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Displacement-based retrofit of existing reinforced concrete frames using alternative steel brace systems

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ABSTRACT

This paper presents an application of the displacement-based method for selecting suitable retrofit strategies for an RC frame building. An overview of different bracing technology is presented. By targeting the ultimate displacement depending on the failure mechanism of existing frames, a displacement-based retrofit of brace systems is implemented. A detailed seismic assessment (DSA) of a pre-1970 four-story RC case study frame is performed using SLaMA (Simple Lateral Mechanism Analysis) and Displacement-based assessment methods. The results indicate that the frame is classified as potentially earthquake-prone, showing joint shear failure mechanism with a seismic capacity of 46%NBS (New Building Standard). X-braced, V-braced (Chevron) concentric braces, and Buckling-Restrained braces (BRB) are used for retrofitting the building. After the retrofit, the failure mechanism of the frame is changed and the seismic capacity of the building exceeds 100%NBS. By evaluating the local and global behaviour of the retrofitted frames, the advantages and disadvantages of alternative braces systems are derived. The results indicate the feasibility and efficiency of Displacement-based retrofit design procedure for concrete frame buildings.

1 INTRODUCTION

Existing RC buildings were constructed prior to the early 1970s and designed according to old seismic codes, underlined a significant vulnerability in high seismic events. A lack of inelastic design consideration has generally led to poor detailing in these types of buildings. Insufficient joint transverse reinforcements, use of plain round bars, hooked end anchorages, inadequate development lengths and insufficient lap splices of

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column and beam reinforcement were common all over the structures. The behaviour is mainly prevailed by brittle local failure mechanisms and generally leading to the possible collapse of the lower or ground floors (Pampanin, 2006). Thus, their seismic performance needs to be carefully assessed and appropriate retrofit intervention chosen.

Steel brace systems are extensively utilised in steel structures and several studies have been carried out to understand the inelastic response of steel bracing members under cyclic loading (Tremblay, 2002). However, the use of steel bracing systems in seismic retrofitting of existing RC buildings is also an attractive technique, as it is often characterised by high architectural and functional compatibility with respect to the original purposes of the existing structure. Typically, a force-based design is used for the retrofit design of the braced frame. However, more recently procedures based on the displacement-based design have been developed such as Vulcano and Mazza (2002) and Bergami and Nuti (2013). In Vulcano and Mazza (2002) the authors suggest a distribution of the braces finalized to maintain strength and stiffness distribution of the original structure and consequently, as the authors suggest, guarantee that modal shapes don’t change after the insertion of the braces. The procedure by Bergami and Nuti (2013) is based on the iterative capacity spectrum method. The needed global energy dissipation due to bracing is estimated as the difference between total damping and hysteretic damping of the structure without braces.

In this paper, a direct displacement-based retrofit design procedure is presented as an effective tool for retrofit of poorly detailed (pre-1970’s) frame building using steel brace systems. An example of a design procedure following the detailed seismic assessment (DSA) of a case study frame is also given to explain design steps and compare the features of selected steel braces for retrofit.

2 PERFORMANCE ASSESSMENT OF EXISTING BUILDING

2.1 Assessment methodologies

Force-based methodology traditionally has been used for seismic assessment of existing buildings because most practitioners are familiar with the background of the force-based seismic design. However, it has been recognised that there is a direct relationship between a displacement imposed on the structure by an external force, due to seismic events or gravity, and structural elements damage (e.g. Priestley et al., 2007). Moreover, for assessing some of the non-structural elements (e.g. partitions) displacement-based parameters such as the inter-storey drift is more efficient.

In New Zealand, the “Seismic Assessment of Existing Building Technical Guidelines” (NZSEE (2017)) is used for assessment of existing buildings. The consequence of the procedure is the appraisal of a seismic rating based on the %NBS (New Building Standard), or Capacity/Demand ratio (Figure 1), i.e. the capacity, in terms of forces and displacements, of the building divided by the demand, in terms of spectral ordinates, acceleration and displacements, of an equivalent newly designed building on the same site.

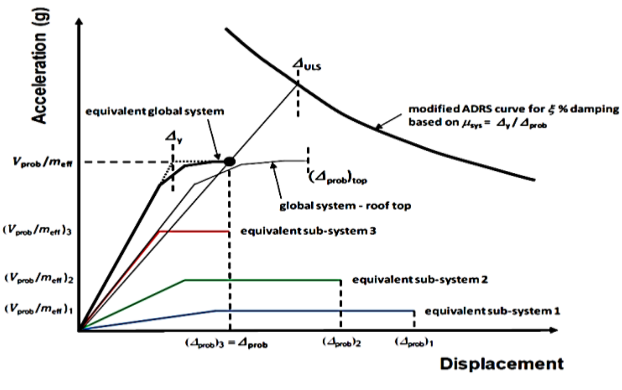


Figure 1: Evaluation of the %NBS as Capacity/Demand ratio within an ADRS Domain (NZSEE2017)

2.2 Performance of pre-1970's buildings

One of the main weaknesses of older beam-column joints in pre-1970's buildings was the insufficient joint shear reinforcement. Actually, in older construction design codes beam-column joints acted either as construction joints or as part of columns (NZSEE2017). Figure 6 indicates the schematic sketch of joint transverse reinforcement in pre-1970s buildings. It can be seen that in Figure 2a and Figure 2b the joint was neglected in design or considered as a construction joint and in Figure 2c, Figure 2d and Figure 2e the joints treated as part of the column, thus the number of joint stirrups depended on column stirrup spacing and beam depth.

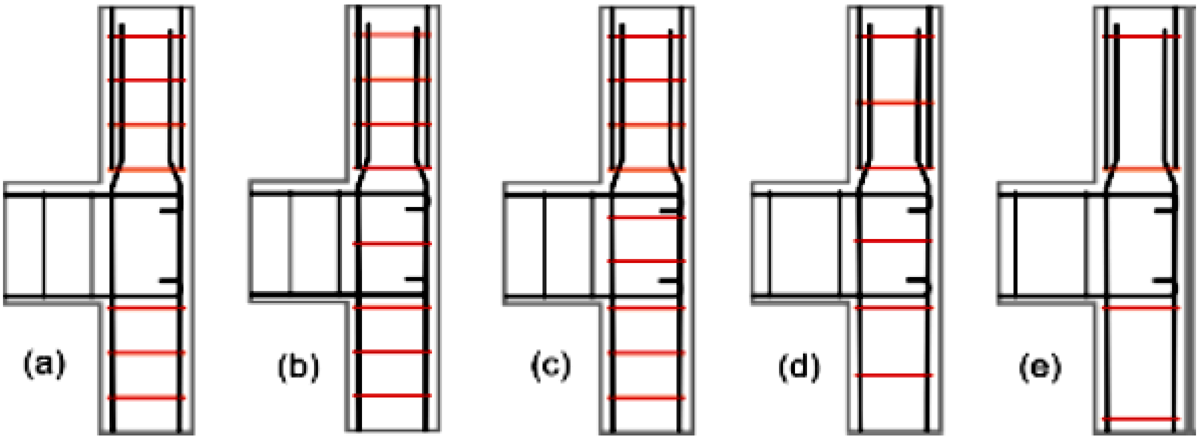
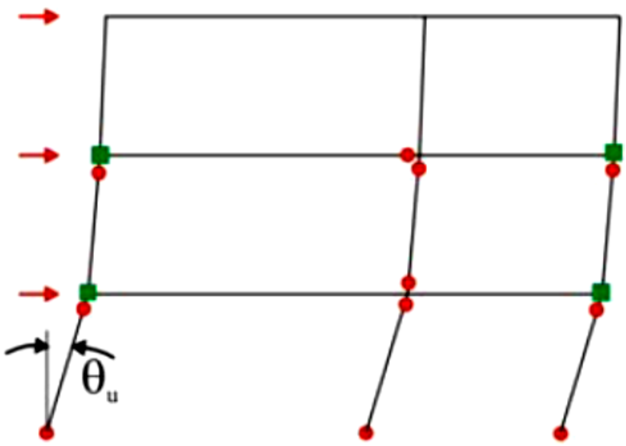
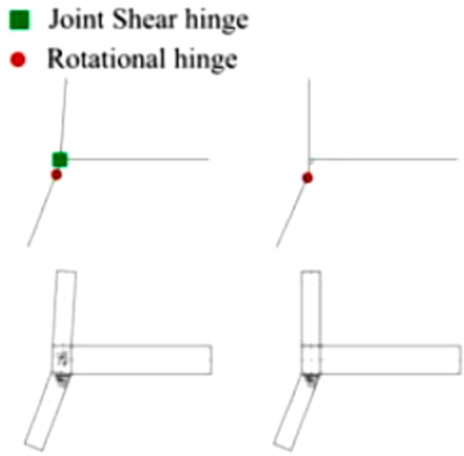


Figure 2: Schematic view of joints reinforcement in pre-1970s buildings (NZSEE2017)

Furthermore, lapping inside the plastic hinge zone and inadequate development lengths are other common issues for these types of buildings' elements. As a result, low ductility capacity of these components is expected due to bar slip, poor confinement, buckling of longitudinal reinforcement and joint shear failure.

In order to define the local mechanism, a comparative strength assessment allows a hierarchy of strength and sequence of events to be evaluated within the beam-column joint subassembly (Pampanin, 2006). The capacity of each structural element (beam, column, and joint) is converted to a common-unit, such as an "equivalent column moment", allowing a direct strength comparison to be made within a moment-axial performance domain (M-N interaction diagram). Both flexure and shear failure modes can be considered as well as strength degradation effects. Figure 3a shows a soft-storey "global mechanism" for a 3-storey building due to the independent local mechanism as shown in Figure 3b (Marriott et al., 2007).





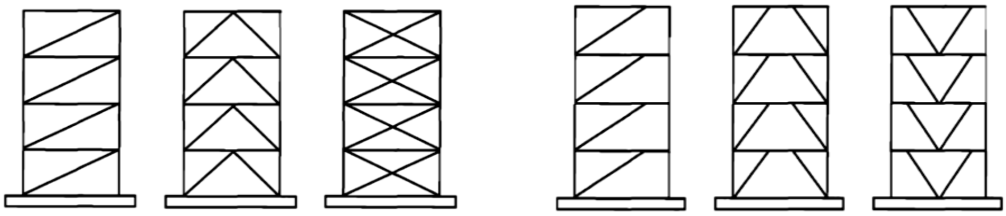
a) Global failure mechanism *b) Local failure mechanism*

Figure 3: Global and local failure mechanisms (Marriott et al., 2007)

3 STEEL BRACES FOR RETROFIT OF RC BUILDINGS

3.1 Non-dissipative steel braces

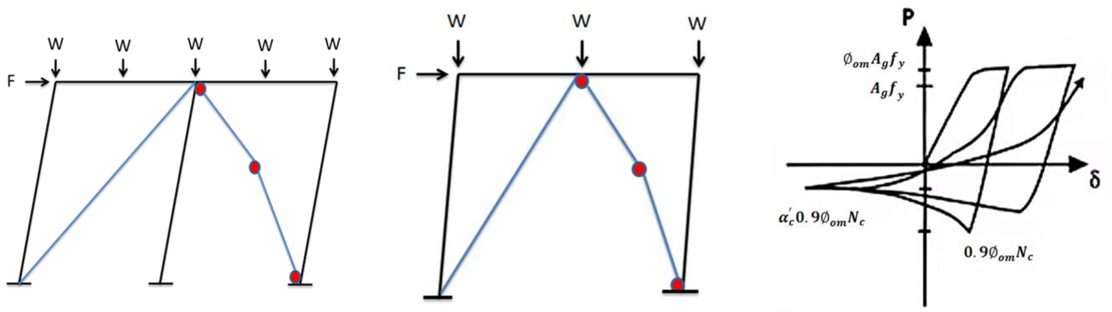
Steel braces are effective structural systems for buildings subjected to seismic or wind lateral loadings. Steel braced frames are generally divided in two main categories: Concentrically Braced Frame (CBF), where there is no eccentricity of connection, and the elastic response may be described by truss action, and Eccentric Braced Frame (EBF), where the eccentricity of connection is deliberately induced to provide a displacement mechanism which involves a mix of truss and flexural action (Figure 4a and Figure 4b). Eccentric brace system is not the concern of this study, but rather a concentric bracing and Buckling-Restrained Braces (BRBs).



a) Concentrically braced frame *b) Eccentrically braced frame*

Figure 4: Different types of Braced frame systems

Stability of the concentric brace systems to earthquake ground motions relies on the capacity of the bracing members to undergo several cycles of inelastic deformations including stretching in tension and buckling in compression. A critical loading condition exists in tension-compression bracing when the sum of the horizontal components of the compression and tension braces reaches its maximum value. The buckling of one or more braces is considered a first non-linear event. Due to strength degradation with axial deformation and the number of cycles in compression brace, the most sever condition typically occurs when the tension brace yields just after the compression brace has buckled. Figure 5(a) and Figure 5(b) show the sketch of Diagonal and Chevron Concentrically braces respectively. Also, the schematic hysteresis behaviour of the system is indicated in Figure 5(c).



(a) *(b)* *(c)*

Figure 5: (a) Diagonal concentric braced-frame. (b) Chevron concentric braced-frame. (c) The hysteretic behaviour of the non-dissipative concentrically braced frame

3.2 Dissipative buckling-restrained braces (BRBs)

To exploit the ultimate capacity of the braces in both tension and compression under seismic excitations, conventional steel concentric braces are improved by preventing them from buckling in compression. The retrofitting systems using these types of braces are called steel BRB (Figure 6) systems. In fact, it has been recognised that conventional braced frames have exhibited certain unfavourable modes of behaviour in past earthquakes, such as a connection fracture, degradation of stiffness and strength under cyclic loading, and excessive bending of beams in chevron-braced frames. In BRBs the compression buckling failure mode in intermediate and slender compression elements can be eliminated to avoid degradation of the stiffness of the steel braces. BRBs achieve stable, balanced hysteretic behaviour by accommodating ductile compression yielding before the onset of buckling. Structural systems designed with these braces can achieve performance superior to that of CBFs in many aspects such as a comparable cost and capacity of energy dissipation.

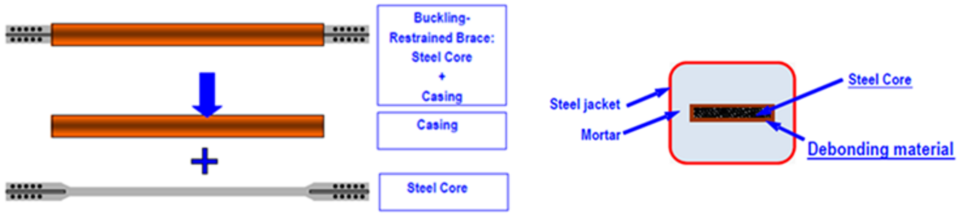


Figure 6: The schematic view of the Bucklin-restrained braces

The hysteretic behaviour of a BRB is stable and the brace is designed to resist the axial tension or compression force without global flexural buckling and the brace exhibits the same behaviour both in tension and compression, as shown in Figure 7.

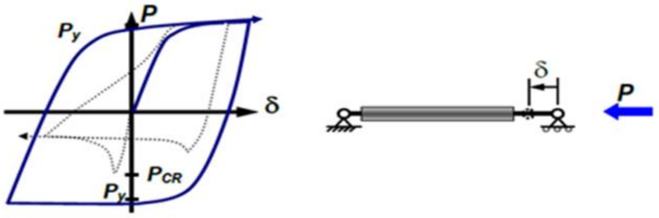
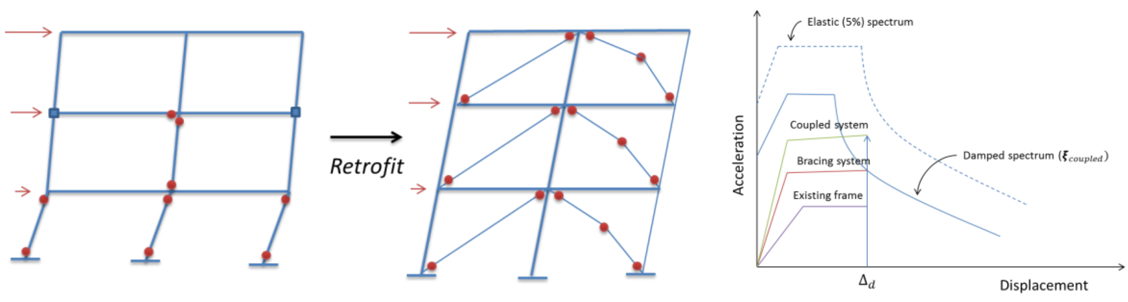


Figure 7: The hysteretic behaviour of a Bucklin-Restrained Braces

3.3 Application of steel braces for retrofit

Steel bracing retrofit systems have the advantage of distributing the inelastic demand up the height of the structure, increasing the total strength, stiffness and energy dissipation of the existing bare frame. The key parameters of retrofit intervention with steel braces are outlined in Figure 8 for the prototype building. Firstly, the brittle failure mechanism of the frame can be eliminated by setting the design drift. Secondly, the desired deformed shape and the required capacity of the frame can be obtained. Finally, within force-displacement in capacity spectrum approach, where the capacity is given by force-displacement and the demand is given by Acceleration-Displacement Response Spectrum (ADRS), the behaviour of the retrofitted frame is evaluated.



(a) *(b)* *(c)*

Figure 8: (a) Prototype bare frame. (b) Retrofitted frame. (c) Acceleration-Displacement Response Spectrum (ADRS) curve of prototype frame before and after retrofit with a non-dissipative bracing system

4 DISPLACEMENT-BASED RETROFIT METHODOLOGY

A displacement-based retrofit design procedure is defined as an extension of the Displacement Based Design Procedure (DDBD) proposed by Priestley and Kowalsky (2000). The main purpose of this method is preventing a building from reaching and exceeding the failure deformation. Following the displacement based assessment, the ultimate displacement of the building is obtaining and is considered as a target displacement. Figure 9 illustrates the basic steps within the procedure.

Step 1): A target displacement is defined based on the result of the assessment of the existing building and permissible deformation limits of critical components i.e. joint rotation.

Step 2): An equivalent single degree of freedom (SDOF) system of the retrofitted braced frame is defined and the effective height and the mass are obtained.

Step 3): The yield drift of braced frame, using Equation 1 proposed by (Priestley et al., 2007), is calculated and the system displacement ductility is computed.

$$\theta_y = \varepsilon_y \frac{L_{bay}}{H_s} \qquad (1)$$

where ε_y = strain of steel; L_{bay} = length of frame bay; H_s = storey height.

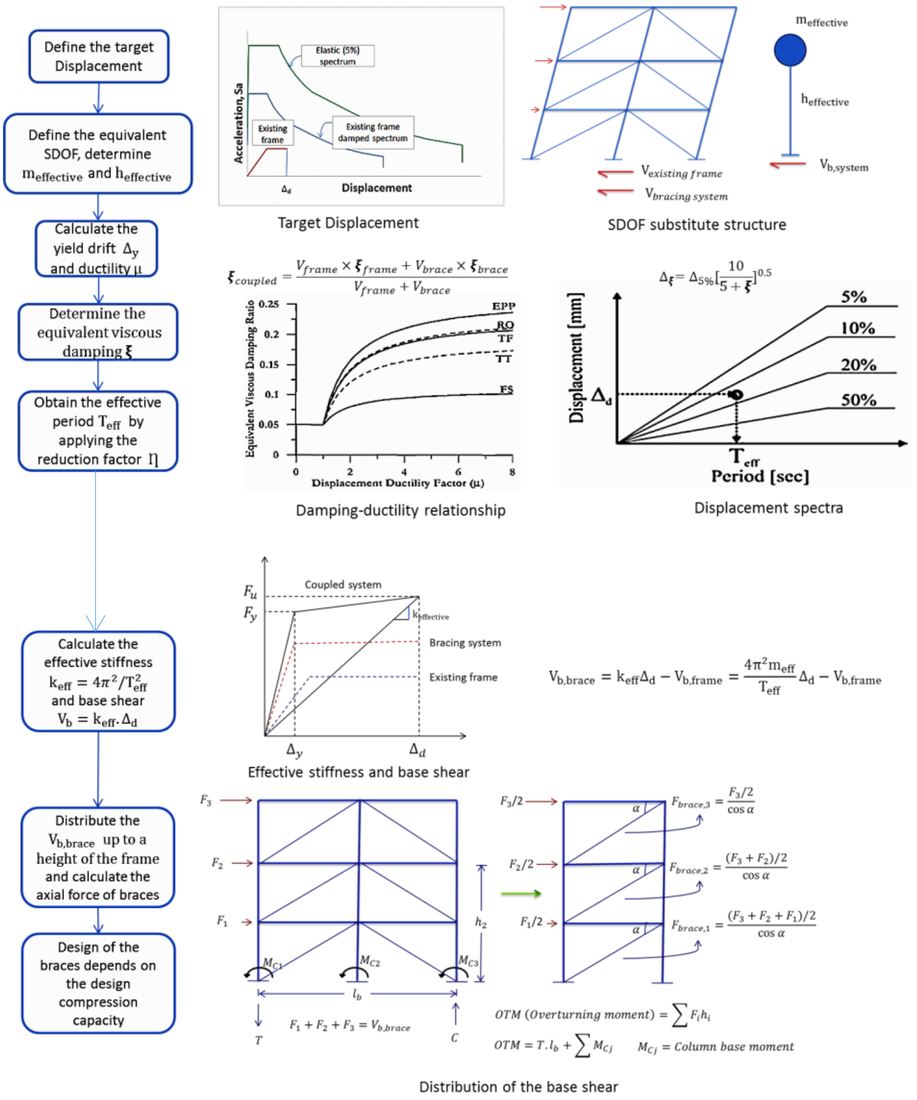


Figure 9: Displacement-based retrofit design procedure

Step 4): The system damping is evaluated based on weighting in proportion to the base shear carried by a bare frame and braces. The equivalent viscous damping for the braced frame can be calculated following the recommendations and equations in Priestley et al. (2007). A bounded Ramberg Osgood rule is used and the equivalent viscous damping is obtained from Figure 10.

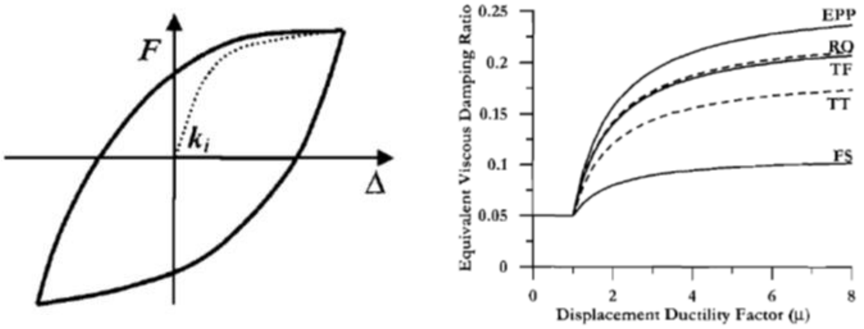


Figure 10: a) Ramberg Osgood hysteresis rule, b) damping-ductility relationship (Priestley et al., 2007)

Furthermore, the basis of substitute-structure analysis by Jacobsen (1960) was built on the energy absorbed by the hysteretic steady-state cyclic response to a given displacement level to the equivalent viscous damping of the substituted structure (Figure 11) and has the outcome of the Equation 2 for calculating the equivalent viscous damping.

$$\xi_{hyst} = \frac{\Delta_h}{2\pi F_m \Delta_m} \quad (2)$$

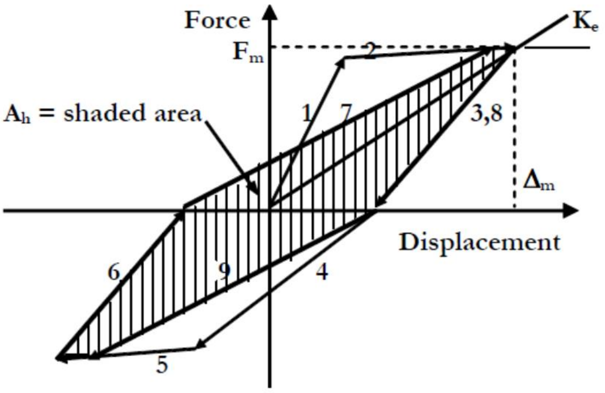


Figure 11: Hysteretic area for damping calculation (Priestley et al., 2007)

Step 5): The effective period is obtained using a displacement response spectrum by applying the reduction factor $R_\xi = [\frac{10}{5+\xi}]^{0.5}$ to the 5% damped spectrum as adopted in Eurocode 8 (2005).

Step 6): The total required base shear of the braced frame is calculated as $V_{b,total} = k_{eff}.\Delta_d$. The required base shear for the brace system based on an analytical approach is obtained as $V_{b,brace} = k_{eff}\Delta_d - V_{b,frame}$.

Step 7): Distribute the $V_{b,brace}$ up the height of the frame and calculate the axial force of the braces based on equilibrium consideration. Finally, the required area for the braces can be obtained, depending on the design compression capacity of the braces.

5 EXAMPLE OF THE PROCEDURE ON THE CASE STUDY FRAME

This section briefly summaries an example of the proposed retrofit design procedure on selected case study frame. The frame is part of four storey reinforced concrete office building (Grenadier House) that was located in Christchurch and due to unrepairable damage, in 2011 earthquake, was demolished.

5.1 Seismic assessment of the case study frame

Detailed Seismic Assessment (DSA) of the case study frame is implemented by a SLaMA and Displacement-based assessment methodology. The purpose of this assessment is to determine the current seismic capacity of the building in terms of % New Building Standard (%NBS) and if the building is classified as being earthquake-prone in accordance with the Building Act 2004.

Information obtained from a review of historic structural drawings indicates that the subject building was designed and constructed circa the 1960s, prior to the introduction of modern seismic codes. Seismic load in the transverse direction is resisted by reinforced concrete frames, extending to level 4 of the building. The external frame in the transverse direction is considered as a case study frame (Figure 12).

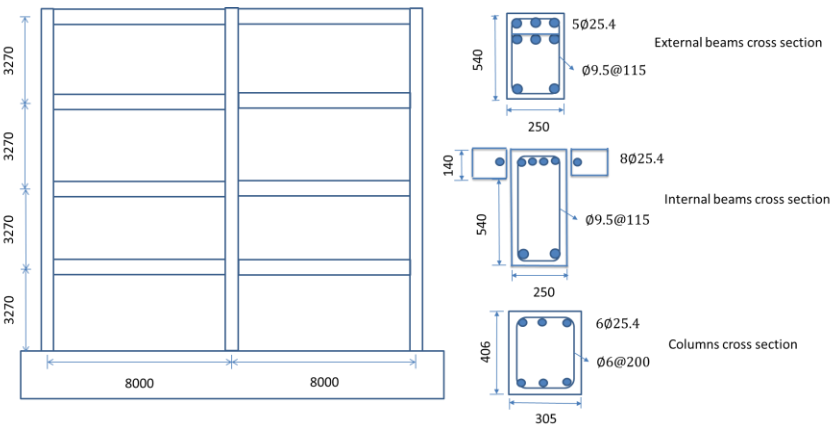
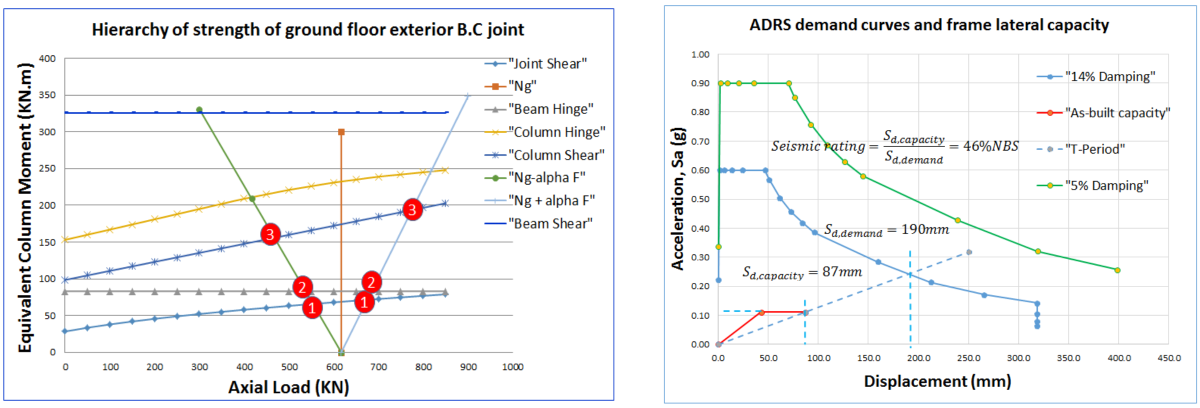


Figure 12: Cross section of the case study frame

The assessment and hierarchy of strength results indicate that the failure mechanism is a mixed sway mechanism with the brittle failure of exterior and interior beam-column joints. A yield drift of $\theta_y = 0.5\%$ and ultimate drift of $\theta_u = 1\%$ is considered according to the beam-column joint limit state (Pampanin et al., 2003). The frame has a seismic rating of 42% NBS and is considered earthquake prone. The results of the assessment are shown in Figure 13a and Figure 13b.



(a) *(b)*

Figure 13: (a) Example of the hierarchy of strength for an external beam-column joint. (b) ADRS demand curves and frame lateral capacity

5.2 Retrofit design of bracing systems and results

For the retrofit of the case study frame, two types of braces are considered. The non-dissipative Concentric Braced Frame (CBF) in a two configuration (Diagonal and Chevron) and the Buckling-Restrained Braces (BRB). The design drift is considered $\theta_d = 1\%$ corresponding to the ultimate drift of the existing frame for preventing a frame joint shear failure and is the same for all the systems and configurations. The equivalent SDOF system features (effective mass, effective height and target displacement) is the same according to DDBD equation (Priestley et al., 2007)

The first important difference starts with defining the yield drift. For the non-dissipative CBF systems, which are working in compression buckling and yielding in tension, the yield drift is calculated according to Equation 1. The strain of the steel is $\varepsilon_y = f_y/E$. The type of the material (e.g. $f_y = 250\ MPa$ or $f_y = 350\ MPa$) that is been using for brace profile, affects the amount of strain. On the other hand, the slenderness limitation should be considered in the selection of the steel material. This is especially more important when the length of the bay is long (e.g. the case study frame bay length is 8m). It can be seen from Figure 14, for the Chevron CBF systems the slenderness limitation is less effective due to the shorter length of braces when comparing to the Diagonal CBF systems.

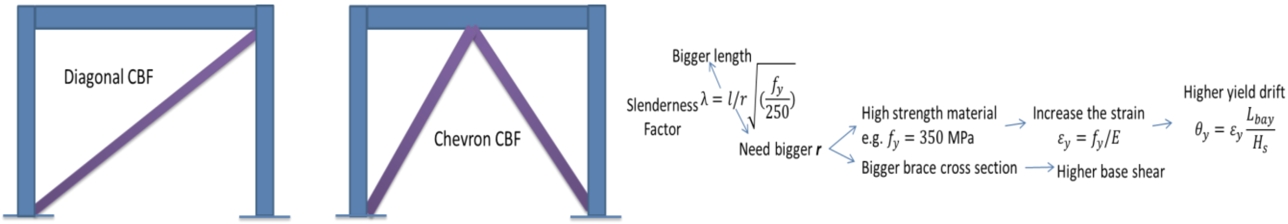


Figure 14: Comparison between the Diagonal and the Chevron CBF systems

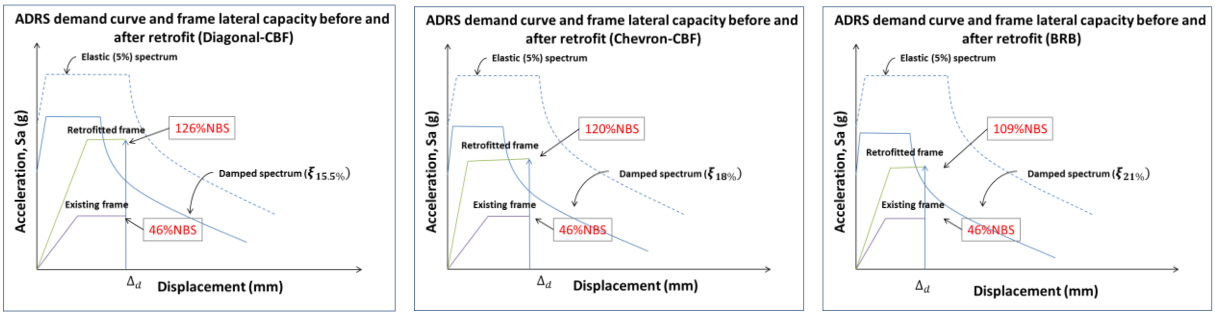
The Buckling-restrained braces do not have a buckling behaviour in compression and the slenderness limitation does not affect the calculation of the yield drift.

Because of the same design displacement for all type of the retrofitting systems, the lower yield drift results in higher ductility ($\mu = \Delta_d/\Delta_y$) and higher equivalent viscous damping (Figure 14). Table 1 summarises the numerical results of the DDBD retrofit examples.

Table 1: Summary of the displacement-based retrofit design examples.

	θ_d	Δ_d (mm)	h_e (mm)	m_e (ton)	θ_y	Δ_y (mm)	μ	ξ_e	V_b (kN)
Diagonal CBF	1%	87	8700	270	0.425%	37	2.35	15.5%	1378
Chevron CBF	1%	87	8700	270	0.3%	26	3.35	18%	1185
BRB Frame	1%	87	8700	270	0.34%	29	3	21%	946

The ADRS curves of the retrofitted frames are shown in Figure 15a, Figure 15b and Figure 15c for the Diagonal CBF, Chevron CBF, and BRB systems respectively. Also, the obtained base shears of retrofitted frames are 1378kN, 1185kN and 946kN for the Diagonal CBF, Chevron CBF, and BRB systems respectively.



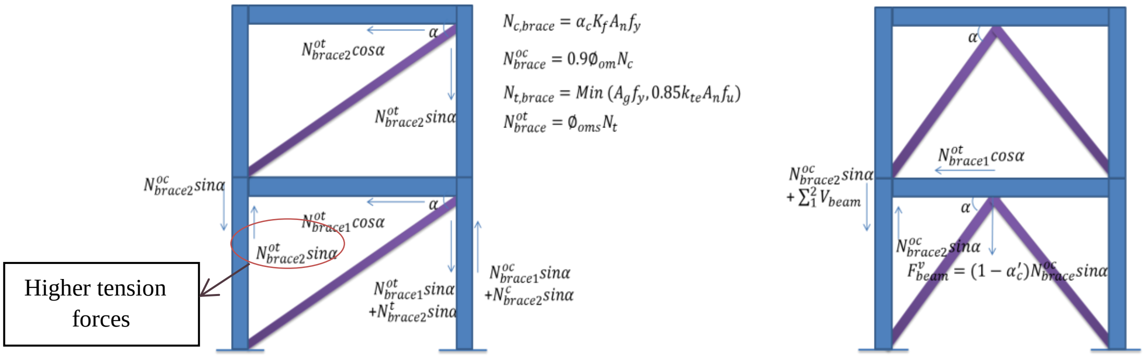
(a) (b) (c)

Figure 15: (a) ADRS curve for the Diagonal concentric braced-frame. (b) ADRS curve for the Chevron concentric braced-frame. (c) ADRS curve for the Buckling-restrained braced-frame

It can be seen from ADRS curves that all of these three retrofit methods (Diagonal CBF, Chevron CBF, and BRB) provide more than 100%NBS. However, the internal action that is imposed from these systems is different and the frame may need local retrofitting intervention as is discussed in the following paragraph.

Figure 16a and Figure 16b demonstrate the load path of Diagonal-CBF and Chevron-CBF systems respectively. For Chevron-CBFs the net vertical seismic force should be considered at the brace/collector beam joint due to compression brace buckling. Moreover, the collector beams must be checked for bending moment due to net vertical force and also the combination of bending and axial load (imposed from braces).

For both systems, seismic axial forces on columns are determined on the basis that the braces at the lowest level affecting the column under consideration achieve their over-strength capacity (N_{brace}^{oc} and N_{brace}^{ot}), while all braces above that level are at their design capacity ($N_{c,brace}$ and $N_{t,brace}$) (NZS 3404).



(a) (b)

Figure 16: (a) Load path of the Diagonal-CBF. (b) Load path of the Chevron-CBF

However, in the left column of the Diagonal braced frame, as recommended on NZS 3404 (2007) and HERA Report R4-76 (Feeney and Clifton, 1995), for calculation of tension force, the over-strength tension capacity of the upper floor is used, while in the Chevron-braced frame the tension force on the same column is calculated depends on the over-strength compression capacity of the upper floor braces.

The tensile force is consequently higher (e.g. 1173kN > 903kN (Column tensile capacity)) in the first-floor column of the Diagonal braced frame and local retrofitting of the column (e.g. steel jacketing) might be required.

The distribution of the load path for the BRB system is similar to the Diagonal-CBF system with one main difference that is, due to the absence of slenderness limitation in the BRB, the lower strength type of material can be used (e.g. f_y = 280 MPa instead of f_y = 350 MPa) with a smaller cross section, and the braces is imposing smaller forces on beams and columns.

6 CONCLUSIONS

In this work, a displacement-based retrofit design procedure for existing concrete frames based on the use of steel bracing systems has been presented. The steel braces can add lateral strength and damping to the structural system while controlling the damage in the as-built frame. Global inter-storey drifts for the frame can be limited, in addition to having the inelastic demand distributed up the height of the frame. The displacement-based retrofit procedure targets a pre-defined displacement, limiting deformations within critical structural elements such as joint rotations or member curvatures.

The Concentric-braced frame with two configurations (Diagonal and Chevron) and Buckling-restrained braces have been used for the retrofit of a case study frame. The results show the effectiveness of the procedure through a non-linear static capacity curve (pushover) within the ADRS domain, and how the global mechanism of the case study frame is changing by different retrofit intervention. Moreover, the investigation of the internal actions for all retrofitted frames consequences how the buckling behaviour of non-dissipative braces affects the amount of internal forces and increase the dimension of braces due to the slenderness limitation. Furthermore, the proposed design methodology is adaptable to many other applications rather than braces and can be considered as a general design approach.

7 ACKNOWLEDGEMENTS

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