons against %*NBS* require careful consideration of the value of S_p. Although S_p can be applied to either the capacity or the demand it is generally applied to the demand. This means that potentially brittle mechanisms (and shear limited mechanisms are considered to be brittle) should be compared against a demand calculated with S_p no less than appropriate for μ =1.25. A more logical approach might be to factor up the capacities - flexural or shear by the reciprocal of the appropriate value of S_p.

5 CONCLUSIONS

From our experience, the Displacement-Based seismic Assessment (DBA) methodology provides a better understanding of the likely behaviour and performance of the structure in an earthquake. The deformation capacity and seismic deformation demand are assessed and quantified on a component-by-component and a global system basis. This is a fundamental departure from assessing lateral strength demand-to-capacity which is based on an elastic period and an assumed structural ductility reduction factor for the structures.

The paper outlines the DBA procedure that we have adopted for the seismic assessment of a number of real complex buildings. In the process, we have found a number of challenges and issues with the current DBA procedure which requires further thinking and consideration. There is an opportunity to align the direct DBA procedure as outlined herein and elsewhere (Priestley, 1996; NZSEE, 2006, EAG 2012) with the computer-analysis based DBA procedure (e.g. ASCE-41, 2007) as there are some similarities in the assessment of the displacement capacities of various inelastic mechanisms.

In our opinion, the DBA procedure whilst simplistic provides a rational methodology to form a good understanding of the probable inelastic mechanism and the critical load path, and therefore a better seismic assessment outcome that necessarily possible from complex, elastic based computer analysis. It is a myth that a refinement of the analysis with computer-aided simulation will necessarily improve the end result.

REFERENCES

American Society of Civil Engineers (ASCE) Standard ASCE/SEI 41-06 *Seismic Rehabilitation of Existing Buildings*. 2007 USA.

Blume, J. A., Newmark, N. M., and Corning, L. (1961). Design of multistory reinforced concrete buildings for earthquake motions, Portland Cement Association, Chicago, IL.

EAG (2012). *Guidance on detailed engineering evaluation of earthquake affected non-residential buildings in Canterbury. Part 3.9 - Reinforced Concrete Wall Buildings. Revision 0.5*, Engineering Advisory Group (EAG) - Commercial, Christchurch.

Kam, W.Y., Pampanin.S., Elwood, K. Seismic performance of reinforced concrete buildings in the 22 February Christchurch (Lyttelton) Earthquake. *Bull. of NZSEE*. Vol **44** (4). Dec 2011.

Kam, W.Y., *Selective weakening and posttensioning for the seismic retrofit of non-ductile RC frames*. PhD Dissertation. University of Canterbury. 2011 Christchurch.

New Zealand Society for Earthquake Engineering (NZSEE). *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*. New Zealand June 2006.

Pampanin, S., Kam, W. Y., Tasligedik, A. S., Quintana-Gallo, P., and Akguzel, U. Considerations on the seismic performance of pre-1970s RC buildings in the Christchurch CBD during the 4th Sept 2010 Canterbury earthquake: was that really a big one? *Proc., 9th Pacific Conf. on Earthquake Eng.* 2011 Auckland.

Priestley M.J.N., Calvi G.M., Kolwasky M.J. *Displacement-based seismic design of structures*. IUSSS Press. 2007 Pavia, Italy.

Priestley M.J.N., Displacement-based seismic assessment of existing reinforced concrete buildings. *Bull. of NZSEE*. Vol **29** (4). Dec 1996.

Sullivan, T., Calvi G.M. Considerations for the seismic assessment of buildings using the direct displacement-based assessment approach. *Proc. Of Italian Annual National Conference (ANIDIS 2011).* Available online at: http://ww2.integer.it/Web_1/database_locale/ATTI%20ANIDIS_2011/Papers/SS04/298%20996-2044-1-RV.pdf