THE LIMITATIONS AND PERFORMANCES OF DIFFERENT DISPLACEMENT BASED DESIGN METHODS

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Displacement based design (DBD) methods are emerging as the latest tool for performance based seismic design. Of the many different DBD procedures proposed in recent years there are few that are developed to a standard suitable for implementation in modern design codes. This paper presents the findings of a study that uses eight different DBD methods to undertake the seismic design of five different case studies. Some significant limitations with the eight methods have been identified through their application to realistic design examples. The study also shows that despite all of the DBD methods using the same set of design parameters, a large variation in design strength is obtained. Finally, through non-linear time history analyses the performance of each method is assessed. The performance assessment indicates that each of the eight DBD methods provide designs that ensure limit states are not exceeded. It is hoped that by presenting the limitations and comparing the required strength and performance of the methods, developments will be made that will enable designers to undertake DBD with ease and confidence.

1. Introduction

It is important that current displacement based design (DBD) methods are developed further to ensure wider acceptance of their value and to enable their implementation in modern design codes. To drive this development, eight displacement based design methods have been applied to five different buildings. The aim of these case studies in displacement based design is threefold. Firstly, the investigations aim to assess the relative ease or difficulty with which the design methods can be applied and any apparent limitations the methods may have. Secondly, the investigations aim to compare the design strengths required by each method. The final aim of these investigations is to consider the performance of the methods for each case study by comparing the predicted deformation with that obtained through time-history analysis. The significant features of each method that account for the variation in design strengths and performance of the methods are clearly identified and discussed to provide a complete evaluation of the methods.

There have been many different DBD procedures proposed in recent years. These investigations have selected eight methods that are expected best represent the range of methods available. A brief description of these methods will be provided after introducing the case studies and outlining the investigation procedure adopted by this study.

2. Description of the Buildings Considered

Five different buildings of similar height but with significantly different characteristics were selected to assess the performance of the displacement-based design methods. The five case studies considered include three wall structures and two frame structures. Case Study 1 is a three storey wall structure with regular layout on a rigid foundation as shown in Figure 1 (a). Only one earthquake direction is considered and the contribution of walls perpendicular to the earthquake direction is neglected. The second case study is identical to the first except that a flexible foundation beam has been introduced. This case study was useful in identifying any methods that have difficulty incorporating foundation flexibility in design. The third case study is also a wall structure, however, the walls are arranged in an irregular layout on a rigid foundation as shown in Figure 1 (b). The irregular layout causes the building to twist during an earthquake and therefore assesses each design method's ability to design for torsion problems. Case Study 4 is a seven-storey regular frame structure on a rigid foundation as shown in Figure 2 (a). The case study was taken from the SEAOC Seismic Design Manual [1997]. The frame member sizes and the individual floor masses are presented in Table 1. The fifth case study examines an eight-storey frame building with a vertically irregular layout. The geometry, including beam and column dimensions, are shown in Figure 2 (b). This case study considers the performance of design methods with application to a vertically irregular but realistic structural shape.

Table 1 Details of Case Study 4 - the regular moment frame

	Height	Frame Floor mass	Beam depth	Beam width	Beam Length
Level	(m)	(T)	(m)	(m)	(m)
7	26.22	387	0.91	0.61	9.144
6	22.56	456	0.91	0.61	9.144
5	18.90	456	0.91	0.61	9.144
4	15.24	456	1.07	0.76	9.144
3	11.58	456	1.07	0.76	9.144
2	7.93	456	1.32	0.76	9.144
1	4.27	460	1.22	0.76	9.144
		3125			

3. Investigation Procedure

The investigations proceed by using the DBD methods to develop design forces for each of the case studies. By applying the methods to each case study and obtaining design forces, the study achieves two of its three aims. The final aim of assessing the performance of the methods is achieved through time-history analyses. This section presents the design criteria and general design assumptions that were necessary to develop design forces for each case study. Also presented is a brief description of the non-linear time-history models and assumptions used for the performance assessment of the methods.

3.1. Design input

Demand spectra for the case studies were taken from the SEAOC *blue book* [1999]. The decision to use spectra from the SEAOC *blue book* was made arbitrarily and does not indicate a limitation of the methods since any suite of spectra can be used. SEAOC provide displacement response spectra (DRS), acceleration response spectra (ARS) and acceleration-displacement response spectra (ADRS) for four different level earthquakes; EQ-I to EQ-IV. For design, the case studies utilise EQ-I, corresponding to a frequent earthquake and EQ-IV, corresponding to a maximum earthquake.

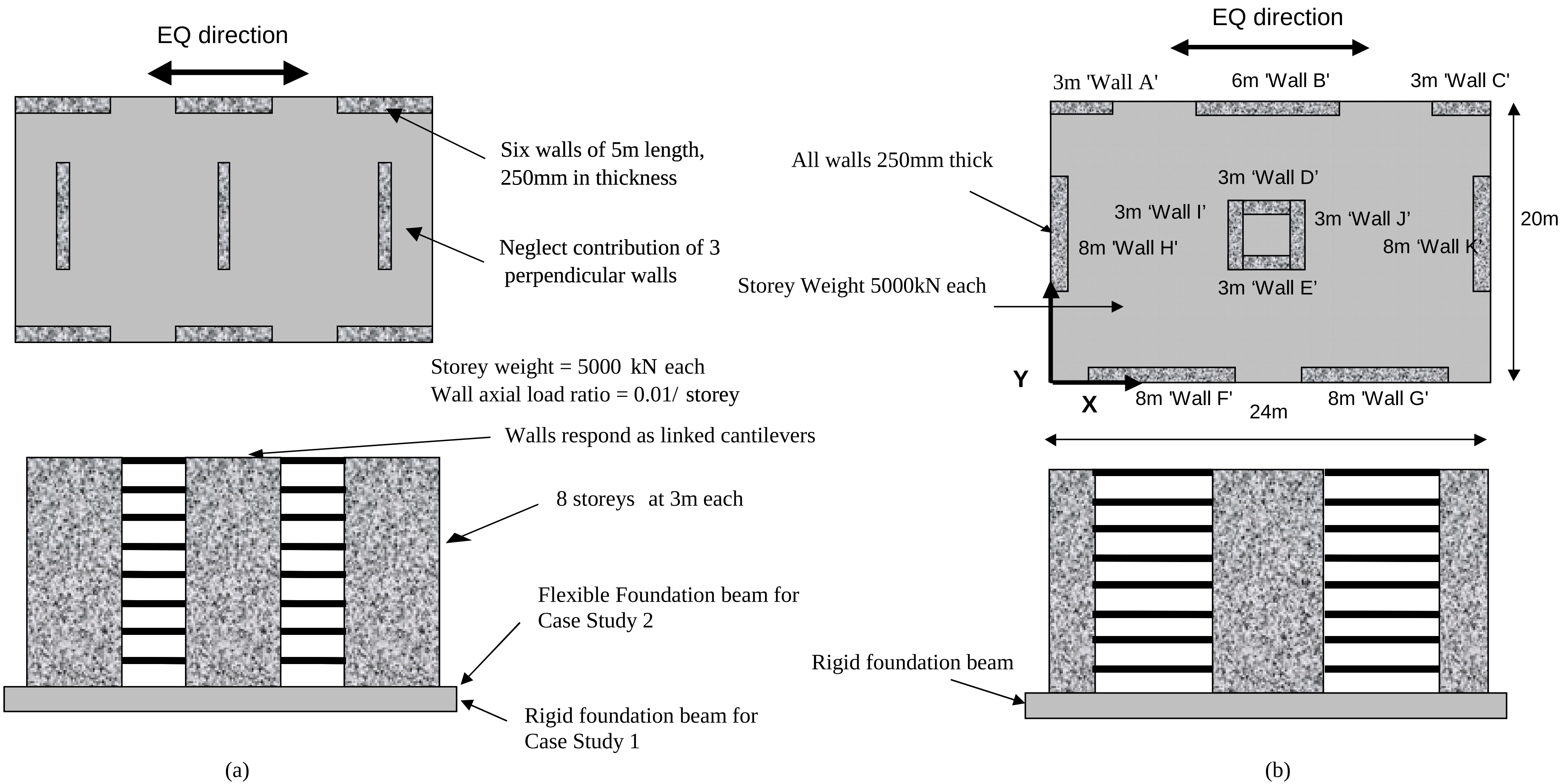


Fig. 1. Details in plan (top) and elevation (bottom) of (a) Case Studies 1 and 2, Wall Structures With and Without Flexible Foundation beam, and (b) Case Study 3 – Wall Structure with Irregular Layout

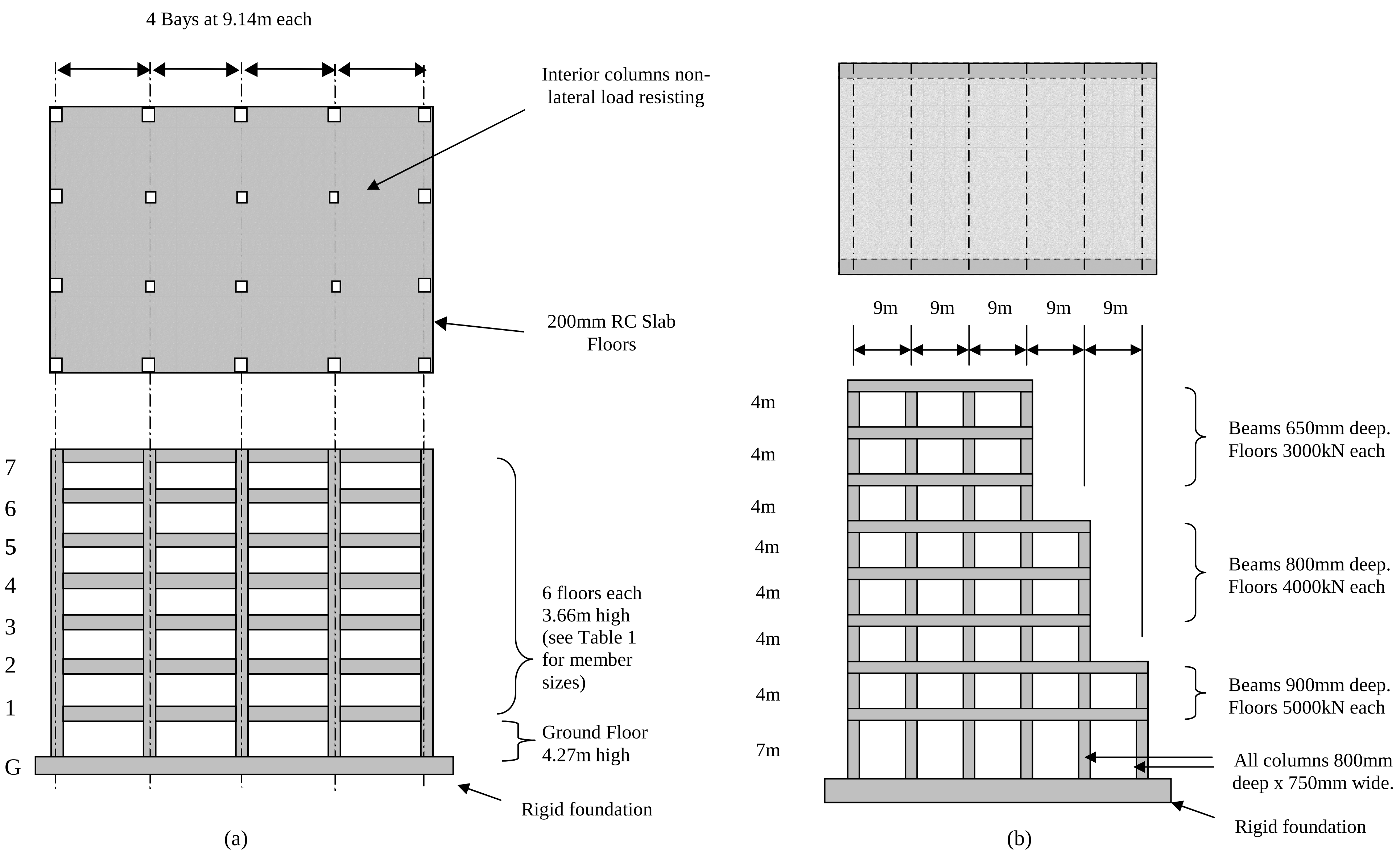


Fig. 2. Details in plan (top) and elevation (bottom) of (a) Case Study 4 - Regular moment frame structure and (b) Case Study 5 - Vertically Irregular moment frame

Seventy percent of the SEAOC EQ-I ground motion has been used for all case studies except Case Study 2 for which the full EQ-I was used. The decision to use a reduced EQ-I spectra was made after the design of Case Study 2 showed that the EQ-I event was controlling the design in all methods. It appears that the SEAOC EQ-I ground motion currently overestimates typical spectra of frequent earthquakes. It was considered that the use of a strong EQ-I motion would not reveal the benefits of effective displacement based design methods. The PGA values associated with the different spectra used for design are:

- 70% EQ-I Spectra PGA = 0.11g
- EQ-I Spectra PGA = 0.16g
- EQ-IV Spectra PGA = 0.66g

The case studies consider two load combinations; (i) G+EQ-I and (ii) G+EQ-IV. The gravity loads (G) are only used as axial loads for the wall case studies and are applied as uniformly distributed loads along the beams of the frame case studies. Load cases other than earthquake combined with gravity are not considered.

Design drift limits and system displacement ductility values were also selected from the SEAOC blue book. The target values relevant to the case studies are shown in Tables 2 and 3. Note that the longest wall for each of the case studies has an aspect ratio of three.

Table 2. SEAOC Recommended Drift Limits associated with Basic Safety Objective for Standard Occupancy structures

	System Drift Values related to Earthquake Event			
Structural System	EQ-I	EQ-II	EQ-III	EQ-IV
Shear wall H/L=1	0.003	0.0055	0.008	0.010
H/L=2	0.004	0.008	0.012	0.015
H/L=3	0.010	0.019	0.028	0.035
Moment Frame	0.005	0.015	0.030	0.040

Table 3. SEAOC Recommended Displacement Ductility Limits

	System Displacement Ductility Limits for EQ Level	
Structural System	EQ-I	EQ-IV
Shear wall (1 < H/L < 5)	1.0	5.0
Shear wall (H/L = 10)	1.0	2.5
Moment Resisting Frame	1.0	8.0

3.2. General design assumptions

Various assumptions were necessary for the designs. Assumptions that could be considered as a limitation to a DBD method are presented in Section 4. General assumptions that could reasonably be expected for any design, whether using force-based design or displacement-based design methods, are detailed below.

For design to EQ-I it was intended that the structural system yield mechanism could be partially developed but damage would generally be negligible. For design to EQ-IV it was accepted that the damage could be major. For structural systems around 80% of the usable inelastic displacement of the structure could be expended. Consequently, concrete compressive strain limits of 0.004 and 0.018, and steel tensile strain limits of 0.015 and 0.06 were adopted for design to EQ-I and EQ-IV respectively, as recommended by Priestley and Kowalsky [2000]. It was assumed that these design limit strains would first be attained in the longest wall for the wall case studies and the first floor beam for the frame case studies. Note, however, that some methods do not incorporate strain limits directly in the design calculations and instead rely on ductility or drift values to limit inelastic deformations.

The concrete and steel material properties adopted for design are values that could typically be found in building practice. Values for the concrete include; (i) f'_c = 27.5 MPa and (ii) E_c = 28100 MPa for Case Studies 1 & 2 and 32000 MPa for Case Studies 3, 4 and 5. Design values used for the reinforcing steel include: (i) f_y = 400 MPa and (ii) E_s = 200 000 MPa. Note that material strengths are not factored to dependable strength levels for design and instead, the expected strengths and stiffnesses associated with the given material properties were adopted. Where capacity design was required (refer Section 4.1.2), an overstrength factor of 1.4 was assumed.

To enable clear comparison between methods, the case studies maintain the same dimensions and member sizes for each design method. Obviously, this restriction disadvantages DBD methods that optimise design by varying the proportions of a structure. Methods affected by this restriction are identified later in the paper.

3.3. Time-history analysis assumptions

Time-history analyses are undertaken to evaluate the actual response of the case studies with strength as prescribed by the DBD methods. Some results of the time history analyses will be presented in Section 6, to demonstrate the performance of the methods. As many simplifying assumptions are made in the modelling process for the time history analyses the assessment can only be considered as an indication of true performance. The assumptions made for the time history analyses are outlined next.

3.3.1. *Time-history records*

Three spectrum-compatible time-histories were generated using SIMQKE that is part of the non-linear time-history analysis program, *Ruaumoko* [Carr 2001]. The response spectra for the three time-histories generated to match EQ-I are shown in Figure 3. A time step of 0.01s and duration of 20s were chosen for the accelerograms. It is expected that the case studies would benefit from the use of real recorded time-history records of earthquake as artificial records generally include motions with different phase than real records. However, because of the nature of these case studies, it was decided that artificial time-histories would best match the design spectra and would therefore most clearly demonstrate the performance of each method. The difference between artificial and real time-history records may affect the inelastic hysteretic response of the concrete structures, although it is considered that the spectrum compatible records provide a good indication of the relative performance of the design methods.

The plot of the displacement response spectra (DRS) shows that the artificially generated time-histories do not match the design spectra very well with fairly large deviations at longer periods. However, by considering the average and the peak of the maximum values of response from the three time histories it is expected that the design spectra will be adequately represented. Therefore, in the following sections an “average” and a “peak” value of response for the three time history analyses are presented.

3.3.2. *Modelling approximations*

The *Ruaumoko* time history analysis program is used to subject each of the structures to the three spectrum compatible accelerograms. Strengths obtained for each method are input into separate models, assuming that the actual strength provided in practice would exactly match the design strength required. The models use effective section properties, obtained by taking the design strength and dividing by the yield curvature. Approximations for the yield curvatures were obtained from the expressions provided by Priestley and Kowalsky [2000].

Yielding elements of the concrete structures were modelled using the Takeda hysteresis behaviour, with 5% post-yield displacement stiffness, using the unloading model according to Emori and Schonbrich [1978] with an unloading stiffness factor of 0.5, reloading stiffness factor of 0.0 and reloading power factor of 1.0. An explanation of these factors and the shape of the hysteresis model is presented in the Ruaumoko user manual by Carr [2001]. The plastic hinge lengths associated with the yielding elements were calculated using the recommendations from Paulay and Priestley [1992].

Elastic damping is modelled for the structures using tangent stiffness Rayleigh damping of 5% applied to the 1^{st} and 2^{nd} modes. It was assumed that the floor system is adequately connected to the structure and provides an efficient diaphragm action (rigid diaphragm) in order to introduce inertia forces to the structure at different levels. P-delta effects are not considered and all lateral forces were resisted by the structural walls and frames of the case studies, identified in Section 2.

Other modelling assumptions, particular to each case study are described below:

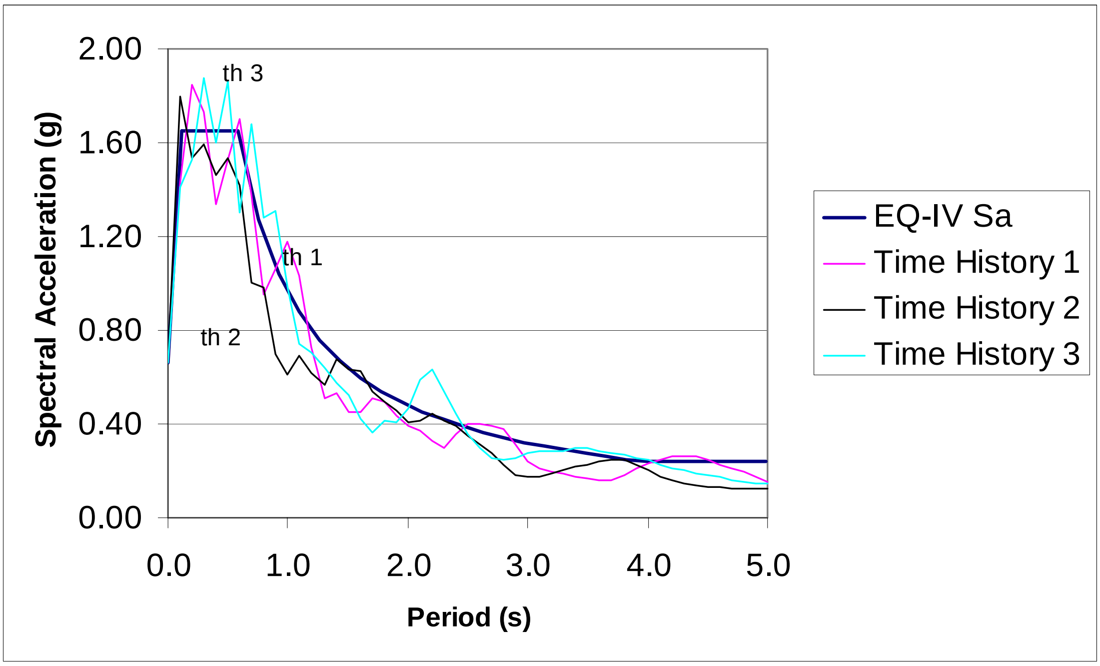
- *Case Study 1 – wall structure.* In modelling the wall structure, masses were placed at floor levels assuming the floors to be flexible out-of-plane, and infinitely stiff in-plane. The strength required for the bottom storey was continued up the full height of the building and a constant effective stiffness was used over the structure’s height.

- *Case Study 2 – wall structure with flexible foundation.* Modelling of the wall structure with the flexible foundation required introduction of a base restraint with finite rotational stiffness. Note that all the other case studies applied base restraints with infinite stiffness assuming rigid foundation response.

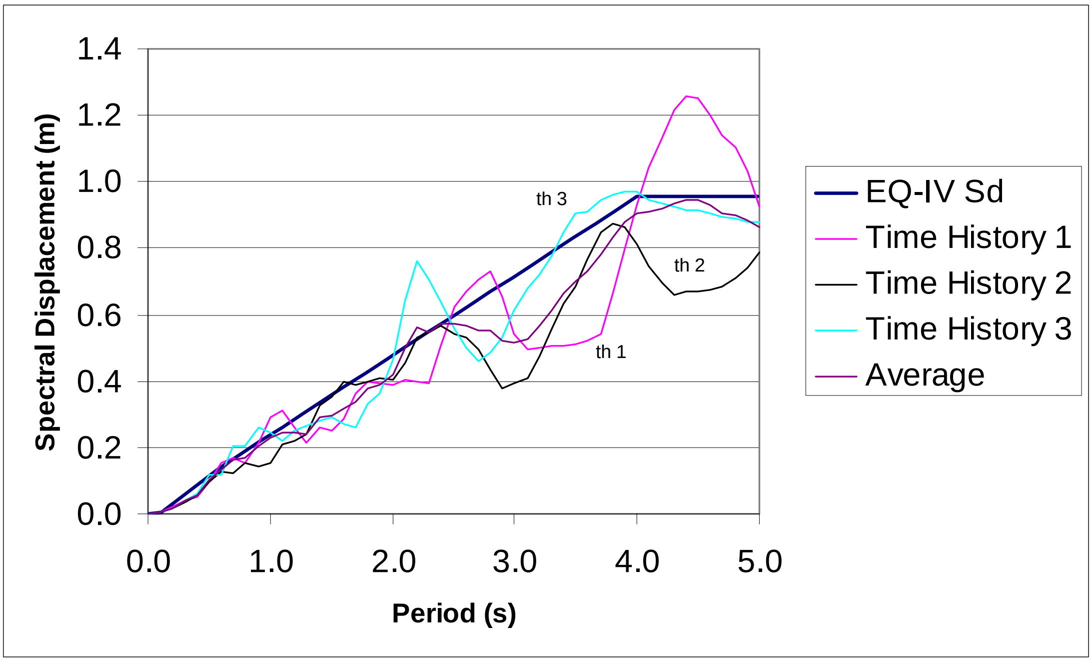
- *Case Study 3 – wall structure with irregular plan and rigid foundation.* Ruaumoko 3D was used to develop a model for the time history analysis of Case Study 3 with assumptions similar to those of Case Study 1, but with the design strength provided for each level. Walls perpendicular to the principal earthquake direction were modelled with elastic section properties.

- *Case study 4 – Regular RC frame structure.* A model was developed in Ruaumoko that includes base storey columns with axial load interaction diagrams, and effective stiffness before yield estimated as 50% I_g using recommendations from Paulay & Priestley [1992]. The columns above the ground floor were modelled as elastic members as yield should be confined to the base columns and beams by the principles of capacity design. These elastic columns were modelled with cracked stiffness of 60%I_g, in accordance with Kappos [2001] and Paulay and Priestley [1992] recommendations.

- *Case Study 5 – vertically irregular RC frame structure.* The model used for the vertically irregular frame structure was also developed in Ruaumoko making assumptions similar to those of the regular frame model of Case Study 4.



(a)



(b)

Fig. 3. Comparison of spectra obtained from the artificial time-histories vs the design spectrum in (a) Acceleration Response Spectra format, and (b) Displacement Response Spectra format.

4. Utilising Displacement Based Design

This section describes the main differences between the displacement based design methods that are currently available and identifies the eight methods that have been selected for use in these case studies. After broadly describing the methods, the difficulties that designers may face in applying them to realistic design examples are discussed.

4.1. *Selection of the DBD Methods*

The eight DBD methods selected for the case studies, including references to the papers that describe the methods in full, are listed below:

- **ISDC** method = *Initial Stiffness Deformation Control* method by Panagiotakos & Fardis [1999].
- **ISIP** method = *Initial Stiffness Iterative Proportioning* method by Browning [2001].
- **YPS** method = *Yield Point Spectra* method by Aschheim & Black [2000].
- **INSPEC** method = *Inelastic Spectra* method by Chopra & Goel [2001].
- **CASPEC** method = *Capacity Spectrum* method by Freeman [1998].
- **SEAOC** method = DBD "method a" from SEAOC recommended lateral force requirements [1999].
- **DDBD** method = *Direct Displacement Based Design* method by Priestley & Kowalsky [2000].
- **T-HIST** method = Advanced Analytical Techniques method that utilises *Time-history* analyses, by Kappos & Manafpour [2001].

Note the abbreviated names assigned to each of the methods as these abbreviations will be used throughout the remainder of this paper.

The eight methods were chosen from a suite of contributions as shown in Table 4. The individual contributions shown in Table 4 generally refer to a smaller set of distinct design procedures. For example, the methods by Freeman [1978], ATC [1996], Paret [1996], and Chopra and Goel [1999] refer to the Capacity Spectrum approach. Given the space limitations associated with this paper each method is not described in detail. However, the methods shown in Table 4 have been grouped according to major differences in the design approaches. The table can be considered as a matrix where the role of displacement in the design process is shown on the horizontal axis, and the type of analysis utilised is shown on the vertical axis. Even though other criteria could be used to distinguish between the methods these are the major differences and are therefore elaborated on in the following paragraphs. For the details of a particular method, readers are directed to the individual contributions referenced or to the summary provided by Sullivan [2002] or *fib* TG7.2 [2002].

Table 4. Matrix of Design Procedures

	Deformation-Calculation Based (*DCB*)	Iterative Deformation-Specification Based (*IDSB*)	Direct Deformation-Specification Based (*DDSB*)
Response Spectra: *Initial Stiffness Based*	Moehle (1992) FEMA (1997) UBC[1] (1997) **Panagiotakos & Fardis[1,2] (1999)** Albanesi *et. al*. (2000) Fajfar (2000)	**Browning[1] (2001)**	SEAOC (1999) **Aschheim & Black (2000)** **Chopra & Goel (2001)**
Response Spectra: *Secant Stiffness Based*	**Freeman (1978)** ATC (1996) Paret *et. al*. (1996) Chopra & Goel (1999)	Gulkan & Sozen (1974)	Kowalsky (1995) **SEAOC[1] (1999)** **Priestley & Kowalsky[1] (2000)**
Direct Integration: *Time History Analysis Based*	**Kappos & Manafpour[2] (2000)**	N/A	N/A

1. *Method has been developed for particular structural types and is not intended for application to other structural types.*
2. *Method has been developed with specific limit states in mind that must be checked during design.*

4.1.1. *Role of displacement in the design process*

The various design procedures can be considered to fall into one of three basic categories based on the role that deformation plays in the design process. The three categories are described as (i) Deformation-Calculation Based (*DCB*), (ii) Iterative Deformation-Specification Based (*IDSB*), and (iii) Direct Deformation-Specification Based (*DDSB*).

Deformation-Calculation Based (DCB)

The DCB methods involve calculation of the expected maximum displacement for an already designed structural system. Detailing is then provided such that the displacement capacity of the system and its components exceeds the calculated maximum displacement. As a result, no attempt is made to induce a change in the system to alter the maximum displacement demand, but rather, the demand is taken as a design quantity which is dealt with through proper detailing.

Iterative Deformation-Specification Based (IDSB)

The IDSB methods are similar to the DCB methods in that they involve analysis of an already designed system to evaluate the expected maximum displacement. However, unlike the DCB methods, a limit to the maximum displacement is enforced, and as a result, changes are made to the structural system such that the analysis displacements are kept below the specified limit. Consequently, the design procedure is iterative.

Direct Deformation-Specification Based (DDSB)

The DDSB methods utilise as a starting point a pre-defined target displacement. The design of the structure then progresses in a direct manner whereby the end result is the required strength, and hence stiffness, to reach the target displacement under the design level earthquake. These procedures are not iterative, and do not require a preliminary design.

4.1.2. *Type of Analysis Used in the Design Process*

The second criteria used to classify the methods relates to the type of analysis used in the design process. The methods have been grouped into three categories; (i) Response Spectra - Initial Stiffness Based, (ii) Response Spectra - Secant Stiffness Based, and (iii) Time History Analysis Based.

Response Spectra - Initial Stiffness Based

Initial stiffness based procedures utilise elastic stiffness (or a variation thereof) coupled with approximations between elastic and inelastic response, such as the equal displacement approximation or other R-μ-T relations (refer Miranda and Bertero [1994]) to evaluate the maximum response.

Response Spectra - Secant Stiffness Based

Secant stiffness based procedures utilise the secant stiffness to the maximum response level and the concept of equivalent viscous damping to characterise the non-linear response of structural systems.

Time History Analysis Based

Time-history methods solve the equations of motion by direct integration for a specific earthquake time history to evaluate the maximum response. The analysis may be elastic or inelastic, although there is little advantage in conducting elastic time history analysis. Time history analysis may be based on frame members where assumptions on section hysteretic characteristics are required. Analysis may also be based on fibre models where individual materials that comprise the structural system follow an assumed non-linear response.

It is possible to quickly describe further details of the methods to distinguish between those methods that fall into the same region of Table 4. As implied by the names, the YPS method proposed by Aschheim and Black [2000] includes the use of Yield Point Spectra whereas the INSPEC method proposed by Chopra and Goel [2001] utilises Inelastic Response Spectra in DRS format. The most distinguishing feature between the SEAOC method and the DDBD method of Priestley and Kowalsky [2000] is that designers using the SEAOC method refer to a set of tables for design parameters whereas the DDBD method provides equations and recommendations by which the designer can proceed.

4.1.3. *General restrictions associated with some design methods*

Indicated in Table 4 are the DBD methods that are applicable only to certain structural types as well as the methods that have been developed for specific limit states that must be checked during design. These restrictions are explained below.

The ISDC method by Panagiotakos and Fardis [1999] includes tables of factors that scale elastic chord rotations to inelastic chord rotations. The method only provides these factors for the ultimate inelastic rotations of frame elements and is therefore restricted to frame type structures and to one specific limit state.

The ISIP method by Browning [2001] is also restricted to certain structural types because the method is only intended to be applied to regular frame type structures.

The SEAOC and DDBD (Priestley & Kowalsky [2000]) methods both incorporate an assumed displaced shape for design. Therefore, these methods are restricted to structures for which the displaced shape is fairly well known.

The T-HIST method, by Kappos and Manafpour [2001], requires that the initial step in the method is to design for a “serviceability” type earthquake. This limit state is used to provide the initial strength to the structure and an inelastic model is then developed that is used in inelastic time-history analysis to design for other limit states.

Note that many of the methods were applied to all the case studies despite these restrictions. This was done to determine firstly whether the methods could be used to design structures for which they were not intended and secondly, to investigate how the methods perform when applied to these structural types.

4.2. *Current limitations of the DBD methods*

Through application to the various case studies several limitations associated with the methods have been identified. This section proceeds by outlining the limitations that are common to several methods and then describes the difficulties that relate to each of them.

4.2.1. *Common design decisions*

The recommendations provided by each method and the consequent difficulties with design associated with two common design decisions are summarised in Table 5 and similarly for irregular structures in Table 6. By reviewing the tables the completeness of each method can be quickly gauged. Where no recommendations are provided then assumptions have been made as described below. Any difficulties associated with a particular method are discussed in section 4.2.2.

Table 5. Summary of the recommendations and difficulties associated with common design decisions

Method	**Vertical Distribution of Base Shear**		**Design of Structures with Flexible Foundations**	
	Recommendations Provided?	Difficulty in Design?	Recommendations Provided?	Difficulty in Design?
ISDC	Yes[1]	No	No	No
YPS	Yes[1]	No	No	Yes
INSPEC	No	No	No	No
CASPEC	Yes	No	No	No
SEAOC	Yes	No	No	Yes
DDBD	Yes	No	Yes[2]	No
T-HIST	Yes[3]	No	Yes[3]	No
ISIP	No	No	No	No[4]

1. *Recommendation is to use current code guidelines as utilised in force based design.*
2. *Recommendations are limited – relate to appropriate system damping, not overall procedure.*
3. *For EQ-I the method considers the design to be elastic, thereby allowing common code recommendations. For other earthquake levels the structural characteristics are allowed for in the inelastic time-history model.*
4. *Difficulties are not anticipated, however, the case study with flexible foundations is a wall structure and the method therefore could not applied to this case study.*

Table 6. Summary of the recommendations provided for Irregular Structures and the consequent difficulty in design

Method	**Design of Structures with Irregular Layout**		**Design of Vertically Irregular Structures**	
	Recommendations Provided?	Difficulty in Design?	Recommendations Provided?	Difficulty in Design?
ISDC	No	No	No	No
YPS	No	No	No	No
INSPEC	No	Yes	No	No
CASPEC	No	No	No	No
SEAOC	No	No	No	No
DDBD	Yes	No	No[1]	No
T-HIST	Yes[2]	No	Yes[2]	No
ISIP	N/A	N/A	N/A	N/A

1. *Procedure acknowledges that method is not expected to be effective as the inelastic displaced shape is unknown for irregular structures.*
2. *For EQ-I the method considers the design to be elastic, thereby allowing common code recommendations. For other earthquake levels the structural characteristics are allowed for in the inelastic time-history model.*

Where a method does not provide recommendations for the vertical distribution of base shear it was assumed that the forces should be distributed with respect to mass and height, in line with most modern code approaches.

Foundation flexibility was allowed for in the design methods by either using a model with the appropriate foundation flexibility, or adopting an iterative procedure whereby the displacement due to foundation rotation was initially assumed and then checked at the end of the design process.

Where recommendations were not provided for the design of structures with irregular layout, assumptions in line with most modern code approaches were adopted. Consequently, the procedure described in Paulay & Priestley [1992] was utilised to distribute the design base shear to individual walls of the structure in Case Study 3.

Despite the lack of design recommendations for vertically irregular structures, difficulties were not encountered during design. The structures were treated essentially the same as vertically regular structures. It was expected that the effect of this approximation would be seen in the performance assessment.

Other design decisions that arose in several of the methods relate to appropriate values for cracked stiffness, and estimates for the structures' yield displacements. Estimates for the cracked stiffness values of walls, beams and columns were taken from Paulay & Priestley [1992]. Where a method recommended a more accurate value, the demand strength, M_n, was divided by the yield curvature, ϕ_y, to give $EI = M_n/\phi_y$. With knowledge of the concrete elasticity, E, the cracked value of the second moment of inertia, I, could be obtained directly. Approximate values of yield curvature as provided by Priestley and Kowalsky [2000] were utilised for design. These values for yield curvature were also used to estimate the yield displacement of the structure in combination with an estimate for the structure's effective height.

Finally, each method places different value on the use of capacity design. Table 7 presents the recommendations and assumptions made by each method for capacity design. The capacity design procedure with an overstrength factor of 1.4 described by Paulay and Priestley [1992] is adopted for the case studies.

Table 7. Capacity Design and dynamic magnification recommendations

Method	Capacity Overstrength Factor Recommended?	Dynamic Magnification Required?	Comments or Other scaling factors Required?
ISDC	Yes	Assume Yes – as part of capacity design approach.	Method uses dependable rotation capacity factors.
ISIP	Yes	Assume Yes	No recommendations made regarding higher mode effects.
YPS	Assume Yes	Assume Yes	Method recommends use of the conventional force based design process.
INSPEC	Assume Yes	Assume Yes	No recommendations.
CASPEC	Assume Yes	Assume Yes	Method is essentially an assessment procedure - suggests if pushover ok capacity design not required.
SEAOC	Yes	Yes	Recommends Paulay & Priestley [1992] capacity design procedure.
DDBD	Yes	Yes	Examples provided show that capacity design & allowance for higher modes should be made.
T-HIST	No – allows for strain hardening in process but not for material strengths higher than the dependable value.	No – inherent in the time history analysis.	Factors demand shears by 1.1 to allow for larger EQ than predicted.

4.2.2. *Difficulties related to individual methods*

The following section describes the difficulties related to individual methods.

ISDC Method

The ISDC method proposed by Panagiotakos and Fardis [1999] estimates inelastic deformations using initial stiffness with response spectra and elastic to inelastic amplification factors. The method allows for checking of a target ductility (equal to 1.0) for a frequent earthquake (equivalent to SEAOC EQ-I) and then requires that permissible inelastic rotations are not exceeded for a very rare earthquake (SEAOC EQ-IV).

As a performance based design tool the method could appear restrictive, in the sense that only two different limit states can be checked and that non-structural damage (affected by drift) is not controlled.

Amplification factors incorporated in the method are a relatively easy and fast way to obtain inelastic chord-rotation demands. However, the scaling factors are not provided for wall structures. For the case study it was assumed that the amplification factors for ground storey columns could be used.

Although not examined as a case study in this report, Panagiotakos and Fardis [1999] investigate frames with infill panels and provide several design recommendations for this irregular structural form.

ISIP Method

The ISIP method proposed by Browning [2001] is a target period method that aims to achieve a pre-defined average drift limit. Browning's method is relatively fast and simple to use, although Browning [2001] writes that it is only applicable to regular reinforced concrete frames. Neither inelastic rotation demands nor ductility limits are controlled in the design process.

In determining minimum base shear strength Browning [2001] provides an expression that includes an acceleration factor and a strength reduction factor. It is unclear how sensitive the design would be to assumptions for the amplification factor. The case study used the value of 15/4 provided by Browning for systems with 2% damping.

YPS Method
The YPS method presented by Aschheim and Black [2000], permits design to a number of performance criteria relatively quickly. The method involves development of yield point spectra, which are used to define a permissible design region considering target drift and ductility values.

To enable design for various risk events in one step, the permissible design regions for the different earthquakes can be plotted on the same axes. Then, with knowledge of the structure's yield displacement, the strength required to satisfy all ductility and drift limits can be obtained from the plot in one step. The single design step means the method is relatively fast, however, an exception occurs when the design procedure is applied to structures with flexible foundations.

In applying the method to Case Study 2, iterations were performed to allow for foundation flexibility. The system yield displacement, consisting of the sum of displacements due to the structural deformation and those due to foundation rotation, was varied until the resulting design base shear caused the same displacement to the system as that assumed. It was revealed that it is difficult to use the method in this way for structures with flexible foundations. This is because the system ductility and yield displacement change with foundation rotation. To use the yield point spectra correctly the designer must shift the drift control branch every iteration since the branch is a plot of the displacement, which multiplied by the system ductility will give the target displacement. For an increase in target displacement due to foundation rotation, the branch will be lowered since for the same value of ductility a larger value for yield displacement will be required to achieve the target displacement.

The designer must also recognise that the limiting ductility curve changes with each iteration. This is because for a fixed structural ductility capacity, a building with flexible foundations has lower available system ductility, as Priestley and Kowalsky [2000] discuss in the presentation of the DDBD method.

As noted previously, the YPS method uses the yield displacement to obtain the design base shear. In contrast with other methods such as the DDBD method, the design is relatively sensitive to the yield displacement assumed. Because the method uses the yield displacement to obtain a base shear coefficient directly from demand spectra, a small difference in yield displacement can result in large differences in design base shear. As the estimates for yield displacement will become less accurate for irregular structures for which the response is difficult to predict, the pushover analysis suggested by Aschheim and Black [2000] could be used in such instances to obtain a better value for the yield displacement.

INSPEC Method
The INSPEC method proposed by Chopra and Goel [2001] utilises the initial steps of the method presented by Priestley and Calvi [1997] to determine a target displacement and

design ductility. The method then enters inelastic displacement response spectra, to obtain a period and initial stiffness. With the yield displacement and initial stiffness known, the yield force can be determined. This method thereby designs structures to a target drift level and acceptable plastic rotation. The displacement ductility is not directly controlled in the process.

During the iteration process for Case Study 3, it was noted that the method has difficulty iterating on stiffness for a number of walls. At the end of each iteration the strength is distributed to the walls and their cracked stiffness is determined as $EI = M_n/\phi_y$. As these values of stiffness are used to distribute the base shear at the end of the next iteration it emerges that the shear is distributed totally away from the smaller walls to be carried entirely by the larger walls. For detailing purposes it was assumed that minimum steel would then be provided to the smaller walls for which no demand is expected.

CASPEC Method

The method proposed by Freeman [1998] is best suited to checking the performance of existing structures for which the member sizes and strengths are known. This is because the method requires that a capacity spectrum for the structure is graphically superimposed onto a suite of demand spectra with different ductility/damping levels.

Freeman [1998] does not recommend a particular procedure to develop demand spectra for different levels of damping. For the relationship between ductility and damping, various papers are referenced and for the case studies Equation 1 was utilised. This relationship was obtained from Eurocode 8, CEN [1996]:

$$\Delta_{(T,\xi)} = \Delta_{(T,5)}\left[\frac{7}{2+\xi}\right]^{1/2} \qquad (1)$$

This relationship was necessary for the development of spectra at different levels of damping and in checking the capacity curve against the demand.

The method does not include a recommended procedure for the design of new structures for which the initial strength is unknown. To overcome this in the case studies, the 5% damped EQ-I spectra, for which the structure is required to remain elastic, was used to determine the minimum strength for a known structural yield displacement. The structural yield displacement was estimated using the relationships provided by Priestley and Kowalsky [2000]. Having obtained an initial strength level, a capacity curve beyond first yield could be developed and used to check higher demand events.

During the design process for Case Study 1 it was found that the strength provided to satisfy EQ-I drift and ductility criteria, was insufficient for the EQ-IV criteria. The method does not provide recommendations on how the structure should be improved to satisfy the critical demands of EQ-IV. For the case study it was assumed that the dimensions would not change and that the strength of the structure should be increased uniformly. Because increasing the strength does not affect the yield displacement significantly, the new design could simply scale the forces up until the end of the pushover curve reached the demand curve corresponding to maximum allowable drift or ductility, whichever governed.

SEAOC Method
Application of the first of the displacement based design procedures provided in the SEAOC [1999] blue book showed that the procedure was relatively fast and easy to use to obtain the design base shear.

The method designs for target drift values while ductility demands are not controlled. Four different risk events and drift limits may be considered for design depending on the structural performance objective.

For wall structures, the method is currently limited to three different aspect ratios, and does not advise the designer on what should be assumed in the case of a different aspect ratio. For the case studies it was assumed that interpolation of the data could be performed.

In applying the method to Case Study 1, an inconsistency was noted. The method recommends that the yield strength of the system may be obtained using an overstrength factor divided into the required effective strength. SEAOC suggests a range of overstrength factors from 1.25 to 2.0, however, it does not recommend a procedure through which to obtain these factors. With the effective strength known it was noted that this assumed overstrength factor is likely to predict a yield strength inconsistent with the yield strength obtained using the ductility demand and the post-yield stiffness ratio. For the case studies an overstrength value of 1.4 was assumed for the design to EQ-IV and 1.0 for design to EQ-I.

SEAOC provides no guidance for the design of structures with flexible foundations. Due to the prescriptive nature of the method, it was found that allowance for foundation flexibility could not be made. This is because the method determines a target displacement using prescribed factors and assumes a ductility demand. These values are independent of a likely yield displacement or foundation rotation. If the method had instead calculated the ductility value using the yield displacement, and then determined equivalent damping for this ductility demand, an appropriate effective period could have been obtained. Despite this restriction in the preliminary design stage it is not likely that non-conservative designs would be generated since the method would account for negative effects of foundation flexibility during the pushover analysis.

DDBD Method
The DDBD method proposed by Priestley and Kowalsky [2000] is a relatively fast method that designs a structure to satisfy a pre-defined drift level. The code drift limit and the drift corresponding to the system's inelastic rotation capacity are considered in the design process. The method does not directly control the system displacement ductility demand.

In application of the method to structures with irregular layout, the method recommends base shear strength is distributed to the walls in proportion to their length squared. In development of the design displacement profile it is unclear whether to use the longest wall, or some average length of all the walls. It was assumed that the longest of the walls should be used. In accordance with an example presented by Priestley and Kowalsky [2000] for a structure with varying wall lengths, the equivalent damping of the building was determined using the expected damping of each wall factored by its length squared over the sum of the squared lengths of the walls. It was assumed that transverse walls should not be considered in this evaluation of the effective damping despite the load that they carry due to the twisting of the structure.

Integral to the DDBD method is the assumed displacement profile of the structure at the drift limit. Displacement profiles have not been developed for irregular structures and therefore the method cannot strictly be applied to Case Study 5. However, it was proposed that the method be applied to Case Study 5 using the displacement profile for a regular moment frame with number of bays equal to the average of the vertically irregular system. Design assumptions then followed those as for the regular RC frame structure of Case Study 4.

T-HIST method
Of all the design procedures considered in this project the T-HIST method presented by Kappos and Manafpour [2001] is the most involved. The method uses traditional force-based design to obtain a basic strength level necessary for an elastic response to EQ-I. A detailed model of the structure is then developed in which members are able to exhibit inelastic behaviour. The model is then subjected to two different time-history analyses for hazard levels corresponding to EQ-II and EQ-IV for which drift target values are checked and detailing for plastic rotations is provided.

The method does not design to a target drift for the frequent EQ-I event. However, for the less frequent EQ-II event the method does check that serviceability type drift limits are not exceeded. For the rare EQ-IV event the method does not control the system ductility but rather details the structure to provide sufficient inelastic rotation capacity. In applying this method to the case studies, the time-history analysis for the EQ-II event was omitted. This was done to enable clear comparison of the design methods.

5. Required Strength Comparisons

The flexural strength, shear strength, and reinforcement content values required by each method for each of the case studies, demonstrates that the aforementioned differences in the methods can have a significant effect on design. This section presents the design strengths obtained and highlights the most interesting results.

5.1. *Shear strength*

Values for the building design base shear strength at yield for each of the methods and all case studies are shown in Table 8. While reviewing the design strengths it is worthwhile considering the parameter that governed the design for each method as presented in Table 9. It is apparent that design for EQ-1 was often critical for many of the design methods. In assessing the results it can be seen that those methods that use uncracked initial stiffness for determination of EQ1 forces, such as the ISDC and ISIP methods, attract the highest design shears. Methods that attract intermediate values of design strength, such as the CASPEC and YPS methods, benefit from the use of yield displacements obtained from realistic values of yield curvature at first yield. These yield displacements imply longer periods of vibration and therefore reduced accelerations than the methods using uncracked or large estimates of cracked section stiffness. The direct deformation specification based design methods, such as the INSPEC, DDBD and SEAOC methods tend to require even lower levels of strength because these methods do not require that ductility limits be maintained but instead design to drift and material strain limits associated with acceptable levels of damage. The implication of neglecting arbitrary ductility limits is that structures are often designed for larger target displacements and therefore require lower values of design strength.

Table 8. Total Building Design Base Shear for each of the case studies.

	Building Design Base Shear (at 1st Yield, kN)				
Method	Case Study 1	Case Study 2	Case Study 3	Case Study 4	Case Study 5
ISDC	9480	7200	10987	13406	7131
YPS	3008	5755	4426	3732	4038
INSPEC	3416	3750	2434	3077	6307
CASPEC	4537	5419	5059	4499	4584
SEAOC	4560	4560	3013	3596	3249
DDBD	2900	3494	3417	6136	7623
T-HIST	5400	5562	8044	9627	4464
ISIP	N/A	N/A	N/A	13369	N/A

Table 9. Governing Design Parameter for each Case Study for each method

	Governing Design Parameter				
Method	Case Study 1	Case Study 2	Case Study 3	Case Study 4	Case Study 5
ISDC	EQ-I ductility	EQ-I ductility	EQ-I ductility	EQ-I ductility	EQ-I ductility
YPS	EQ-I ductility	EQ-IV ductility	EQ-I ductility	EQ-I drift limit	EQ-I drift limit
INSPEC	EQ-I drift limit	EQ-I drift limit	EQ-IV inelastic rotation	EQ-I drift limit	EQ-I drift limit
CASPEC	EQ-IV ductility	EQ-I ductility	EQ-IV ductility	EQ-I drift limit	EQ-I drift limit
SEAOC	EQ-IV drift limit	EQ-IV drift limit	EQ-IV drift limit	EQ-I drift limit	EQ-I drift limit
DDBD	EQ-IV inelastic rotation	EQ-IV inelastic rotation	EQ-IV inelastic rotation	EQ-I drift limit	EQ-I drift limit
T-HIST	EQ-I ductility	EQ-I ductility	EQ-I ductility	EQ-I ductility	EQ-I ductility
ISIP	N/A	N/A	N/A	EQ-I Min. strength	N/A

Note that the DDBD method does not follow the trend described above for the frame structures of Case Studies 4 and 5. Instead, for Case Study 5, the DDBD method has the highest design base shear of all the methods. This situation develops from the unusual situation that the code drift limit is less than the drift associated with first yield of the structure. The DDBD method establishes the strength required to maintain the target displacement and then factors this strength allowing for the expected ductility to obtain the yield strength. In the frame case studies the design strength obtained by the DDBD method for the design displacement is increased to the value of yield strength shown because the yield displacement is greater than the design displacement.

The fact that the frame structures of Case Studies 4 and 5 are governed by an EQ-1 drift limit that is less than the yield drift indicates that the drift limits suggested by SEAOC for frames may be inappropriate. However, if displacement based design methods are to be used in a performance based design approach it is important that they can design for any target drift, whether this target drift is less or greater than the yield drift. For instance a performance based design approach should be able to design a building that has special non-structural drift limit requirements that may be less than the yield drift.

5.2. *Flexural strength*

Design bending moments of the wall case studies are presented in Figure 4. For Case Study 3, only the design moments associated with the 3m ‘Wall A’ and the 8m ‘Wall F’ are shown. For the location of these walls, refer to Figure 1. Note the low design strength obtained by the INSPEC method for ‘Wall A’ of Case Study 3. As described in Section 4.2.2, this results from the low stiffness of the wall that is obtained during the iterative design procedure. Also note the DDBD method design strengths in relation to other methods for the 3m and 8m walls. The method assigns greater strength to the shorter ‘Wall A’ than the longer ‘Wall F’ since the DDBD method recommends that strength is distributed in proportion to wall length squared, instead of wall length cubed as assumed by the other methods.

Design actions for the two frame case studies were obtained by applying the vertically distributed design base shear to a simple 2D model in SAP2000. These models included gravity loads and modelled members with cracked stiffness as recommended by each design method as discussed in Chapter 3. The first floor beam design moments for each method are presented in Figure 5 to enable further comparison of the design strengths.

5.3. *Relative steel content and steel distribution*

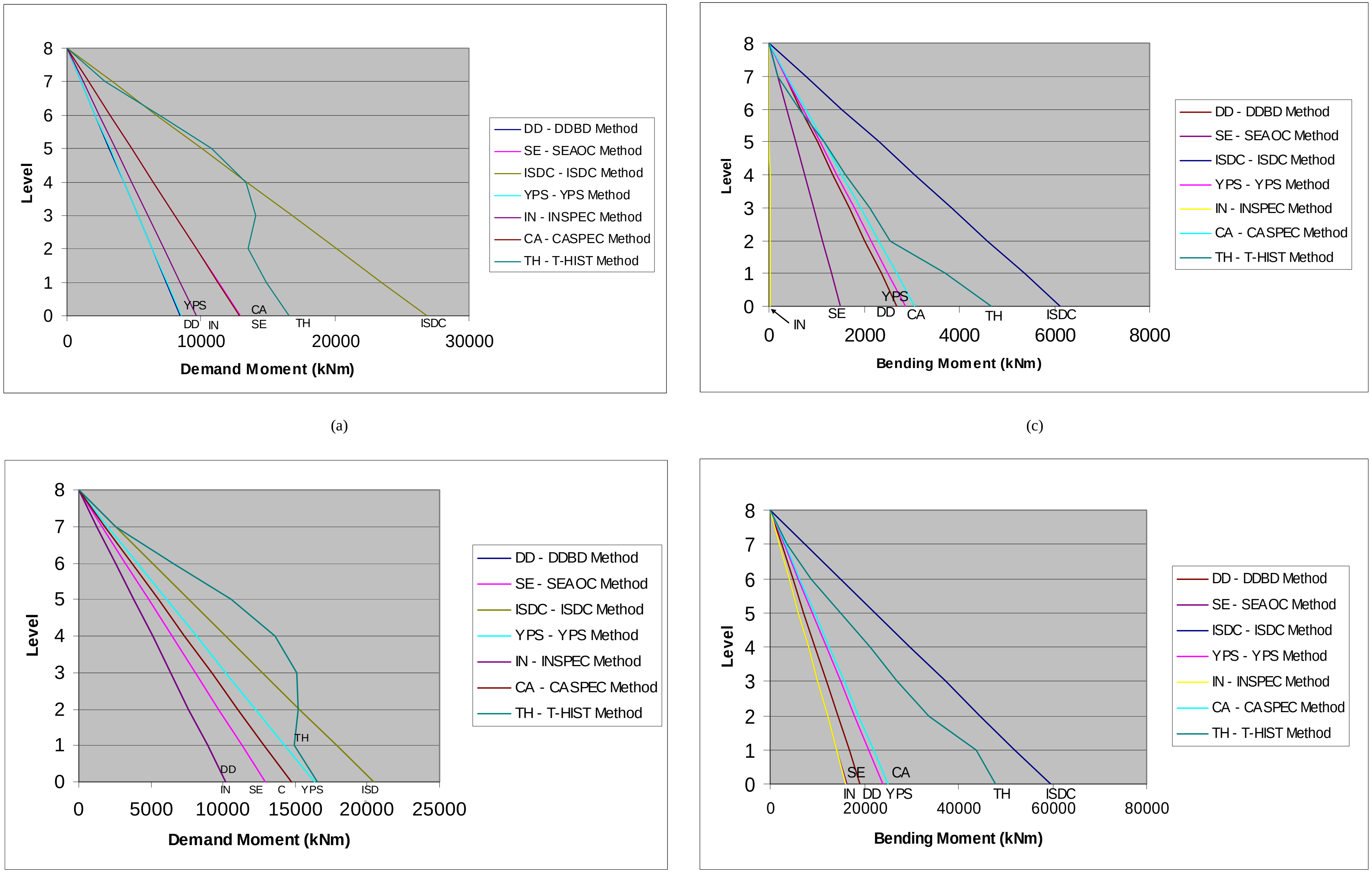
Longitudinal reinforcement ratios are presented for the columns of the frame case studies in Table 10. These reinforcement ratios were determined using RECMAN [*King et. al. 1986, Mander et. al. 1988*] moment-curvature analyses assuming that the reinforcing steel is distributed evenly to the top, bottom and sides of the section. Some of the steel contents are excessive and it is expected that in design different column dimensions would be selected. However, the unrealistic steel contents are presented to highlight the substantial difference in the strength required for each of the methods.

Table 10. Longitudinal Steel Percentages for the columns of Case Studies 4 and 5

	Longitudinal Steel Case Study 4		Longitudinal Steel Case Study 5	
Method	Interior Columns	Corner Columns	Interior Columns	Corner Columns
ISDC	1.4%	2.7%	6.7%	9.9%
YPS	0.3%	0.3%	1.0%	1.8%
INSPEC	0.3%	0.3%	2.3%	3.2%
CASPEC	0.3%	0.4%	1.5%	2.3%
SEAOC	0.3%	0.3%	0.8%	1.3%
DDBD	0.3%	0.4%	2.6%	3.7%
T-HIST	0.5%	1.3%	1.3%	2.1%
ISIP	1.5%	2.6%	N/A	N/A

Min. steel assumed = 0.3%
No max. steel reinforcement content maintained for comparison purposes
Also assumed that area tension steel = side = compression steel
ISDC method includes design for ductility requirements with 50mm stirrup spacing.

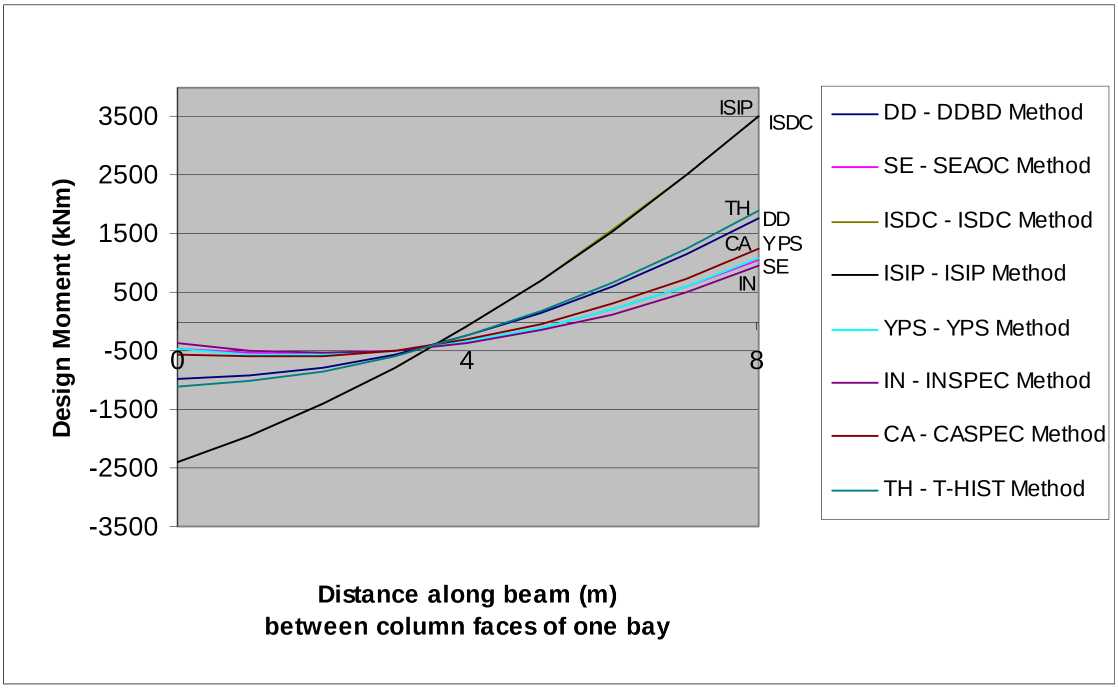
It is interesting to compare the reinforcement contents and beam flexural strengths for the DDBD method in relation to other methods. The DDBD method includes recommendations (Priestley and Kowalsky 2000) for frame structures that enable the designer to choose the design strength for the base storey columns and then determine the beam flexural strengths that satisfy equilibrium for the total building shears and these base storey column moments. Consequently, for Case Study 5, it is apparent that the DDBD method assigns a greater proportion of the design strength to the beams than the columns in relation to other methods.



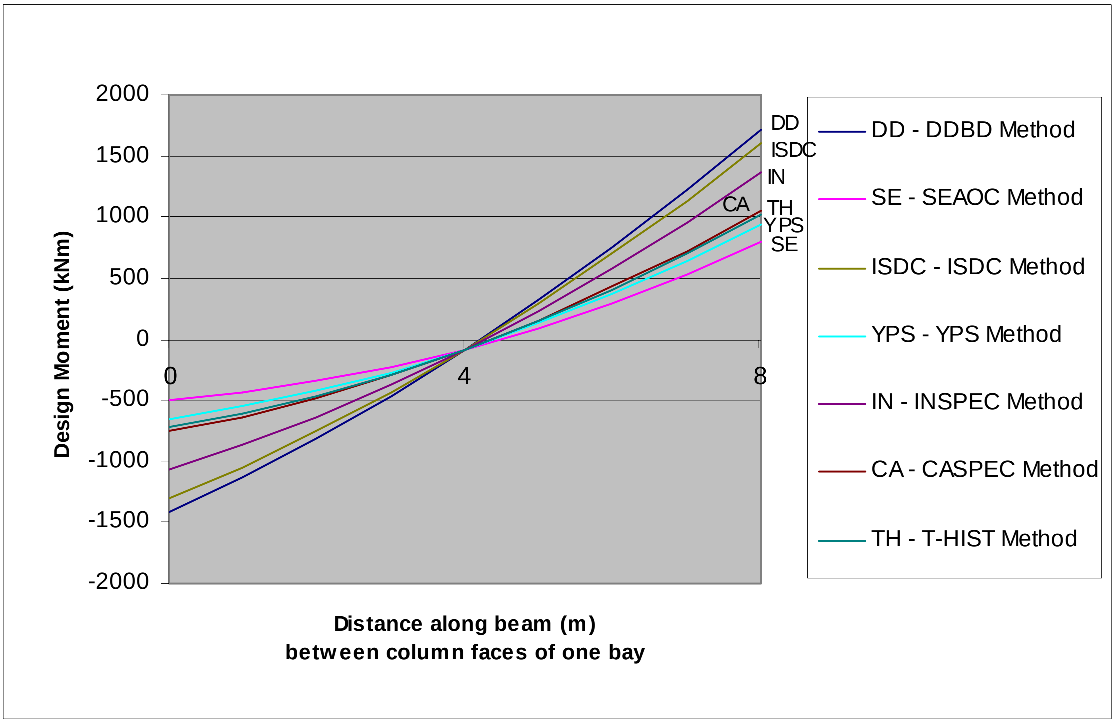
(a) (c)

(b) (d)

Fig. 4. Design Bending Moments for the wall structures; (a) Case Studies 1 (b) Case Study 2 (c) 3m Wall A of Case Study 3 and (d) 8m Wall F of Case Study 3.



(a)



(b)

Fig. 5. Design Bending Moments for the first floor beams of the frame structures; (a) Case Study 4 and (b) Case Study 5.

The longitudinal reinforcement content obtained by the ISDC method appears excessive and out of proportion to the base shear design strengths presented in Section 5.1. This large value is required by the method to maintain the inelastic rotation demands predicted for the EQ-IV design event. These large values do not indicate a limitation of the method as it is expected that in reality the inelastic rotation capacity would be provided by changing the section dimensions and increasing confining steel, rather than increasing the longitudinal steel which is not as effective.

6. Evaluation

In this section some general points related to the performance of all the methods are proposed and then confirmed by presenting the results of the time-history analyses. This section also identifies characteristics of each method that account for the large differences in design actions and the consequent variation in performance of the methods.

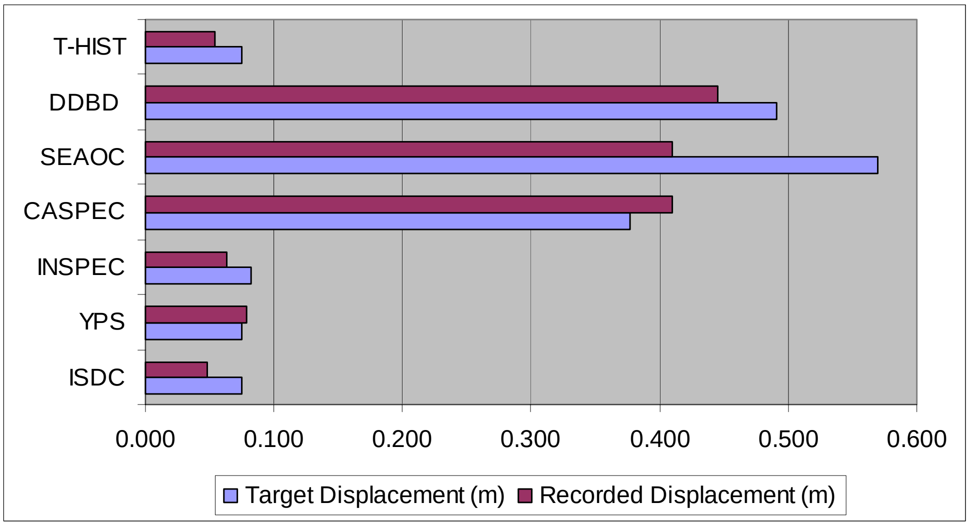
6.1. *General points relevant to all the design methods*

6.1.1. *Target displacements are successfully maintained*

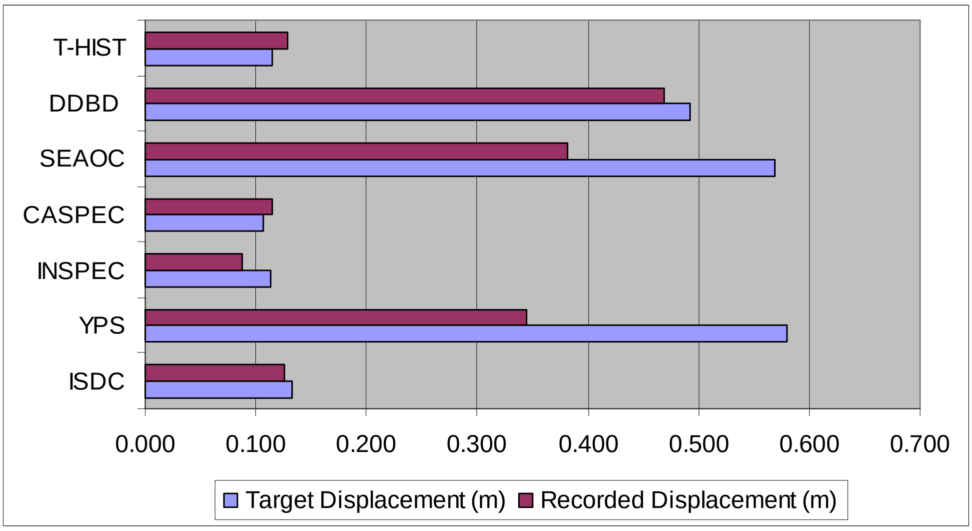
Perhaps the clearest means of demonstrating that the displacement based design methods really do work is achieved by comparing the design or “target” displacements with the maximum displacements recorded for the 3 time history analyses. Figures 6 and 7 show that the maximum recorded displacement rarely exceeds the design displacement. Each bar chart compares the displacements at the seismic level that governed design as identified in Table 9. Therefore, graphs that appear to show a large variation in target displacement include some methods that were governed by EQ-1 and others that were governed by EQ-4. The comparison is made of displacements recorded at an assumed effective height, except in the case of the ISIP method that designs to a roof displacement associated with a drift limit.

Where a method does not directly design for a target displacement, but rather for a certain displacement ductility limit, then the displacement ductility limit is multiplied by the yield displacement to give the appropriate target displacement.

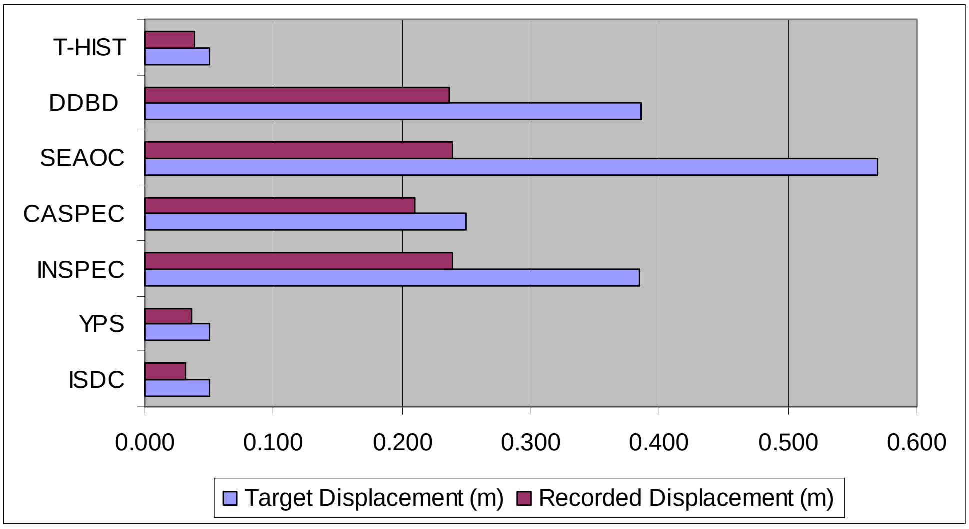
It can be seen in Section 3 that the spectra from the three artificial time histories used for the analyses exceed the design spectra for every period within the range considered for design. Therefore, the maximum recorded displacement should be seen as an upper bound to the peak displacement that would be observed if a time history exactly fitting the demand spectra had been used. Even considering the maximum displacements to be upper bounds, it is evident that all the DBD methods are successful, and that some methods are more efficient than others.



(a)

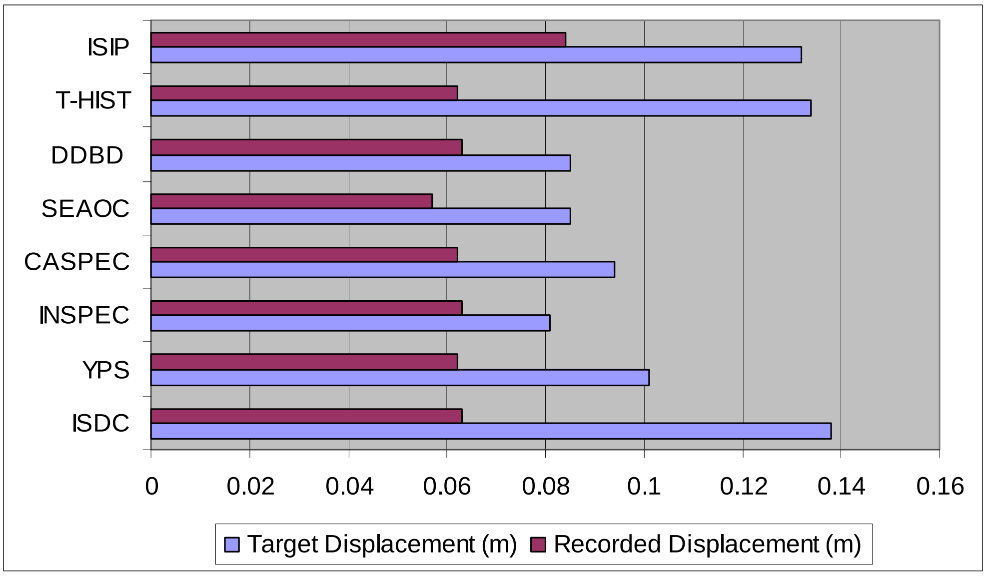


(b)

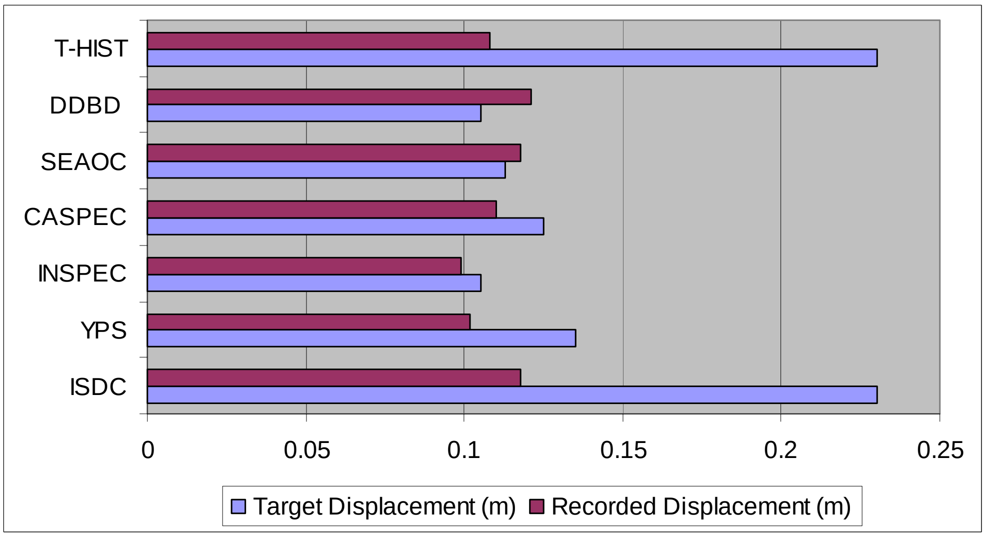


(c)

Fig. 6. Comparison of design displacements with maximum-recorded displacements for the wall structures; (a) Case Study 1, (b) Case Study 2 and (c) Case Study 3.



(a)



(b)

Fig. 7. Comparison of design displacements with maximum-recorded displacements for the frame structures; (a) Case Study 4 and (b) Case Study 5.

6.1.2. *Use of strength to control displacements*

The most striking result provided by the assessment of each method's performance is that the design strength has a low influence on displacements. This is shown most clearly by considering the range of design forces presented in Table 8 with Table 11 that presents the maximum drifts and ductility demands obtained for each method from the time-history analyses for both the EQ-I and EQ-IV levels. It can be seen that despite ratios of strength as great as four between methods, ratios of displacement, drift and ductility demand never exceed two. In fact, the ratio of displacements between two given methods is always less than or equal to half the ratio of the strengths for the same methods. This observation is in line with the relation between strength, stiffness and displacement as explained in the following paragraphs.

Table 11 Maximum Drift and Ductility values obtained from the time-history analyses of the case studies for each DBD method

	Method	EQ-I				EQ-IV			
		Inter-storey Drift		Ductility Demand		Inter-storey Drift		Ductility Demand	
		Peak	Average	Peak	Average	Peak	Average	Peak	Average
Case Study 1 Wall Structure	ISDC	0.47%	0.36%	0.63	0.49	1.6%	1.5%	2.7	2.4
	YPS	0.75%	0.65%	1.05	0.88	3.1%	2.8%	6.1	5.5
	INSPEC	0.63%	0.62%	0.83	0.80	2.7%	2.5%	5.3	5.0
	CASPEC	0.61%	0.49%	0.74	0.63	2.7%	2.4%	5.4	4.6
	SEAOC	0.61%	0.49%	0.74	0.63	2.7%	2.4%	5.4	4.6
	DDBD	0.76%	0.65%	1.06	0.88	3.0%	2.8%	5.9	5.5
	T-HIST	0.56%	0.45%	0.72	0.60	2.2%	1.9%	3.9	3.4
Case Study 2 Wall Structure with Flexible Foundation[5]	ISDC	1.05%	0.74%	0.87	0.63	2.3%	2.0%	3.0	2.7
	YPS	1.02%	0.79%	0.96	0.72	2.6%	2.4%	3.9	3.7
	INSPEC	0.82%	0.76%	0.81	0.76	3.1%	2.7%	5.8	5.0
	CASPEC	0.96%	0.78%	0.92	0.73	2.7%	2.6%	4.8	4.3
	SEAOC	0.86%	0.74%	0.87	0.70	2.6%	2.6%	4.5	4.3
	DDBD	1.01%	0.73%	1.06	0.74	3.1%	2.7%	5.8	5.0
	T-HIST	1.03%	0.79%	0.98	0.72	2.6%	2.4%	3.7	3.6
Case Study 3 Walls A & B[6] Irregular Wall Structure	ISDC	0.45%	0.35%	0.69	0.54	1.72%	1.52%	2.97	2.60
	YPS	0.47%	0.43%	0.70	0.66	2.11%	1.92%	3.73	3.41
	INSPEC	0.46%	0.43%	0.70	0.66	2.20%	2.06%	3.94	3.53
	CASPEC	0.47%	0.43%	0.70	0.66	1.98%	1.85%	3.50	3.29
	SEAOC	0.46%	0.43%	0.70	0.66	2.20%	2.06%	3.94	3.53
	DDBD	0.46%	0.43%	0.70	0.66	2.23%	2.06%	3.90	3.55
	T-HIST	0.51%	0.40%	0.80	0.60	1.70%	1.60%	2.99	2.77
Case Study 3 Wall F Irregular Wall Structure	ISDC	0.21%	0.18%	0.41	0.35	1.04%	0.84%	2.51	1.95
	YPS	0.30%	0.27%	0.58	0.53	1.90%	1.66%	4.34	3.83
	INSPEC	0.30%	0.27%	0.58	0.53	2.03%	1.89%	4.64	4.14
	CASPEC	0.31%	0.27%	0.58	0.53	1.74%	1.58%	4.00	3.64
	SEAOC	0.30%	0.27%	0.58	0.53	2.03%	1.89%	4.64	4.14
	DDBD	0.30%	0.27%	0.58	0.53	2.08%	1.88%	4.57	4.15
	T-HIST	0.28%	0.21%	0.53	0.41	1.09%	1.01%	2.57	2.34
Case Study 4 Regular Frame Structure	ISDC	0.38%	0.30%	0.78	0.60	1.4%	1.3%	3.1	2.9
	YPS	0.42%	0.35%	0.77	0.65	2.7%	2.4%	5.1	4.7
	INSPEC	0.40%	0.40%	0.78	0.75	3.3%	2.8%	5.0	4.8
	CASPEC	0.41%	0.36%	0.77	0.66	2.8%	2.4%	5.0	4.5
	SEAOC	0.40%	0.37%	0.71	0.65	3.1%	2.7%	5.1	4.7
	DDBD	0.40%	0.36%	0.78	0.67	2.4%	2.0%	4.2	3.6
	T-HIST	0.42%	0.36%	0.77	0.66	1.8%	1.4%	3.5	3.1
	ISIP	0.38%	0.30%	0.78	0.60	1.4%	1.3%	3.1	2.9
Case Study 5 Vertically Irregular Frame Structure	ISDC	0.65%	0.48%	0.62	0.49	3.3%	2.7%	2.5	2.2
	YPS	0.58%	0.53%	0.54	0.49	3.5%	2.8%	3.7	3.1
	INSPEC	0.64%	0.53%	0.52	0.45	3.1%	2.8%	2.7	2.4
	CASPEC	0.60%	0.51%	0.58	0.51	3.1%	2.8%	3.2	2.9
	SEAOC	0.69%	0.56%	0.62	0.51	3.6%	2.9%	4.1	3.1
	DDBD	0.62%	0.46%	0.64	0.49	3.2%	2.9%	2.9	2.4
	T-HIST	0.64%	0.54%	0.57	0.50	3.1%	2.8%	3.4	3.0

1. Displacement ductility demand obtained using the maximum displacement and an assumed effective height.
2. Inter-storey drift values obtained from maximum displaced shape.
3. Peak value refers to the largest of the maximum values obtained from the 3 time-history analyses.
4. Average value refers to the average of the maximum values obtained from the 3 time-history analyses.
5. Ductility demand values determined accounting for foundation rotation
6. Ductility demand values are presented for 6m Wall B only because these are larger than for the 3m Wall A.

To understand how displacements may be related to strength, firstly note that the displacement of a structure with a given damping is linearly related to the period as shown in Figure 8 and stated in Equation 2. This linear relation holds for medium range periods applicable to these case studies.

$$S_d \propto T \tag{2}$$

where, S_d = Spectral displacement
and T = Structural period

Secondly, consider the equation for the fundamental period of an SDOF oscillator given in Equation 3.

$$T = 2\pi\sqrt{\frac{M}{K}} \tag{3}$$

where, M = Mass
K = Stiffness

Using the argument, as presented by Priestley and Kowalsky [1998] and Priestley [1998], that stiffness is proportional to strength, relation 4 is written.

$$T \propto \sqrt{\frac{1}{V}} \tag{4}$$

where, V = Strength

Finally, by combining relations 2 and 4 one obtains Equation 5 that shows that the displacement is proportional to the square root of the inverse of the strength.

$$S_d \propto \sqrt{\frac{1}{V}} \tag{5}$$

This final relation corresponds with the results observed in these case studies. For instance, consider the ISDC and YPS results for Case Study 1.

The ratio of design strengths between methods is:

$$\frac{V_{\text{base ISDC}}}{V_{\text{base YPS}}} = \frac{9480}{3008} = 3.15$$

And the ratio of EQ-1 peak drifts is:

$$\frac{\delta_{\text{ISDC}}}{\delta_{\text{YPS}}} = \frac{0.36}{0.65} = \frac{1}{1.806}$$

This ratio, as expected is approximately equal to the square root of the inverse of the strength,

$$\sqrt{\frac{V_2}{V_1}} = \sqrt{\frac{1}{3.15}} = \frac{1}{1.77} \approx \frac{1}{1.81} = \frac{Disp_1}{Disp_2}$$

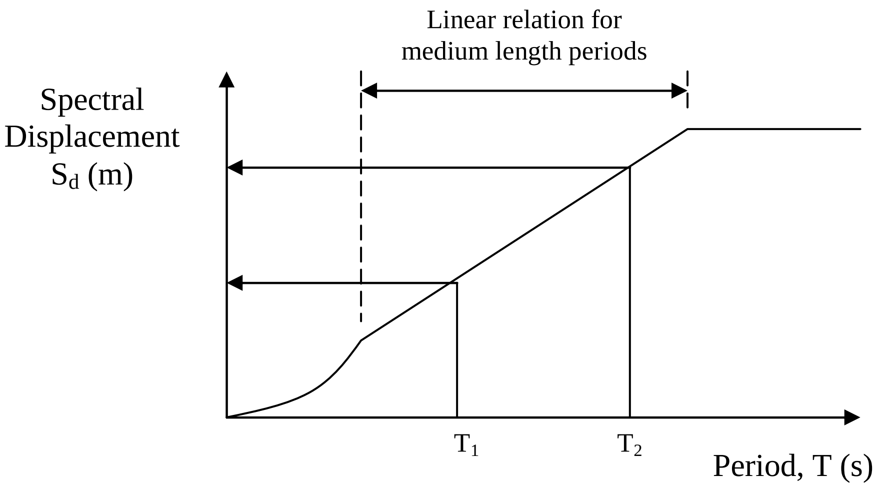


Fig. 8. Displacement response spectra showing linear relation between displacement and period

In other case studies the displacements were even less dependent on strength than Equation 5 implies. For Case Study 2, the wall structure with flexible foundations, Equation 5 would predict that the structure with strength as obtained from the DDBD method would develop displacements 28% greater than the structure with strength as obtained from the YPS method. However, the drifts obtained for the DDBD model were only 12.5% greater than those obtained using the YPS model.

The foundation flexibility, K_{fdtn} associated with Case Study 2 effectively reduces the total building stiffness K_{total} below the value of stiffness, K, associated with an equivalent structure having rigid foundations, as shown in Equation 6.

$$\frac{1}{K_{total}} = \frac{1}{K} + \frac{1}{K_{fdtn}} \tag{6}$$

Since a constant value of foundation stiffness is used for all the methods, it is apparent that the foundation stiffness will act to reduce the ratio of actual stiffness between different methods. For example, consider two structures that have a structural stiffness equal to 4.0 and 2.0 respectively, giving a structural stiffness ratio of 2.0. The ratio of total building stiffness for these two structures on a foundation with a constant stiffness equal to 1.0 is reduced from 2.0 to only 1.2.

The reduction of total stiffness due to foundation flexibility clearly accounts for the reduced influence strength has on displacements for Case Study 2. However, in Case Studies 4 and 5 drift ratios are also less than that predicted by Equation 5. This could be explained by the large elastic periods of these structures. As a consequence of long elastic periods the structures do not need to develop large levels of inelasticity before entering the equal displacement region of the response spectra (seen as the flat portion of the spectra shown in Figure 8). Within the equal displacement region of the spectra the structures are expected to have the same maximum displacements as is observed for some of the methods with relatively low design strengths in Case Studies 4 and 5.

6.1.3. *Inadequate strength distribution procedures for the EQ1 performance level*

The performance assessment of Case Study 3 carried out with time-history analyses using *Ruaumoko* 3D provides drifts and displacements that indicate the design methods ensure target design parameters are maintained, as shown in Table 11. However, preliminary results for Case Study 3 obtained using a fibre element model that could better model cracked section stiffness at low displacements, indicated that the drift and displacement ductility demands for the EQ1 performance level will generally be close to or above the design limits. This observation is understandable when considering the base shear distribution procedures adopted by the design methods. All of the methods adopt well established procedures such as those described in Paulay and Priestley [1992] that distribute the design base shear to each wall in relation to its length cubed (or squared as in the DDBD method) with an adjustment of this shear for torsion effects. The distribution procedures do not consider individual wall yield displacements in relation to the target displacement. However, Figure 9 shows a reasonable target displacement for the EQ1 level in relation to individual wall yield displacements. The "reasonable" target displacement is selected on the basis that some ductility demand can be accepted in the critical element of the structure; in this case the 8m wall. It is of significance that the target displacement is around half the yield displacements of the 3m walls (Walls A and C from Case Study 3).

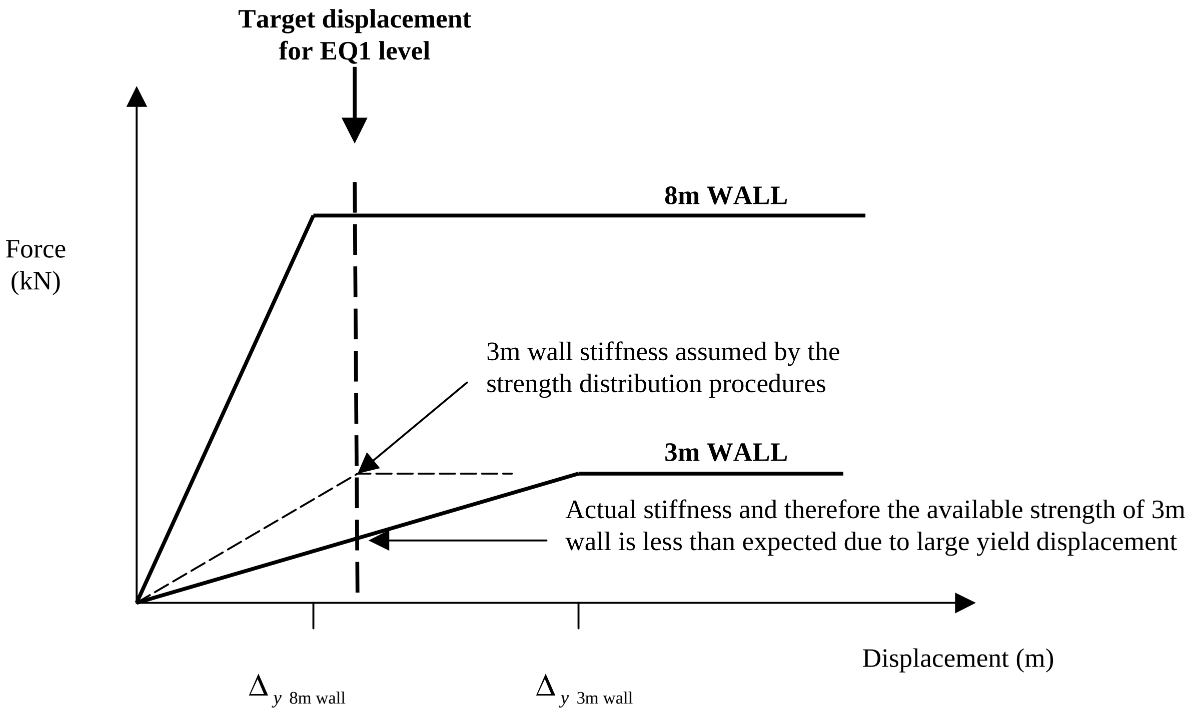


Fig. 9. Possible serviceability target displacement for Case Study 3 in relation to the yield displacements of the different length walls

It can be seen that by neglecting to consider the yield displacements of the walls in the base shear distribution procedure the methods are unable to provide a system with sufficient stiffness to develop the design strength at the target displacement. The design base shear will instead be developed only when all the walls have yielded, i.e. at the yield displacement of the smallest wall, in this case the 3m wall.

As the stiffness of the system is lower than intended with the design strength distributed in this manner, the elastic period increases, as does the peak displacement of the structure. This has the effect of magnifying the displacement ductility demands on the longest walls, for which yielding is expected before the target displacement is attained. It also raises the inter-storey drifts and therefore the non-structural damage increases for the whole of the building.

An improved distribution procedure would include magnification of the distributed base shear for each wall by considering the ratio of the yield displacement to the target displacement as described in further detail by Sullivan [2002].

6.1.4. *Twist induced period lengthening*

It is worth pointing out that no recommendations to account for twist-induced period lengthening were found for any of the methods. This period lengthening occurs in structures such as Case Study 3 because the twist of the structure causes the centre of mass to displace further than the centre of rigidity. For methods that use a target displacement to obtain the required stiffness, it appears that an initial estimate of the twist could be used to increase the target displacement. This larger target displacement would then result in design for a longer period. However, neglecting this twist is unlikely to result in non-conservative design since the structure would essentially be given a shorter period and higher strength than what is necessary to maintain the target displacement.

Those methods that proceed with design using the period of the structure, implicitly allow for the building twist lengthening the period of the structure provided that the model used to obtain the period estimate adequately models the twisting displacement of the mass.

The remaining part of this section discusses features of each design method that are considered to account for the observed variation in the design strengths and performance.

6.2. *ISDC method*

Design actions for the ISDC method developed by Panagiotakos and Fardis [1999] are generally higher than the other methods. This is due to the recommendation that an uncracked model of the structure be used in the initial elastic design to EQ-I. Other methods, including force based design methods, recommend the use of section properties modified to allow for cracking observed in structures at the point of yield. An uncracked model is stiffer with shorter periods of vibration than an identical structure with cracked section properties. Since typical acceleration response spectra are greater at short periods the stiffer uncracked model attracts a high base shear coefficient. This observation explains why the design strength for all case studies was governed by the initial elastic design to EQ-I.

Panagiotakos and Fardis [1999] present a flowchart of the method that indicates that uncracked section properties should be used. However, the issue of using a cracked model is also raised within their paper. After comparing the SEAOC EQ-I and EQ-IV spectra it was interpreted that uncracked section properties were appropriate for these investigations. Through further correspondence with the authors of the method it appears that they did not intend the use of uncracked sections in these case studies.

Note that even though the method does not directly control drifts for the EQ-I earthquake, it was one of only two methods to maintain the average drift below the target value for Case Study 5. This success is attributed to the stiff uncracked model used for elastic design to EQ-I.

The performance assessment indicates that the method may be applicable to wall structures despite being developed for the design of frame structures only.

6.3. *ISIP method*

Design in accordance with the ISIP method presented by Browning [2001] ensured that the drift and ductility values obtained from time-history analyses were well within the design limits.

The method aims to provide only a minimum ‘threshold’ value of strength as a consequence of previous findings [Shimazaki and Sozen 1984; Qi and Moehle 1991; Lepage 1997] that base shear strength has only a small influence on drift control. A similar observation has also been made in the course of these cases studies in displacement based design. It is therefore surprising that the strength provided by this method is more than four times that of other design methods that also satisfy drift and ductility limits after time history analyses. The larger design strength is attributed in part to the use of gross uncracked section properties in determining the structural period that is used with acceleration response spectra to obtain the design base shear coefficient. When values for period that allow for the effects of cracking are used, the design base shear reduces by about 30%. However, the lower base shear is still significantly higher than that obtained from most of the other design methods. The larger design strength may also be due to the use of an acceleration amplification factor intended to allow for a wide range of ground motions for systems with 2% damping. Browning does not present other amplification factors that may be more suitable applicable for the frame structure examined in Case Study 4. Note that the requirement adopted in these case studies to maintain constant member dimensions restricts the efficiency of the ISIP method that relies on an iterative proportioning procedure. It is expected that more cost efficient designs would be obtained if the member proportions were changed during the design procedure.

It is considered that an inconsistency exists in the current design procedure for the ISIP method. Browning [1999] recommends that the member sizes are increased until the target period is less than the initial period of the structure. The next step is to check the strength of the yielding members and columns and increase this strength if required in order to satisfy minimum values. If the procedure accepts that strength controls stiffness at first yield then the dimensions of the structure should be determined at the same time as the strength is assigned. However, it appears that the effect of strength on stiffness is not considered, or at least it is not utilised, when sizing the members because the method uses uncracked section properties.

6.4. *YPS method*

The YPS method presented by Aschheim and Black [2000] is one of two methods examined here that use inelastic spectra in the design process. It is also one of two methods that utilise ADRS format for design purposes. The result of this combination is a method that allows design for several limits states in one step once the spectra have been constructed. The method performs well maintaining the drift and ductility limits and providing low base shear strength and consequently cost effective design in relation to other methods.

Integral to the success of the method is a good estimate of yield displacement. In the case studies the method has benefited from the use of yield displacements obtained using equations for yield curvature presented by Priestley and Kowalsky [2000]. When one considers the shape of the YPS it becomes apparent that yield displacements more than say 20% from the actual displacement could result in an increase or decrease in shear strength provided some 50%. Aschheim and Black [2000] do suggest that designers can obtain the yield displacement from a pushover analysis, however, this requirement would add significant time to an otherwise relatively fast design procedure.

6.5. *INSPEC method*

The INSPEC method presented by Chopra and Goel [2001] successfully limits the drift demands within the design limits, while providing a relatively low level of strength.

The method is not complete as a design tool since it does not provide recommendations for structural types other than SDOF oscillators, does not recommend a procedure for distribution of shear and does not suggest procedures for structures with flexible foundations.

The method does not discuss the principles by which the method achieves success other than to commend the use of inelastic spectra. During the design process it was noted that the level of ductility of the SDOF oscillator does not affect the inelastic displacement spectra at intermediate and long periods. This approximation was shown to be relatively accurate by Miranda and Bertero [2001] through a large number of inelastic time-history analyses. The method takes advantage of this characteristic behaviour of SDOF oscillators by implicitly suggesting that initial stiffness therefore governs the displacement response of a system. The system success then comes from identifying the initial elastic stiffness required to achieve the target displacement and multiplying this by a good estimate for yield displacement.

Considering the important role that the initial stiffness plays in the INSPEC method note that for a structure of given dimensions the yield displacement can be considered constant [Priestley and Kowalsky 2000]. Therefore, for such a structure a unique level of strength will exist that satisfies the minimum initial stiffness as shown in Figure 10. After a designer chooses an acceptable plastic rotation the target displacement can essentially be fixed and known immediately (an exception occurs for structures with flexible foundations). Therefore, only one level of strength will satisfy the initial stiffness that is obtained from the inelastic displacement response spectra.

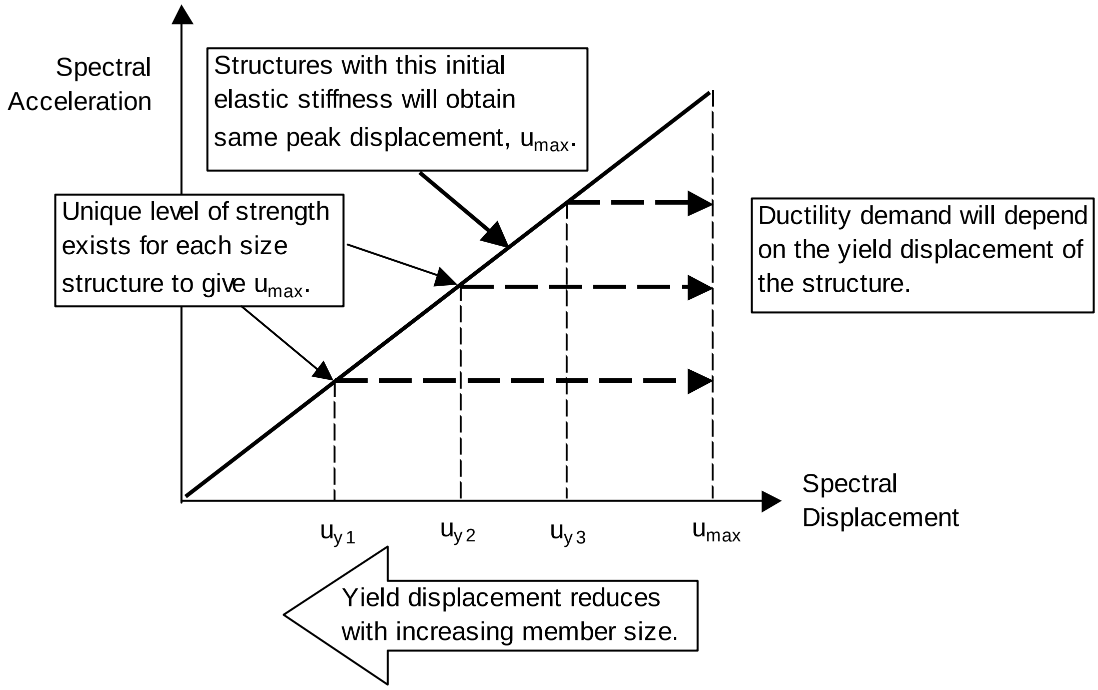


Fig. 10. Relationship between the structural dimensions, yield displacement and ductility developed considering the role of initial stiffness used in Chopra's design method

Keeping the above points in mind, it is also worth noting that there is no recommended procedure through which to estimate member sizes. Interestingly, considering that initial elastic stiffness is believed to control the displacement response, it should be possible to quickly optimise the strength and proportions of the structure such that the drift limit corresponds to the inelastic rotation limit. For instance, if structural drift controls the target displacement, dimensions could be increased and a higher level of ductility developed. Alternatively, if inelastic rotation capacity is controlling, then dimensions could be reduced and the strength and yield displacement increased. A complication with this procedure is that the inelastic rotation capacity for larger members is less and as a consequence even though they can afford lower strength they would need a higher percentage of confinement steel. Obviously for the smaller walls the reverse is true, in that the percentage of confining steel required for inelastic rotation could be reduced even though the longitudinal steel for strength would be increased.

6.6. *CASPEC method*

The CASPEC method developed by Freeman [1998] is intended for the assessment of existing structures. This project utilised the method for design by using the EQ-I demand spectra to set the initial base shear strength. Results indicate that the procedure performs well since target design parameters are not exceeded and the required strength is not excessive. For wall structures with irregular layout it is important that the pushover analysis identifies the critical elements of the structure rather than considering the system drift and ductility values. After the assignment of strength initially, pushover analysis for the EQ-I limit state for irregular structures is likely to improve the performance of the design procedure. However, considerable computation time can be saved where a simple structure is being considered and pushover analysis is not required to obtain a reliable capacity curve.

6.7. *The SEAOC method*

The SEAOC [1999] method performs relatively well giving cost efficient design and in general maintaining the target design parameters. One point where the method may be improved could be through the identification of an approximate yield displacement for the structure.

Target displacements for the design drift limit could be checked against the yield displacement and the required effective stiffness adjusted if necessary. It was seen in Case Study 5, where the system yield drift of 1.0% was twice that of the design drift of 0.5%, that the design strength calculated should have been factored to the required yield strength by considering the yield displacement in relation to the target displacement. By multiplying the calculated effective stiffness by the displacement corresponding to the target drift, the effective stiffness provided was actually equal to half of that intended.

Yield displacement values would also enable the design of structures with flexible foundations since system ductility and damping values could be calculated relatively quickly. Furthermore, yield displacements would enable accurate calculation of the required initial stiffness and therefore the required yield strength for the structure, rather than relying on an estimate for the overstrength factor.

6.8. *DDBD method*

The DDBD method developed by Priestley and Kowalsky [2000] performs well giving cost efficient design while maintaining the target design parameters. Performance appears to be excellent in all case studies. However as already noted, preliminary time-history analyses for EQ1 of Case Study 3 suggested that for low intensity earthquakes the method may benefit from alternative procedures for the distribution of strength to walls of differing length.

The suggestion that the DDBD method develops higher displacements in relation to other methods for the serviceability earthquake is not surprising given the points made in Section 6.1.3 and considering the recommended procedure of distributing the design base shear to the walls in proportion to their length squared rather than their length cubed as done in the other DBD methods. While this distribution procedure is most rational [Priestley and Kowalsky 2000] for structural response where all walls are expected to be yielding, it can cause larger displacements and ductility demands for the frequent EQ-I than other methods with the same total design base shear. By distributing the design base shear in proportion to the wall length squared, more strength is assigned to the shorter 3m walls and less to the longer 8m walls in comparison to other design methods. As the strength of the shorter walls cannot be fully developed at the EQ1 design displacement (refer Section 6.1.3) a smaller percentage of the design base shear can be developed in comparison to other methods at the design displacement. The structure is consequently provided with much lower elastic stiffness than anticipated and therefore develops larger displacements and ductility demands than desired.

Another point regarding the DDBD method relates to the use of an assumed displacement profile that is integral in the design process. Currently, displacement profiles are provided by

Priestley and Kowalsky [2000] for wall structures assuming that the code drift limit governs design. Indeed, it has been shown by Kowalsky [2001] that a code drift limit of 2.5% will be critical for the design of walls with aspect ratio greater than one. However, the data presented by Kowalsky [2001] also shows that if the code drift limit is 3.5%, as recommended in the SEAOC blue book, then walls of aspect ratio around three to five may be governed by inelastic rotation capacity. Since this inelastic rotation demand is likely to be developed at the base of a cantilever wall it is assumed that a linear displacement profile would be utilised. However, a linear profile and perhaps the current displacement profile recommended for cases where the code drift governs, would not account for higher mode effects that can be rather significant as shown by the time history results for the T-HIST method presented in Section 5.

Despite the dependence of the method on an assumed displacement profile, the method performed well for the vertically irregular frame structure of Case Study 5. It is noted however, that the design of Case Study 5 was controlled by the EQ-1 drift limit and it would be interesting to consider the performance of the method applied to a structure with even greater irregularity that is governed by the EQ-IV event.

6.9. *T-HIST method*

Design in accordance with the T-HIST method [2001] ensured that the drift and ductility values obtained from time-history analyses were well within the design limits.

It appears that the design procedure for EQ-I could be made more efficient since throughout the case studies it was always the strength for EQ-I that governed the design. To improve the design procedure for EQ-I the method should also take into account the drift limit. The potential benefit of this was seen in Case Study 5 where the average maximum drift of the structure exceeded the recommended maximums even though the ductility demand was well within the acceptable value. The method could combine the drift check it performs for the fairly low intensity EQ-II event. The fact that the drift is checked for EQ-II but not EQ-I suggests that the method was disadvantaged by the SEAOC blue book EQ-I drift limits that were shown to be less than the yield drift for the frame case studies. The method could also adopt a check of material strains associated with acceptable damage at the serviceability limit state rather than a ductility limit of 1.0 as done in other methods.

This method may be considered unnecessarily complex and time consuming for most design situations since multiple time-history analyses are required. However, the method does provide a thorough procedure that can be used when the likely inelastic response of a structure appears difficult to predict.

7. Conclusions

Eight different displacement based design procedures have successfully been applied to 5 different structural forms. Application has highlighted the strengths and weaknesses of each of the methods. We see that all the DBD methods successfully maintain the target design parameters even though significant variation in design strength exists.

It is considered that many of the DBD methods could benefit from the use of alternative target design parameters that can still ensure accurate performance based design. The different target design parameters adopted by the methods cause differences in design strengths and yet have relatively little influence on performance. It would be interesting to compare the strengths obtained by the DBD methods when they utilise a common target displacement, associated with an agreed set of design parameters.

The large variation in design strengths between methods has a relatively low influence on peak displacements due to the relationship between stiffness and displacement. The influence was observed to reduce with the inclusion of foundation flexibility and where the response entered the equal displacement range of the spectra.

Limitations have been identified for all of the eight displacement based design methods considered. These limitations can be considered as minor in some instances and rather major in others. However, it is also considered that all of these limitations can easily be overcome now that they have been identified.

Since there are developments that can be made to all of the design methods it is currently difficult to propose one method over another. However, it is suggested that for regular structures, designers can refer to the relatively fast DDBD method that provides the most complete set of recommendations and obtains good performance. These recommendations could also assist designers using the INSPEC method that is again relatively fast and performed well in the case studies but does not provide a comprehensive set of guidelines. The YPS method appeals because of its speed in designing structures to a number of limit states, although it relies on a good estimate of the yield displacement. The DDBD, INSPEC and YPS methods appear to perform adequately when applied to irregular structural forms, however, given the limited scope of these investigations further study is needed to verify this observation. Methods that appear better suited to irregular structural forms include the CASPEC and T-HIST methods and the YPS method when it incorporates a pushover analysis to obtain an accurate value for the yield displacement. The ISIP and ISDC methods provide designers with relatively simple means to design for frame structures, noting that the use of cracked section properties in these methods is expected to give safe and cost-efficient designs.

This investigation concludes that the future for performance based design is bright. Designers have a range of displacement based design methods available to them, all of which have been shown to perform well in real life design examples.

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